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OPTIMAL POLLUTION CONTROL MODELS FOR INTERCEPTOR SEWER SYSTEMS

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To Mum, Dad and Eleanor

and

In Memory of Aunty Mair

ABSTRACT

Interceptor sewer systems are designed to alleviate environmental impacts of raw sewage discharge into receiving waters by intercepting combined sewer outfalls and diverting the flows to a treatment works prior to ultimate discharge into the receiving waters. However, they are designed to overflow during medium/heavy storm events and over-spill to the receiving waters through combined sewer overflow (CSO) structures. These intermittent discharges of CSOs to receiving waters can be significant.

Currently in the UK, most interceptor sewer systems are locally controlled, where the flows diverted to treatment are restricted to maximum settings (often to 'Formula A') based on sensed data at each outfall location. This type of control procedure is deficient because it does not ensure full utilisation of the sewer system before over-spills occur. The use of volumetric-based or pollution-based global control reduces overflows by making more efficient use of the entire sewer system. There has been a significant amount of research in volumetric-based global control, as the literature review in the study shows, but very little in pollution-based global control, which this study focuses upon.

The aim of this study is to develop novel methods of controlling large interceptor sewer systems to minimise total pollutant over-spill loads to the receiving waters, using a unique 'slug flow' approach. This approach allows the pollution-based control actions to be determined very quickly, enabling the application of the developed optimal pollution control (OPC) model in real time.

The results have shown the viability of the slug flow approach and the developed OPC model. Furthermore, the validity of the slug flow approach has been demonstrated where the interceptor sewer water profiles generated from the OPC strategies were generally conservative. Additionally, the interceptor sewer hydrographs from the OPC model correlated well with those generated from the WALLRUS sewer flow simulation package.

Extensions to the OPC model have shown its robustness by including non-linear equations that govern the flow through CSO chambers without significantly affecting its computational efficiency. To illustrate this, the pollution-based control strategies for an entire month's rainfall data were determined in minutes by the application of OPC to a case study Liverpool Interceptor Sewer system. Therefore, the OPC model is entirely suitable for application in real time.

The study has shown that significant reductions in pollutant over-spill load were achieved with volumetric-based global control when compared to fixed local control, although further improvements were achievable with OPC. Extensive results from the application of the OPC model in a typical year of rainfall in Liverpool have shown these further improvements to be significant. The OPC model therefore potentially offers the ultimate in interceptor sewer performance.

CONTENTS

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LIST OF FIGURES

CHAPTER 2

CHAPTER 3

CHAPTER 4

CHAPTER 5

CHAPTER 6

CHAPTER 7

CHAPTER 8

LIST OF TABLES

LIST OF NOTATIONS

A list of symbols and abbreviations used in this study is given below, although most of the symbols used in the literature review sections are not included.

α	Pollutant concentration factor [typically mg/l]
αv	Chamber pollutant concentration factor [typically mg/l]
a	Area of Orifice [m^2]
A	Cross-sectional area of flow [m^2]
ADWP	Antecedent dry weather flow period [hours]
A_{stor}	Storage Chamber Area [m^2]
C	Interceptor sewer capacity [cumecs]
c_d	Coefficient of discharge of orifice
CSO	Combined Sewer Overflow
d_I	Decision variable in DP
DOE	Department of Environment
DP	Dynamic Programming
DWF	Dry Weather Flow [cumecs]
Δt	Solution time step [seconds]
EQO	Environmental Quality Objective
FLC	Fixed Local Control
FWR	Foundation of Water Research
g	Acceleration due to gravity [m/s^2]
h	Head [m]
h_0	Height above mid-point of orifice [m]
HRS	Hydraulic Research Society
i	Intercept point
IDF	Intensity-Duration-Frequency
j	Time step
L	Length of slug of fluid [m]
LP	Linear Programming

MEPAS	Mersey Estuary Pollution Alleviation Scheme
n	Number of intercept points
O	Overflow from chamber [cumecs]
OPC	Optimal Pollution Control
P	Wetted perimeter [m]
PEAKEDNESS	Peak rainfall intensity/average rainfall intensity
P_w	Penstock width [m]
P_h	Penstock height [m]
P_{top}	Maximum level of penstock [m]
q	Flow rate to interceptor sewer [cumecs]
Q	Inflow rate from catchment [cumecs]
Qc	Continuation flow into interceptor [cumecs]
$Qcmax$	Maximum possible continuation flow rate [cumecs]
R	Hydraulic radius of interceptor sewer [m]
r_i	Cost of decision variable in DP
R	Hydraulic gradient [m]
S	Surface slope
S_i	State variable in DP
SSO	Storm Sewage Overflow
Stor	Storage volume [m^3]
t_i	Time step position within the interceptor of intercept point i
RTC	Real Time Control
T	Time step
Tc	Time of concentration
Tr	Time step (run-off)
TSR	Time Series Rainfall
TSS_p	Peak suspended solids concentration [mg/l]
$TSS(t)$	Concentration of suspended solids at time t
UH	Unit Hydrograph
v	Mean flow velocity [m/s]
V	Storage chamber water volume [m^3]
VLC	Variable Local Control
$Vmax$	Maximum possible chamber retention volume [m^3]
y	Chamber level above invert [m]

y_1	Upstream water level of slug [m]
yi_1	Upstream invert level [m]
y_2	Downstream water level of slug [m]
yi_2	Downstream invert level [m]
y_{mean}	Mean water level of slug
WRc	Water Research Centre
WwTW	Wastewater Treatment Works

CHAPTER 1

INTRODUCTION

1.1 GENERAL INTRODUCTION

The development of the modern sewer system coincided with the industrial revolution and the associated urbanisation. The enormous population influxes into towns and cities created many public health epidemics, such as cholera and typhoid. Consequently, sewer systems were constructed to discharge the raw sewage away from the urban areas into watercourses. The sewer systems could additionally be utilised to drain the cities during storm events.

Sewer systems can be categorised into two types, 'separate' or 'combined'. The separate sewer systems consist of two networks: one network transports the surface runoff directly into the receiving waters; and the second network transports the foul sewage to a treatment works prior to discharge to the receiving waters.

Combined sewer systems transport the raw sewage and surface runoff through a common network. Many of the urban drainage systems have been inherited from the 19th Century and are combined sewer systems because of financial limitations. Historically, the combined sewer flow was discharged directly into watercourses without prior treatment.

It is well recognised that the discharge of combined sewer flow into receiving waters has significant environmental implications (Ellis and Marsalek, 1996; Ellis and Hvitved-Jacobsen, 1996). The major impact is the increased levels of pollution in the water, with the consequential decrease in dissolved oxygen. This discharge has resulted in the devastation of many waterways with resultant fish deaths.

Interceptor sewer systems were conceived to alleviate environmental impacts of raw sewage discharge into receiving waters. They are designed to intercept the existing combined sewer outfalls and divert the flow to a treatment works prior to discharge into receiving waters. However, they are designed to overflow during medium/heavy storm events and spill to the receiving waters through combined sewer overflow (CSO) structures to prevent overloading of the downstream sewers and/or treatment works.

The intermittent discharge of combined sewer overflows to receiving waters can be significant. It has been estimated that the overflows contribute about one third of pollution load to urban streams and watercourses (Andoh, 1994). These inputs can exert both acute and accumulative impacts upon receiving water quality (Harresmoes, 1988).

Generally interceptor sewer systems consist of pipes, storage chambers and pumping stations. Such a system has been installed in the Liverpool area as part of the Mersey Estuary Pollution Alleviation Scheme (MEPAS). In this specific case, the interceptor sewer runs parallel and close to the banks of the Mersey Estuary where overflow structures form the junction between the existing combined sewer outfalls and the lower level interceptor. The interceptor system is very complex being twenty-nine kilometres long with twenty-six interception points. Therefore, the sewer flow time within the interceptor is very long and there are many control points.

1.2 OUTLINE OF THE PROBLEM

Sewerage systems have historically been designed using passive technology, where the system performance criteria was established at the design stage and thereafter fixed for the life of the scheme. An example of this approach is the Sewerage Rehabilitation Manual (WRc, 1983), which provides planning and design procedures for the management of flooding and structural dereliction. In these conventional solutions the dynamics and flexibility in operating the system were neglected. Theoretically, if no dynamic control is applied, the capacity of the system will only be fully utilised when it is loaded with the design load. Therefore, by definition, for all other loadings the system will perform sub-optimally.

Interceptor sewer systems have critical locations (control points) where the sewer flow may divert to treatment or over-spill to the receiving waters, after possible retention in overflow chambers. These locations may be passive or active; active control devices can move (and are therefore controlled) whereas passive devices have no moving parts. Currently in the UK, most interceptor sewer systems that have active control devices are locally controlled, where the flows (from the original outfalls) diverted to treatment are restricted to maximum values (a fixed setting) based on sensed information at each outfall location. This type of control has improvements over traditional passive control because the maximum flow setting is achievable longer. However, it is likely that some overflow structures or outfalls will spill during storm events even though there may be spare storage at other overflow structures or within the interceptor itself.

The current fixed local control systems satisfy the requirements of the regulatory authorities. Conventionally in the UK, each overflow structure or outfall would have its own consent imposed by the Environmental Agency, which is often the 'Formula A' setting. Additionally, Environmental Quality Objectives (EQO) have been introduced that take the receiving water body and its particular characteristics into account for the definition of discharge criteria (FWR, 1994).

Considerable environmental improvements, in terms of pollutant load reduction to the receiving waters, can be made with the use of global control strategies. Here the control devices in the interceptor system would be operated more effectively in response to the prevailing circumstances by utilising the full capabilities of the system. Hence, volumetric global control uses information on flows throughout the system to generate control strategies which will keep the interceptor as full as possible for as long as possible. There has been a considerable amount of research published on the control of sewer systems in general, although much of the work has been based on the use of detailed hydraulic models for sewer simulation. The disadvantage of such methods is that considerable computer time is required to investigate the consequences of a single set of control decisions. The determination of an optimal set of control decisions increases this computer time by several orders of magnitude.

It is hypothesised that pollution-based global control can make further environmental improvements by developing control strategies for the entire system so that the total pollution over-spill load to the receiving waters is minimised. The objective of this system ensures that the 'dirtiest' sewer flows are retained in the system and the 'cleanest' flows over-spilled to the receiving waters. Such a control system has to predict when and where 'dirty' flows from contributing catchments will reach the interceptor sewer and arrange that there is sufficient capacity in the sewer to accommodate them. A pollution-based global control system represents a move beyond the local discharge consents for each outfall, towards the possibility of a global consent for an entire interceptor sewer system. This study therefore aims to make a contribution to this area, where there has been very little research published, by developing a robust, computationally efficient optimal pollution control model.

1.3 OBJECTIVES OF THE STUDY

The primary objective of this research study is to develop novel methods of controlling large interceptor sewer systems to minimise total pollutant over-spill loads to receiving waters. The key aspect of such a model is computational efficiency since many control iterations are required to determine the optimal control strategy. To illustrate this, an interceptor diverting sewer flows from twenty outfalls has over one million possible control states at any point in time, if each control device can be either open or closed. An entire control strategy must evaluate these at many points in time.

Contrary to many other investigations in this area, this study focuses on developing a control model from a formal optimisation perspective using a relatively crude hydraulic representation of the sewer, which is termed the 'slug flow' approach. It is envisaged that this approach will allow the optimum pollution-based control decisions to be found very quickly. The model is based on the 'slug flow' approach in which sewage diverted to the interceptor at any point is represented as a 'slug' of water advecting at pipe-full velocity down the sewer. Slugs do not interact with the immediate upstream or downstream slugs and are incremented by sewage diverted to the interceptor at successive intercept points.

The control models developed in this study are tested on simple interceptor sewer systems to confirm the viability of the approach adopted. A thorough verification of the slug flow approach is conducted to evaluate its validity. The relevant hydraulics of interceptor sewer systems are included to develop a control model for real life interceptor sewers. A case study interceptor system is used with historical rainfall series to illustrate the potential reductions in pollutant over-spill loads. Considerable reductions are achieved with the optimal pollution control model compared to the current fixed local control and volumetric-based global control procedures.

1.4 LAYOUT OF THE THESIS

Chapter 2 gives a literature review of the principles of storm drainage. It gives a description of hydrologic and hydrodynamic theories used in urban drainage system design and simulation, from precipitation through rainfall-runoff relationships. The processes of sewer water quality modelling are described with a discussion on the 'first flush' effect, including a methodology for its peak prediction. An overview of some of the major commercial computer packages modelling these processes is given in Appendix 1. The impacts of discharges from sewer systems on the aquatic environment are presented, illustrating the necessity for the improved performance of sewer systems. *Chapter 2* also gives a description of the limitations of current sewer system control and the potential of real time control.

Chapter 3 gives a literature review of the principles of operation and control and gives an insight into general control concepts and procedures. A description of the formulation of an operational problem is given with a review of the methods used for its solution. A thorough description of Linear and Dynamic Programming is given as these methods are used within this study. Finally, a description is given of the uncertainty in mathematical modelling.

The remaining chapters in the thesis describe the innovative work undertaken in this study using the unique 'slug flow' approach. *Chapter 4* presents the development of an idealised interceptor control model from first principles and provides a complete description of the unique 'slug flow' modelling approach used throughout this study.

The formulations of the optimisation modules are shown and the initial model assumptions are explained. The models are tested on a fictitious test case to confirm the viability of the slug flow approach and the optimisation procedures.

Chapter 5 describes the verification of the slug flow approach in the control models using a post-processing hydraulic verification routine developed in this study, which is validated against WALLRUS (HRS, 1991). The uncertainties regarding sewer flow dynamics are also described.

Chapter 6 describes the extensions of the idealised interceptor sewer control models to better represent real-world systems. A thorough description is given of the control model extensions to include combined sewer overflow chambers. The extended model is tested on a test case interceptor sewer system.

Chapter 7 presents the testing of the full interceptor sewer control models on the Liverpool sewerage system as a case study in historical rainfall events. A historical background is given on the Liverpool Sewer System with a description of the Mersey Estuary Pollution Alleviation Scheme (MEPAS). The case study is described and the optimal pollution control model is applied in a typical year of rainfall and compared to the existing control system (fixed local control) and an alternative volumetric-based control system (variable local control). A brief evaluation of the sensitivities in the control procedures to moving storms is given. The overall results are presented and discussed.

Chapter 8 presents some conclusions from the study and recommendations for future work.

Various appendices are presented at the end of this thesis showing: an overview of some of the commercial computer packages in use for sewer simulation *(Appendix 1)*; detailed results of the idealised interceptor control model *(Appendix 2)*; examples of operational problems solved using linear and dynamic programming *(Appendix 3)*; calculations of the peak concentrations in suspended solids in a typical year

(Appendix 4); a User Guide for the OPC model *(Appendix 5)*; and a list of supporting papers *(Appendix 6)*.

CHAPTER 2

REVIEW I - STORM DRAINAGE

2.1 INTRODUCTION

This chapter gives a review of the methods used in storm drainage technology. The chapter is divided into the following sections:

- Hydrologic and Hydrodynamic Criteria;
- Water Quality;
- Impacts from Sewer Systems; and
- Potential of Real Time Control.

The hydrologic and hydrodynamic criteria section describes the rainfall-runoff processes and presents a historical development of the methods used by engineers for their simulation. An overview of some of the major commercial computer packages modelling these processes is given in Appendix 1.

The water quality section describes the processes involved in sewer water quality modelling. The 'first flush' effect in sewer systems is discussed with several definitions, including a methodology to predict its peak concentration. The common modelling approaches are also discussed. Finally, the complexities in sewer water quality processes are discussed explaining the limited success in the modelling of them. An overview of software for some of the major commercial computer packages modelling these processes is given in Appendix 1.

The impacts from sewer systems section describes the effects of discharges from sewer systems on the aquatic environment. These impacts illustrate the necessity for the improved performance of sewer systems.

Finally, the potential of real time control section discusses the inefficiencies of conventional sewer system design and operation. It is explained how the dynamics within the sewer system offers the potential for improved efficiency with the use of control techniques.

2.2 HYDROLOGIC AND HYDRODYNAMIC CRITERIA

There are examples of drainage systems built more than 2000 years ago during the Roman Empire in Europe and in China during the Han Dynasty. However, the major developments in storm drainage technology began in the 19th Century, as a consequence of the industrial revolution. Since then, engineers have employed various methods to determine the sizing requirements of the drainage systems. O'Loughlin *et al.* (1996) describes the rainfall-runoff processes and the historical development of methods used by engineers, which are summarised below.

The scientific advancement in urban storm drainage design began around 1850 when there was scientific quantification of the drain size, related to the area and the location to be drained, using tables or simple formula. A typical example is the drainage tables for sewer sizes and slopes prepared by a London surveyor, John Roe in 1852 (Chow, 1962). The next step was the separate consideration of the quantity of storm water to drain, a hydrological problem, and methods of draining it, a hydraulic problem.

For almost a century studies on urban storm drainage have been focused on techniques to determine a peak discharge for sizing sewers and other auxiliaries. Following the turn of the century, the point rainfall depth was considered as a function of the rain duration. The development of frequency analysis to establish the Intensity-Duration-Frequency (IDF) relationship for point rainfall began in 1910.

Historically, the main hydrologic criterion in urban drainage design was expressed by the return period of the design storm, which was estimated from an IDF relationship or from historical rainfall data. Fundamentally, the idea was that once a drain was sized to handle a design rainstorm it would be able to cope with all smaller rainstorms.

From the 1960's, the nature of the urban drainage problem changed from just draining the water to disposing it in an acceptable sanitary way. This change was brought about because of the unacceptable pollution of receiving waters. The detention and retention of sewer flow offered possible remedies to reduce the pollution on receiving waters. Therefore, it was no longer sufficient to just determine the peak discharge; information on the temporal variations of the storm runoff, i.e. the runoff hydrograph, was required.

There have been many methods proposed for the determination of the runoff hydrograph. These methods include modified versions of the Rational Method, the Unit Hydrograph method, hydrologic routing and more complex hydrodynamic routing.

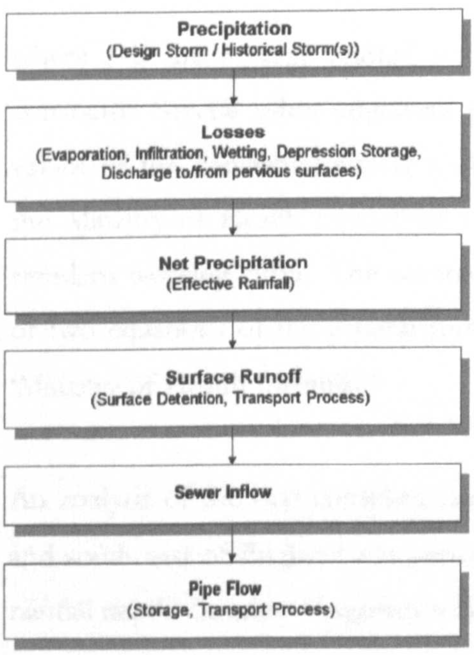


Figure 2.1: Typical Rainfall-Runoff Model (Modified from Nelen (1992)).

The complete scheme of a rainfall-runoff model is shown in Figure 2.1, which forms the basis for the structure of this sub-chapter. It is easy to visualise the various components of a mathematical model for rainfall-runoff relationships but they are complicated physical processes. For example, the runoff is a function of rainfall intensity, the storm duration, area of the catchment, the infiltration capacity of the soil, the type of vegetation, distribution of storm temporally and spatially and other factors. Moreover, the hydrological system is non-linear (Amorocho, 1967; and Prasad, 1967). Therefore, various assumptions are needed in the model and Dooge (1968) stated that the problem of prediction is virtually insoluble if no assumptions are made about the nature of the system. Obviously, the accuracy of the model will decrease with more assumptions but the solution becomes easier.

2.2.1 Precipitation

Before the 20th Century, drainage systems were designed on the basis of an average rainfall intensity, which was assumed to be independent of duration. The collection of information on heavy rainfalls in short periods by the British Rainfall Organisation, with their publication of statistical summaries in 1888 and 1908, led to the inverse relationship between average rainfall intensity and duration. Lloyd-Davies (1906) analysed five years of records and this resulted in a rainfall intensity-duration relationship, which subsequently became known as the 'Birmingham Curve', of the general form:

$$I = x_1 /(D + x_2) \tag{2.1}$$

where I is the average rainfall intensity within the duration D, and x_1 and x_2 are constants. Several other engineers produced equations (each differing slightly in the values of the constants) similar to (2.1) from their local rainfall records. This led to the Ministry of Health convening a committee for the purposes of recommending a standard working curve. The committee produced a report in 1930 proposing the use of two equations of the general form of (2.1). These equations became known as the 'Ministry of Health formulae'.

An analysis of the first complete decade of rainfall data from 12 sites in the Midlands and south east of England was performed by Bilham (1935) who derived the following rainfall depth-duration-frequency relationship:

$$N = c_b D(r_b + 0.1)^{-3.55} \tag{2.2}$$

where N is the number of occasions in 10 years on which a rainfall depth r_b is recorded within a duration D, and c_b is a constant.

From these early beginnings various investigations have been conducted (for example, the Flood Studies Report, NERC (1975)). The synthetic design storm has since been used to check the hydraulic behaviour of a sewer system, especially when flooding is the main criterion. The rainfall intensity is generally derived from historical rain data and the IDF relationship from which the storm obtains its recurrence interval. They are easy to construct and use for any location in the U.K. An example of this approach is the rainfall generator of the WALLRUS (HRS, 1991) model, which generates a symmetrical rainfall hydrograph with the peak at the mid-point of the hyetograph. However, it is recognised that this rarely occurs in practice (FWR, 1994) and has been identified as being particularly inappropriate when considering storage or runoff volume estimation. The full spectrum of the naturally occurring precipitation must be used (Clifforde *et al.*, 1986) to effectively simulate the full range of the quantitative performance of a CSO with a mathematical model. Time Series Rainfall (TSR) (Henderson, 1986) was developed to address some of the limitations of the synthetic design storms.

2.2.1.1 Time Series Rainfall

Time Series Rainfall (TSR) is a sequence of historical rainfall events statistically representative of the annual (annual Time Series Rainfall) or long term rainfall patterns for a given location. Henderson (1986) described the development of a TSR for application at several sites in the UK. The analysis consisted of selecting a typical month from the rainfall records at particular locations and ordering the selected months to form the annual series.

A TSR series may be used in a chronological order to simulate the hydraulic performance of the system in the order that the storms are likely to occur. The series may be ranked by severity or, alternatively, it may be a ranked as a seasonal series (summer and winter).

The application of these series allows investigations of the hydraulic performance and the behaviour of existing systems under day to day rainfall conditions. In particular, the series are appropriate for sewer quality modelling, overflow analysis, detention tank design. The main limitations of TSR are that the regionalisation procedure was relatively crude and that the series represents a typical year of rainfall so does not contain any particularly extreme events (Cowpertwait *et al.*, 1991).

Many of these limitations can be overcome by working directly from the long rainfall time series. Historically, this option has not been possible for the ordinary user because of either the lack of suitable rainfall records or the lack of software to handle the data. To a large extent, these problems have been addressed in the STORMPAC rainfall processing package (WRc, 1994). The main component of STORMPAC is the stochastic rainfall generator.

2.2.1.2 *Stochastic Rainfall Generation*

A stochastic rainfall generator (Cowpertwait *et al.*, 1994) attempts to overcome the TSR shortcomings by producing a synthetic rainfall data set of several years, seeking to represent all the rainfall events during that time. The method is considered to give good results although, because many events have to be run, it is generally restricted to the later stages of a scheme design where specific proposals are being checked for compliance.

One of the limitations of this method, and all previous methods, is that the spatial distribution of the rainfall is not considered. A discussion on the limitations of historical rainfall series is given in Einfalt *et al.* (1998). This is recognised as being one of the situations where most benefits from real time control (RTC) (see Chapter 3) implementation could be encountered. This can be explained in the first instance by the lack of sufficient data from a dense network of rain gauges, which is required to investigate this phenomenon.

Willems (1998) also studied the stochastic generation of spatial rainfall by using a deterministic structure for the physical description of individual rain cells and cell clusters, and a stochastic structure for the description of the intrinsic randomness in the sequence of different rain events.

2.2.1.3 *Rainfall Estimation and Measurement*

So far in this section, various methods for the generation of the rainfall hyetograph have been discussed, which are used for the design and analysis of sewer systems. However, for the operational management of sewer systems it is essential to have an accurate estimation of the volume and distribution of precipitation in a storm event to determine operations within the system. It is impossible to measure the amount of rainfall falling onto an entire catchment, but this information is required before any control actions can be determined. Historically, hydrologists have used raingauges that provide point measurements across the catchment and are generally considered to be ground truth. For practical application, domains within the catchment are assigned to each raingauge forming a raingauge network. It is assumed that the rainfall falls uniformly across each domain at the rate measured by the raingauge.

Significant work has been undertaken in the development of techniques that incorporate the different data sets into rainfall fields that are presumed to be more descriptive of areal rainfall. These techniques include:

- *Simple Averaging of Data* – The individual gauge data values are added and the total is divided by the number of gauges.
- *Application of Thiessen's Polygons* – This technique divides the catchment into polygons constructed around each raingauge and ascribes the rainfall data measured by the raingauge to that area of catchment.
- *Isohyetal Methods* – These methods apply contours to the data and uses the areas inside the contour levels to define the sub-catchment receiving rainfall at a stated rate.

The advantage of the raingauge is that it measures the amount of rainfall directly. However, there are some shortcomings of using raingauges for the measurement of rainfall. One drawback is that they are unable to fully catch and monitor the dynamics, both spatial and temporal, of the storm event, particularly in the case of convective cells, which can stay unseen by ground measurements (Guarnieri, 1998; Seed and Austin, 1990). The raingauge will only measure the time during which the rain falls over the gauge site and will give no indication of the storm duration over the catchment. Not only will this result in the predicted storm being under-estimated but it will also ignore other aspects of moving storms. An example of this phenomenon is associated with the effect that the direction of a travelling storm has on the shape of the runoff hydrograph (Shepherd, 1998). Simplistic work carried out by Shepherd (1987) showed that storms depositing the same quantity of rainfall can produce very different flow hydrographs depending upon the direction of the storm movement.

Other error producing effects on the raingauge records are wind effects, rain shadow from buildings and trees, and blocking from debris.

Often the data from the gauge is modified by the application of an Areal Reduction Factor (ARF) when simulation models are used. This factor is used to compensate for presumed areal variability of the rainfall event and has the form (D.O.E., 1981):

$$ARF = 1 - f_1 D^{-f_2} \tag{2.3}$$

where f_1 and f_2 are functions of the drainage area and D is the duration of the rainfall event.

The shortcomings of raingauges are largely overcome with weather radar, which remotely senses the rainfall, and is more effective at representing the spatial and temporal characteristics of the rainfall. Johann and Verworn (1997) investigated the influence of various time/space resolutions of radar rainfall data on the results of rainfall-runoff simulations and stated that in dynamic management of urban catchments rainfall-runoff simulations are very sensitive to the resolution of the input data.

Austin (1998a,b) describes the history and theory of weather radar. A comprehensive description of weather radar systems is given in Collier (1989). Weather radar radiates electromagnetic energy in the microwave range and measures the back-scattered radiation or reflectivity coming from the encountered objects. The observed reflectivity Z [mm^6/m^3] is usually converted into rainfall rate R [mm/hr] making use of an exponential law of the form:

$$Z = aR^b \tag{2.4}$$

where the values of the parameters a and b depend on the kind of precipitation but are usually assumed to be 200 and 1.6 respectively (Marshall and Palmer, 1948). Operationally, the typical resolution of weather radar is about 5 minutes temporally and 1 km spatially.

Weather radar data is also affected by a significant number of errors. There are problems related to the operational characteristics of the radar and its geographical position. The radar beam may encounter ground targets, which will appear as strong echoes at the receiver. This unwanted signal (the ground clutter) may be wrongly interpreted as intense precipitation. Moreover, whenever some orographic obstacle is encountered, the beam gets partially or totally blocked and rainfall at further distances away remains undetected (Creutin, 1998). Ground clutter maps are useful in minimising this effect. In advanced applications doppler techniques can also be used that discriminate between moving and unmoving targets (Monai, 1998).

There are also problems concerning the way the beam propagates through the atmosphere. Beam attenuation, anomalous propagation and the Earth's curvature effects all contribute to degrading the return signal.

Other sources of error include the variability of reflectivity vertically due to the bright band (a highly reflective layer in the atmosphere near the melting layer) (Tilford, 1998), the natural variability of the Drop Size Distribution (DSD) (Porra, 1998; Uijlenhoet, 1998) and the instability of the radar hardware.

Correction techniques for these errors are discussed in Creutin (1998), Monai (1998), Tiford (1998) and Guarnieri (1998). The weather radar data is normally calibrated to ground truth, the rainfall rate measured at ground level by raingauges. An example of this procedure is described in Moore *et al.* (1994).

The advent of radar rainfall estimates has enabled the development of many different forecasting applications that would not have been possible simply based on raingauge information alone. This rainfall prediction capability is an important feature in the operational management of sewer systems, particularly in predictive real time control (see Chapter 3). Systems such as FRONTIERS and Nimrod have a forecast range to around 6 hours, by combining the radar with other observational and model data.

Radar also provides an opportunity to improve the performance of Numerical Weather Prediction (NWP) models, with impacts on forecasts from 1 to 120 hours (5 days) ahead (Hardaker, 1998).

Weather radar is often used for the predictive control of sewer systems. Examples of these investigations and applications are Marquès *et al.* (1999a), Cluckie *et al.* (1995), Shepherd (1998), Verworn (1998a) and Cluckie *et al.* (1998).

2.2.2 Losses and Surface Runoff

Beyond the necessity for an accurate prediction of precipitation, it is essential to predict the amount of runoff on the drainage areas, which is lower than the amount of rainfall because of certain losses. These include evaporation losses, depression storage, wetting losses at the start of the storm and the discharge to and from pervious areas. Similar losses occur on pervious areas where additional substantial infiltration into the soil often takes place.

Common loss models include:

- an *initial loss* (depression storage) and a *constant continuing loss*;
- an *initial loss* and a *proportional continuing loss* (a fixed proportion of rainfall);
- an *initial loss* with a *diminishing continuing loss* based on Horton's equation (Horton, 1939):

$$f = f_c + (f_0 - f_c).e^{-kt} \tag{2.5}$$

where f is the infiltration capacity [mm/hr]; f_0 and f_c are initial and final rates (constants) [mm/hr]; k is a shape factor [h^{-1}]; and t is the time from the start of rainfall [hours].

After subtraction of the losses, the net rainfall is transported over the surface until it enters the sewer system. Some common modelling approaches are discussed below.

2.2.2.1 Rational Method

The most widely known of the flood estimation procedures is the Rational Method. In the U.S., Kuichling (1889) is credited with the introduction of this approach but the principles of the approach were expounded earlier by Mulvaney (1850).

The Rational Method is based upon the premise that every drainage area has a time of concentration t_c which is defined as the flow time from the most remote point upstream in the catchment to reach the point under design. The peak discharge Q_p [m^3/s] is then assumed to occur when the whole catchment contributes to the flow, a time equal to the time of concentration after the rainfall begins. The magnitude of Q_p is taken to be proportional to the volume of effective (i.e. runoff-producing) rainfall during the time of concentration t_c:

$$Q_p = \frac{1}{360} CIA \tag{2.6}$$

where A is the catchment area upstream of the design point [hac]; I is the average rainfall intensity during the time of concentration t_c [mm/hr]; and C is a dimensionless runoff coefficient.

This formula is sometimes referred to as the Lloyd-Davies method because Lloyd-Davies (1906) first applied it to urban drainage design in the U.K. The Rational Method is therefore a simple design tool but is unable to deal with catchments where there are subdivisions in contributing areas. This led to the introduction of Time-Area Methods.

2.2.2.2 Time-Area Methods

An example of a Time-Area Method is the Tangent Method, which determines the peak discharge from a function of time (the time-area diagram). The peak discharge is the sum of flow contributions from subdivisions in the catchment defined by time contours. These time contours are lines of equal flow time to the outfall where the peak discharge is required. From the time-area diagram, Typical Storm Methods have been derived. These differ from the Time Area Method in producing a complete runoff hydrograph rather than simply an estimate of the peak flow rate and are discussed below.

2.2.2.3 Hydrograph Methods

The development of techniques for estimating the runoff hydrographs began with the Typical Storm Methods. These consisted of the combination of an incremental rainfall profile and an incremental time-area diagram. The method generally assumes an arbitrary shape of storm profile, often constructed from the intensity-duration relationship for a given frequency of occurrence.

In the U.K., the development of the Typical Storm Methods culminated in the introduction of the Transport and Road Research Laboratory (TRRL) Hydrograph Method in 1963 based on research by Watkins (1962).

A more sophisticated approximation of the runoff hydrograph is given by the Unit Hydrograph method. The unit hydrograph is defined as the hydrograph of direct runoff resulting from a unit depth of effective rainfall generated uniformly over the catchment area at a constant rate during a specified period of time. The Unit Hydrograph method is often termed a 'Black-Box' model since it does not describe the inner processes of the system.

The principles of the unit hydrographs developed by Sherman (1932) to convert effective rainfall to a storm hydrograph are:

- linear proportionality of the ordinates of the hydrograph to the depths of effective rainfall;
- equal time bases of hydrographs for equal durations of effective rainfall;
- superposition of hydrographs of incremental runoff to produce a storm hydrograph; and
- time invariance of the rainfall-surface runoff relationship.

An example of the use of the Unit Hydrograph method is that of Burrows and Wenyuan (1991) where the method was used for the long-term synthesis of sewer flow enabling the simulation of storm overflow operation. Here the Unit Hydrographs were obtained from a preliminary application of an advanced hydraulic flow model and the results in application were acceptably accurate. The method also produced considerable computer run time reductions over the application of the advanced models to long duration rainfall records. This work has continued with the development of COSSOM, which is a program developed for the long-term simulation of sewerage systems (Mehmood, 1995). This program executes rapidly making it ideally suitable for the operational management of sewer systems. COSSOM is described in full in Chapter 6 when it is used within this study.

2.2.2.4 *Stochastic Methods*

Several researchers have investigated the use of stochastic models for the simulation of rainfall-runoff processes. A stochastic model represents a dynamic relationship between the observed input and output of a system without describing the inner processes that affect the response of the system itself, i.e. a black-box model. If the input is intended as purely random and uncorrelated (white) noise, the model is said to be an ARMA-type model (Box and Jenkins, 1976), while if the input is an actual observed variable, then the model is said to be a Transfer Function model. The schematic structure of a stochastic model is shown in Figure 2.2, which consists of two parts:

- a transfer of the input rainfall into sewer flow; and
- a noise term, which could be considered as dry weather sewage contributions and disturbances.

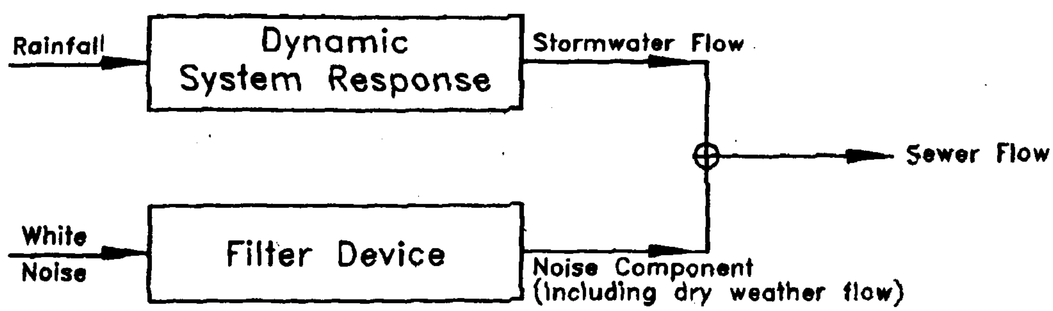


Figure 2.2: Stochastic Representation of a Combined Sewer System (Zheng and Novotny, 1991).

Detailed descriptions of the development of stochastic methods are given in Capodaglio and Fortina (1996), Novotny and Zheng (1989), Capodaglio *et al.* (1990), Zheng and Novotny (1991) and Cluckie *et al.* (1998). According to Capodaglio and Fortina (1996), stochastic transfer function relationships may be used for forecasting flows in urban drainage systems with an accuracy that matches or even surpasses that of far more complex deterministic models. Stochastic models execute very rapidly and are particularly applicable for real time control of sewer systems (see Chapter 3).

2.2.3 Sewer Flow and Flow Routing

Uniform (steady) sewer flow can be adequately described by several flow formulae, for example:

Manning-Strickler:
$$v = n^{-1} R^{2/3} S^{1/2} \tag{2.7}$$

Chezy:
$$v = C.R^{1/2} S^{1/2} \tag{2.8}$$

where v is the mean flow velocity [m/s]; n is the Manning roughness coefficient [$m^{1/3}$/s]; R is the hydraulic radius (= wetted area/wetted perimeter)[m]; S is the surface slope [-]; and C is the Chezy resistance coefficient [$m^{1/2}$/s].

However, for most design and operational purposes, it is necessary to know the temporal and spatial variations of flood waves arising from storm rainfall or changing foul flow through the system. Here, flood routing methods are used which can be divided into two categories: hydrologic routing and hydrodynamic routing.

2.2.3.1 Hydrologic Routing

Hydrologic routing involves the balancing of inflows, outflows and volumes of storage through the use of the continuity equation and an equation of motion (a storage discharge relationship). This can be written as:

$$\frac{\partial Stor}{\partial t} = I(t) - O(t) \tag{2.9}$$

where $Stor$ is the storage volume [m^3]; I is the inflow [m^3/s]; O is the outflow [m^3/s]; and Δt is the solution time step [s]. The general form of a finite-difference equation for equation 2.9 for two points in time is:

$$Stor(t + \Delta t) = Stor(t) + \left[\frac{I(t) + I(t + \Delta t)}{2}\right] - \left[\frac{O(t) + O(t + \Delta t)}{2}\right] \tag{2.10}$$

In the special case of reservoir routing, where the discharge is only related to storage, a simple approach is possible. The general form of a non-linear reservoir model is:

$$q = \left(\frac{Stor}{k}\right)^n \tag{2.11}$$

where q is the discharge [m^3/s]; *Stor* is the storage [m^3]; and k [s] and n (dimensionless) are reservoir constants that have no strict physical meaning. Equation 2.11 represents a linear reservoir model when n equals 1.

2.2.3.2 *Hydrodynamic Routing*

Hydrodynamic routing is more complex than hydrologic routing since it is based on the solution of the continuity equation and the momentum equation for unsteady flow in open channels. Three different levels of hydraulic descriptions can be distinguished in the Saint-Venant equations, i.e. the kinematic wave, the diffusion wave and the full dynamic wave:

Continuity Equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{2.12}$$

Momentum Equation:

$$\underbrace{\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left[\beta\frac{Q^2}{A}\right] + \underbrace{gA\frac{\partial h}{\partial x} + \underbrace{gAI_f = gAI_o}_{kinematic\ wave}}_{diffusion\ wave}}_{full\ dynamic\ wave} \tag{2.13}$$

where Q is the flow rate [m^3/s]; A is the cross-sectional area [m^2]; h is the flow depth [m]; g is the gravitational acceleration [m/s^2]; x is the longitudinal axis [m]; t is the time [s]; β is the Boussinesq velocity distribution coefficient [dimensionless]; I_o is the bottom slope [dimensionless]; and I_f is the friction slope [dimensionless]. The coefficient β is defined as:

$$\beta = \frac{A}{Q^2}\int^{A} u^2 dA \tag{2.14}$$

where u is the flow velocity [m/s].

The Saint-Venant equations can be conveniently generalised to include flow in full pipes (i.e. pressurised flow) by introducing a fictitious slot in the top of the pipe, the 'Preismann slot'.

The simpler approximations of the momentum equation (2.13) may be used if certain assumptions are made in the modelling of unsteady flow. Often it is sufficient to use a one-dimensional continuity equation and a uniform and permanent flow relationship (i.e. the kinematic wave), meaning that only the frictional and gravitational forces are considered. The pressure forces are accounted for in the diffusion wave approximation, which is therefore able to compute backwater and surcharge effects in many cases. All the terms of the Saint-Venant equations are used in the dynamic wave approach, which is better at computing sudden changes in the runoff.

The solution of the Saint-Venant equations is conventionally achieved by numerical methods and is computationally demanding. Many commercial computer packages have become available that solve these equations (e.g. HYDROWORKS and MOUSE). A review of software for rainfall-runoff processes is given in Appendix 1.

2.2.3 Storage Structures

The majority of sewer systems have storage volume available, either as storage tanks or as in-sewer storage. Additionally, combined sewer overflows (CSOs) are common features in many systems, which allow relief discharges into watercourses preventing the overloading of the sewer system further downstream. These structures are designed through guidelines from the Sewerage Rehabilitation Manual (WRc, 1983) and the Urban Pollution Management Manual (FWR, 1994). Saul (1998) fully describes the historical development of CSO chamber design. Storage is a prerequisite to the use of real time control (RTC) (see Chapter 3) which makes deliberate use of this storage volume to achieve operational objectives.

The storage tanks and combined sewer overflow (CSO) structures can be categorised into two types, off-line or on-line. Off-line tanks remain dry during dry weather periods and the storage is only utilised in storm conditions when the wet weather flow is diverted to the structures. The sewage continuously flows through on-line tanks during dry weather conditions.

Many of the storage tanks allow CSO spillage at certain flow settings . In the U.K., the criteria for CSOs was reviewed by the Technical Committee on Storm Overflows and Disposal of Storm Sewage in 1955. This review led to the introduction of 'Formula A' as the overflow setting (FWR, 1994):

$$\textit{Formula } A = DWF + 1360P + 2E \tag{2.15}$$

where *Formula A* is the overflow setting [litres/day]; *DWF* is the average daily rate in dry weather including infiltration and industrial discharge [litres/day]; P is the population and E is the average daily rate industrial discharge [litres/day]. This is the general requirement that CSOs will not spill until the incoming flows exceed that calculated by Formula A. The spills should then be of sufficient dilution not to have an impact on the receiving waters.

2.3 SEWER WATER QUALITY

The previous section briefly described the general regulations for CSO discharges in the U.K. However, further regulations are enforced on each CSO structure, beyond the general requirement of 'Formula A' pass forward rate, and are based on spill frequencies depending on the sensitivity of the receiving waters, which define the river use class (DoE, 1994; and NRA, 1994). This is because of the difficulties in assessing the pollution spill load to the receiving waters. This section discusses sewer water quality processes but a more thorough review is given in Gupta (1995).

Sewage contains many pollutants, some of which are organic, while others are not. The determinants often used in sewer water quality modelling are BOD (Biochemical Oxygen Demand), COD (Chemical Oxygen Demand), ammonia, suspended solids, heavy metals and organic micropollutants. Heavy metals are generally associated with industrial effluent. The impacts on the environment from these pollutants are discussed in Section 2.4.

In storm events the rainfall and associated runoff collect various pollutants, which are then transported to the sewer inlets as dissolved loads, suspended loads, or bed loads. There are also contributions within the sewer from foul sewage inflows and re-suspension of in-sewer sediment deposits before the flows are finally discharged to the receiving waters, either through CSOs or after treatment in a wastewater treatment works (WwTW).

The main processes (FWR, 1994) in sewer water quality modelling can be listed as:

- foul inputs;
- build-up and wash-off of surface sediments;
- deposition and erosion of sewer sediments;
- sediment transport in sewers;
- sediment partitioning in tanks;
- advection and dispersion of pollutants; and
- biochemical reactions.

Foul inputs include domestic, commercial and industrial inputs to sewers, which can vary spatially and temporally.

Sediments build-up on roads, gully pots, roofs, etc. during dry weather periods. Their quantity and characteristics depend on many factors including the length of the dry weather period. Therefore, more sediment is collected on the surface during longer periods of dry weather. During storm events these sediments and associated pollutants are washed off the surface into the sewer systems. The quantities washed off depend on the intensity of the rainfall and the erosion capability of the surface runoff.

Suspended sediments tend to settle out of the sewer flow when the velocities are low, and settle onto the invert of the sewer. The deposition process depends on the size and density of the sediment particles amongst many other factors. Therefore, coarser sediments from catchment surfaces tend to deposit more readily than the finer organic sediments from foul inputs. These deposited sediments have associated pollutants and act as a store of pollutants within the sewer system. The deposited sediments may be eroded again when the sewer flows and velocities increase. The erosion rate depends on the flow velocity, the width of the sediment bed and the characteristics of the sediment deposits, particularly the shear strength, amongst other factors. During the erosion process the pollutants within the sediments are released into the sewer flow.

The sediment within the sewer flow may be transported either as suspended or as bed load. Finer, lighter particles tend to travel in suspension whereas the heavier particles travel as bed load. Thorough descriptions of the processes in sediment transport are given in Ashley and Verbanck (1996) and Ashley *et al.* (1998).

Babaeyan-Koopaei *et al.* (1999) described an experimental programme that investigated the advection behaviour of artificial gross solids, which showed that the water depth and flow shear stress were important parameters in the advection behaviour of the solids. Rushforth *et al.* (1999) undertook a laboratory-based study that examined the erosion and transport of sediment mixtures with varying proportions of granular inorganic and fine-grained organic materials. The results demonstrated that for similar hydraulic conditions, the addition of granular material into an in-pipe deposit significantly increased the amount of organic material eroded in comparison to that eroded from a deposit composed entirely of organic material.

Fraser *et al.* (1999) developed a proactive approach for sewer sediment control, based on sediment trapping structures. A deposition model was developed that estimated the masses of sediments for the City Centre of Dundee, Scotland, which showed reasonable correlation with observed data.

Storage tanks in sewer systems have the effect of reducing the local flow velocity, which encourages the suspended sediment to settle. Therefore, the pollution concentration of the suspended sediment in CSO spills from a tank tends to be less than that in the tank itself. This partitioning effect clearly helps to reduce the polluting impact from the spill flows. These deposited sediments may create a short term load on the WwTW when they are re-entrained into the sewer flow.

Dissolved pollutants and suspended sediments are transported through a sewer system by two main processes – advection and dispersion. Advection is the movement of pollutants and suspended sediments in the same direction and at the same velocity as the water movement. Dispersion refers to the movement of pollutants and suspended particles due to random water motion and mixing. It has a tendency to minimise differences in concentration by moving pollutants from regions of high to low concentrations. Additionally, dispersion spreads out the pollutants.

Finally, on their way through a sewer system pollutants can undergo biochemical processes (e.g. degradation of organic substances), which can significantly change the quantity and quality of the pollutants within the sewer system.

At the start of a storm event, a substantial increase in the suspended solids concentration can be observed in some sewer systems. This 'first flush' effect is discussed in the next section.

2.3.1 The First Flush

The first flush is identified as the relatively high proportion of total storm pollution load that occurs in the initial part of the combined sewer runoff. Despite the evidence of first flushes in sewerage systems (Pearson *et al.*, 1986; Thornton and Saul, 1986 and 1987; Ashley *et al.*, 1992), they are not universal (Geiger, 1986) and, moreover, the phenomenon is a controversial subject (Saget *et al.*, 1996).

The first flush of pollutants observed at the onset of a storm flow in many combined sewer systems has been attributed to the scouring/re-entrainment of in-pipe sediments deposited during extended periods of dry weather (Saul and Thornton, 1989; Geiger, 1987; and Verbanck *et al.*, 1994).

One approach to define the first flush is based on the relationship between the percentage of total load and the percentage of cumulative event flow. This is shown in Figure 2.3, where Geiger (1987) suggested that a first flush was observed when this curve had an initial slope greater than 45°. The 45° line represented constant concentrations of suspended solids throughout the runoff and a line of gradient less than 45° represented dilution. The percentage deviation of the cumulative load curve from the diagonal was a measure of the strength of the pollutant concentration and the point of maximum divergence from the equilibrium line defined the volume and load of the first flush.

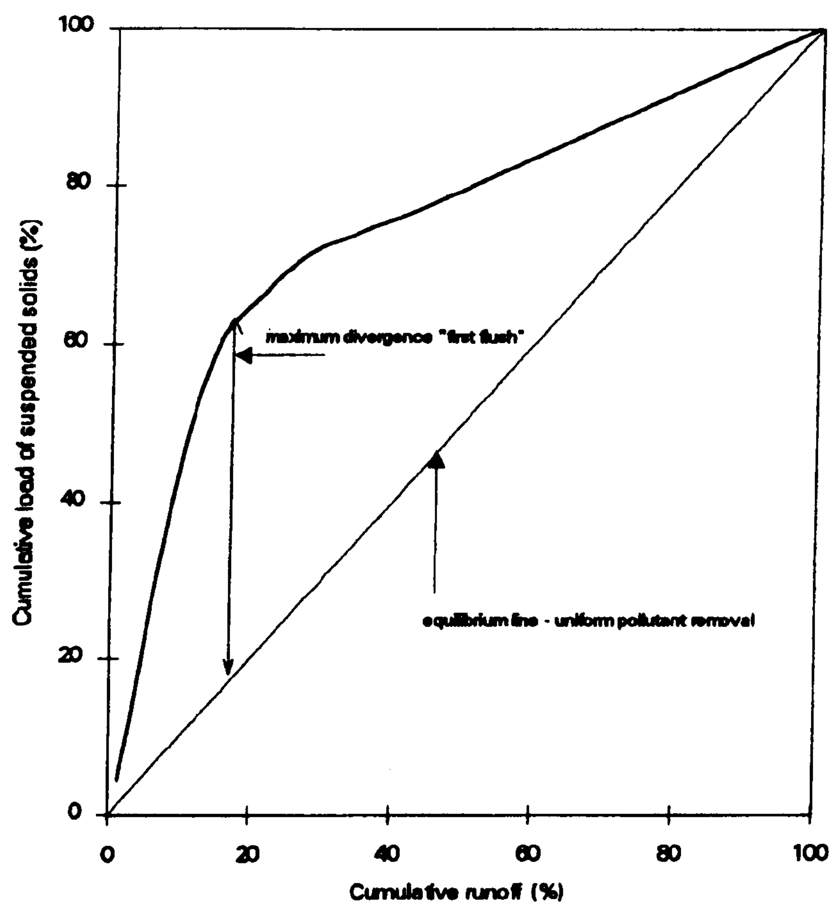


Figure 2.3: First Flush as Defined by Geiger (1987).

A similar approach was used by Saget *et al.* (1996) where the resulting curves could be represented by equation (2.16):

$$Y = X^{a} \tag{2.16}$$

where Y is the fraction of discharged pollution load (-); X is the corresponding fraction of flow volume (-); and a is exponent to be calculated from data (-). The first flush was defined when a was less than 0.185, which occurs when at least 80% of the pollution load is transferred in the first 30% of the volume. In a study of 197 events from 14 French catchments, there was considerable variability in the parameter a and the first flush phenomenon rarely occurred. In fact, the author stated that this conclusion was not sensitive to the definition of the first flush.

2.3.2 Water Quality Modelling

With regard to the above descriptions of sewer water quality processes, it is necessary to quantify the performance of CSOs, in terms of polluting loads to the receiving waters, to mitigate their impacts on the water body.

There are various approaches that can be used for the water quality modelling of sewer systems as described below.

- *Detailed Deterministic Models* - These models attempt to represent most of the processes listed in Section 2.3. Examples of these models are HYDROWORKS QM, MOUSE TRAP and MOSQUITO (an overview of these and other software packages is given in Appendix 1). They are based on detailed sewer flow models and have additional modules to represent the sediment generation, transport and advection processes. Most deterministic models simulate foul inputs, surface wash off, pollutant and sediment behaviour in pipes, and pollutant and sediment behaviour in tanks. They are capable of producing pollutographs (time-varying graph of pollutant determinand) for any part of the sewer system during a simulated event.
- *Sewer Flow Models and Event Mean Spill Concentrations* – This modelling approach uses detailed hydraulic models to predict spill volumes. These volumes are then multiplied by standard values for event mean concentrations to give the total spill loads and, therefore, the approach does not model quality processes in the sewer system. Threlfall *et al.* (1991) recommended average determinand concentrations for combined storm sewage in combined sewer systems.
- *Simple Tank Simulation Models* – In these models the flow processes are represented by a number of tanks in series and in parallel. Each tank receives foul flows and runoff from a different sub-catchment. Pollutants are modelled in different ways but in SIMPOL (an example of these types of models described in Appendix 1) a BOD sediment store is represented in the sewer tanks and is eroded by runoff during storms. Attenuation and sedimentation parameters are the main calibration

parameters to adjust the performance of a SIMPOL model to obtain good agreement with the results from detailed simulation models.

Several authors have attempted to develop statistical regression models for the simulation of water quality processes. The development of these models is based on forming relationships, using regression analysis, between the important parameters and the sewer water quality.

Gupta (1995) gives a review of statistical regression studies undertaken for the simulation of water quality processes. He undertook regression studies on two catchments in the North West of England (Great Harwood and Clayton-le-Moors) to establish regressional relationships between the cumulative load of suspended solids in the first flush, the hydrological parameters and the sewer flow characteristics. Initially, an analysis was carried out on the recorded pollutographs by classifying them by storm duration, antecedent dry weather period (ADWP) and maximum rainfall intensity (Gupta and Saul, 1996a). In this study it was possible to assign a maximum peak concentration of suspended solids that was associated with each category of peak rainfall intensity. These may give an indication of the maximum suspended solids concentration that may be expected from a storm associated with the particular rainfall intensity.

Gupta (1995) then developed site specific regressional relationships to predict the first flush load of suspended solids in combined sewer flow. A summary of the development and results is given in Gupta and Saul (1996b). In the study, it was stated that the first flush load of pollutants ($LOAD_{ff}$) could be expressed as a function of one or more of the following independent variables:

$$LOAD_{ff} = f\begin{pmatrix} EMC_f, EMF, RFINT_{avr}, QIN_{max}, \\ RFINT_{max}, STDURN, ADWP, FLOW_{tot} \end{pmatrix} \tag{2.17}$$

where EMC_f is the flow weighted event mean concentration [mg/l]; EMF is the event mean flow [m^3/s]; $RFINT_{avr}$ is the average rainfall intensity [mm/hr]; QIN_{max} is the maximum inflow [m^3/s]; $RFINT_{max}$ is the maximum rainfall intensity [mm/hr];

STDURN is the storm duration [min]; *ADWP* is the antecedent dry weather period [hr]; and $FLOW_{tot}$ is the total inflow [mm].

The results from the regression analysis showed that the first flush load correlates well with the peak rainfall intensity ($RFINT_{max}$), the storm duration (*STDURN*) and the antecedent dry weather period (*ADWP*). The equations were verified against observed values from Clayton-le-Moors site where there was reasonable agreement but up to 20% differences. It must be stressed that the equations developed were site specific but they could be used to establish an approximate estimate of the pollutant load within the first flush in the sites. An example of a site specific regressional equation is shown below (2.18), which is specific for Great Harwood for summer storms:

$$LOAD_{ff} = 1.35(STDURN)^{0.68}(RFINT_{\max})^{0.68}(ADWP)^{0.28} \tag{2.18}$$

Gupta (1995) then hypothesised that the peak concentration of total suspended solids (TSS_p) could be expressed as a function of one or more of the following explanatory variables:

$$TSS_p = f\begin{pmatrix} ADWP, PEAKEDNESS, RFINT_{\max}, \\ RFINT_{avg}, QIN_{\max}, STDURN, RF_{tot} \end{pmatrix} \tag{2.19}$$

where *ADWP* is the antecedent dry weather period; *PEAKEDNESS* is defined as $RFINT_{max}/RFINT_{avg}$; $RFINT_{max}$ is the peak rainfall intensity [mm/hr]; $RFINT_{avg}$ is the average rainfall intensity [mm/hr]; QIN_{max} is the peak flow [m^3/s]; *STDURN* is the storm duration [min]; and RF_{tot} is the total rainfall [mm].

The results from the regression analysis showed that the peak suspended solids concentration of pollutants could be represented by an equation of the form:

$$TSS_p = K(PEAKEDNESS)^a(ADWP)^b \tag{2.20}$$

where *K*, *a* and *b* are coefficients that are site specific and a function of the catchment and sewer system characteristics.

Gupta (1995) also studied the recession limb of the pollutographs and concluded that the equation of the recession curve was of the form:

$$TSS(t) = At^{-k} \tag{2.21}$$

where *TSS(t)* is the concentration of total suspended solids [mg/l] at any time *t*; and *t* is the time from the start of the storm [min]. *A* and *k* are coefficients that are again site specific and a function of the system. It was concluded that the shape of the recession curve was not sensitive to the values of these coefficients. Gupta (1995) states that while this equation (2.18) provided an adequate representation for the shape of the recession limb of the pollutograph, it does not provide any indication of the time of occurrence of the peak suspended solid concentration.

The computation of the time to peak (t_p) was an iterative process. Firstly, the $LOAD_{ff}$, TSS_p and *TSS(t)* must be calculated from equations (2.18, for example), (2.20) and (2.21) respectively. As a first approximation, Gupta (1995) assumed that the time to peak corresponded to the time of occurrence of the TSS_p on the recession curve. The rising limb of the pollutograph was assumed to be linear from the pollutant concentration of the dry weather flow to that of the peak concentration. The load in the first flush of pollutants is established, as defined in Figure 2.3, as the maximum divergence between the cumulative percentage of pollutants and the cumulative percentage of flows. The time to peak (t_p) is then adjusted until this load is identical to the load calculated from equation (2.18, for example). The pollutograph methodology is shown as a flow chart in Figure 2.4.

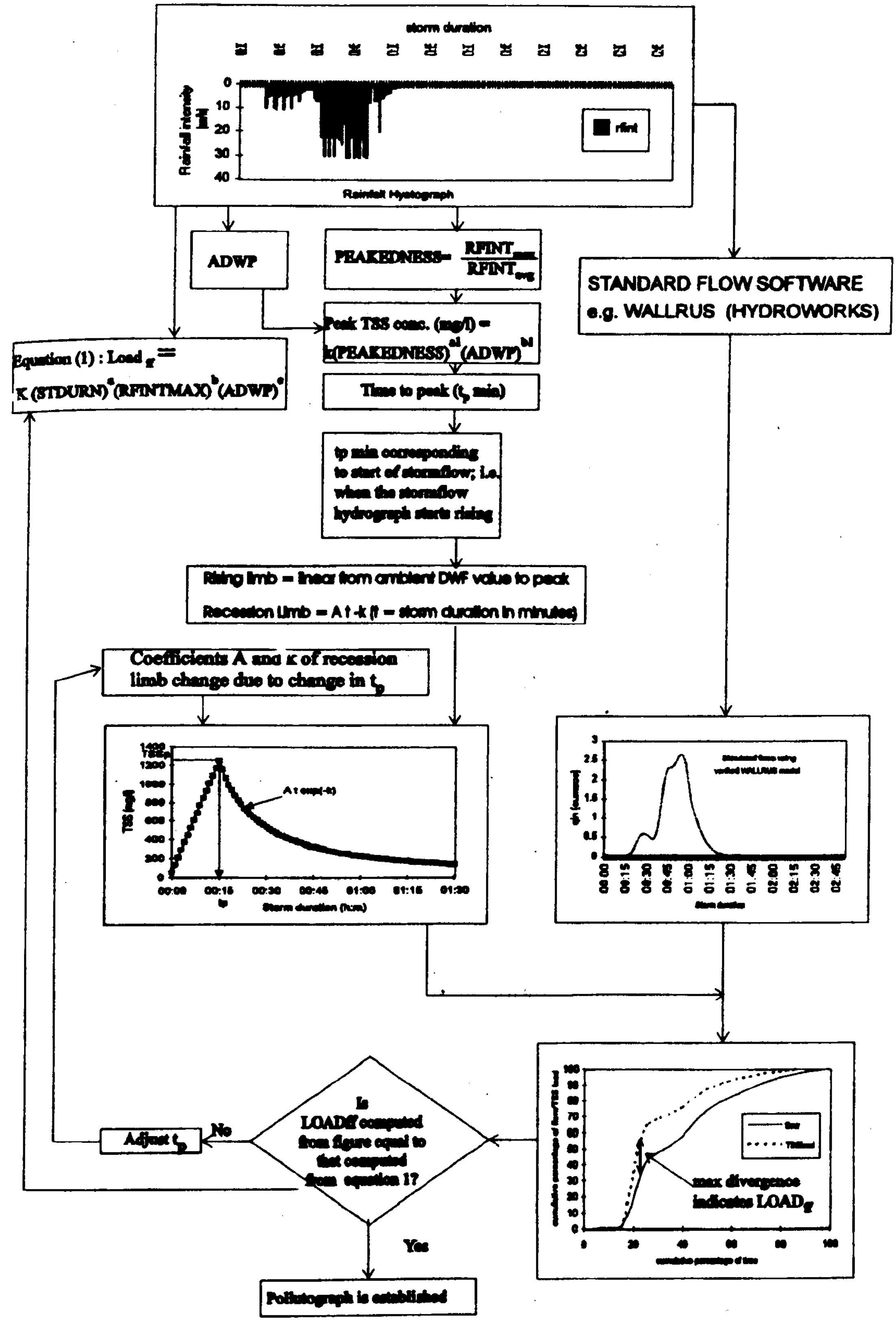


Figure 2.4: Pollutograph Methodology by Gupta (1995).

The major limitation of the work by Gupta (1995) is that the equations are extremely site specific and were developed on limited data sets. Obviously, the equations were developed for sites where first flushes occurred and, as mentioned earlier, the first flush is not accepted to be a universal phenomenon. Therefore, the methodology only has a limited application requiring significant data gathering before it can be applied to any new site.

Chiew and Vaze (1998) also investigated the use of regression equations for the estimation of pollution load from urban areas. Here, the modelling exercise was carried out using total suspended solids, total phosphorous and total nitrogen data from two catchments in eastern Australia. The equations for estimating event polluting loads were power functions of total rainfall, total runoff, rainfall intensity and runoff rate. The results suggested that the rainfall intensity and runoff rate were important variables governing the washoff of particulate pollutants. Again, this work is site specific and has limited application.

Saul *et al.* (1999) described an investigation into the prediction of aesthetic pollutant loadings from combined sewer overflows by conducting a socio-economic survey and field evaluation. A methodology was also presented to improve the prediction of CSO retention efficiency and to improve the selection of the most appropriate based on the anticipated gross solid distribution of the particulate that enters the chamber.

The complexities in the sewer water quality processes make it very difficult to develop accurate models for the simulation of sewer water quality. Jack *et al.* (1996) state that the main limitations of knowledge that have so far confounded attempts at accurate sewer flow quality modelling consist of:

- the inadequate modelling of gully pot performance;
- the lack of knowledge about inputs of gross solids and their interaction with sediments;
- the significant temporal and spatial variability of sediments and pollutants attached to sediments within even a single sewerage network;
- the transformation mechanisms of an 'active' sediment layer into a consolidated/storage layer and vice versa;
- the temporal variability of sediments and pollutants in sewerage networks in terms of short and long timescales, i.e. daily and seasonally, suggest models can only be calibrated and not be verified;

- the problems of modelling sediment transport and associated pollutants; and
- the important dilution of dry weather flow pollutants associated with infiltration (for systems where this is significant).

2.4 IMPACTS FROM SEWER SYSTEMS

The effluent in a sewerage system will eventually be discharged (either treated or untreated) into a receiving water body. The substances within the discharged effluent will interact with the environment of the receiving water and will therefore have an impact. The level of impact, however, depends on the quantities and nature of pollutants released, and presents a problem if the assimilative capacity of the receiving environment is exceeded. Unfortunately, industrialisation and urbanisation have inevitably resulted in the assimilative capacities of receiving environments close to centres of population being exceeded, resulting in observed pollution incidents. It is therefore critical to assess the impacts from the discharge to receiving waters to quantify the detrimental effect of the sewerage system. Detailed reviews on impacts of urban discharges on receiving waters are given by Andoh (1994), Ellis and Marsalek (1996), Ellis and Hvitved-Jacobsen (1996) and House *et al.* (1993).

Principally, there are two forms of discharge into receiving waters from combined sewer systems during storm events:

- intermittent discharges from CSOs and storm tanks; and
- continuous discharges from WwTW effluent.

The key pollution problems associated with these discharges are:

Oxygen Depletion – A reduction in DO (dissolved oxygen) levels in the receiving waters following rainfall are generally associated with intermittent discharges and are a result of (FWR, 1994): low DO levels in CSO and storm tank discharges; degradation of dissolved BOD; degradation of BOD attached to sediments; and the resuspension of polluted bed sediments exerting an additional oxygen demand. During a storm event, soluble and fine particulate organics are transported in the water phase and exert an immediate DO depletion. Settleable solids accumulate on the bottom of the water body and result in a delayed DO depletion due to an increase in the SOD (sediment oxygen demand), (Hvitved-Jacobsen and Harremoes, 1982; Hvitved-Jacobsen, 1986; Harremoes, 1996a). Such delayed effects may last for 1-2 days (Ellis and Hvitved-Jacobsen, 1996). Fish kills are the most apparent effect of acutely reduced DO levels. Sublethal effects on fish (e.g. reduced growth) may be a result of reduced DO concentration level, Figure 2.5. It should be noted that the oxygen concentration in a river varies diurnally due to photosynthesis and respiration from instream vegetation and this, of course, affects the sensitivity to oxygen depletion.

Eutrophication – Discharges of nutrients, including nitrogen and phosphorus, can cause excessive algae growth (eutrophication) in the receiving waters, which may dramatically change the ecosystem and cause secondary oxygen depletion in stagnant waters. According to Ellis and Hvitved-Jacobsen (1996b), this is normally a problem with stagnant or semi-stagnant waters.

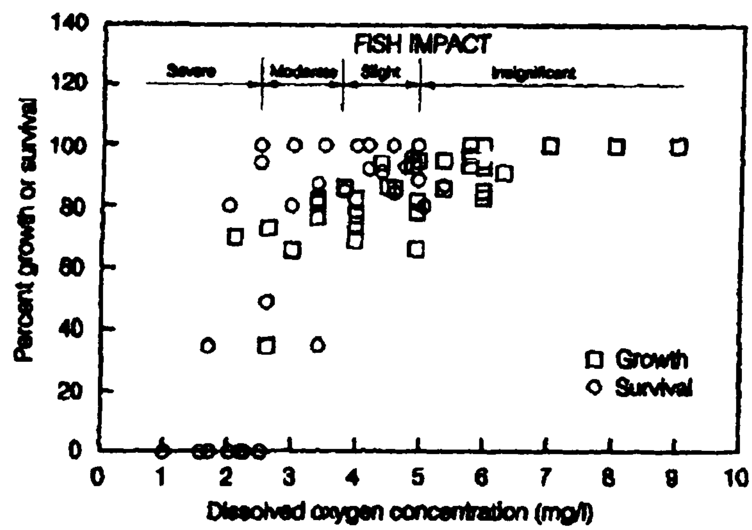


Figure 2.5: Impact of Long Term DO Concentrations on Fish Growth and Survival (Limo-Tech Ltd, 1987).

Sediment and Toxic Pollutant Impacts – Sediments constitute a sink and a potential source of pollutants in receiving water ecosystems. Substances discharged from CSOs may contribute a range of absorbable and settleable pollutants derived from sewer deposits, wastewater and urban surfaces. Due to the nature and amount of biodegradable organics, anaerobic conditions may prevail in such receiving water sediments and accumulated metals (mainly from trade effluent into the sewerage system). Hydrocarbons and bacteria can then impose long-term impacts on the sediment community. Localised acute effects may also follow storm flow induced scour and resuspension of toxic substances such as hydrocarbons, heavy metals and ammonia (which is a strong fish toxicant), in addition to SOD.

Public Health Risks – The design of CSOs means that untreated sewage and contaminated effluents will discharge to receiving waters and this raises public health risks related to potential exposure, particularly if the receiving waters are used for recreational purposes. In addition to bathing and sailing activities, shellfish harvesting in areas of urban runoff is a potential health risk. It is well recognised that urban runoff contains a wide variety and frequently high numbers of pathogenic bacteria and viruses. Mandatory bacteria levels are often violated in urban receiving waters, especially during the first flush period of storm events (House *et al.*, 1993). Bacteria can also become encapsulated in bed sediment where survival times become considerably extended. With a return period for CSO discharge of 1-3 months, sediments near outfalls are potentially permanently contaminated with *E.coli, faecal coli and faecal streptococci* (Ellis and Hvitved-Jacobsen, 1996).

Aesthetic Impacts – This list of impacts is concluded with the effects from the discharge of materials, such as debris or oil, which form part of aesthetic pollution noticed by the general public.

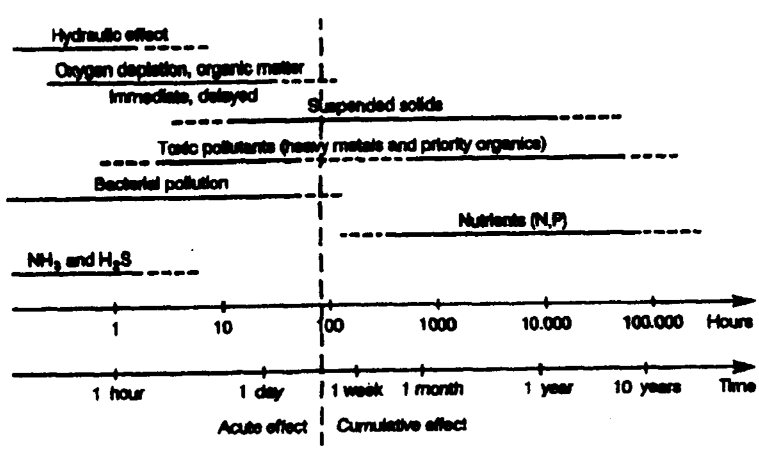


Figure 2.6: Time Scale for Receiving Water Effects from Intermittent Pollutant Discharges (Ellis and Hvitved-Jacobsen, 1996).

Time Scales - The time scale of the pollutant effect on the receiving water is also an essential factor to be considered and Figure 2.6 shows the time scale of the various impacts on the receiving water. As presented in Figure 2.6, the effects on the receiving waters can be categorised into two types:

- *Acute* – The pollution effects last for a period comparable to that of the storm event. Oxygen depletion in rivers lasts for slightly longer than the storm but the fish may be killed in this process. Toxicity of ammonia and some toxic trace organics and faecal bacteria contamination belong to this category. Performance criteria have to be formulated as extreme events to be meaningful.
- *Accumulative* – A pollutant that accumulates in the receiving water will gradually build up to a level that can be toxic. Typically, this applies to the accumulation of metals in sediments or to the accumulation of nutrients in lakes. The performance has to be measured as the cumulative effect in the receiving water for a characterised period (e.g. a season or a year). In this case the variability of the pollutant load from storm to storm is not important.

In 1994 the Urban Pollution Manual (FWR, 1994) was published to give a recommended practice in the U.K. for the control of the impacts from discharges using an integrated approach. It brought together the different modelling tools including rainfall modelling, sewer quality modelling, sewage treatment modelling and river quality modelling within a comprehensive planning framework. This approach was referred to as the UPM procedure.

2.5 POTENTIAL OF REAL TIME CONTROL (RTC)

As mentioned earlier, urban drainage systems were historically designed to prevent surface flooding within the drainage areas. A change in attitude towards environmental protection led to an increase in regulations for the discharge of sewage into receiving waters. The discharge of foul sewage is no longer accepted and there are now established regulations to limit the discharge quantity, often to 'Formula A' in the U.K. The Sewerage Rehabilitation Manual (WRc, 1983) was published in 1983 providing planning and design procedures for the management of flooding and structural dereliction. The conventional solution, within this framework, to these urban drainage problems was to provide sufficient system capacity. Often storage structures were installed to attenuate the flows, preventing the overloading of the system downstream. However, the solutions were based on passive technology (i.e. where system performance criteria are established at the design stage and are thereafter fixed for the life of the scheme). The dynamics of the system and flexibility in operating the system were therefore neglected. Theoretically, if no dynamic control is applied, the capacity of the system will only be fully utilised when it is loaded with the design load. Therefore, by definition, for all other loadings the system will perform sub-optimally, meaning that there will be storage and discharge capacity available in the system.

The potential exists to operate an urban drainage system more efficiently and effectively in response to the prevailing circumstances by utilising the full capabilities of the system. This can be achieved through active or real time control (RTC), which was introduced concurrently with the environmental protection philosophy as a method to minimise capital expenditure (IAWPRC, 1989). RTC attempts to make efficient use of the storage within the system, thereby possibly preventing the costly upgrade of the system. An urban drainage system is operated in real time if the current state of the system is used to operate the regulators (e.g. pumps and valves) during the actual storm event. The sewerage system is a dynamic environment, in both inputs and outputs, and RTC uses this to its advantage. The dynamics of the system are discussed below:

System Inputs – The loading of the urban drainage system is variable in time and space, due to the heterogeneity of rainfall, the variations in response characteristics of the sub-catchments and the temporal and spatial variations in dry weather flow (DWF). Therefore, with such a varied loading, optimum performance can only be achieved if the regulators are adjusted in response to the actual conditions experienced.

System Response – Urban drainage systems usually consist of several sub-catchments that have different response characteristics. Therefore, even under a homogeneous loading, the system will not respond uniformly. Moreover, some parts of the system will have more capacity available and this will lead to an uneven use of the system, meaning that some parts of the system may be overflowing while other parts have capacity available. Hence, optimum performance can only be achieved if the regulators are adjusted using information about the response of the entire system.

System Output and Impacts on Receiving Waters – The effects of overflowing into receiving waters also vary spatially and temporally. Generally, overflows into larger watercourses have less impact than overflows into small streams. Also, the sensitivity of the receiving waters to overflows varies temporally. For example, the DO level overnight is lower than in the daytime in receiving waters because of the respiration of the vegetation. Therefore, discharges overnight will have more impact on the fish because the consequential oxygen depletion may reduce this reduced DO level to a lethal level.

2.6 CONCLUDING REMARKS

This chapter has described the processes involved in urban drainage systems and the effects that discharges from them have on the aquatic environment. It has been shown that the impacts of intermittent discharges can be significant and, consequently, firmer regulations now exist to enforce improvements in the performance of sewer systems. Conventionally, a passive solution has been used to solve these urban drainage problems based on the concept of providing greater system capacity, often by the inclusion of storage chambers. This approach has deficiencies since it lacks the flexibility in the operation of the system under dynamic loading. Implementation of real time control techniques would significantly improve the efficiency of the system since the approach has the flexibility of taking advantage of the dynamics in the system in the operation.

It is noted that many sewer systems in the U.K. are equipped with active flow regulators (e.g. pumps and valves) and, therefore, the cost of implementation on improved operating techniques would not be too great.

CHAPTER 3

REVIEW II - OPERATION AND CONTROL

3.1 INTRODUCTION

The previous chapter mainly described the underlying processes considered for an uncontrolled (passive) sewerage system, illustrating the potential benefits of active control by operational management which influences the state of the system. This type of operation is often termed real time control (RTC), which Schilling *et al.* (1996) defines as:

> "A wastewater system is controlled in real time if process data such as water level, flow, pollutant concentration, etc. is continuously monitored in the system and, based on these measurements, regulators are operated during the actual flow and/or treatment process."

Thorough reviews of real time control techniques are given in IAWPRC (1989), Schilling *et al.* (1996) and Schilling (1994).

Initially, this chapter gives a brief description of the control elements used in a controlled sewer system. The various control concepts are then described illustrating the advantages and disadvantages of each. The main aim of this chapter, however, is to describe the formulation of an operational problem and review the methods used for its solution. Two methods, Linear Programming and Dynamic Programming, are used within this thesis and are therefore more thoroughly described, with examples of the solution procedures in Appendix 1. Finally, a description is given on the uncertainty in mathematical modelling.

An overview of commercial computer software packages that have facilities for the real time control of sewer systems is given in Appendix 1.

3.2 CONTROL ELEMENTS

Before describing the procedures used in operational management, it is necessary to have an understanding of components within a sewer system that can be controlled. Therefore, this sub-chapter briefly describes the hardware requirements for the active control of sewer systems. Figure 3.1 shows a schematic of a controlled process or 'control-loop', which consists of various elements including a controller, a regulator and information links. The control loop shown in Figure 3.1 has a decision-maker that enables the process to be dynamically controlled. Each element is discussed briefly below.

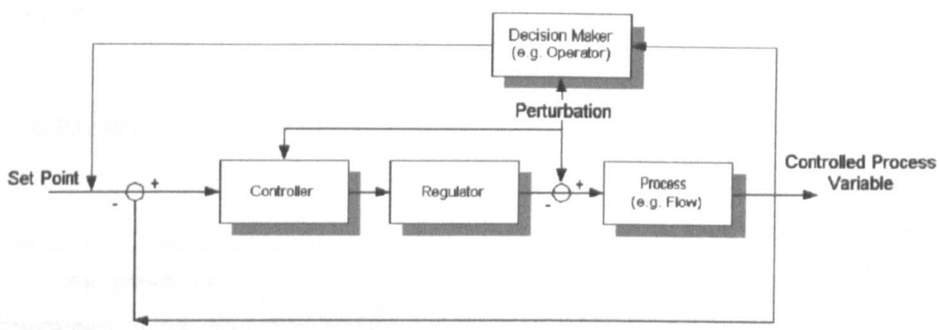


Figure 3.1: Schematic of a Controlled Process (Modified from Nelen, 1992).

3.2.1 Sensors

Sensors are required to monitor the state of the system and variables that are used to predict the disturbances. The amount of information required depends on the system configuration, the operational objective and the control level. Examples of sensors include:

- Rainfall (e.g. rain gauges and weather radar);
- Water Level (e.g. pressure and ultrasonic sensors);
- Flow (e.g. electromagnetic and sonic flow meters); and
- Pollutants (e.g. optical sensors (Ruban, 1995)).

3.2.2 Regulators

Regulators manipulate the process variable (normally flow) in order to obtain a desired state in the system, which is often defined by water levels or flow rates through time. Examples of regulators in a sewer system include pumps, gates, moveable weirs, sluices and valves.

3.2.3 Telemetry

A communication system or telemetry system is required to transmit the information between the various elements of the control process. For example, the information from the sensors is transmitted to the decision-maker in Figure 3.1, which then transmits instructions to the controller that operates the flow regulator. Often data is transmitted to personal computers (either locally or centrally) for operation supervision or data archiving.

3.2.4 Controllers

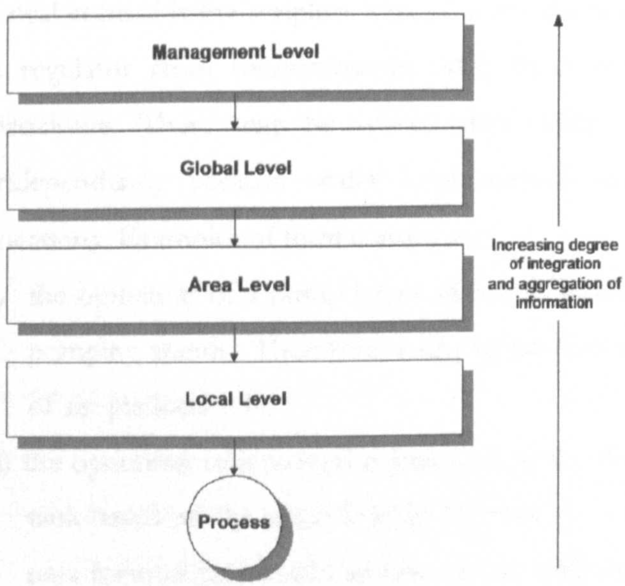


Figure 3.2: Levels of Information and Control (Taken from IAWPRC (1989)).

Controllers activate the flow regulators using the information from the sensors and (possibly) decision-maker. Fundamentally, the controller sends instructions to the regulator depending on the deviation of the actually measurements from the desired state of the system (set point). The desired state of the system through time is referred to as the *control strategy*. Examples of controllers include the two-point controller, the Proportional Integral Differential (PID) controller and the predictive controller.

3.3 CONTROL CONCEPTS

3.3.1 Control Levels

Control systems differ in complexity. Figure 3.2 shows the levels of control that can be distinguished and the levels of information they require. Information is required to control a system and, therefore, these levels of control also represent levels of information.

3.3.1.1 Static Control

Static control, or passive control, is the traditional form of control where the system is designed and constructed to operate automatically without the need for any form of control intervention. Here, the set points of the process are constant through time and, therefore, a decision maker is not required. For example, flow over fixed weir crests (advantage of no moving parts) will divert into storage, often to 'Formula A' (~6DWF) in the UK, prior to discharge of combined sewer overflows (CSOs) into receiving waters.

3.3.1.2 Local Control

Local control is the simplest level of active control and is described as the operation of a regulator from measurements made by a sensor at the same location or close proximity. There may be several sites using local control but they still operate independently because under local control there is no communication between locations. Examples of local control are:

i) the operation of a pump based on water levels in the wet well, as in a conventional pumping station. Therefore, many urban sewer systems already contain some form of simple local RTC.

ii) the operation of a moveable penstock at the downstream end of a detention storage tank based on the water level in the tank. In a storm event, using a fixed orifice, the pass forward rate would increase as the tank filled. The maximum pass forward rate would only be achieved when the tank was full, and then rates would reduce as the tank emptied. However, the moveable penstock would allow the maximum pass forward rate to be sustained throughout the storm event. Additionally, the tank would empty more rapidly. Therefore, local control has significant advantages over static control.

Local control forms optimal solutions for systems that have only one regulator and one decision variable. Usually, however, sewer systems contain several regulators and the local control of such systems do not give optimal solutions. Better operation is possible with the use of area control. Local control of sewer systems implies decentralised control.

3.3.1.3 Area Control

This type of control is defined in various ways. It is termed as *Area Control* by HR Wallingford (1996) and *Regional* or *Unit Process Control* by Nelen (1992) and IAWPRC (1989). Despite these variations in terminology, this type of control is an extension of local control where parts of the sewerage system are controlled in a coherent manner. Generally, the operation of regulators is defined by the process measurements taken at locations other than at the regulator site.

Area control is better able to control systems that have several regulators but the control solutions are not normally optimal. In systems that have several regulators, optimal control solutions can only be achieved with global control. However, in comparison to global control, relatively little data processing is necessary. In general, area controlled systems are decentralised systems.

3.3.1.4 Global Control

Global control determines set points and control actions that are specified in accordance with process measurements taken throughout the system. Global control systems are generally designed as centralised systems obtaining the process data from decentralised control units by means of a communication network. Therefore, this procedure involves substantial use of telemetry systems and process hardware.

Global control is the only control level that allows flexible reaction to the rainfall runoff process in every operational situation. Moreover, optimal performance of the sewer system can only be achieved when global control is applied.

3.3.1.5 Integrated Control

In recent years there has been some research into the integrated control of wastewater systems. Comprehensive reviews can be found in Harremoës (1996b) and Schütze (1998a). Generally, integrated control of wastewater systems is the operation of one part of the wastewater system (e.g. the sewerage system) taking into consideration the operational objectives in another subsystem (e.g. pollution reduction in the receiving water). Schütze (1998a) defines that integrated control of urban wastewater systems is characterised by two aspects:

"*Integration of objectives:* Objectives of control within one part of the urban wastewater system may be based on criteria measured in other subsystems (e.g. operation of pumps in the sewer system aiming at minimum oxygen depletion in the receiving water body)."

"*Integration of Information:* When taking a control decision within one part of the system, information about the (present and predicted) state of another subsystem may be used (e.g. considering treatment plant effluent concentrations when performing control in the sewer system) – hence state information is transferred across subsystem boundaries."

Therefore, integrated control attempts to find an optimum solution within various subsystems where there may be conflicting objectives between each subsystem. Of course, the importance of each subsystem operational objective needs to be carefully analysed before an overall integrated control objective can be assigned to the system.

Integrated control has been applied in a pilot study in Venice (Pretner *et al.*, 1999), where MOUSE and STOAT were used to model sewer processes and the treatment plant. MIKE 21 was used for the river modelling. In future developments of this study, the models are to be integrated and the study will also use real time control strategies to control the quantitative and qualitive performance of the system.

Integrated control has also been applied on a wastewater system in Trondheim/Norway (König *et al.*, 1999) where the model included wastewater production, surface runoff, infiltration, transport and treatment. Real time control has also been simulated. The objective of the investigation was to minimise pollution discharges to receiving waters and to define design loads for the extension of the treatment plant. The results from this simulation showed that it was possible to achieve a reduction in the hydraulic load to the treatment plant by 12% without reducing the pollution transport to the plant. Simulaneously, discharges from upstream CSOs could be reduced by half.

Alex *et al.* (1999) developed an integrated modelling system within a MATLAB/SIMULINK environment using PLASKI, SIMBA-sewer and SIMBA for the simulation of wastewater production, transport and treatment, respectively. SIMBA simulated biological and chemical treament processes, e.g. activated sludge and biofilm processes, chemical precipitation and sedimentation. PLASKI is a hydrological water balance model used for the continuous simulation of runoff processes. SIMBA-sewer modelled the transport processes for simulation of both water flow, dissolved and solid pollutant concentrations. Additionally, the receiving water was simulated within the SIMBA-sewer model. This integrated model was applied to the municipality of Fredrihstad in Norway (Risholt *et al.,* 1999) to get an overview of the pollutant discharges to the receiving waters. The second objective was to determine the potential reduction of pollutant discharges by pollution based real time control. Phosphorus was used as the pollutant determinand. In a comparison between local control and pollution based RTC, the results from the simulation showed that the CSO volume increased but there was a decrease in total phosphorus discharged. Surprisingly, in a comparision between local control and pollution based RTC, the results showed that the CSO discharge volumes were equal but the pollution based RTC had a higher discharge of total phosphorus. In this simulation only the main pumping stations closest to the treament plant were included in the pollution based real time control and so had little effect on the overall phosphorus discharge because of the alleviated concentrations. Risholt *et al.* (1999) stated that the control of upstream inflows, where the sensitive receiving waters were also located and concentration differences were greater, would give better performance improvements.

An integrated simulation and optimisation tool SYNOPSIS was developed by Schütze (1998a) allowing water quantity and quality processes in the urban wastewater system to be simulated. A brief description of SYNOPSIS is given in Schütze *et al.* (1999a). A global optimisation procedure was used offline applying a Controlled Random Search

(Price, 1979), a Genetic Algorithm (Goldberg, 1989) and a derivative-free local optimisation procedure (Powell, 1964). The main building blocks of SYNOPSIS were based on existing models: KOSIM for the simulation of the sewer system; a simplified version of the IAWPRC Activated Sludge Model No. 1 was used for the simulation of the treatment plant; and DUFLOW (IHE, 1992) was used for the simulation of the river. Schütze (1998a) concluded that integrated control can lead to some improvement of the performance of the urban wastewater system depending on the characteristics of the case study site.

Hernebring *et al.* (1999) describe three Swedish pilot studies that have attempted to develop and validate an integrated methodological and technological framework for the pollution impact analyses of complete urban wastewater systems on the receiving waters. An Integrated Catchment Simulator (ICS) was developed to allow interactions between MOUSE for simulation of the sewers, STOAT for the simulation of the wastewater treatment plant and MIKE11 for the simulation of the receiving waters. Further descriptions of these studies can be found in Mark *et al.* (1999) and Clifforde *et al.* (1999).

3.3.1.6 Management Level

A large amount of data is collected on the performance of the system with centralised control, which can then be used for the general management of the system. These include further data analysis, performance statistics, and maintenance planning.

3.3.2 Optimal Control

As previously mentioned, a system can only be operated optimally if global control is used. All other control levels generally generate sub-optimal control strategies, except in rare cases. The optimal control of a system generates optimal solutions at all operational states within the constraints of the control problem. Of course, optimal performance is achieved with respect to the criteria specified in the operational problem. The optimal solutions can be readily determined for simple sewers (e.g. by 'trial and error') but for complex systems it is impossible for an operator to determine optimal solutions because there are many decision variables and/or many control permutations. In this situation a mathematical model is essential to the calculation of optimal control decisions, where various solution techniques (see section 3.4.4) can be used.

3.3.3 Mode of Operation

A control system can be operated in different ways, each of which is called the mode of operation. A control system can be operated manually, in supervisory mode or in automatic mode. A manually operated control system is when the regulators are operated directly by the operators. The operators would therefore need a good understanding or 'feeling' for the hydraulic behaviour of the control system and the sewer system.

In a supervisory control system the regulators are activated by local automatic controllers but the set points are specified by the operators. A decision support system (DSS) is often used to assist the operators in their decision making. These could be simulation models that allow the operator to try possible strategies before implementation. The operator may have a decision model, for example an optimisation model, that suggests control strategies. This has the advantage that optimum solutions can be determined but the operation still remains with the operator. The system must have the facility for manual operation in emergencies or during maintenance. They are often the first step towards fully automatic global control of sewer systems.

The system is operated automatically when the decision operation strategy and the execution is fully automatic. Again, the control system should allow for manual control in cases of emergency.

3.4 THE OPERATIONAL PROBLEM

It has previously been described that a controller adjusts a regulator to achieve a minimum deviation of the regulated parameter from the set point. The term *control strategy* was also introduced, which was defined as the time sequence of all regulator set points in a control system. This section describes methods of determining these set points or control strategy.

The simplest control strategy is to keep the set points constant. This option may be advisable for certain problems but in most cases the optimum set points vary with each flow pattern. Moreover, in sewer systems the flow patterns are transient in storm flows and pollutant loads, which show no regular pattern. Therefore, a flexible method has to be used to react to whatever transients will occur.

3.4.1 Operational Objectives

Clearly, before the control strategy can be determined the objectives of controlling the sewer system have to be specified. An outline of the planning and assessment phases of RTC in waste water systems is given in HR Wallingford (1996). The objectives should be relatively easy to outline because they usually relate to the solving of the initial problem which instigated the investigation. It is possible, even probable, that there will be multiple objectives and here they should be divided into a hierarchy of importance. It is important to be able to evaluate the performance of the control strategy so an 'ideal' operation can be specified and 'costs' assigned to sub-ideal operation. Often the objectives are said to be conflicting, where they cannot be satisfied simultaneously. For example, during a storm event CSOs can be reduced by storing the storm flows in the sewers and storage tanks but this increases the risk of flooding. Therefore, a trade-off between the conflicting objectives has to be defined and a best compromise strategy established.

The objectives in the optimisation problem may be included using three options. Firstly, all the objectives can be incorporated with related weights to include their relative importance. This is achieved by using the principle of *nonpreemptive Goal Programming*, where for each objective a specific goal is set and an objective function is defined. The optimal solution is found by minimising the weighted sum of deviations of objective function values from their respective goals. The second option is termed *Preemptive Goal Programming*, where the objectives are divided into different priorities. For each objective a numerical goal is set and a objective function is defined. Initially, the focus of the optimisation is to approach the goals corresponding to the first priority objectives as closely as possible. If the optimal solution for the first priority objectives is not unique, the second priority objectives can be taken into acount, while maintaining optimality for the first priority objectives until a unique solution is found. The last option, termed a *constraint method*, can be used if one objective is essential and the other less important objectives are included as constraints.

Often one or more of the following are chosen as objectives for the control of sewer systems and often prioritised in the order shown:

- Prevention of flooding;
- Reduction of CSO discharges (Criteria often used to assess the CSO spillage are volumes discharged, frequency of overflow events and pollutant loads discharged);
- Uniform utilisation of storage capacity within the sewer system;
- Quick provision of storage capacity for subsequent storm events by emptying storage as quickly as possible at the end of rainfall; and
- Minimisation of operation and maintenance costs.

3.4.2 Physical Constraints

The control strategy has to be physically achievable, otherwise it will never be realised in the actual system. Sewers and storage tanks have capacities that cannot be exceeded in the control strategy and, therefore, the problem has to be restricted by these 'static' constraints. Valves and gates move at certain rates and so these also have to be introduced into the problem as constraints.

The control strategy has to obey to the physical laws of motion in a sewer system, i.e. continuity and energy balances - the dynamic constraints. For example, the dynamic constraint of a storage tank is:

$$Storage(t+1) - Storage(t) = [Inflow(t+1) - Inflow(t)] - [Outflow(t+1) - Outflow(t)] \tag{3.1}$$

The control strategy would not be physically feasible without these constraints.

3.4.3 System Loading

It is actually rather simple for an engineer to determine the response characteristics of a sewer system to a sufficient degree of accuracy, but it is more difficult to determine the loading of the system in real time. It is very difficult to measure the loads, determine a control strategy and implement the control actions in real time. A prediction of the loads can be very useful to aid this process. The following are examples of options that can be used to determine the inputs of a sewer system:

- Measuring flow and water levels in upstream sewers allowing a control reaction within the travel time of the sewage;
- Measuring rain and applying a rainfall-runoff model that extends the available reaction time by the time of concentration on the surface of the catchment; and
- Forecasting rain that gains additional time depending on the forecast time horizon.

It should be noted that the response characteristics and the control procedure govern the selection of the appropriate option. For local control it would often be sufficient to use the first option because the control procedure only reacts to the local sewer system state. However, on the other extreme, global control reacts to the state of the entire sewer system and therefore the decisions would take longer to determine making the other options more appropriate.

Obviously, the measurement and prediction of inputs may contain uncertainties and errors. Therefore, it is advisable that the control strategy is generally conservative to avoid failures, e.g. surcharging. Furthermore, it is sensible to have a good understanding of the behaviour of the control model and an analysis should be undertaken to check the sensitivity of the model to various types of errors.

3.4.4 Solution Techniques

Several methods can be used for the solution of operational control problems. Fundamentally, all the methods are optimisation methods and attempt to find the best control strategy. The control strategies can be determined on-line or off-line. An on-line system determines the control strategy during the process that is controlled.

Often optimisation methods are used to determine on-line control strategies and are generally automatic systems. An off-line system determines the control strategy prior to the actual control process and is often used in a supervisory mode. The methods can be listed as:

- heuristic methods;
- rule based scenarios;
- neural networks; and
- mathematical optimisation.

Gonwa *et al.* (1993) give comparisons of methods used in RTC (see, also, Khelil *et al.*, 1993b) and also sewer simulation methods. The above methods are descibed below.

3.4.4.1 Heuristic Methods

Heuristic methods determine the control strategy from experience (gained by trial and error). They range in complexity from simple rules on paper to rules implemented by computerized knowledge-based systems. Potentially, heuristic control can use sources of information that are not accessible to a computer such as intuition or 'view out of the window' etc. An experienced operator will probably carry out near-optimum control by disregarding all options that are possible but not advisable. The method does not ensure optimal control actions.

A major disadvantage with heuristic control is that the experience, gained by trial and error, will be lost once the operator leaves the job. The successor will make mistakes all over again. Additionally, heuristic control is only valid for one system since the experience gained at one catchement is not transferable to another.

Simulation tools are often used in heuristic approaches for the testing and comparison of a variety of control strategies. Examples of simulation tools are Fitasim (Einfalt, 1993; Einfalt *et al.*, 1994), Hydroworks (Ashley *et al.*, 1995), SAMBA-CONTROL (Jakobsen *et al.*, 1993), HYSTEM-EXTRAN (Khelil *et al.*, 1993a) and MOUSE ONLINE (Williams *et al*, 1994). However, the approach can only test a finite number of simulations so the optimum solution cannot be guaranteed.

Schütze *et al.* (1999b) developed a formalised trial-and-error procedure to determine RTC strategies offline and applied it to the city of Aachen. Firstly, the optimum static control strategy was determined. From this base case, local control actions were defined at points where improvements were possible. Finally, global control was incorporated, adding rules relating the interdependency of the different control devices where an improvement could be achieved.

3.4.4.2 Rule Based Scenarios

Rule based scenarios can be interpreted as being a heirachy of *if...then...else* statements relating input variables (e.g. forecasts) to output variables (e.g. control actions) by means of boolean logic, i.e. true or false. Such rule based systems are easy to understand although they require an extensive amount of development work because the outputs need to be specified in advance for all possible states of the system. Decision trees (or matrices) may be used to organise the enumeration process.

Rule based scenarios have the advantage of being able to develop control strategies very rapidly. However, the optimality of the strategies cannot be guaranteed. Similar to heuristic methods, rule based scenarios are application orientated.

Almeida *et al.* (1993) developed *if...then...else* rules for a simplified version of the Lisbon sewer system. The aim of this investigation was to combine the benefits of optimisation and heuristic approaches. An optimisation procedure, NOUDS (Neugebauer *et al.*, 1991), was used offline to produce optimal strategies corresponding to historical events. A heuristic procedure was then used to condense the strategies to a set of *if...then...else* rules. It was concluded that the results from an application of this procedure to the simplified Lisbon sewer system showed that it was possible to almost reach the performance of the optimised system.

In some investigations into RTC elements of fuzzy theory (Zadeh, 1965) have been implemented in control rules (Fuchs, 1997). Information can be included in an imprecise way, for example "the water is high" rather than the more usual "the water level is 3.45m". However, approaches based on the theory of fuzzy sets require careful definitions of the membership functions, which relate the linguistic terms to those terms used in the inference process.

Fuchs *et al.* (1995, 1997 and 1999) used HYSTEM-EXTRAN in combination with a rule based control device using fuzzy-logic for the RTC of a sewer system. A rule base for the rules interpreter, which processed the rule base and used fuzzy-logic, was created based upon optimisation calculations and the results of the simulated state of the system. In an application on part of the sewer system in Flensburg, this control study reduced overflows by 90% compared to the uncontrolled system.

Worm *et al.* (1999) used rule-based control strategies to equalise the hydraulic loading of a wastewater treatment plant. In this application, a stochastic modelling concept was used. The authors concluded that the rule-based control strategies had a great potential to equalise the loading of the plant.

If...then...else rules can easily be implemented in *knowledge based systems*, which are often called *expert systems*. This type of system was used by Khelil *et al.* (1993a) and Fuchs *et al.* (1987) where meta rules (the learning process) were developed in the knowledge based system, in conjunction with a hydrodynamic sewer model, which evaluated its own performance and modified its rule set from time to time. The shortcoming of this method is that many rules are generated so that the computer storage capacity may be exhausted (Khelil *et al.*, 1993a).

3.4.4.3 Neural Networks

Neural networks replicate the behaviour of the human brain by emulating the operations and connectivity of biological neurons. They are often regarded as black box methods. In a neural network, a series of connecting weights are adjusted in order to fit a series of inputs to another series of known outputs. When the training set of a neural network is large enough, the system is capable of reproducing an output for a given input, if this input is included in the original range of validity.

The training time for a neural network is long but its response time is generally short. Fundamental to the application of neural networks is the range of validity. Inputs outside of this range of validity, e.g. certain sewer system loads and states, should be prevented. If the sewer system is altered in any way, then a new training set is required to re-train the neural network. This is probably the major disadvantage of this method. An additional disadvantage is that the the reasoning behind the control decisions cannot be traced easily. However, neural networks are very fast at determining the solutions once they are trained and they do not need to know the problem structure.

Loke *et al.* (1997) gives a thorough description of neural networks and discusses applications in urban drainage systems.

Vazquez *et al.* (1999a) used a neural network in the application of the Muskingham model for real time management of sewer systems. Here, the Muskingham model was parameterised, calibrated to the St. Venant equations and a neural network was used to assess the parameters according to the length, slope and diameter of the sewer. The authors stated that the use of this model was more accurate than the classic Muskingham model.

3.4.4.4 Mathematical Optimisation

The four main mathematical optimisation procedures of relevance to the present context are:

- Dynamic Programming (DP);
- Linear Programming (LP);
- Non-linear Programming (NLP); and
- Network Programming (NP).

Comprehensive descriptions of these methods are given in Mays *et al.* (1992), Smith *et al.* (1983) and Templeman (1982).

The choice of optimisation method depends on two factors:

- the characteristics of the system under consideration; and
- the required modelling accuracy of the objectives.

These methods are described below.

Dynamic Programming

Dynamic Programming (DP) is a decision process (not an algorithm) for solving sequential problems and originates from work undertaken by Bellman (1957). The principle upon which DP is based is termed *Bellman's Principle of Optimality*, which states that 'an optimal policy has the property that whatever the initial state and decision are, the remaining decisions must constitute an optimal policy with regard to the state resulting from the first decisions.' In a sequential system the principle means that at any stage the only information required to determine an optimal solution for the remaining stages of the system is the input information to the present stage. There are no standard solvers for DP problems and, therefore, any solvers developed are problem specific. DP has been used within the present study and is described in greater depth in Section 3.6.

Linear Programming

Linear Programming (LP) is probably the most widely used of the mathematical optimisation methods. In Linear Programming, the objective function and constraints all have to be linear. The main advantages of LP are that the solutions are found very rapidly using readily available solvers. LP has been used within the present study and is discussed in greater depth in Section 3.5.

Non-Linear Programming

Non-Linear Programming (NLP) determines solutions to problems that have non-linearity in some or all functions. NLP problems are very difficult to solve, and are also computationally slow. Sometimes there may not be a single unique solution, but many local optima. Additionally, there are few general purpose methods or programs to solve them.

One approach to the solution of NLP problems is *sequential linear programming.* The non-linear optimisation problem is approximated by a series of linear optimisation problems. After finding the solution to the linearised problem a new approximation to the non-linear relationships is determined at the optimal values of system variables found. The updated LP problem is then solved and the process is repeated. The linearisations become more accurate as the iterative procedure continues. If applied well, the process converges and the final linearisation point closely matches a local optimal solution of the non-linear problem. Nelen (1992) and Lobbrecht (1997) use this approach, where a network flow algorithm is used, NOUDS (Neugebauer *et al.*, 1991), for the LP solutions.

The LOCUS modelling package was developed for the real time control of urban drainage systems and uses the sequential linear programming technique (Nelen, 1992; and Nelen, 1993). The problem was formulated to minimise an objective function of variables storage, continuation discharge and overflow, each with unit costs. The continuity equations were added as constaints. The unit costs of overflow were higher than those of storage and transport. The unit costs were modified at each time step, for example the unit cost of storage would increase with increasing filling degree. LOCUS was applied (Jørgensen *et al.* 1995) on a number of case studies to determine the potential reduction of CSOs by means of RTC. It was also applied to the Hague catchment successfully (Allitt *et al.*, 1994), where pollution load spilled was reduced.

Non-linear problems can be solved using search methods. One search method is the *genetic algorithm (GA)*, which is a stochastic search technique applying the process of biological evolution to find a near optimal solution in a search space. The conceptual development of the technique by Holland (1975) was inspired by the ability of natural systems for adaption. GAs mimic some of the observed processes in genetics in order to retain the robust performance of natural systems in their search for improvement. A description of the application of these types of techniques in urban drainage modeling is given in Rauch *et al.* (1998).

Rauch *et al.* (1999) used a GA technique to minimise pollution from urban wastewater systems, which included discharges from overflow structures and the treatment plant, i.e. integrated control. In this study, the SAMBA (DHI, 1996) model was used to simulate the processes in the sewer system, the IAWQ activated sludge model No. 1 (Henze *et al.*, 1987) was applied for the conversion processes in the treatment plant, and FOXTROTT was used for the modelling of the water quality in the river, which has been integrated with SAMBA (Harremoës *et al.*, 1996). It was stated that a reduction in overflow volume was not directly linked to an increase of the oxygen concentration in the river and that superior performance would be obtained with improvements in the description of the processes in the system rather than an improvement in the optimisation algorithm.

Yagi *et al.* (1998) described the use of GAs and fuzzy logic to achieve advanced pump operation in a combined sewer pumping station. The pumping rates were determined by fuzzy inference and fuzzy control rules and the GAs were used to automatically improve these fuzzy control rules.

Vetri *et al.* (1999) used a GA technique for the calibration of urban drainage models. An alternative technique was also used, *Genetic Programming (GP)* (Koza, 1992). GP is described in detail in Babovic (1999). GP is an extension of the computational simulation of natural genetics and induces a process of selection to identify a relation between input and output values. Both methods were used satisfactorily on a case study in Italy.

Controlled Random Search (CRS) is also a search technique for the solution of non-linear optimisation problems. CRS was used by Schütze *et al.* (1998b) who stated that it was the most successful optimisation procedure within an integrated control model. CRS can be categorised as a stochastic search procedure where the algorithm starts with a randomly generated initial population of trial points. After evaluation of the corresponding objective function values, a new population is generated and this evaluation-generation procedure is repeated until the algorithm terminates. New populations are generated by reflections on the centre of gravity of subsets of the current population.

Network Programming

Networks are structures that can be described by arcs and nodes. A network optimisation problem is actually a special type of LP problem where there are further restrictions in the functions of the problem. The functions have to be not only linear but also the coefficients have to be either 1, -1, or 0. The solution algorithms for such problems are very fast.

Neugebauer *et al.* (1991) developed a network algorithm for the optimum control of urban drainage systems, which was tested on a hypothetical sewer system. The results showed that the computational speed of the network algorithm was much higher than the speed of LP algorithms.

3.5 LINEAR PROGRAMMING

Linear programming (LP) is probably one of the most successful and widely applied of the mathematical optimisation methods. It may be used to solve a wide variety of civil engineering problems and can be rapidly programmed for solution on computers. However, several conditions must be met before LP can be adopted. Firstly, the problem under consideration must be concerned with the specification of non-negative values and a set of variables that optimise a linear function expressed in terms of these variables. Secondly, the optimisation of this function must also satisfy one or more linear constraints which mathematically describe the availability or requirements of the resources.

The general form of a LP problems reads:

Minimise $$\sum_{j=1}^{n} c_j x_j \qquad (3.2)$$

subject to $$\sum_{j=1}^{n} a_{ij} x_j = b_i \qquad \text{for } i = 1,2, \ldots ,m \qquad (3.3)$$

$$x_j \geq 0 \qquad \text{for } j = 1,2, \ldots ,n \qquad (3.4)$$

where c_j is the objective function coefficient, and a_{ij} and b_i are known constants of the constraints.

A simple example of a two-dimensional LP problem is shown below (3.5 to 3.11).

Minimise $$f = 6x_1 + 4x_2 \qquad (3.5)$$

subject to $$2x_1 + 2x_2 \geq 60 \qquad (3.6)$$

$$2x_1 + 4x_2 \geq 80 \qquad (3.7)$$

$$4x_1 \geq 60 \qquad (3.8)$$

$$4x_2 \geq 20 \qquad (3.9)$$

$$3x_1 + 2x_2 \leq 120 \qquad (3.10)$$

$$x_1, x_2 \geq 0 \qquad (3.11)$$

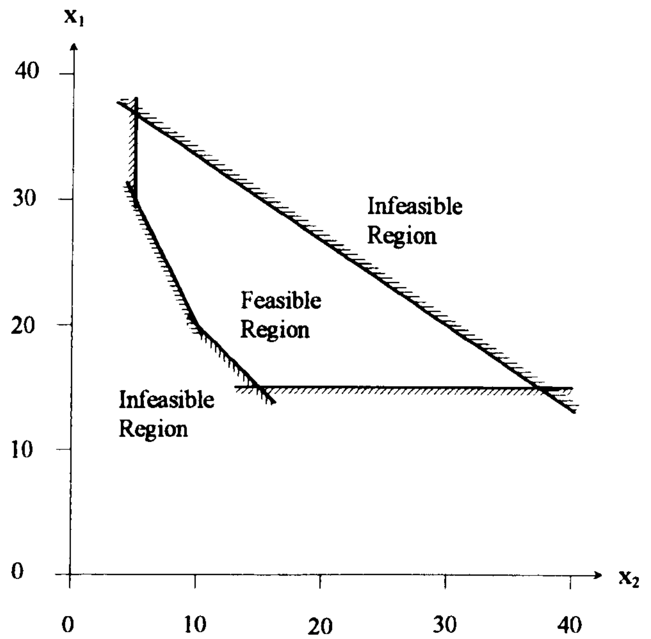


Figure 3.3: Complete Set of Constraints for the LP problem (3.6 to 3.11).
(Taken from Templeman (1982))

The complete set of constraints (3.6 to 3.11) marks out a *feasible region* and an *infeasible region*, as shown in Figure 3.3. A point within the feasible region or on its boundaries satisfies all the constraints and so is an acceptable solution to the problem. However, this point may not be an optimal solution to the problem, which is located at the minimum (or maximum) cost of the objective function. The optimum solution will always fall on a constraint vertex or boundary in LP problems.

3.5.1 Forms of Linear Programming

LP models can be presented in a variety of forms (e.g. maximisation, minimisation, ≤, =, ≥) and it is necessary to modify these forms to fit a particular solution procedure. There are basically two types of linear programming model formulations used: *standard form* and *canonical form.*

The *standard form* is used for solving the LP model algebraically. Its characteristics involve the following:

- all constraints are equalities except for the non-negativity constraints associated with the decision variables which remain inequalities of the ≥ type;
- all the right hand side (RHS) coefficients of the constraint equations are non-negative, that is, $b_i \geq 0$;
- all decision variables are non-negative; and
- the objective function can be either maximised or minimised.

The *canonical form* is useful in presenting the duality theory of the LP model. It possesses the following features in the model formulation:

- all decision variables are non-negative;
- all constraints are of the $\leq$ type; and
- the objective function is of the maximisation type.

It should be noted that a negative RHS coefficient in the canonical form is permissible.

Often, the LP model originally constructed does not satisfy the characteristics of a standard form or a canonical form. The following operations enable the transformation of an LP model into any desirable form.

- Maximisation of a function $f(x)$ is equal to the minimisation of its negative counterpart, that is, Max $f(x)$ = Min$[-f(x)]$.
- Constraints of the $\geq$ type can be converted to the $\leq$ type by multiplying by -1 on both sides of the inequality.
- An equation can be replaced by two inequalities of the opposite sign. For example, an equation $g(x) = b$ can be substituted by $g(x) \leq b$ and $g(x) \geq b$.
- An inequality involving an absolute expression can be replaced by two inequalities without an absolute sign, e.g. $|g(x)| \leq b$ can be replaced by $g(x) \leq b$ and $g(x) \geq -b$.
- If a decision variable x is unrestricted-in-sign, i.e. it can be positive, zero, or negative, then it can be replaced by the difference of two non-negative decision variables.
- A non-negative variable can be added or subtracted to transform an inequality into an equation.

3.5.2 Solution Algorithms for Linear Programming

3.5.2.1 Graphical Method

The simplest way to solve an LP problem is by using the graphical method, although this method is limited to LP problems involving at most two decision variables. An example of the graphical solution method is given in Figure 3.4, which is a representation of the LP problem in equations 3.5 to 3.11. The constraints of the

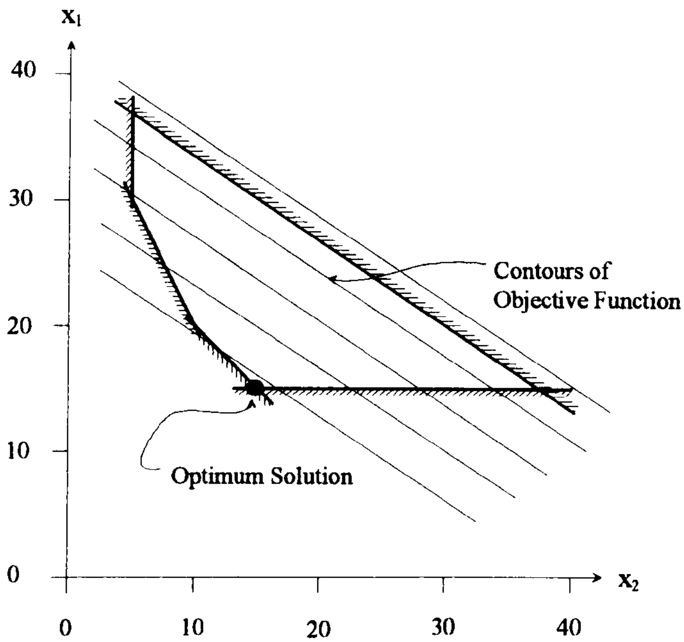


Figure 3.4: Graphical Solution to the LP Problem (3.5 to 3.11).

problem can be drawn defining the feasible region (as in Figure 3.3). It should be noted that the non-negativity constraints (3.11) are not active in this problem. The contours for the objective function (3.5) can then be drawn, which is to be minimised. Therefore, by inspection it can be seen that the optimal solution to the problem is located at the lower left constraint vertex in Figure 3.4. The corresponding values of the decision variables, x_1 and x_2, can be determined from the axes.

3.5.2.2 Simplex Method

The simplex method is the most widely used solution method for LP problems. As demonstrated, a LP problem can easily be solved using the graphical method, or a heuristic approach, when only two variables are involved. However, these methods cannot be practically implemented for the solution of larger problems. Furthermore, the problem must be expressed algebraically for the implementation of a solution algorithm on computers. The simplex method was developed for this procedure and is described below. However, it is easier to understand the method by example, one of which is given in Appendix 2.

It is generally more convenient to use equations rather than inequality relationships in problem solving. Therefore, the simplex method first transforms the model into a standard form. Each constraint that is not an equality is converted into an equality constraint by adding or subtracting a non-negative variable, a *slack variable*, which is different for each constraint.

The LP model is then rearranged so that for each constraint a variable (usually the slack variable) is expressed as a linear function of the other variables. The objective function is introduced and the model can be written in matrix form. A starting point from the feasible region is selected and the values of the variables are determined. The value of one of the variables is then altered so that the solution point moves to a constraint boundary. The LP model is *pivoted* so that the solution point is altered so that it is located at a constraint vertex. This pivoting operation is continued so that the solution point moves to another constraint vertex until the optimum solution is determined.

3.5.2.3 Alternative Solution Procedures

There are some alternative solution methods that can be used in LP models, which are more computationally efficient than the simplex method. The simplex method restructures the matrix at each pivot operation but only a few elements are used and, therefore, many of the calculations are redundant. A variety of alternative simplex-based methods can be used such as the *revised simplex method*.

Two other methods use a completely different algorithmic philosophy: Khatchian's ellipsoid method (Khatchian, 1979) and Karmarkar's projective scaling method (Karmarker, 1984) seek the optimum solution by moving through the interior of the feasible region.

3.5.4 Applications of Linear Programming

A RTC system was developed using LP and applied successfully to the Bremen combined sewer system (Schilling *et al.*, 1987). Schilling *et al.* (1987) advise to account for the simplifications in the LP formulation with detailed verification and that the forecast model should not be biased towards underforecasting. The formulation assigned variables for each process in the sewer system and had to be solved for the entire time horizon to achieve a global optimum solution. This approach is considered inappropriate for the large interceptor systems investigated in this study because it would be computationally inefficient. The 'slug flow' approach adopted here allows for a computationally efficient solution and enables the operational problem to be solved in discrete time steps through the time horizon.

A pollution-based RTC module was developed by Weinreich *et al.* (1997) using a rules interpreter or a mathematical optimisation model. The optimisation model was formulated using LP for volumes only, assigning a unit cost for overflows, storage and discharge. A completely mixed reactor model (i.e. non-linear) was used for the calculation of the pollution concentrations, which was determined between time step solutions. Pollution discharges were minimised by modifying the unit costs of the overflow volumes according to the actual concentrations calculated. Therefore, a new optimisation problem was solved at each time step. This procedure was applied successfully in a case study, having more than a 10% reduction in the discharge of pollution compared to volume based RTC. The formulation of LP problem differs from the approach adopted here in that the objective function includes variables for overflows, storage and discharge. This objective function requires more computational effort to solve since there are more variables in each time step. Therefore, this approach has not been adopted in this study.

Verworn *et al.* (1998b) describe the upgrading of the hydrodynamic rainfall runoff model HYSTEM/EXTRAN and the decision finding model INTL for real time performance. INTL developed online control strategies from an extension of the simplex algorithm solving a linear optimisation problem. Pollution based control could be taken into account by varying the objective function in consideration of a rule base containing relations between pollution parameters and their influence within the control system (Kolbinger, 1996). This approach is considered inappropriate for very large interceptor systems on which this study focuses upon because of the computational inefficiency of applying detailed simulation models in determining the system's state in real time.

Vazquez *et al.* (1999b) developed a real time management algorithm based on an alternative LP procedure, graph theory, to reduce the pollution discharged into the environment. In heavy rainfall conditions the amount of pollution discharged could be reduced by four times, despite higher volumes of discharges. The application of the theory in this research is inappropriate because of the scale of interceptor system studied.

To overcome the problems associated with system simplification Rohlfing (1993) linked an optimisation program, which was based on LP, with a hydrodynamic flow routing program EXTRAN to ensure that the effects of the control actions were taken into account accurately for the optimisation of the subsequent time step. A similar approach was suggested by Lobbrecht (1997). Again, the use of detailed simulation models is considered inappropriate in this study because of their computational inefficiencies when modelling large systems.

3.6 DYNAMIC PROGRAMMING[1]

As defined earlier, Dynamic Programming (DP) is a decision process (not an algorithm) for solving sequential problems and originates from work undertaken by Bellman (1957 and 1962). The principle upon which DP is based is termed *Bellman's Principle of Optimality*, which states that 'an optimal policy has the property that whatever the initial state and decision are, the remaining decisions must constitute an optimal policy with regard to the state resulting from the first decisions. In a sequential system, as in Figure 3.5, the principle means that at any stage the only information required to determine an optimal solution for the remaining stages of the system is the input information to the present stage.

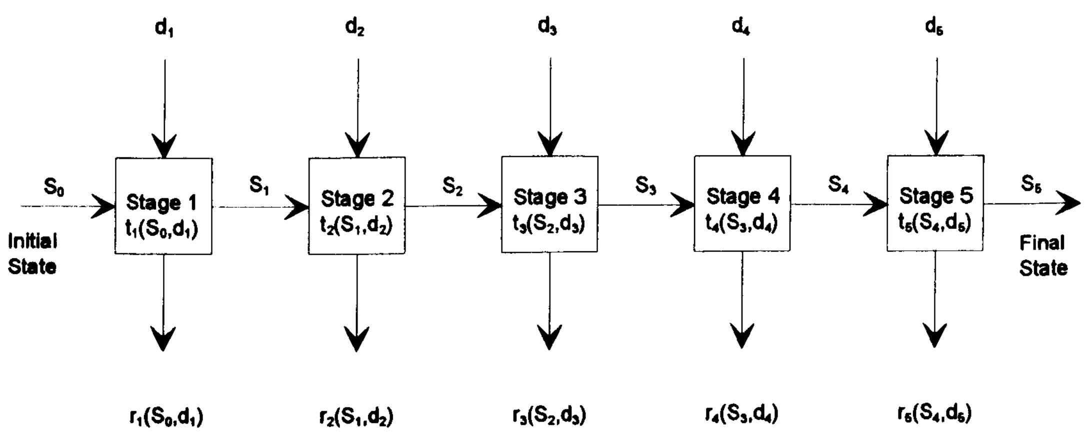


Figure 3.5: A General Serial System.

Each stage in Figure 3.5 is connected by a state variable, a variable or parameter that can change in value, which passes through the entire system. Initially the state variable, S, has the value of S_0. Some decisions are made as it passes through stage 1, d_1 causing the value of the state variable to change to S_1, the output state from stage 1. The change in value of the state variable is represented by the transition function $t_1(S_0,d_1)$ so that

$$S_1 = t_1(S_0, d_1) \tag{3.11}$$

In words, the value of the output state from stage 1, S_1, is a function of the input state to stage 1, S_0, and the decisions made in stage 1. Some costs (or returns) are generated by stage 1 as a consequence of the decisions made and the change in the value of the state variable. These costs are functions of the stage 1 input state and decisions represented by $r_1(S_0,d_1)$.

The output state from stage 1, S_1, now becomes the input state to stage 2 and here decisions d_2 are made which alter the value of the state variable from S_1 to S_2 by the transition function:

$$S_2 = t_2(S_1, d_2) \tag{3.12}$$

The decisions and resultant alterations to the value of S in this stage again generate costs given by $r_2(S_1,d_2)$. The process continues as S_2 enters stage 3 and eventually stage 5 is reached where the state variable emerges with a value of S_5.

The general DP process has been presented above in that each stage is examined sequentially and a set of discrete output state values is postulated. For each discrete output state, cumulative total costs (or returns) associated with achieving that state value are examined and the minimum (or maximum) value is selected from the possible candidate values. This cumulative approach for possible output states is the essence of the DP method and is represented as:

$$C'_{S_i} = \min_{d_i} (or \max) \left\{ C'_{S_{i-1}} + r_i(S_{i-1}, d_i) \right\} \tag{3.13}$$

[1] Most of the DP description is extracted from Templeman (1982).

In words, the cumulative cost C' to be associated with a particular output state from the i^{th} stage is equal to the least value (or greatest if the problem is one of maximisation) obtained by adding the cumulative cost associated with each possible input state, C'_{Si-1}, to the appropriate i^{th} stage return $r_i(S_{i-1},d_i)$. This relationship (3.13) is sometimes referred to as the *dynamic programming recurrence relationship.*

An example of the DP method is given in Appendix 2.

Labadie *et al.* (1980) used DP in the optimal control of unsteady combined sewer flow and applied it successfully to part of San Francisco's sewer network. Here a fully dynamic, unsteady flow model was included within a deterministic DP formulation of the control model. Labadie *et al. (1980)* state that convergence to a local optimum, let alone a global optimum, cannot be guaranteed and, therefore, the approach is considered inappropriate for this study.

3.7 UNCERTAINTY IN MATHEMATICAL MODELLING

A deterministic mathematical model attempts to replicate the processes in a physical system where it is assumed that the cause-effect relationships are known enabling predictions with certainty. However, Harremoës *et al.* (1998) state that this is an ideal that is never reached in practise. Uncertainty is defined as the occurrence of events that are beyond human control. Nevertheless, decisions still have to be made under various kinds of uncertainty, whether it is in the planning, design or operation phase of a system.

Beck (1991) states that in the first instance it is important to determine whether the uncertainty in a problem is significant or not. The options here are:

- the level of uncertainty is not significant, enabling the use of the model in an entirely deterministic fashion; or
- the level of uncertainty is significant and some account of uncertainty attaching to the model's predictions must be given.

It should be noted that significance might not solely be a function of the magnitudes of the prediction errors. For example, a small amount of uncertainty close to an acceptable boundary may be more significant than a large amount of uncertainty further away from this boundary.

Errors may derive from three sources:

- the estimated initial state of the system;
- the assumed patterns of future variations in the input disturbances of the system; or
- the model.

If errors in the initial state of the system were dominant, a more extensive survey of the field system would be called for. Beck (1991) states that such dominance is unlikely and that prediction errors are more probably dominated by uncertainty in the system's inputs.

The most dominant source of uncertainty is the model itself where it is important to quantify the uncertainty and find the means of its reduction. The model's performance should be exposed to a set of in situ field data in order to establish whether it is fit for the making of predictions. This model verification does not quantify the uncertainty in the model but just examines its validity. There are four purposes to which exposure of the model's performance to field data may be put:

- acceptance or rejection of the model as a valid instrument;
- estimation of the values of those parameters that cannot otherwise be determined;
- estimation of both the values of the parameters and their uncertainty; or
- identification of the correctness of the structure of the mathematical relationships among the system's variables.

In LP it is possible to explore the sensitivity of the solution to changes in the parameters of the model, which is known as *Sensitivity Analysis*. Several methods have been proposed that allow the effects of changes in the objective function coefficients, constraint coefficients and constraint bounds to be calculated without completely re-solving the problem. This procedure illustrates the significance of uncertainties within the inputs of the LP model.

Investigations into the uncertainties within sewer modelling include Einfalt *et al.* (1993), Lei *et al.* (1996), Friedler *et al.* (1996) and Schilling *et al.* (1986).

3.8 CONCLUDING REMARKS

This chapter has described the various techniques that are available for the real time control of sewer systems. The various control levels have been distinguished illustrating that the optimal control of sewer systems can only be achieved with global control. The formulation of the operational problem has been described with descriptions of the various methods for its solution. Much of the research conducted so far in this field has been based on heuristic methods in conjunction with sewer simulation software. The optimum solution cannot be guaranteed using such methods.

Mathematical optimisation procedures have generally been used for the online real time control models. Linear Programming is the most widely applied of these procedures although the simplifications in the problem formulation have led to scepticism about the physical accuracy of the control actions. Therefore, it has been advised to fully verify the results from the optimisation models.

Despite the research and development in the real time control of sewer systems, there are only a very few in operation. These include, for example: Barcelona, Spain (Quer *et al.*, 1993; Marquès *et al.*, 1999a and 199b); Bolton, UK (Williams *et al.*, 1994); Copenhagen, Denmark (Hansen *et al.*, 1997); Ense-Bremen, Germany (Khelil *et al.*, 1991) Seattle, USA (Vitasovic, 1995; and Vitasovic, 1993). The primary reasons for the limited application of real time control techniques, particularly global control techniques, are that there is a general scepticism about the reliability and accuracy of the techniques, the legislation consents are not flexible (e.g. global consent), and there is no definitive control technique available for adoption.

Most of the RTC investigations reviewed in this chapter have been based on volumetric criteria. In the last few years, there have been a few studies into pollution based RTC of sewer systems and also integrated control of wastewater systems. This thesis attempts to further develop the understanding of pollution based control of sewer systems by developing a verified optimal pollution control for interceptor sewer systems. The formulation of the model is described in the next chapter, which actually uses a rather unique approach. The model has been developed from a systems point of view, adding hydraulic criteria when necessary, and therefore avoids the use of computationally inefficient sewer flow simulation software within the control procedure.

CHAPTER 4

IDEALISED INTERCEPTOR CONTROL MODEL-DERIVATION AND RESULTS

4.1 INTRODUCTION

This chapter describes the formulation of the optimal pollution control models for idealised interceptor sewer systems from first principles, and provides a complete description of the modelling approaches in this thesis. The model is formulated using a slug flow approach where the 'slugs' are tracked through the interceptor and the control model determines the amount of sewage that should be added from the individual catchments based on the appropriate time delays and their respective pollution loadings. This optimisation problem is solved using two procedures, Linear Programming (LP) or Dynamic Programming (DP). Both methods are described in full. The development of the models is also described in Thomas *et al.* (1998; and 2000). The limitations of the slug flow approach are enumerated in Section 5.2.

The control models are first tested on a fictitious interceptor sewer system to confirm the viability of the slug flow approach and the optimisation procedures. Even though this is a short chapter the slug flow approach is a fundamental aspect of the optimal pollution control models investigated herein and remains key to further developments described later in the thesis (Chapter 6).

4.2 MODEL DEVELOPMENT

The optimal pollution control model formulation allows the optimum solution to be obtained using Linear Programming (LP) or Dynamic Programming (DP). The control models were developed using a slug flow approach and determine optimum inflow rates for an interceptor sewer system based on estimated pollutant concentrations. The slug flow approach and LP formulation are described below.

Various assumptions are made in the formulation of the control models including:-

i) all inflows and their respective pollution concentrations are known;

ii) hydrographs are piecewise constant within time steps;

iii) flows in excess of the controlled interceptor inflows are spilled to the river;

iv) all pipe flows are in the downstream direction; and

v) there is complete control over the proportion of flow diverted into the interceptor from the CSOs.

4.2.1 Initial Linear Programming Model

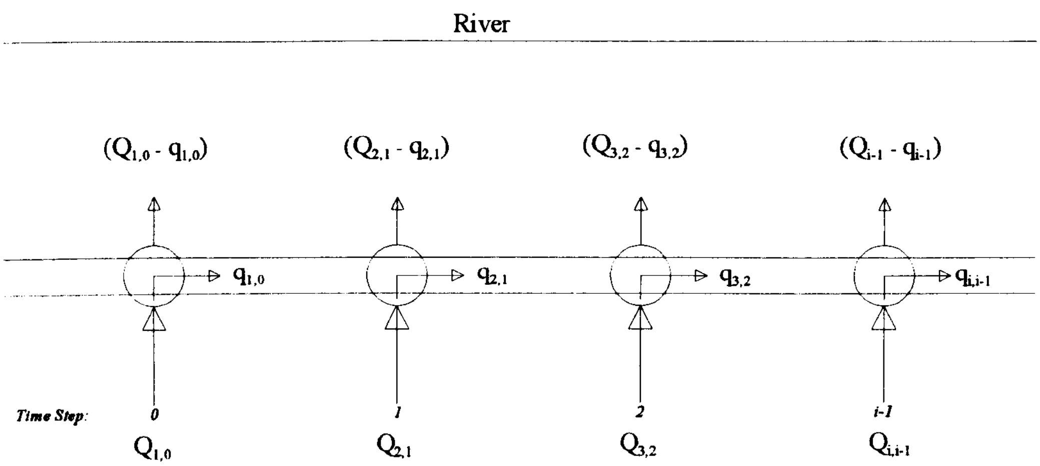


Figure 4.1: Theoretical Basis of the Model.

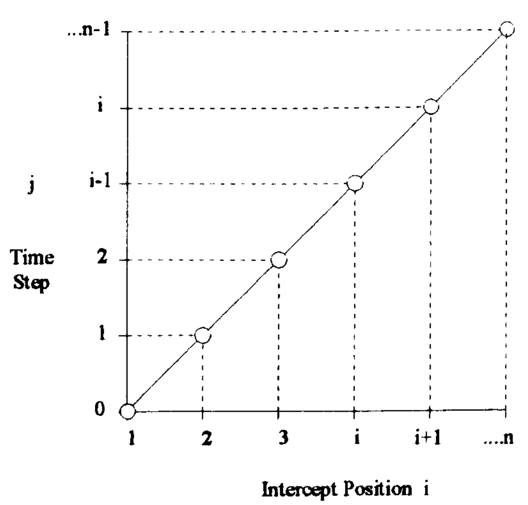


Figure 4.2: Chain of Water Commencing at Time Step 0.

The fundamentals of the optimal pollution control model are shown in Figure 4.1. Figure 4.1 represents a decision to be made at each intercept point where q_i is the control variable. (Q_i - q_i) is the spill rate as a result of the control decision q_i. Q_i are hydrographs so Q_i and q_i are functions of time. Therefore, a second subscript is included in the notation to allow for this, j, corresponding to a particular time step. For example, $Q_{1,2}$ corresponds to the inflow at intercept point 1 in time step 2.

For the purposes of model development, some further assumptions are initially imposed (and later removed). It is assumed that interceptor points are equally-spaced along the interceptor, flow velocity is constant, and therefore the time step can be chosen to be equal to the time of flow in the interceptor sewer between any two intercept points. As a consequence of this assumption, the model can be described as a chain of water travelling down the interceptor system as shown in Figure 4.2. A slug of water, $q_{1,0}$, enters the interceptor at the extreme upstream end (intercept point 1) at time t = 0 and travels down the interceptor in the first time step. At t = 1 it arrives at intercept point 2 where it is incremented by slug $q_{2,1}$. The combined slug moves on down the sewer, accreting slugs $q_{3,2}$, $q_{4,3}$ etc., until the outfall is reached. The slugs of water are therefore treated as being separate in time and space. Therefore, an explicit constraint for hydraulic continuity between time step solutions within the optimisation is not needed. This approach has hydraulic deficiencies since the slugs will of course interact to some degree but it enables a highly efficient computation of the control actions (or control strategy).

A pollutant concentration factor, $\alpha_{i,j}$, is introduced into the model to facilitate the optimisation of pollution levels. For computational convenience here the pollutant concentration factor is defined as a coefficient assigned to each time step inflow at each time step. For general illustration this coefficient can be considered to range from 0 to 1, i.e. absolutely 'clean' through to absolutely 'dirty' inflows. A standard LP optimisation model can now be set up.

Objective Function: Minimise pollutant load to the receiving water over variables $q_{i,j}$.

Thus,

$$Min\sum_{i=1}^{n}\alpha_{i,i-1}(Q_{i,i-1}-q_{i,i-1}) \tag{4.1}$$

where n - the number of intercept points.

Since the total pollutant load $\sum_{i=1}^{n}\alpha_{i,i-1}Q_{i,i-1}$ is invariant for any storm event, Eq. (4.1) can be written as:

$$Max\sum_{i=1}^{n}\alpha_{i,i-1}q_{i,i-1} \tag{4.2}$$

subject to capacity constraints.

Constraints:

$$q_{i,i-1} \le Q_{i,i-1} \qquad \forall i \tag{4.3}$$

$$\sum_{j=1}^{i} q_{j,j-1} \le C_i \qquad \forall i \tag{4.4}$$

$$q_{i,i-1} \ge 0 \qquad \forall i \tag{4.5}$$

where C_i - Interceptor capacity at point i.

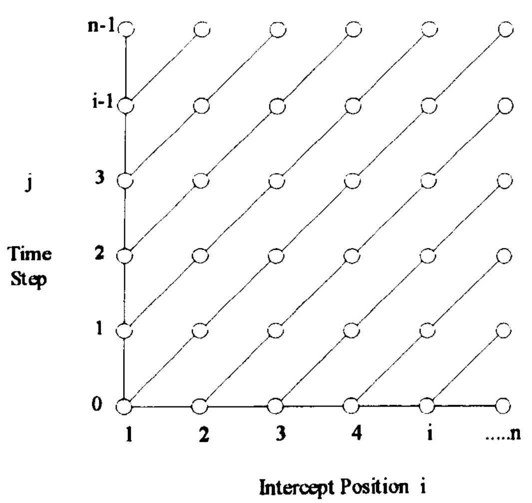


Figure 4.3: Complete Model with Chains from All Time Steps.

The objective function (4.2) has been developed in the LP model for the chain of water shown in Figure 4.2. This LP problem can be solved by any standard LP solver, in this case a computer program written in FORTRAN code using the Simplex Method. However, this is just one chain of water running through the interceptor and there are other chains at different time steps. The complete chain model over an entire storm event is shown in Figure 4.3. This can be solved by the same method as described above but each chain is solved in sequence (i.e. j+1, j+2,…, n-1).

Alternatively, the LP model could have been formulated to solve the complete control strategy, which would lead to a much larger LP model. This has not been developed in this research and is not discussed further.

The solution procedure commences at the lower right hand corner of Figure 4.3, i.e. at intercept point n in time step 0. In fact, there is no decision at this point since this represents the initial state of the sewer (cumulative DWFs). The chain of water to the left is the next solution to be determined. Here, the optimisation routine determines how much inflow to add to the initial state of the interceptor at intercept point n (based on the pollutant concentration of that inflow). Further chains are processed in sequence. The methodology tracks the slugs through the interceptor and determines the amount of sewage that should be added from each catchment based on the pollutant concentrations at the appropriate time steps. The procedure continues until all the chains are solved throughout the time horizon and the full control strategy is obtained. Since, in this approach, the successive slugs of water are assumed not to interact, then the sequence of optimal controls derived in each time step also represents the optimal control strategy for the entire event.

4.2.2 Enhanced Linear Programming Model

An extended version of the LP model relaxes the initial assumption of equi-spaced interception points along the interceptor sewer pipe. The interceptor pipe is now divided into time steps and there are intercept points on only some of these steps. This is shown in Figure 4.4 and is better able to represent a realistic system. The model can now control interceptor systems where the intercept points are at irregular intervals but the times of flow between intercept points remain equal. This version of the model differs slightly from the original version in the definitions of the inflow rates and pollutant factors at the appropriate time steps.

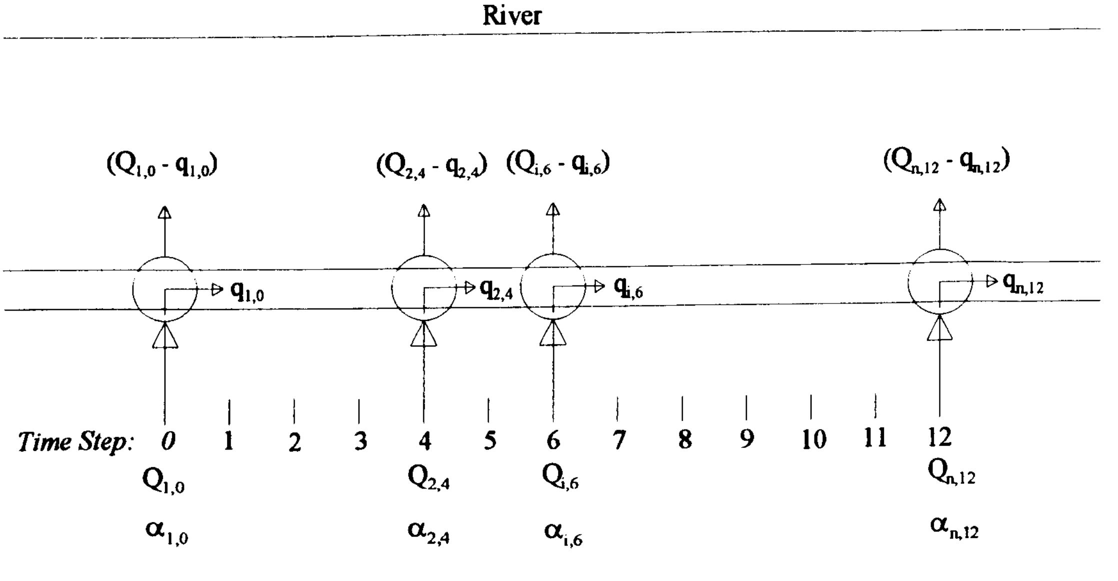


Figure 4.4: Enhanced Model Formulation.

The positioning of the time steps and the intercept points is based on the pipe full velocity of each section of the interceptor sewer. This time step 'grid' on the interceptor pipe is fundamental to the model procedure where the slugs of water are tracked through the pipes at these assumed velocities. The model can now control interceptor sewers where the intercept points are at irregular intervals but the time of flow between intercept points remain constant.

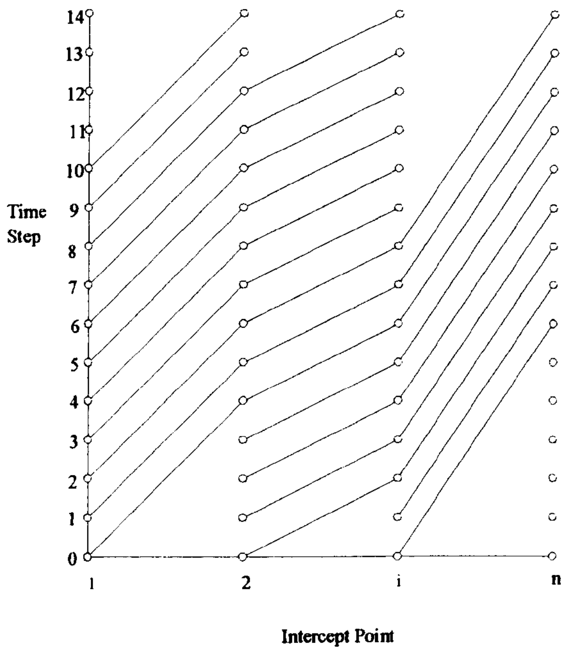


Figure 4.5: Solution Procedure for Enhanced Model.

The solution procedure can be seen graphically in Figure 4.5 where the change in gradient of the lines between control points differs from Figure 4.3. However, the gradient remains constant between successive intercept points because of the necessity of a constant flow velocity, as described earlier. In reality of course the gradients would vary in each time step depending on the depth and surface slope of each slug of fluid (assuming that the slugs do not interact). The modified LP model is little changed:

$$Max \ \sum_{i=1}^{n} \alpha_{i,t_i} q_{i,t_i} \tag{4.5}$$

where n - the number of intercept points;

t_i - the time step position within the interceptor of intercept point i;

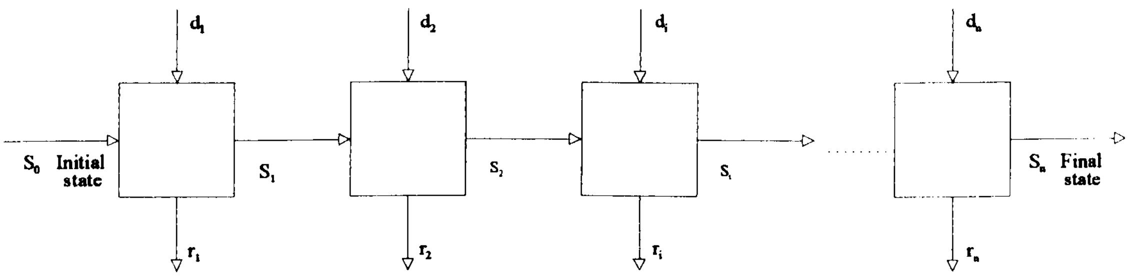
$\alpha_{i,ti}$ - the pollutant concentration factor at intercept point i in time step t_i; and

$q_{i,ti}$ - the interceptor sewer flow rate after intercept point i in time step t_i.

The constraints for the solution are adjusted to coincide with the appropriate time step t_i. This LP model is solved using the Simplex method. Again, the complete strategy is determined by using the procedure where the time step positions are adjusted in sequence (i.e. t_i+1, t_i+2,...etc.).

4. 3 Dynamic Programming Model

An alternative procedure for determining the optimal solutions in the control model is using Dynamic Programming (DP). DP is an extremely fast and efficient method of solving multi-stage sequential optimisation problems. DP models are relatively difficult to formulate and, unlike LP, there is no standard solver such as the Simplex Method. Each DP problem is formulated from first principles and is solved using a computer program unique to that problem.



where S_i - state variable at intercept point i (i.e. Interceptor flow rate);
d_i - decision variable at intercept point i (i.e. How much inflow to intercept?); and
r_i - cost of decision at intercept point i (i.e. Pollutant load to river).

Figure 4.6: Fundamentals of DP Solution Method.

The DP model is formulated in a similar fashion to the LP model, in that the model determines the optimum interceptor sewer flow rates for the chains in Figure 4.5. However, the difference between the models lies in the solution method. Figure 4.6 shows the fundamentals of the DP model. The slug of water travelling though the interceptor sewer is the DP state variable, S_i (limited by the interceptor capacity), and the algorithm determines the quantity of inflow from each catchment, d_i (the decision variable), to add to this slug based on the pollutant concentrations of all the inflows at the appropriate time steps. The cost of the decision, r_i, is the resultant pollutant load spilled to the receiving water. The optimum solution is reached when the total pollutant load spilled from the entire interceptor sewer, $\sum_{i=1}^{n} r_i$, is a minimum.

Figure 4.7 shows an example of the DP solution method. In this example there are ten possible interceptor flow rates, each of which is a proportion of the sewer capacity.

This capacity is, in fact, the capacity at the extreme downstream section of the interceptor system. The selection of this proportion depends on the accuracy required and might be considered to represent the dynamic restrictions in setting of the control gates. However, the increase in accuracy will have a consequential increase in computational time. The solution method shown in Figure 4.7 appears to be a complicated process, and would indeed be complicated if the complete 'network' had

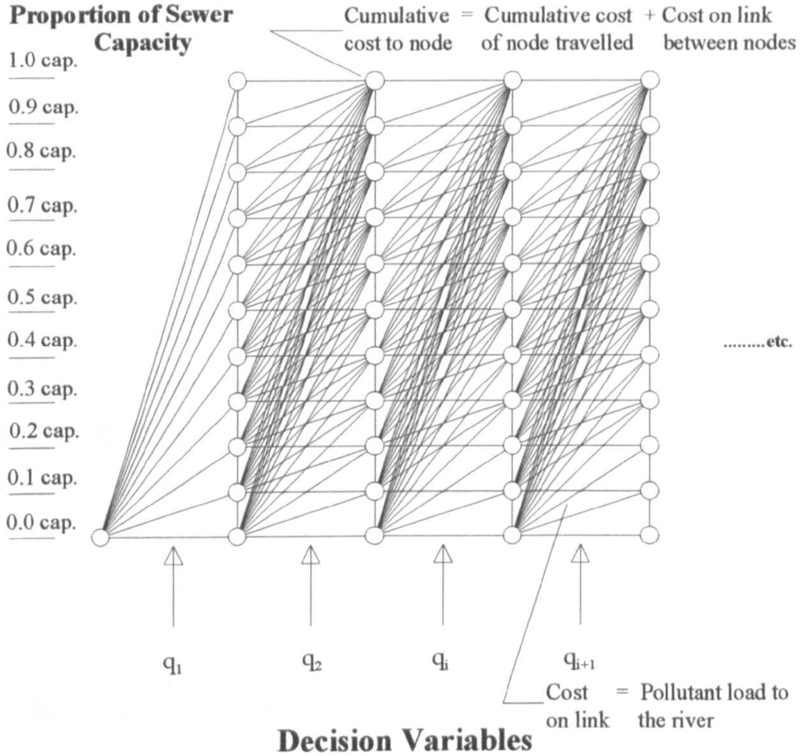


Figure 4.7: Example of a DP Solution Procedure.

to be determined and stored in memory. However, the DP method only stores two stages in the memory at any one time. This is because the cumulative cost at any node is determined by adding the cumulative cost of the previous node to the link cost connecting those nodes. All preceding costs are then discarded.

The DP solution method in Figure 4.7 is described as follows:

1. The DP method works through the system from left to right.
2. The starting cost (at the extreme left-hand node) is 0 since there are no spills.
3. The cost on the link for the decision q_1 to get to node proportion 0.0 times sewer capacity is the spill volume multiplied by the pollutant concentration factor (i.e. pollutant load to river). Clearly, there is no increase in interceptor

sewer flow; therefore the entire inflow volume is spilled.

4. The cumulative cost for the decision q_1 to get to node proportion 0.1 times sewer capacity is the cumulative cost at the node travelled from, plus the link cost between the nodes. As the cumulative cost equals 0 then the cumulative cost to 0.1 times sewer capacity equals the link cost. Note that for this node there is only one route possible but at other nodes further down the network there will be other routes possible.
5. Clearly, the cumulative costs to all the nodes in decisions q_1 equal the link costs that can easily be obtained by the above method. These values are stored in the memory.
6. The method moves on to the next stage (q_2).
7. At any particular node within this stage there are various routes possible to arrive there. For example at interceptor flow 0.6 times sewer capacity for decision q_2 there are six possible routes possible. These six link costs can be determined by calculating the resultant pollutant load to the river. All the cumulative costs in the previous stage are known, therefore the cumulative costs of each of the six routes can be determined. The 'cheapest' (least cumulative pollutant load to the river) is stored in the memory, and the route which attained this cost is indexed. All other costs are discarded.
8. The solution process is continued throughout the system until the 'cheapest' cumulative cost at the final node is attained.
9. A traceback procedure determines the strategy used in attaining this 'cheapest' final cumulative cost.
10. The optimum control strategy is determined by calculating the required inflows necessary to generate the interceptor flow rates in the DP solution.

4.3 IDEALISED TEST CASE - RESULTS AND DISCUSSION

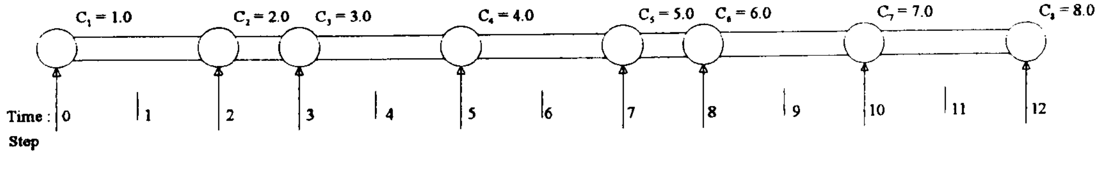


Figure 4.8: Idealised Interceptor Sewer System.

A simple fictitious interceptor sewer system, which is shown in Figure 4.8, has been used to test the viability of the slug flow approach and the optimisation techniques used in the optimal control models. The test system has eight intercept points, each with intercepted dry weather flows (DWF) of 0.1 cumecs. In this application, the DWFs were deducted from the pipe capacities to obtain the effective sewer capacities. The interceptor pipe capacity C_i increases by one cumec at each intercept point with a final capacity of eight cumecs. This is also the WwTW (at intercept point 8) treatment capacity. Each catchment is identical in layout and hydraulic design. One thousand possible proportions (settings) of the control gate at each intercept point have been considered for the DP method.

A fictitious rainstorm is assumed to hit all the catchments at the same time and the resulting runoff hydrograph and pollutant concentration factors are shown in Figure 4.9. The runoff hydrograph has several peaks to thoroughly confirm the validity of the capacity constraints within the control model. In fact, the full hydrograph is a duplication of the first three peaks of the runoff rates. The resultant control strategies should contain a large mix of interceptor states that will give a better comparison between control procedures. No attempt was made to synthesise runoff or pollutant concentrations but the concentrations roughly follow a first foul flush relationship. The model was run using the idealised interceptor system data and runoff hydrographs with pollutant concentration factors.

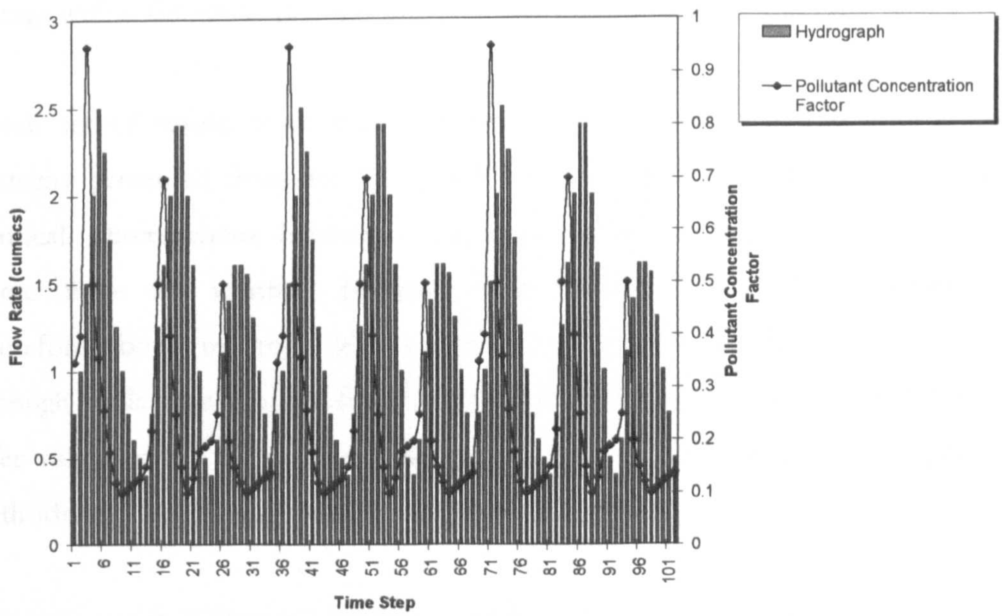


Figure 4.9: Runoff Hydrograph and Pollutant Concentration Factors.

Four different control procedures were considered:

Fixed Local Control: Intercepted flows are determined from local conditions at each independent intercept point. Flows to the interceptor are governed by the use of flow restrictors (e.g. vortex devices) which restrict the inflow to a pre-set maximum. In this system the maximum flow rate was set at one cumec at every intercept point. Clearly, the sum of these maximum flows must be less than or equal to the downstream interceptor capacity and treatment works. No account is made of the conditions in the interceptor system or conditions at other intercept points. The method is volumetric based and no account is taken of the pollutant load of the flows.

Variable Local Control: This method determines intercepted flows using information about the conditions locally in the interceptor system. That is, if there is spare capacity locally in the interceptor, intercept flows up to this capacity can be permitted. The method is volumetric based and no account is made of the quality of the flows.

Global Control (LP): This method uses global information, including pollutant concentrations, to determine optimum strategies using the LP model.

Global Control (DP): This method uses global information, including pollutant concentrations, to determine optimum strategies using the DP model.

All the control procedures use the slug flow approach within the interceptor sewer and only the decision criteria differ. Volumetric optimisation has not been included explicitly because the variable local control objective also fully utilises the available storage within the sewer (though spills will occur at different locations).

A full set of results from this application is shown in Appendix 3. The control strategies presented from the DP global control method and LP global are generally identical. Discrepancies between these global control strategies occur in time steps where there are identical pollutant concentrations at several intercept points. Therefore, both methods give optimal solutions within the model constraints although spills would occur from different intercept points. Significantly, the results offer validation of the computation of the control strategies from the optimisation methods.

A sample result is shown in Figure 4.10 that shows the control strategies from the various control procedures at all intercept points in time step 21. It shows the inflows, their respective pollutant concentration factors and the control strategies. The global control strategies in Figures 4.10 and 4.11 were obtained using the LP model. The local control strategies were obtained on volumetric criteria as defined above and no account was taken of the pollutant concentration factors.

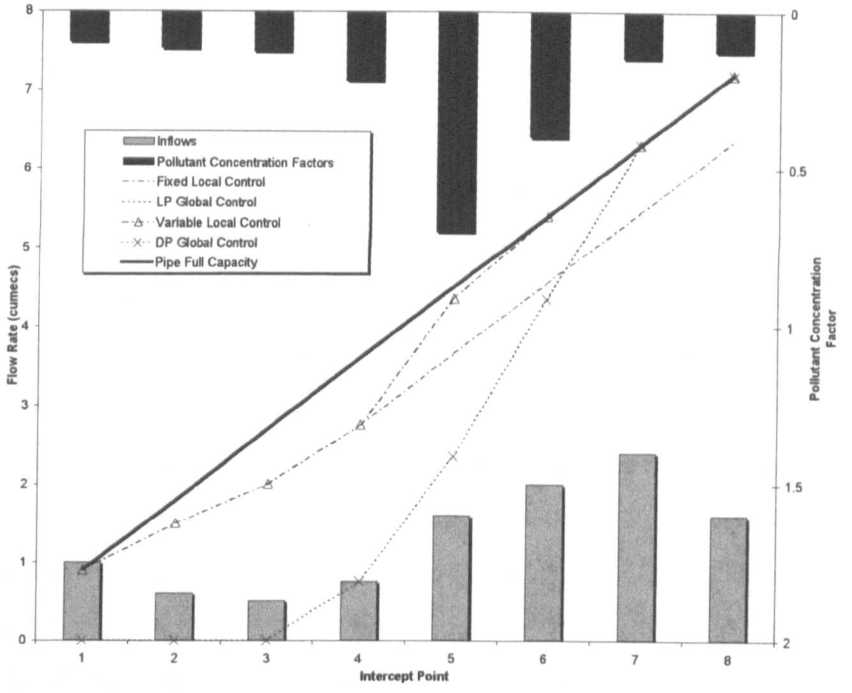


Figure 4.10: Sample Result showing Strategies for Chain of Water - Chain 21.

Figure 4.10 clearly illustrates the deficiencies, in terms of pollutant overspill load reduction, of using fixed local control. In this case, the inflows at certain upstream interception points were less than the pre-set maximum flow rate and the interceptor storage was not fully utilised. The control procedure could not make allowances for this and, consequently, there were unnecessary overflows at the downstream interception points. This is shown in Figure 4.10 where the fixed local control flow rate line is below the pipe full capacity line at all times. The environmental efficiency of the control actions was improved using the variable local control strategy since it uses information about the interceptor system state. Therefore, this strategy would always fully utilise the capacity available locally within the interceptor system. Again, this is shown in Figure 4.10 where the variable local control flow rate line is at the interceptor sewer capacity when possible. There is, however, an inherent danger of bias within the control decisions using this procedure. It may cause the downstream interceptor points to throttle back (i.e. closing of control gates) more readily than the upstream points. This is because the interceptor may already be at capacity because of decisions made upstream. Therefore, there may be operational or environmental difficulties with more frequent spills, or flooding, in the downstream sections of the sewer.

Figure 4.10 also illustrates the potential of the optimal pollution control models. There are two particularly polluted inflows at intercept positions 5 and 6. The global control strategies did not intercept the flows at points 1 and 2 (relatively 'clean' flow) in order to have sufficient capacity to intercept these 'dirty' flows. This is an important feature of the global control strategies that is common to all time step solutions in Appendix 4. The models control the sewer so that storage is made available (by restricting inflows upstream) allowing the interception of more polluted inflows downstream. However, the outflow from the interceptor sewer is always at capacity (if possible) despite the availability of storage upstream. The local control strategies did not make such allowances and as a consequence overflowed at intercept point 6. It was by pure chance that there was sufficient spare capacity within the system that the variable local control strategy did not spill at intercept point 5. In this time step the global control strategies showed a sixty-nine percent improvement over the fixed local control strategy, and a forty-four percent improvement over the variable local control strategy. This improvement was measured in terms of pollutant load reduction to the river.

Figure 4.11 shows comparisons of the solutions in each time step. They show that both the variable local control strategy and the global control strategies give significant improvements over the fixed local control strategy. At worst the global control strategy spills the same amount of pollutant load as the variable local control strategy. Minor deviations between the DP and LP control procedures are explained by rounding errors inherent in the DP solution. There are one thousand proportions of sewer capacity in the solution and the extreme downstream sewer capacity is eight cumecs. Therefore, the resolution is to 0.008 cumecs since there are one thousand sewer proportions of 0.008 cumecs. This means that in certain circumstances the global control strategy will appear to spill approximately 0.01 cumecs more than the other control strategies. Obviously, the discrepancy would be less significant if a finer setting for the discretisation was adopted.

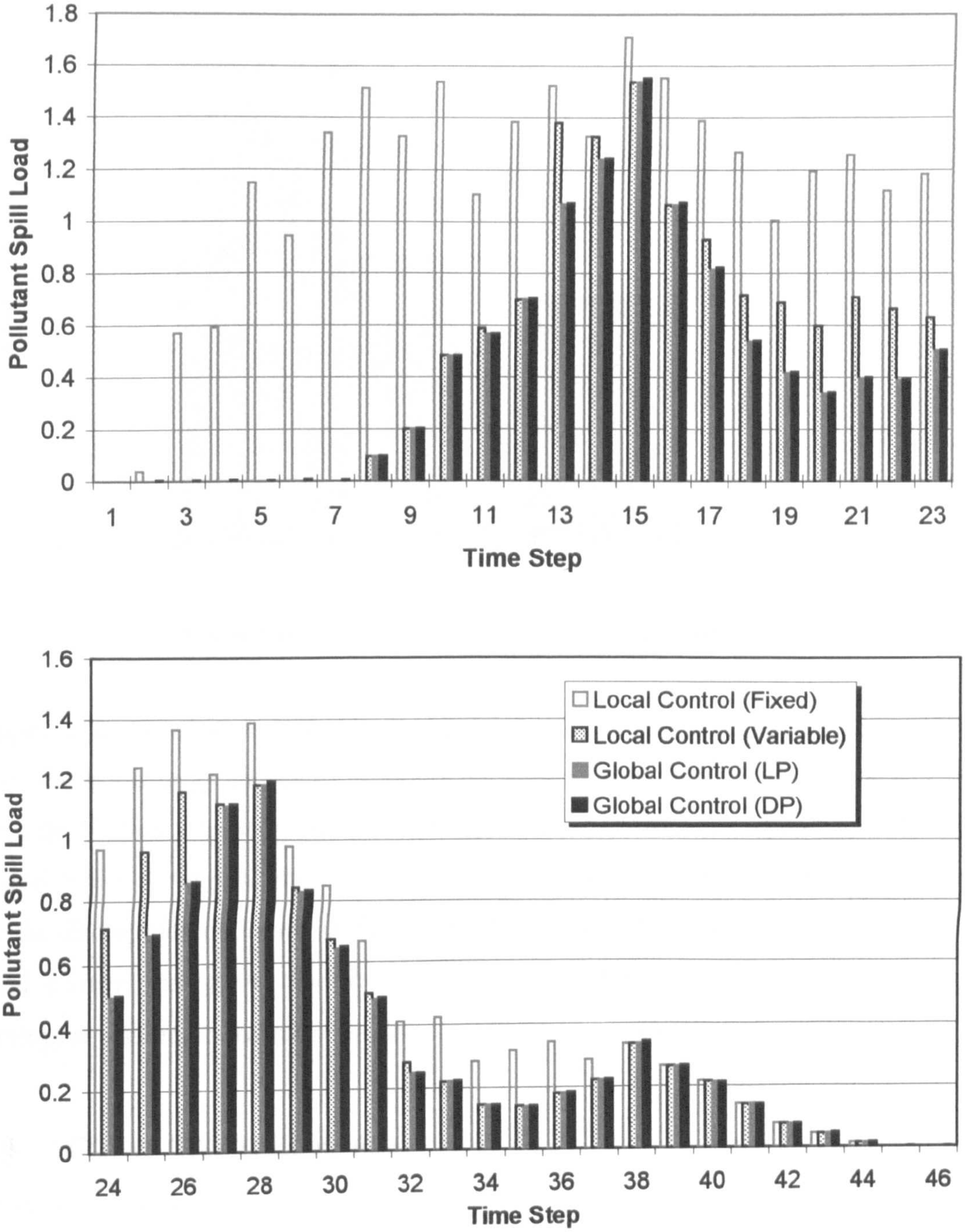


Figure 4.11: Comparison of Local and Global Control Strategies within the Control Horizon.

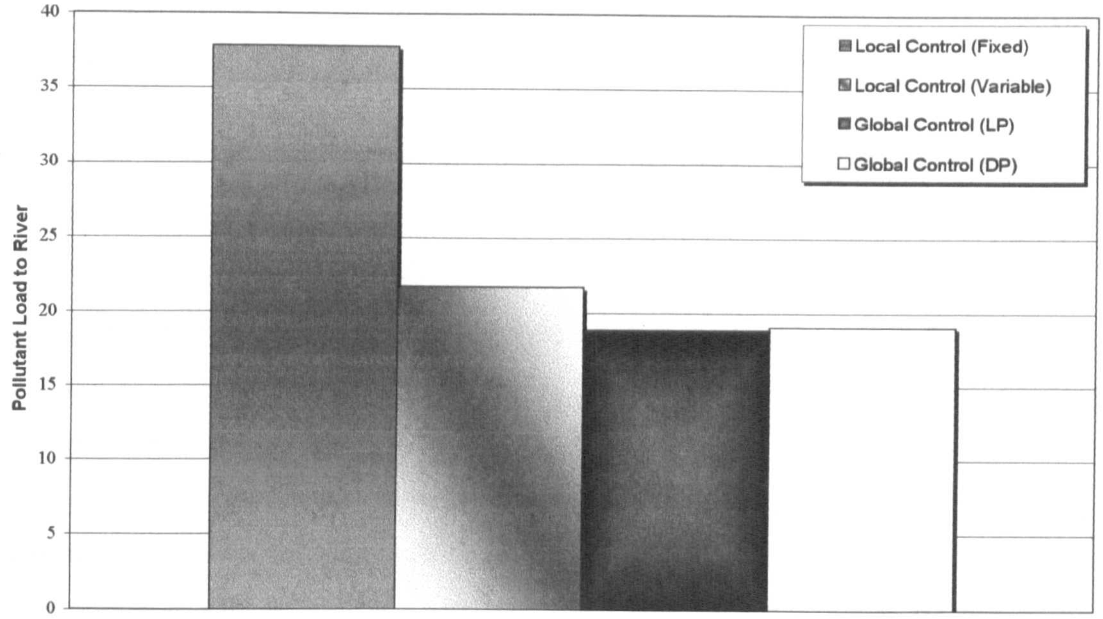


Figure 4.12: Overall Comparison between Control Strategies.

Figure 4.12 shows an overall comparison of the four control strategies. The variable local control strategy provides a considerable improvement over the fixed local control strategy. Further improvements are achieved with the global control strategies. The LP model offers a marginal improvement over the DP model, however, this is explained by the discrepancies inherent in the DP model. The good performance of the variable local control strategy in this test case example is somewhat fortuitous since this strategy is volumetric and takes no account of pollutant load.

4.4 CONCLUDING REMARKS

The results from the idealised test case clearly illustrate the potential of optimal control models in interceptor sewer operation. The results have shown that there are significant deficiencies in using fixed local control resulting in needless overspills when there is spare storage within the interceptor sewer. Local control strategies that use information about the interceptor system state (variable local control) can significantly improve the performance of these systems (in terms of pollutant load spills reduction). In fact, little effort would be required to implement such a change in operation since the method does not use mathematical optimisation models. It must be stressed that the pollutant load overspill improvements from this method presented in this chapter are fortuitous. However, there would be worthwhile benefits of using this method because it would always fully utilise the sewer's storage.

The results show that a further improvement can be achieved with the use of optimal control models using global information. The method always determines optimal control decisions and, depending on the inflows and pollutant concentrations, the control strategies prove never to be worse (in terms of pollutant overspill loads) than the fixed or variable local control strategies. Additionally, the global control models make more efficient use of the available storage within the interceptor system. There is an inherent danger of downstream interception points spilling, and possibly flooding, more frequently with variable local control strategies. This is because the method fills the sewer as quickly as possible and, therefore, the sewer is likely to be full at the downstream sections more readily. This would not normally be the case with volumetric optimisation which sometimes uses coefficients within the objective function to control the spill locations.

The idealised test case application illustrates the viability of the slug flow approach and the optimisation models. The slug flow approach proved a theoretically sound formulation for the simulation of sewage advection through the interceptor sewer. The validity of this approach in terms of hydraulic dynamics is tested in Chapter 5. Both optimisation models generally give identical results offering an independent validation of the computations of the control strategies. Therefore, both models offer potential for further development although the LP model runs considerably faster and yields exact solutions. However, the DP model may become more efficient as the complexity of the systems increase. In LP the computational time increases considerably as the number of variables increase. The computational time for DP solutions increases at a slower rate because more variables (intercept points) merely add stages to the method. At some point the relative computational demands for each model may reverse.

CHAPTER 5

VERIFICATION OF MODELS

5.1 INTRODUCTION

The viability of the developed optimal pollution control models has been demonstrated in the previous chapter, illustrating that significant reductions in overspill pollution load can be achieved when compared with all other control procedures. However, the models were developed using a hydraulically crude slug flow approach, as described in Section 4.2, in order to allow for a computationally efficient solution of the control actions. This chapter tests the validity of the slug flow approach using a post-processing hydraulic verification routine, which is validated against the WALLRUS (HRS, 1991) sewer flow simulation software.

Initially, the shortcomings of the slug flow approach are discussed with possible improvements and the complexities of their implementation. A description of the post-processing hydraulic verification routine and the WALLRUS software is given. Finally, the optimal pollution control model and hydraulic verification routine are tested on an interceptor sewer system with various storm profiles to analyse the sensitivity of the control model and to verify that the control actions generated from the model give physically feasible results.

5.2 INTERCEPTOR SEWER FLOW DYNAMICS

The optimal pollution control models were developed using a slug flow approach, which can be seen as a graphical illustration in Figure 5.1. The illustration is of course physically impossible and the slugs will all interact and the discontinuities in the surface will merge by, for example, formation of surge waves or backwater effects. In reality, when fluid enters a sewer a wave is produced, travelling upstream or downstream depending on the water level, until steady state flow conditions are obtained. This will have implications on the sewer flow velocities on which the slug flow approach is based. Generally, the flow velocity will increase with flow. Therefore, even if the slugs do not interact they would travel through the sewer at different velocities because of their varying depths.

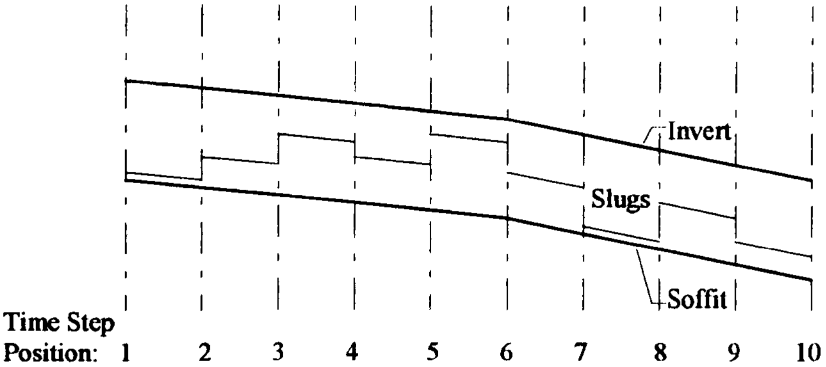


Figure 5.1: Graphical Illustration of the Slug Flow Approach.

There are therefore two major assumptions in the formulation of the optimal control models:

- the slugs of fluid advect through the sewer system at a constant velocity – the pipe full velocity. Historically, this velocity has been used by engineers for the design of sewer systems with little adverse effects. For example, the Rational Method is based on a time of concentration, which is calculated using the pipe full velocities. However, from a control perspective it is imperative that the control model has an accurate understanding of the advection of flows in the sewer system. For example, in the optimal pollution control model storage may be made available by restricting 'cleaner' inflows upstream earlier. Adverse effects may be encountered (e.g. surcharging) if the slug velocities deviate from the assumed pipe full velocity; and
- the slugs do not interact with each other as they advect through the interceptor sewer system. This allows for the computationally efficient calculation of the control strategies but it is hydraulically impossible.

Ideally, the flow velocity assumption would be thoroughly tested by calculating the actual flow velocities after the control strategies have been determined. These velocities could then be used in a second iteration of the optimal control model to determine new control strategies. This iterative procedure could be continued until the solutions do not change. However, the complexities of this approach, which are described below, imply that it is beyond the scope of this thesis and is now included as a recommendation for further work.

It is computationally simple to calculate the actual steady-state flow velocities after the control strategies have been determined, by, for example, the Manning equation (2.4). It is then simple to implement an iteration process where the optimal control model determines new control strategies with the actual velocities. However, this then generates complexities in the slug flow approach, where the slugs will now interact. This is shown in Figure 5.2a.

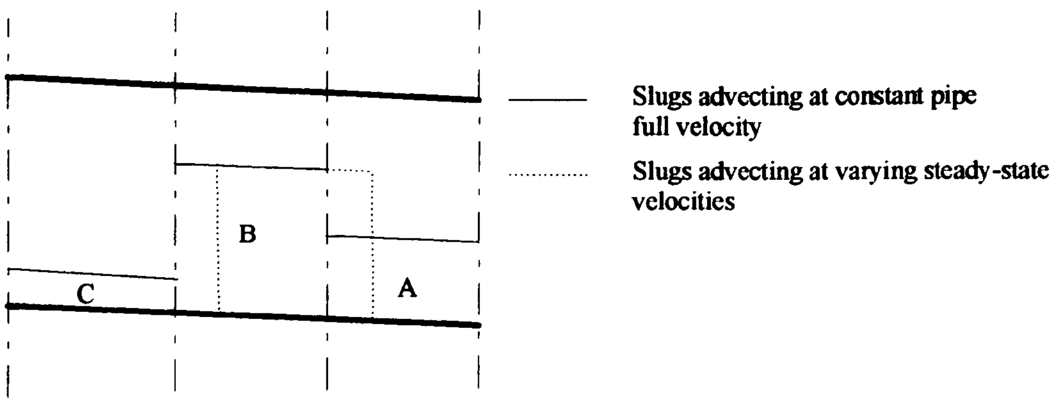


Figure 5.2a: Illustration of the Complexity of Implementing Varying Slug Flow Velocity.

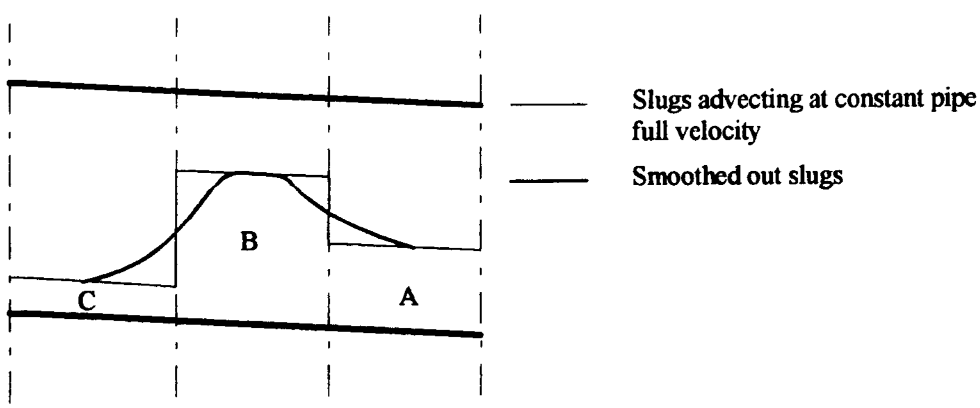


Figure 5.2b: Illustration of the Smoothing Effects from Surge Waves.

Figure 5.2a shows the implications of a slug advecting at a faster velocity (slug B) than the slug ahead of itself (slug A) which was determined from the previous time step solution in the optimal control model (see Figure 4.5). Theoretically, the mean velocities of the slugs follow the proportional pipeflow relationships, where a slug of a greater depth will have a higher mean velocity. Therefore, B moves faster than A so there is an overlap area at the head of B/tail of A, as shown by the dotted lines in Figure 5.2a. It is unclear what actually happens here, but there is probably a surge wave towards A. B also moves faster than C so at the tail of B/head of C there is apparently a gap of missing water. Again, it is unclear what actually happens but there is probably a surge wave towards C as tail B water surges back to fill the gap. Either way, the surge effects will tend to reduce the depth of the faster-moving slug B and so reduce its mean velocity.

The interactions in Figure 5.2a are simply smoothing effects on the discontinuities between A/B and B/C as water from B surges backwards and forwards to smooth out the discontinuities. This effect is shown in Figure 5.2b. This again tends to lower the peak depth of the faster moving slug B and so reduce its mean velocity.

Within the scope of the present study, it is extremely difficult to formulate the model to allow interactions between the slugs since the control decisions were already determined for the slug downstream. The control model would need to be formulated to allow interaction in the vertical direction in Figure 4.5. Potential solutions to this problem are discussed in the Recommendations for Further Work section.

It must be stressed that it is very unlikely that the slugs would form such a discontinuous profile. Generally, the inflow hydrographs and pollutographs from the sub-catchments would not contain any rapid changes and would have smooth build-ups to peak values. Of course, the control strategies may generate rapid changes in flow rates because of the spatial distribution of rainfall amongst other factors. Also, it is envisaged that the discontinuities in the surface of the slugs might tend to cancel themselves out, i.e. the likely forward dispersion of a frontal 'wave' potentially being cancelled by the upstream dispersion (backwater) of the rear of the preceding slugs.

After describing the limitations of the assumptions in the slug flow approach, the rest of this chapter describes the verification of the slug flow approach by introducing a post-processing hydraulic verification routine into the models, which is verified using the WALLRUS sewer flow simulation package (HRS, 1991). A brief description of the development of the hydraulic verification routine is given in Thomas *et al.* (1999a).

5.3 HYDRAULIC VERIFICATION

The post-processing hydraulic verification routine determines the approximate water profiles within the interceptor sewer in each time step solution. The procedure is shown in Figure 5.3 and may be interpreted as 'snapshot' water profiles from each time step throughout the control time horizon.

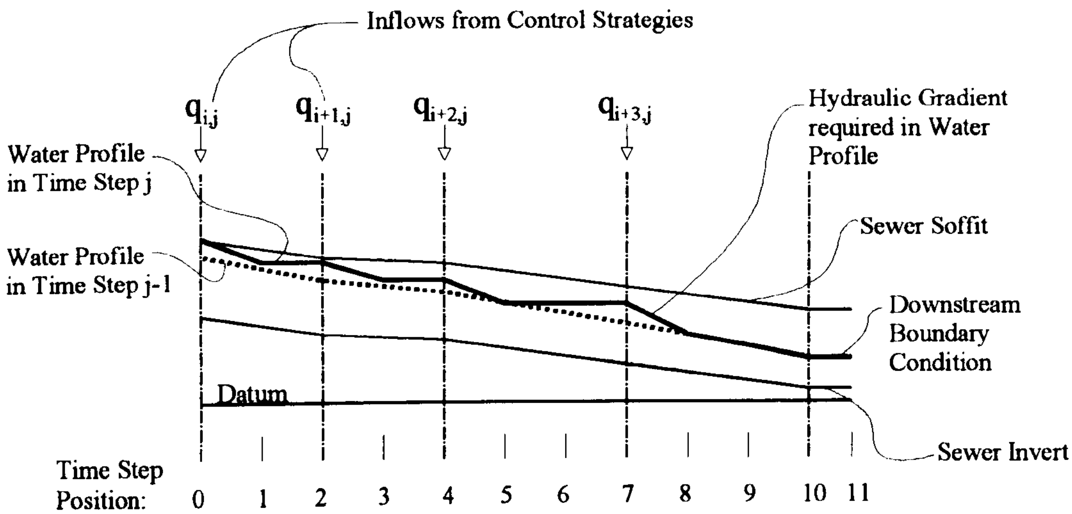


Figure 5.3: Schematic of Water Profile Approximation.

The water profiles are determined by a quasi-steady approach using the Manning equation (2.4). It is assumed that inflow $q_{i,j}$ will have reached one time step position downstream in the interceptor at the end of the time step. Therefore, the position (based on pipe full velocity) and size (from the optimisation module) of the 'slugs' are known. The Manning equation is then used to determine the hydraulic radius required to advect the flow through the reach. Any hydraulic inconsistencies and positions of surcharging are then illustrated.

The procedure commences at the downstream boundary point and determines the water level at the next time step position upstream so that there is sufficient hydraulic radius for that flow rate. The routine then determines the water level for the next time step position and so on. The critical depth is calculated at positions where pipe invert is discontinuous such as where the diameter of the sewer alters. If the water surface level from the Manning equation (2.4) is lower than this depth, then the critical depth is used in the subsequent calculation of the upstream water profile. Transitions in water profile can therefore be determined. The verification routine continues this procedure for all the control strategies throughout the control time horizon.

5.3.1 Derivation of Post-Processing Hydraulic Verification Equations

As mentioned above, the post-processing hydraulic equation uses the Manning equation (2.4) to determine the hydraulic gradients of each slug in the interceptor in each time step during the storm event. In fact, the Manning equation is slightly modified from that presented previously (2.4) to compute the water depths, y, from the slug flow rate, q:

$$\underbrace{q}_{\text{known}} = \underbrace{\frac{1\,A\,R^{2/3}\,S^{1/2}}{n}}_{\text{all functions of } y} \tag{5.1}$$

where A is the cross-sectional area of flow [m^2]; R is the hydraulic gradient [m]; S is the surface slope [-]; and n is the Manning roughness coefficient [m$^{1/3}$/s].

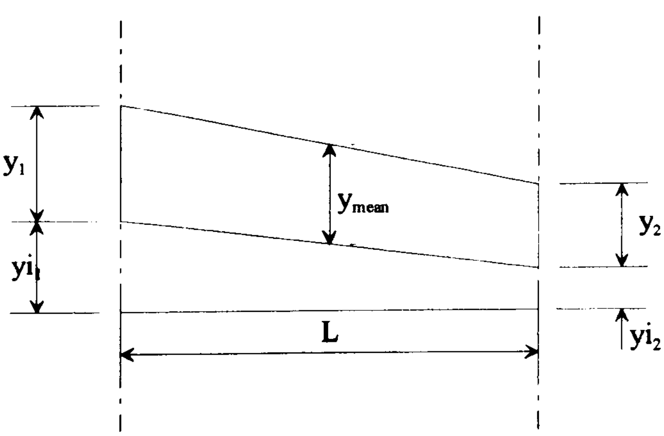


Figure 5.4: Typical Slug of Fluid.

Consider the motion of a typical slug of fluid as shown in Figure 5.4. The mean water level, y_{mean} is assumed to be the water depth required in (5.1) to generate the slug flow rate q. Thus:

$$y_{mean} = \frac{y_1 + y_2}{2} \tag{5.2}$$

and

$$S = \frac{(y_1 + yi_1) - (y_2 + yi_2)}{L} \tag{5.3}$$

where S is the surface slope of the slug [-]; and L is the length of the slug of fluid [m].

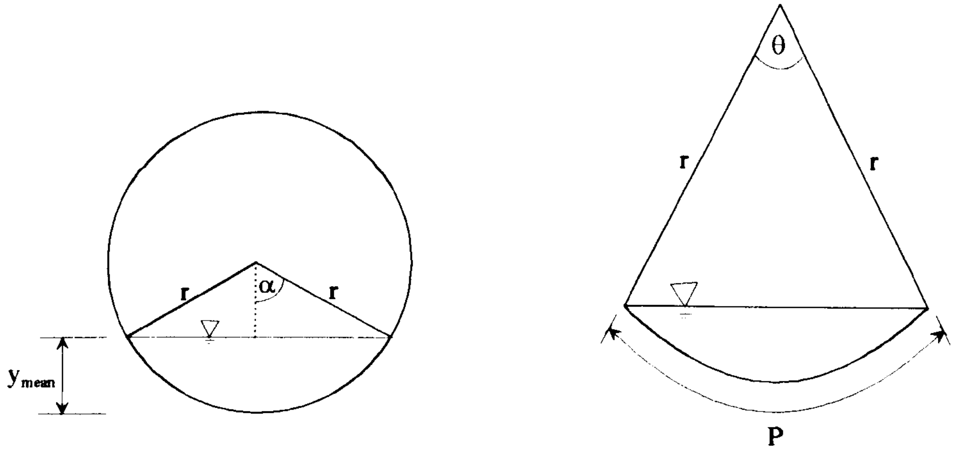


Figure 5.5: Basis for the Calculation of the Area and Wetted Perimeter of Flow in a Circular Pipe.

For a circular sewer pipe as shown in Figure 5.5, the derivation of the equations for the cross-sectional area of flow (A) and wetted perimeter (P) as a function of y_{mean} are as follows:

$$\alpha = \cos^{-1}\left(\frac{r - y_{mean}}{r}\right) \tag{5.4}$$

Thus,

$$2\alpha = 2\cos^{-1}\left(\frac{r - y_{mean}}{r}\right) \tag{5.5}$$

Also,

$$A = \frac{r^2}{2}(\theta - \sin\theta)$$

and

$$P = \theta r \tag{5.6}$$

where A is the cross-sectional area of flow [m^2]; P is the wetted perimeter [m]; and θ = 2α.

Therefore, $$A = \frac{r^2}{2}\left\{2\cos^{-1}\left(\frac{r - y_{mean}}{r}\right) - \sin\left[2\cos^{-1}\left(\frac{r - y_{mean}}{r}\right)\right]\right\} \tag{5.7}$$

and $$P = 2r\cos^{-1}\left(\frac{r - y_{mean}}{r}\right) \tag{5.8}$$

The hydraulic radius $R = A / P$ so:

$$R = \frac{r\left\{2\cos^{-1}\left(\frac{r - y_{mean}}{r}\right) - \sin\left[2\cos^{-1}\left(\frac{r - y_{mean}}{r}\right)\right]\right\}}{\cos^{-1}\left(\frac{r - y_{mean}}{r}\right)} \tag{5.9}$$

The complete Manning equation when substituting in (5.1) is therefore:

$$q = \frac{r^2}{2n}\left\{2\cos^{-1}\left[\frac{r - \left(\frac{y_1 + y_2}{2}\right)}{r}\right] - sin\left(2\cos^{-1}\left[\frac{r - \left(\frac{y_1 + y_2}{2}\right)}{r}\right]\right)\right\}$$
$$\times\left\{\frac{2r\cos^{-1}\left(\frac{r - \left[\frac{y_1 + y_2}{2}\right]}{r}\right) - r\sin\left(2\cos^{-1}\left(\frac{r - \left[\frac{y_1 + y_2}{2}\right]}{r}\right)\right)}{\cos^{-1}\left[\frac{r - \left(\frac{y_1 + y_2}{2}\right)}{r}\right]}\right\}^{2/3}$$
$$\times\left\{\frac{(y_1 + yi_1) - (y_2 + yi_2)}{L}\right\}^{1/2} \tag{5.10}$$

Since the hydraulic verification routine commences at the downstream condition, where y_2 is known, the only unknown within (5.10) is y_1, which is the upstream water level of the slug flow. This equation (5.10) must be solved implicitly for y_t and this is not computationally efficient. However, the equation (5.10) is only used after the control strategies are determined and has no implication on the efficiency of the control model computations.

5.4 WALLRUS VERIFICATION

WALLRUS (HRS, 1991) is a microcomputer model for the simulation of sewer flows in dendritic systems developed by Wallingford Software, U.K. The package consists of several programs including design methods, a simulation method and ancillary programs. The main programs are:

- *Modified Rational Method* – This is the simplest method in the package and is used to design pipes or channel sizes and gradients using a modified version of the Rational Method. Gradients are designed to give adequate self-cleansing velocities and the sizes are designed to take the peak flows using the rational method. Overflow structures and detention storage can also be included.

- *Hydrograph Design Method* – This method is a hydrograph routing method that designs the pipes to take the peak flow. The method can be used to size pipes or channels for observed or synthetic rainfall events in a network with defined layout and levels. Overflows, storage tanks and pumping stations can be included.

- *Simulation Method* – This method is used to simulate flow in an existing sewer system for given rainfall and catchment conditions. Surcharging and surface flooding are included. This method is used to verify the water profiles from the post-processing hydraulic verification routine.

A thorough description of the methods used in the Wallingford Procedure is given in D.O.E. (1981) and is not described further here. However, it should be noted that WALLRUS does not solve the full dynamic wave equations (2.10). Instead, the Muskingham method, a storage routing model, proposed by Cunge (1969) is used. Therefore, WALLRUS is not ideally suited for this application because of the rapid changes in sewer flow generated from the control actions but was deemed adequate given the level of resources available.

The implementation of the WALLRUS suite for the verification of the water profiles was actually rather difficult. WALLRUS was coded under the obsolescent DOS environment requiring the input data to be in special formats. Moreover, WALLRUS was designed to simulate rainfall-runoff processes and it is therefore not fully equipped to simulate rapid changes in sewer flows generated from control actions. In the implementation, WALLRUS was applied innovatively where the control strategies were input as inflow hydrographs and the hyetographs were considered to be zero.

5.5 MODEL SENSITIVITIES TO STORM PROFILES

This section presents the results of the optimal pollution control model sensitivities to storm profiles using the interceptor sewer system shown in Figure 5.6. The corresponding input data for the models is shown in Table 5.1.

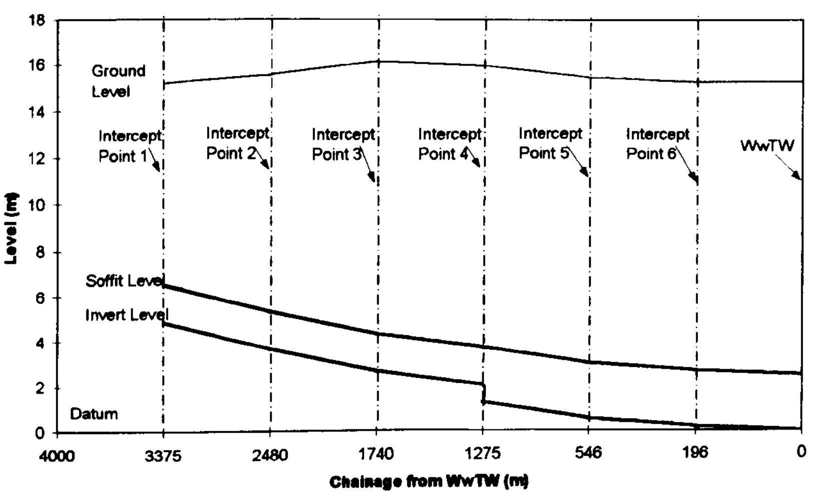


Figure 5.6: Longitudinal Section of the Test Case Interceptor Sewer System (Not to Scale).

Intercept Point	*Sewer Diameter (m)*	*Sewer Gradient*	*Sewer Capacity (cumecs)*	*D.W.F. (cumecs)*	*Fixed Inflow Setting (cumecs)*
1	1.66	1/750	3.26	0.30	1.24
2	1.66	1/750	3.26	0.09	0.25
3	1.66	1/750	3.26	0.04	0.97
4	2.44	1/1000	7.72	0.64	2.82
5	2.44	1/1000	7.72	0.02	0.29
6	2.44	1/1000	7.72	0.09	0.31

Table 5.1: Input Data for Test Interceptor Sewer.

Intuition suggests that storms of increasing severity generate increasingly rapid control actions with higher inflow rates. Therefore, it is hypothesised that there is a greater risk of surcharging with severe storms with a reduction in the validity of the slug flow approach. Additionally, the assumption that the slugs of fluid advect only in the downstream direction may not be able to suitably represent the hydraulics in storm conditions where there are localised peak inflows, especially in the downstream sections. In this case, there would be considerable storage available upstream of the control points, which may not be utilised in the optimal control model. Therefore, a range of storm inputs are used to fully confirm the accuracy of the slug flow approach in the optimal pollution control models.

The runoff hydrographs consisted of six hypothetical storm events, three of varying intensity: a low storm event (~1.5-2 times 'Formula A' setting), a medium storm event (~3-4 times 'Formula A' setting), and a high storm event (~7-10 times 'Formula A' setting). The three other storm events were localised peak inflows, one to the upstream intercept point, one to the middle intercept point and one to the downstream point. In each of the test cases the dry weather flows from each sub-catchment were added to the runoff hydrographs to obtain the total combined sewer flow. For convenience, the inflow hydrographs for the low intensity storm event were used as a base case and the medium and high intensity storm event inflow hydrographs were merely increasingly severe events based on the base case. Also, the pollutant concentrations of the inflows from each contributing catchment were identical and synchronised but the characteristics of the test interceptor sewer produce a temporal distribution in the control strategies. The sensitivities of the control model to the spatial distribution of pollutant concentrations and inflow hydrographs is presented in Chapter 7.

5.5.1 Storm Profile One – Low Intensity and Highly Synchronised

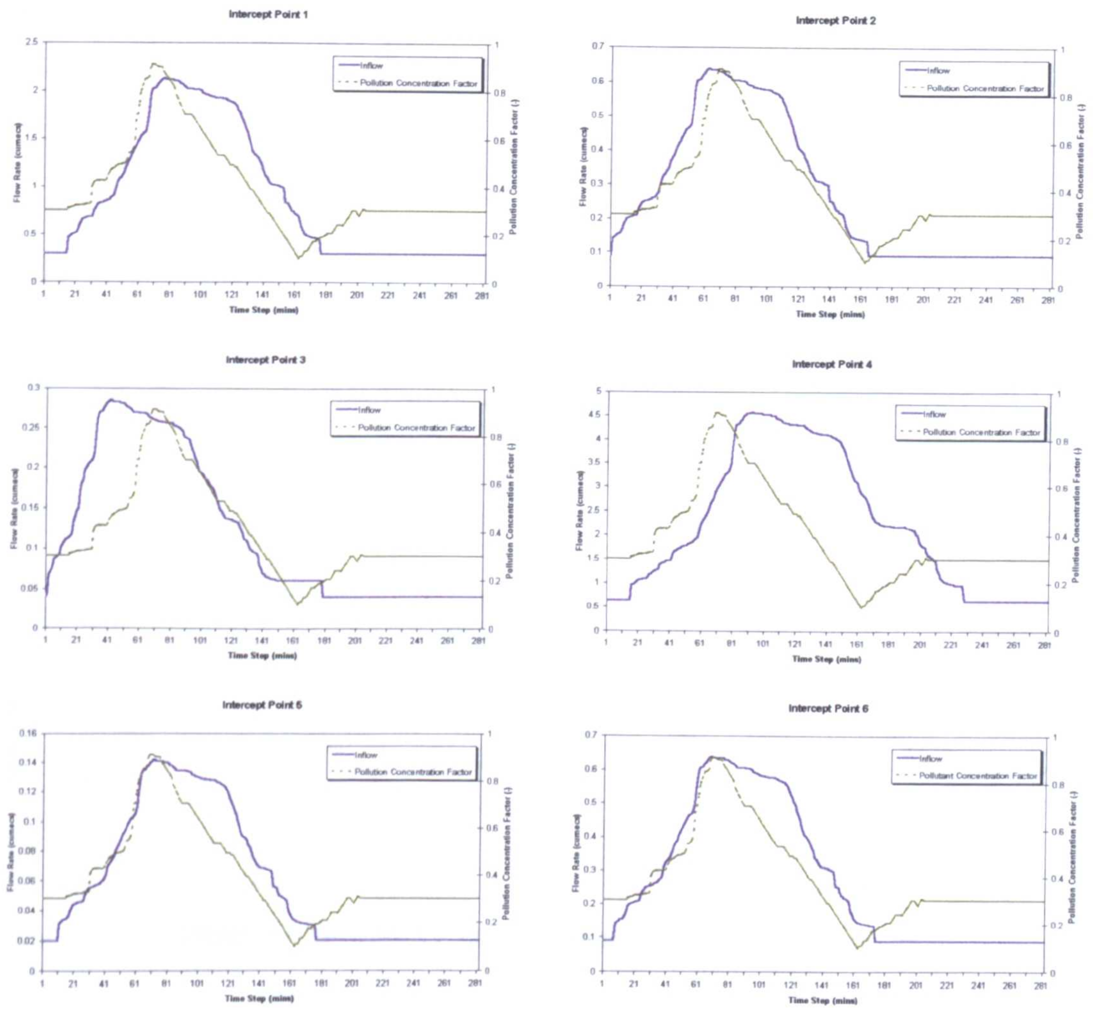


Figure 5.7: Inflow Hydrographs and Pollutographs for Low Intensity Storm Event (Base Case).

The inflows in Figure 5.7 were input into the optimal pollution control model using the interceptor sewer in Figure 5.6 as the test case. The control strategies from the model generated the interceptor sewer hydrographs in Figure 5.8, which show a comparison between the slug flow and WALLRUS hydrographs in each interceptor sewer leg.

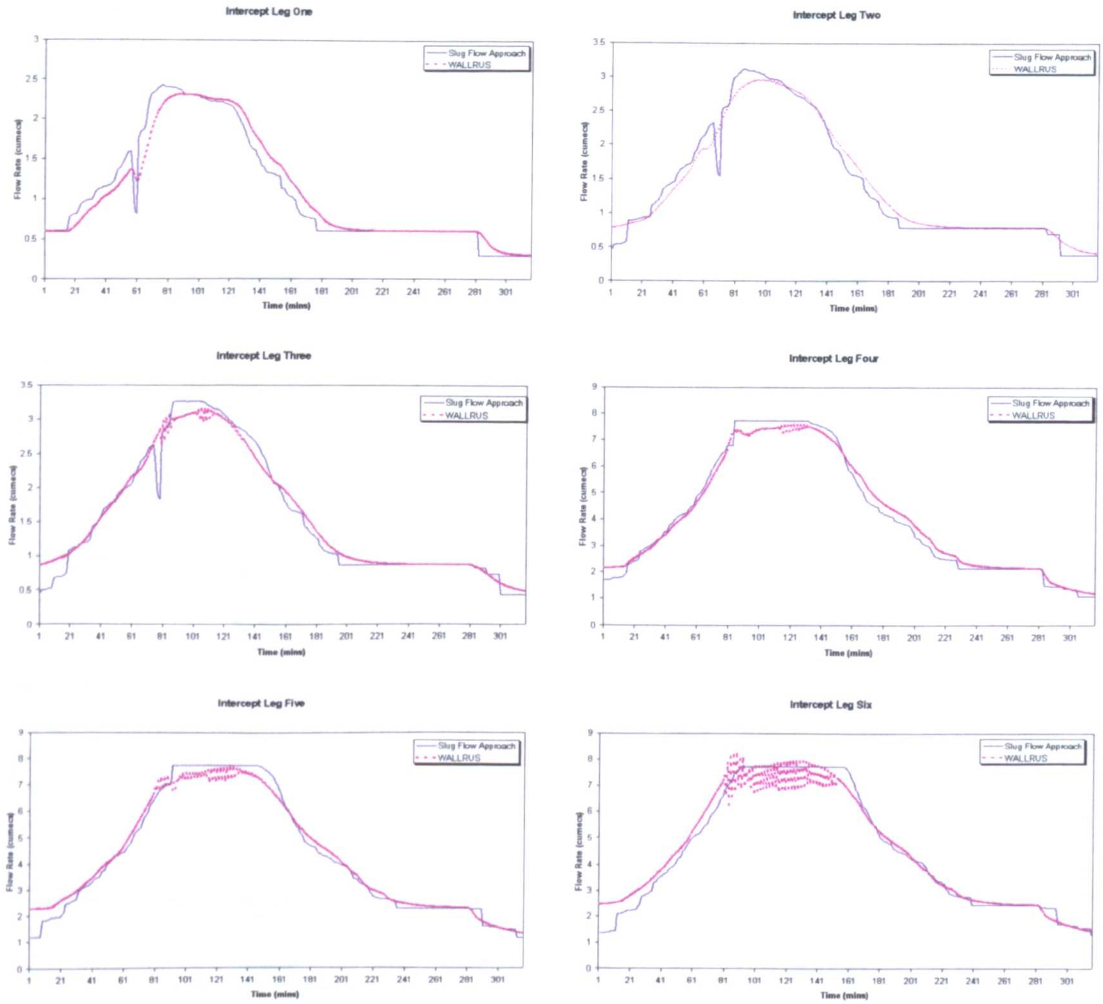


Figure 5.8: Interceptor Sewer Hydrographs for the Low Storm Event.

The hydrographs in Figure 5.8 show that there were very few control actions during the low storm event as expected. The discontinuities can clearly be seen in the slug flow hydrographs where there are instantaneous changes in flow rates. The WALLRUS hydrographs do not show these rapid changes but compare well. Overall, the sewer hydrographs from the slug flow approach and WALLRUS correlate well illustrating the validity of the slug flow approach in the optimal control model. The WALLRUS hydrograph also shows some instability, particularly when the sewer pipe is close to full, and this probably occurs because of the numerical methods used in the WALLRUS code. This instability is common to many of the results from WALLRUS but its significance is considered negligible because the flow volumes within the instability are minimal, the solution recovers and the overall results compare well.

The control strategies were also input into the post-processing hydraulic verification routine to develop approximate sewer water profiles, which were validated against WALLRUS. Some sample water profiles are shown in Figure 5.9, which are taken from various time steps during the storm event.

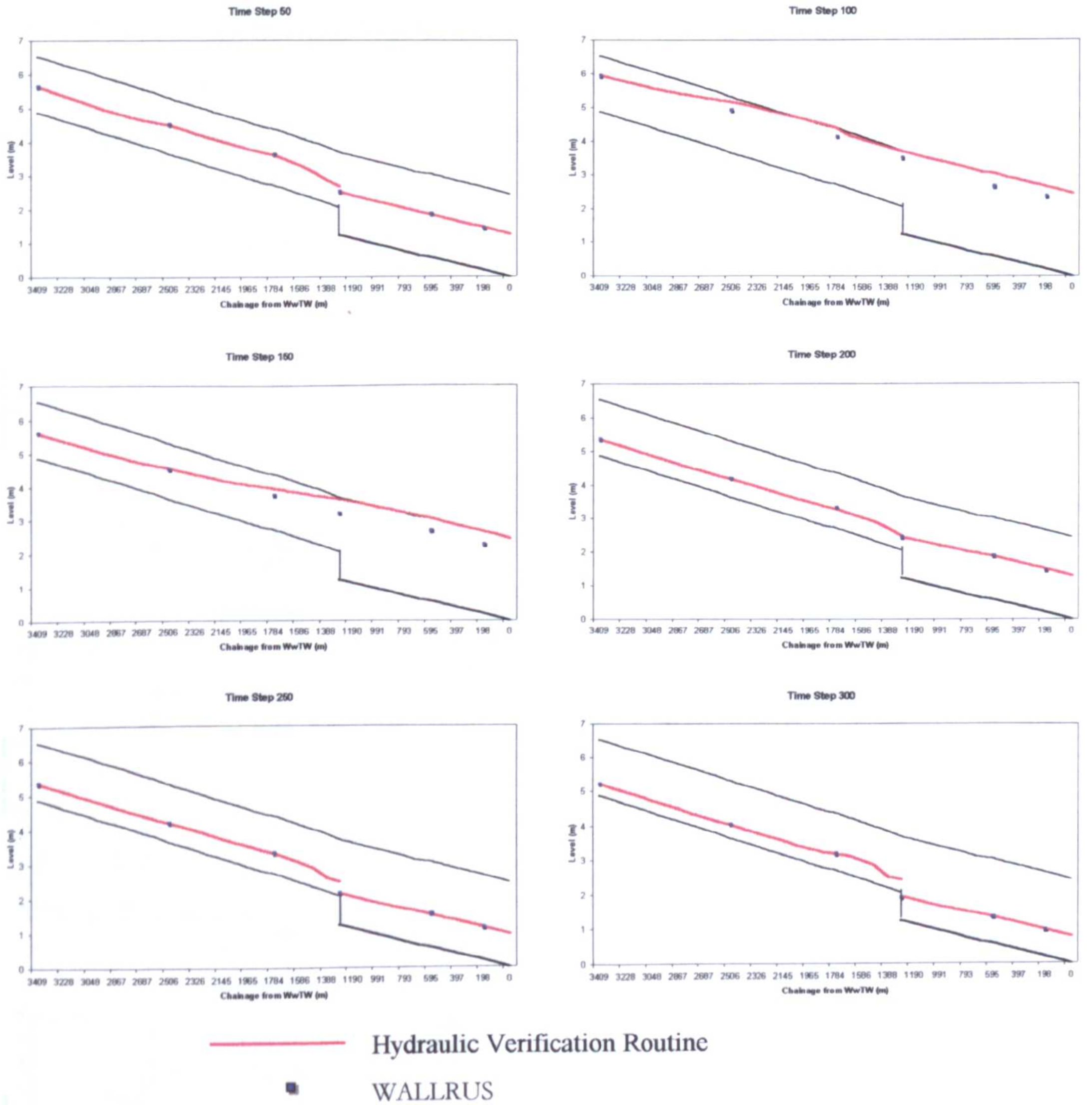


Figure 5.9: Sample Water Profiles from the Low Storm Event.

The water profiles determined by the hydraulic verification routine in Figure 5.9 show that the control strategies from the optimal control model were generally conservative. These profiles illustrate the control model's idealisation of the interceptor sewer system state, i.e. slugs of flow travel through the interceptor sewer at the pipe full velocity irrespective of the water depth and surface slope. The points in Figure 5.9 show depths of flow, calculated by WALLRUS, in each section of the interceptor and provide validation of the hydraulic verification routine's water profiles. There are some discrepancies in the time step 150 but the water profile is conservative in this situation.

5.5.2 Storm Profile Two – Medium Intensity and Highly Synchronised

The low storm inflow hydrographs were multiplied by two to obtain a medium intensity storm and the optimal control model control strategies generated the sewer hydrographs in Figure 5.10 for the same interceptor sewer system.

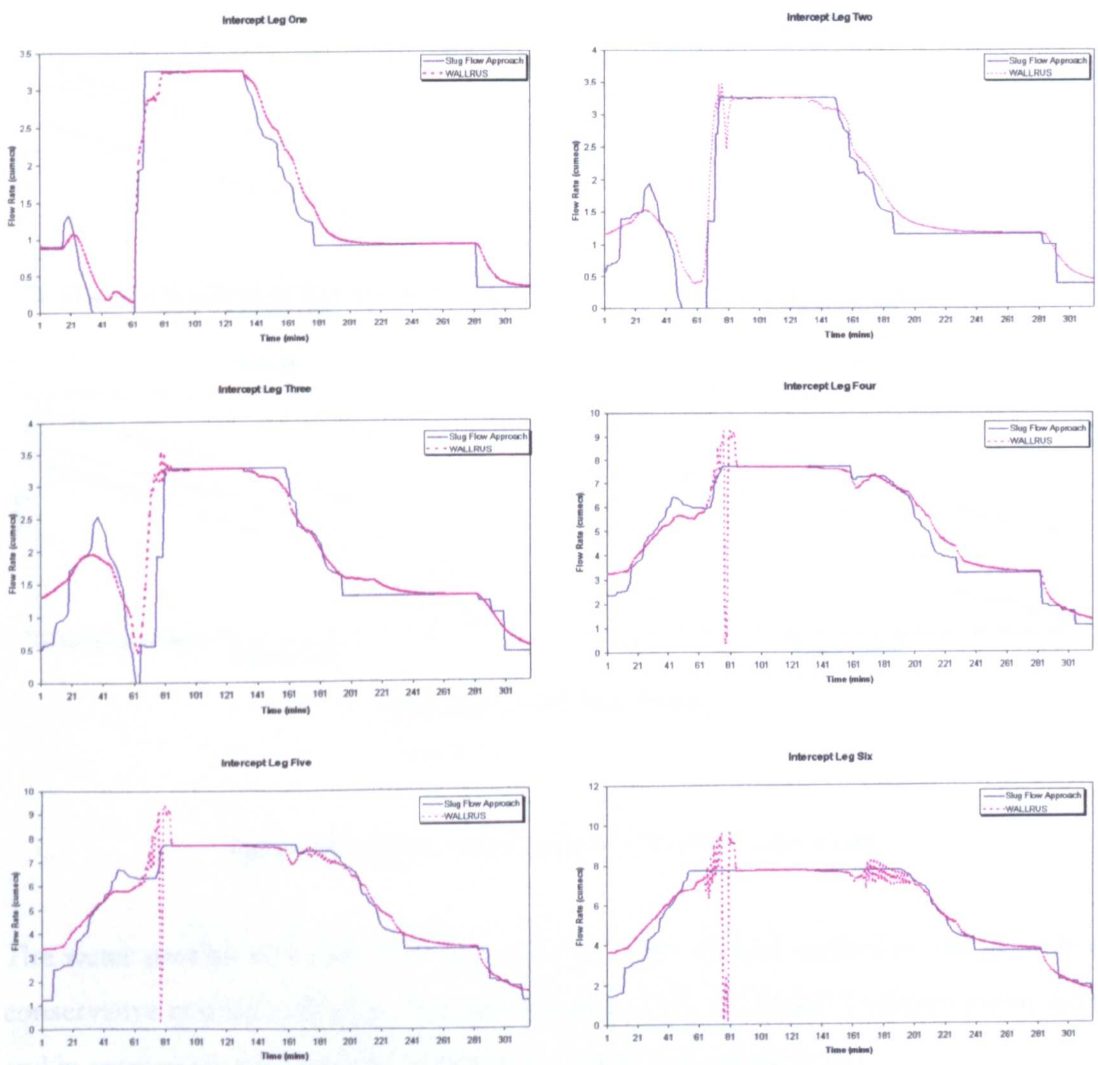


Figure 5.10: Interceptor Sewer Hydrographs for the Medium Storm Event.

It can clearly be seen in Figure 5.10 that there are considerably more control actions for this storm event. The hydrographs from the slug flow approach and WALLRUS compare well although there is more instability in the WALLRUS outputs in this were input into the hydraulic verification routine to obtain the water profiles. Sample results can be seen in Figure 5.11.

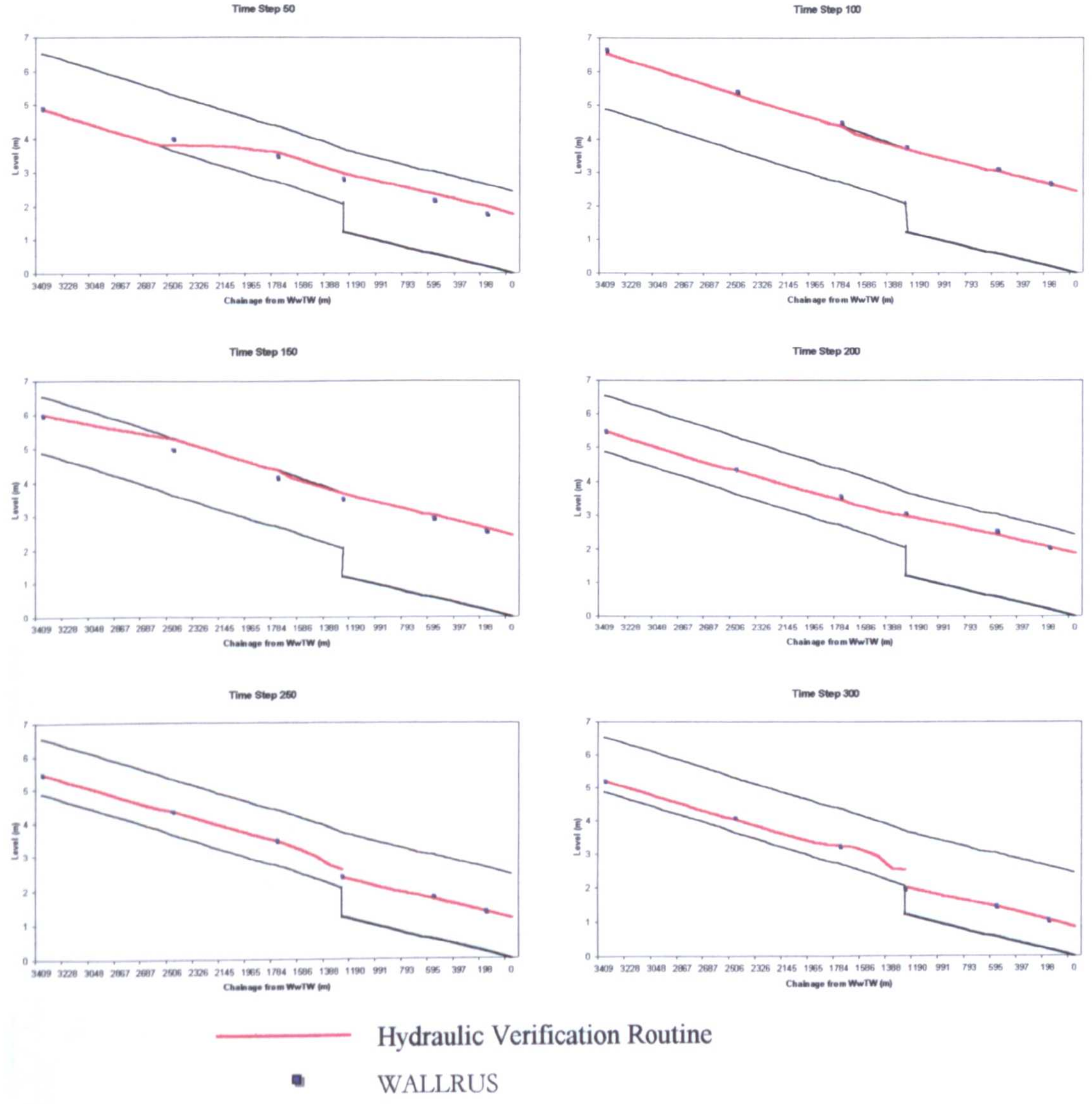


Figure 5.11: Sample Water Profiles for Medium Storm Event.

The water profiles in Figure 5.11 also show that the optimal control model generated conservative control strategies. The water depths from WALLRUS validate the profiles and in most cases the hydraulic verification routine was conservative.

5.5.3 Storm Profile Three – High Intensity and Highly Synchonised

The low storm inflow hydrographs were multiplied by a factor of five to obtain the high intensity storm profiles. These were input into the optimal pollution control

The low storm inflow hydrographs were multiplied by a factor of five to obtain the high intensity storm profiles. These were input into the optimal pollution control model for the interceptor sewer system in Figure 5.6 and the sewer hydrographs in Figure 5.12 were generated from the control strategies.

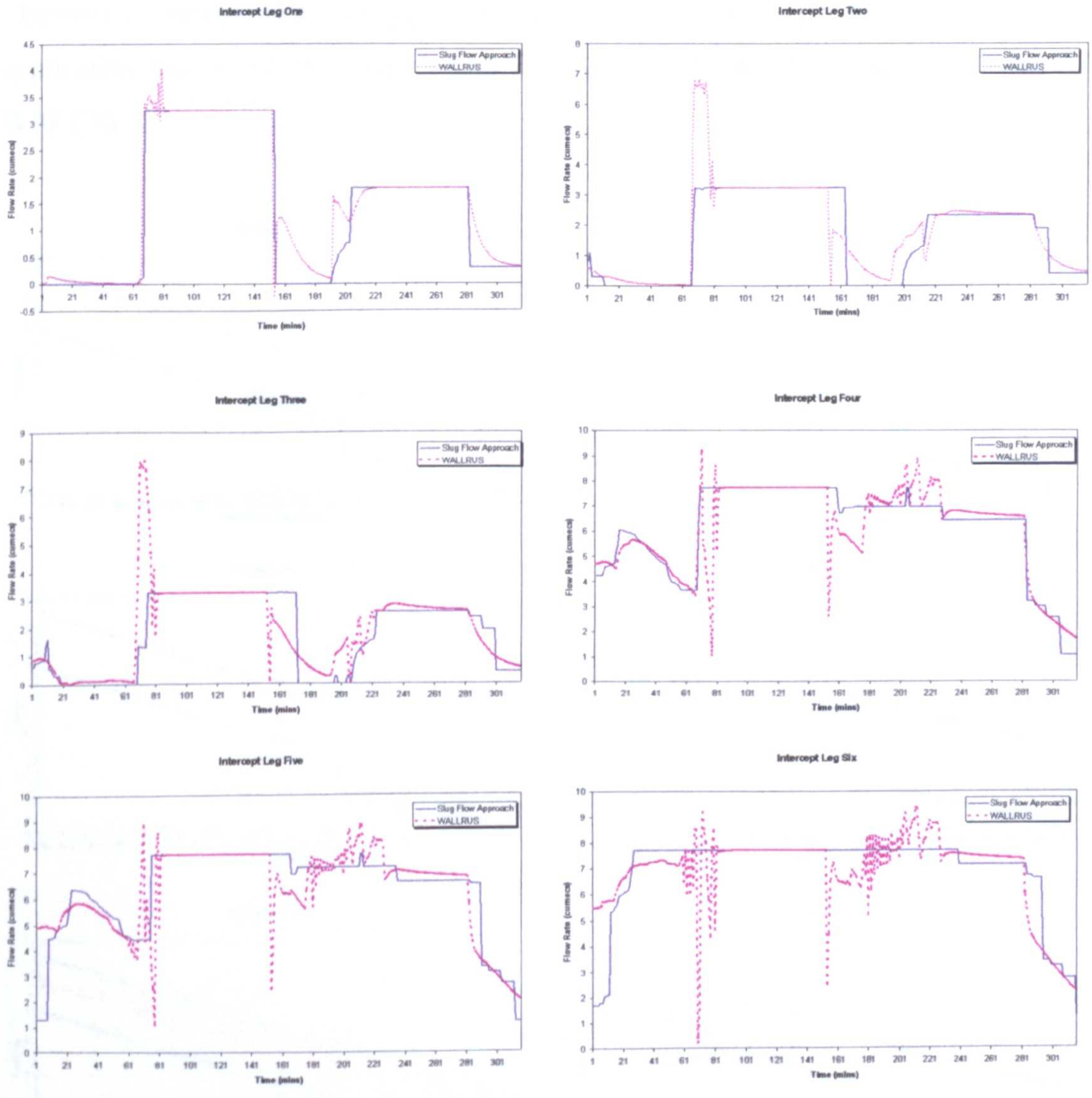
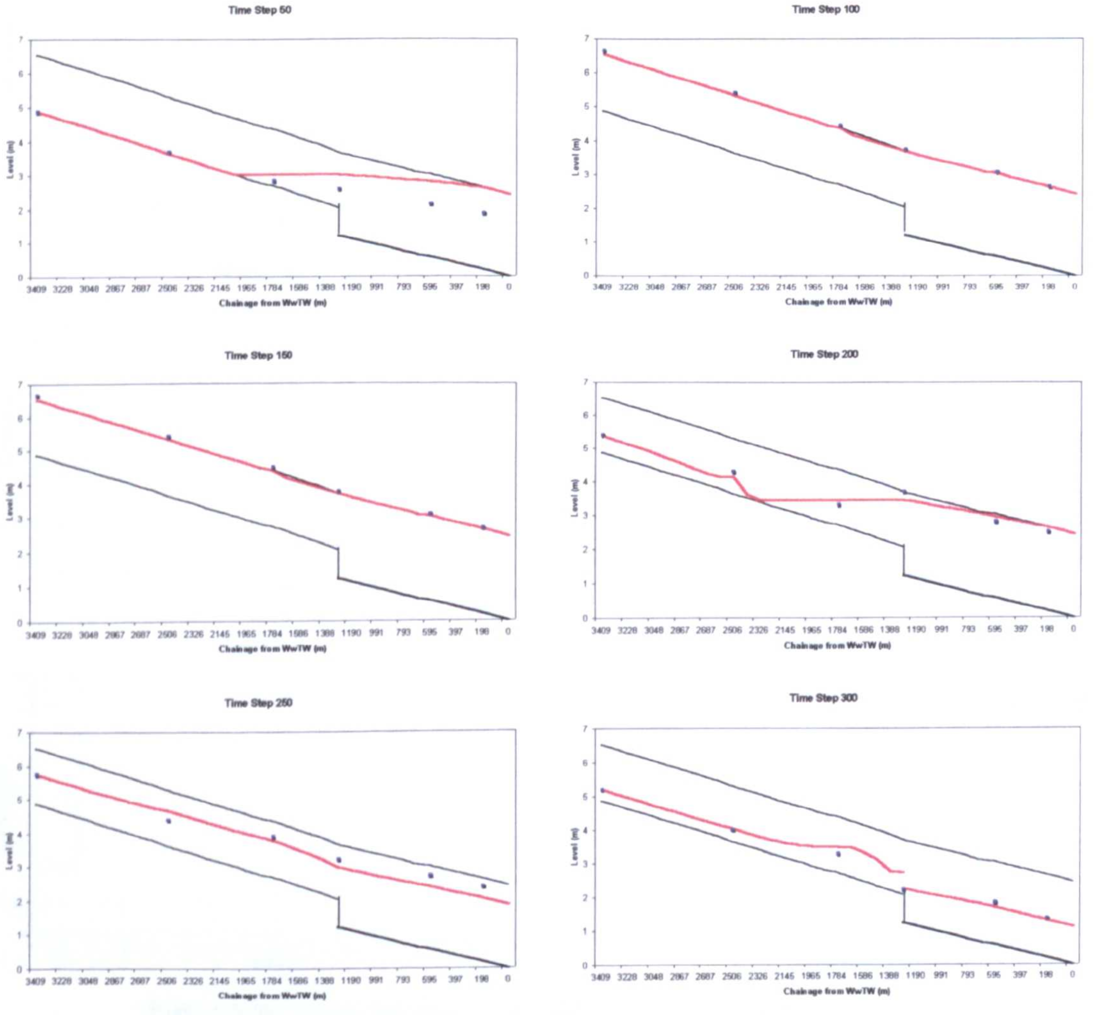


Figure 5.12: Interceptor Sewer Hydrographs for the High Storm Event.

The hydrographs in Figure 5.12 clearly show a considerable amount of control activity arising from the optimal pollution control model and also the difficulty WALLRUS has in simulating them. There is a considerable amount of instability in these WALLRUS results, particularly when there are rapid changes in sewer flow, showing the inadequacy of the Muskingham method (as used in WALLRUS) for these applications. Generally, however, the hydrographs compare reasonably well offering the control strategies were input into WALLRUS with initial values of zero. However, this approach considerably increased the levels of instability in the results although it did solve the initialisation discrepancy. Therefore, the approach was not adopted because the results presented are more stable after the initialisation.

The control strategies were input into the hydraulic verification routine, which obtained the sample water profiles in Figure 5.13. There are a few discrepancies in this application but overall the water profiles compare well with the depths calculated by WALLRUS.



Hydraulic Verification Routine

WALLRUS

Figure 5.13: Sample Water Profiles for the High Storm Event.

5.5.4 Storm Profile Four – High Intensity Localised Peaks in Upstream Section

In this test case a runoff hydrograph was input into intercept point 1 of the interceptor system in Figure 5.6 and all other intercept point runoff hydrographs remained at zero. The control strategies from the control model generated the sewer hydrographs in Figure 5.14.

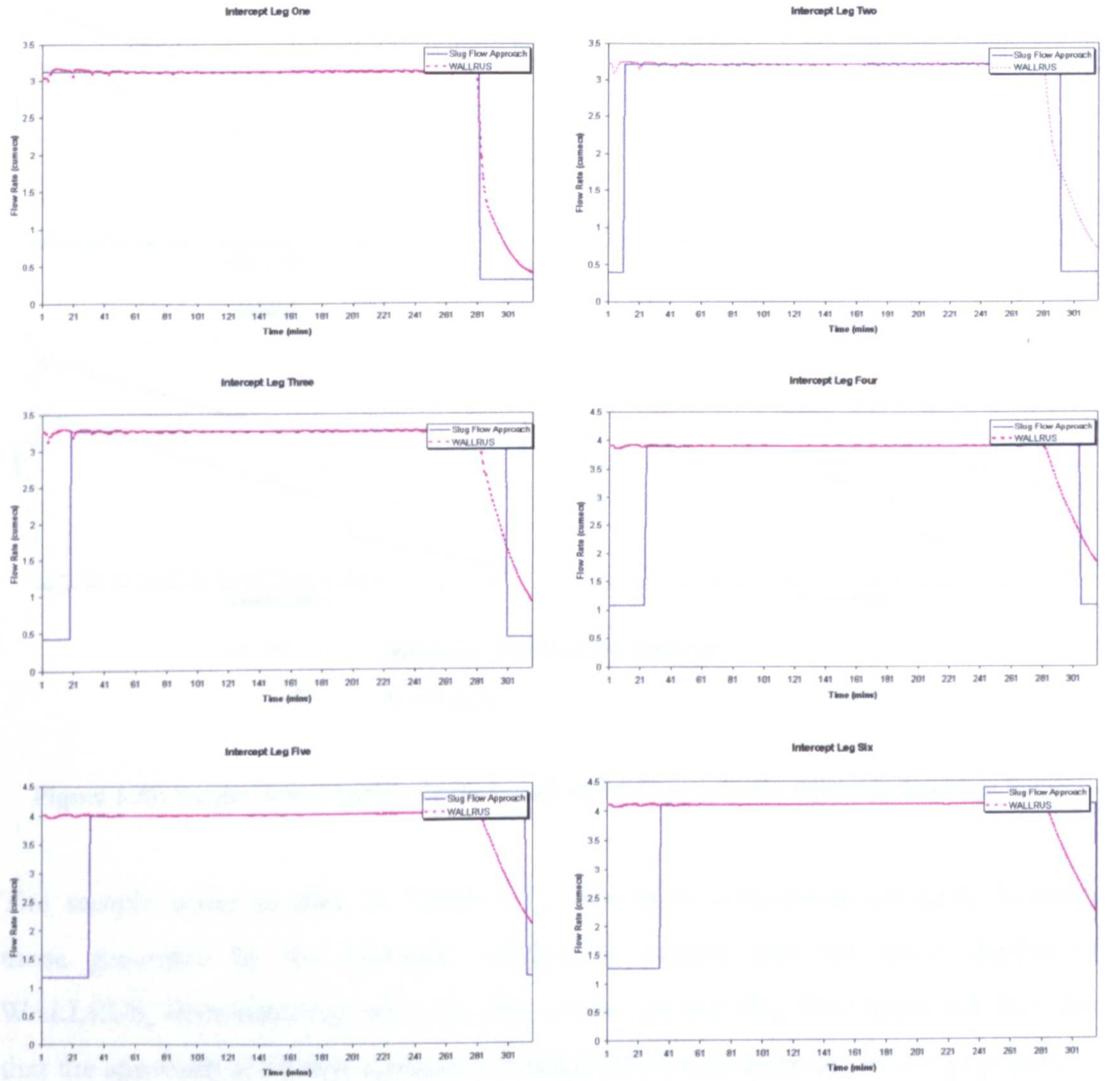


Figure 5.14: Interceptor Sewer Hydrographs for the Localised Peak Storm in the Upstream Intercept Point.

It is clear that there is still close agreement between the slug flow and WALLRUS hydrographs in Figure 5.14. However, as described earlier, there is an initialisation discrepancy with the WALLRUS simulation but to compensate this the results remain stable.

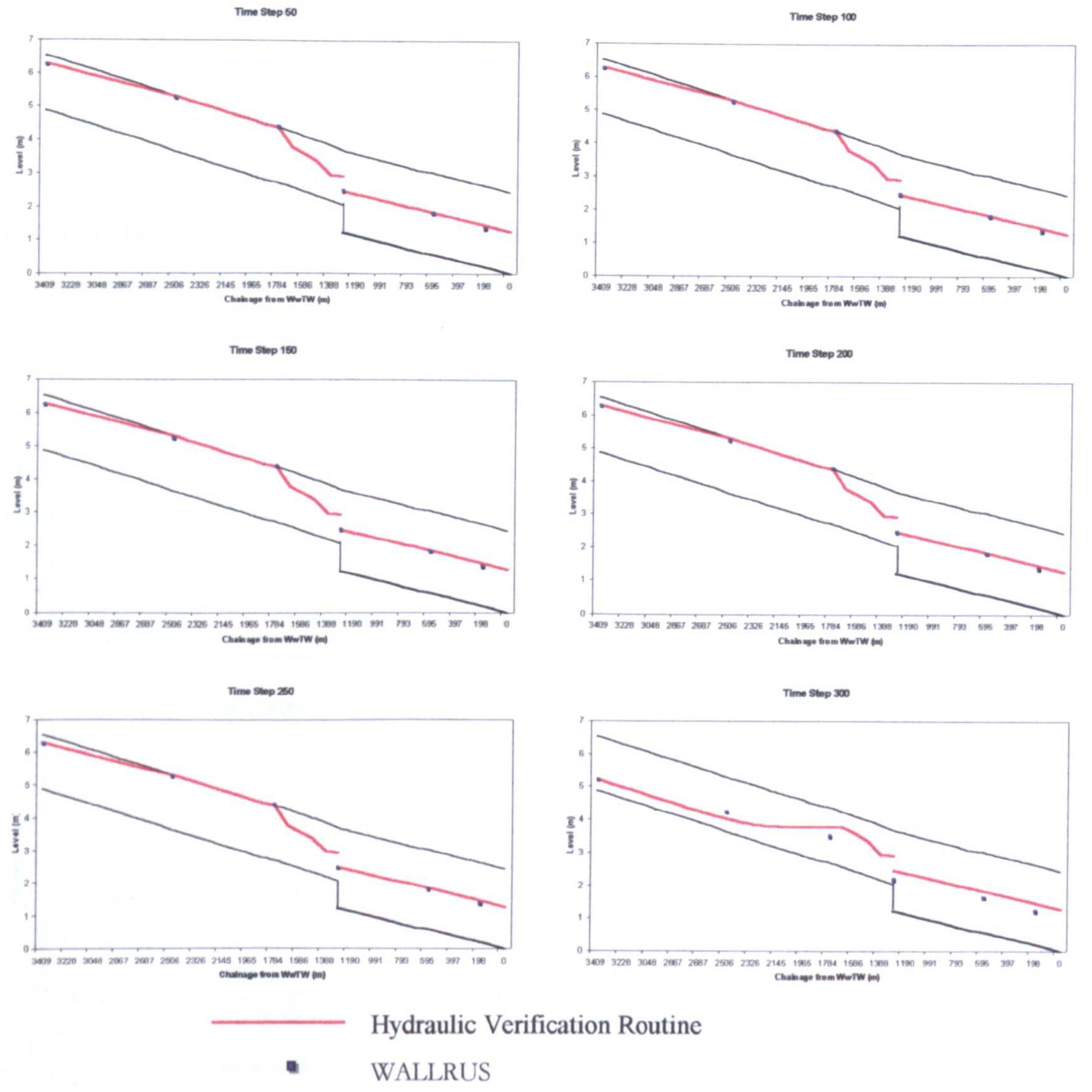


Figure 5.15: Sample Water Profiles for the Localised Peak Storm in the Upstream Intercept Point.

The sample water profiles in Figure 5.15 also show remarkable similarity between those generated by the hydraulic verification routine and the water depths of WALLRUS, demonstrating not only the validity of the slug flow approach but also that the approach accurately represents localised storms in upstream intercept points.

5.5.5 Storm Profile Five – High Intensity Localised Peaks in Middle Section

For this storm profile a runoff hydrograph was input into intercept point 4 (the middle intercept point) when all other intercept point runoff hydrographs remained at zero. The sewer hydrographs that were generated from the control strategies are shown in Figure 5.16.

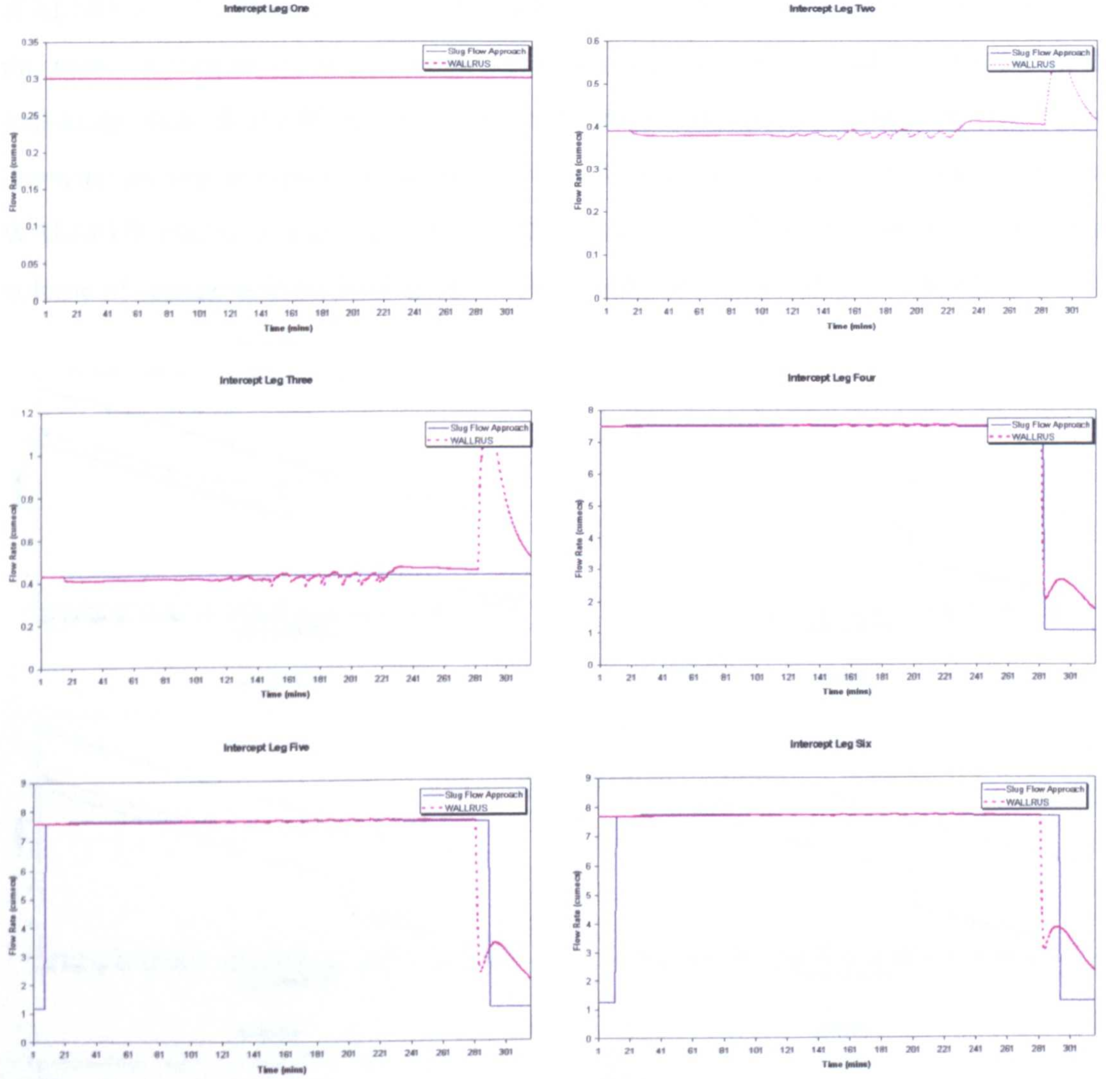


Figure 5.16: Interceptor Sewer Hydrographs for the Localised Peak Storm in the Middle Intercept Point.

On the whole the sewer hydrographs, shown in Figure 5.16, from the slug flow approach and from WALLRUS compare well, apart from the initialisation discrepancy. However, in this test case there is a noticeable adverse effect, which can be seen in the hydrograph for intercept point 3. The WALLRUS results deviate from the slug flow hydrographs towards the end of the storm event showing increased flow in the downstream sections of the interceptor there would be a significant amount of storage available upstream. Therefore, in this situation backwater effects would be significant. Unfortunately, WALLRUS does not solve the full dynamic equations and is not able to simulate such phenomena. The WALLRUS results in Figure 5.16 are hard to explain. It was anticipated that the storage upstream would be utilised early in the storm; the discrepancies are therefore probably due to shortcomings in the WALLRUS code. There are signs of instability in the WALLRUS hydrographs both in the form of oscillations and a 'surge wave' (see intercept legs 2 and 3 in Figure 5.16) indicating that WALLRUS recognises that there are the backwater effects from downstream but is unable to simulate them. Another indication of the errors in the WALLRUS results is that, by virtue of the 'surge wave', there appears to be a larger volume of sewage in these hydrographs than is indicated by the slug flow results.

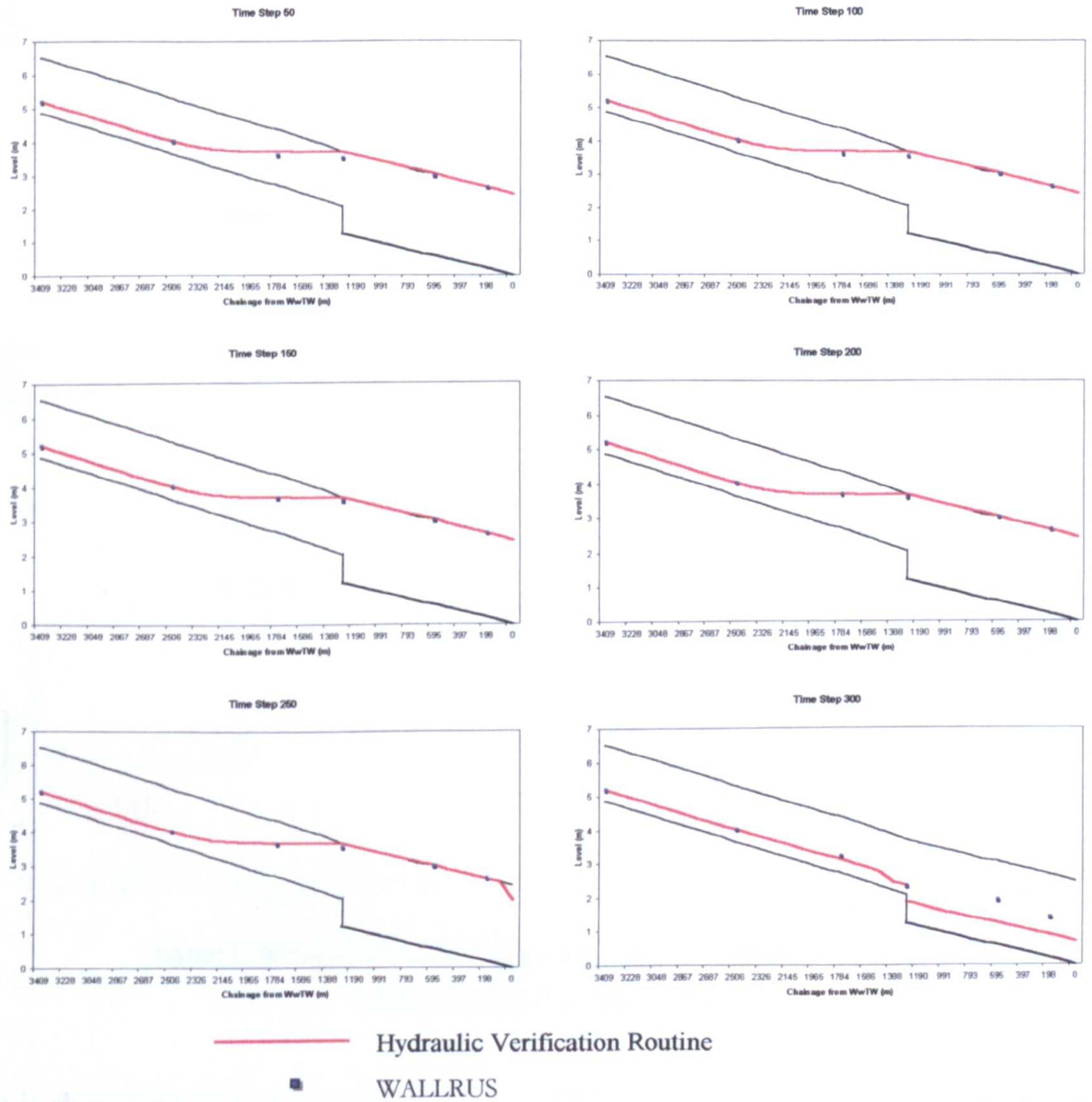


Figure 5.17: Sample Water Profiles for the Localised Peak Storm in the Middle Intercept Point.

The sample water profiles in Figure 5.17 from the hydraulic verification routine correlate well with the water depths from WALLRUS and offer further validation of the slug flow approach.

5.5.6 Storm Profile Six – High Intensity Localised Peaks in Downstream Section

In this storm profile a runoff hydrograph was only input in the downstream intercept point of the interceptor sewer and the hydrographs generated from the control strategies are shown in Figure 5.18.

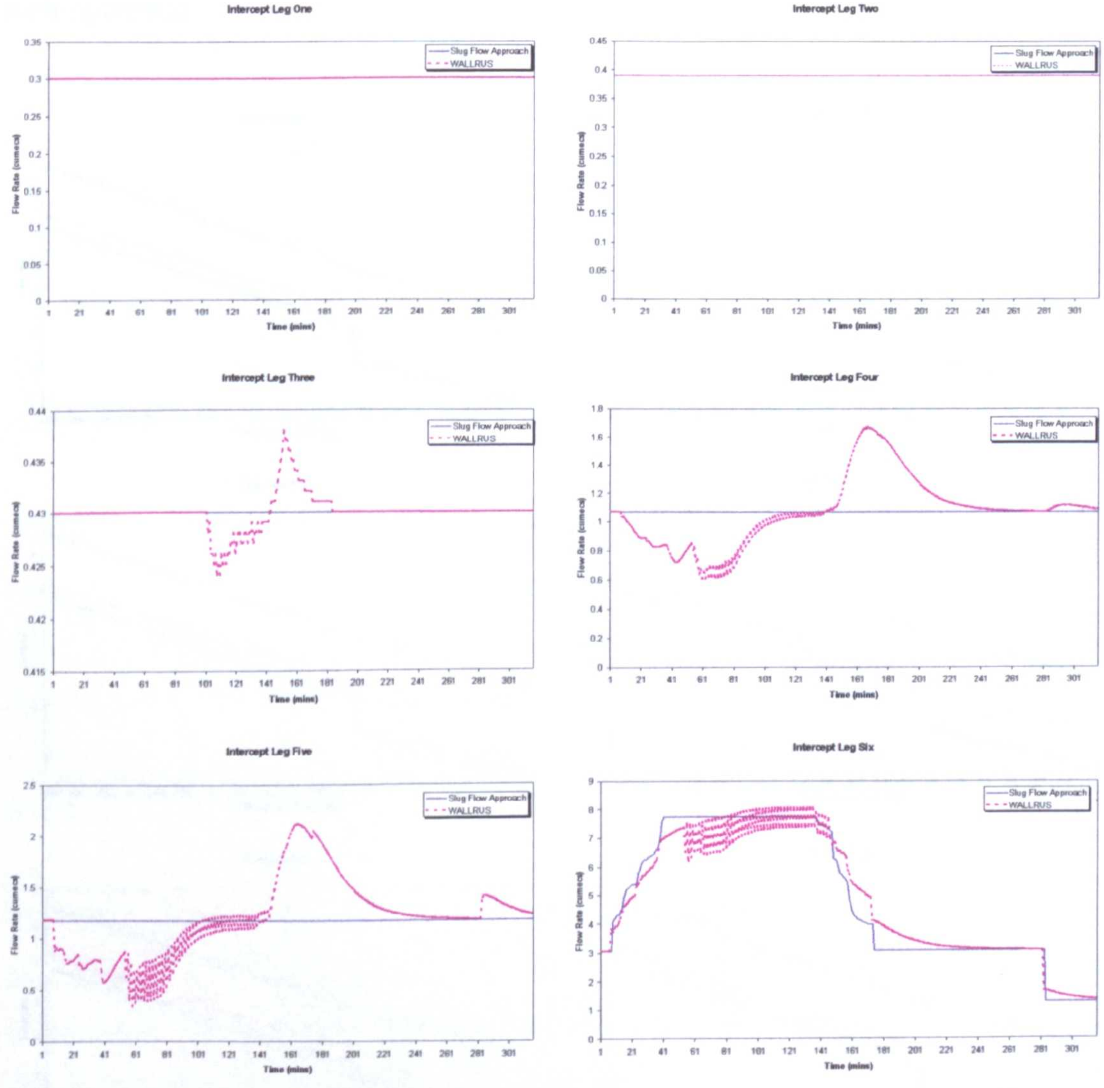


Figure 5.18: Interceptor Sewer Hydrographs for the Localised Peak Storm in the Downstream Intercept Point.

It is clear from the hydrographs in Figure 5.18 that backwater effects are significant in localised storm events in the downstream section of the interceptor. The slug flow approach does not represent any of these effects but WALLRUS, although unable to simulate them accurately, recognises that some of the sewage volume travels upstream utilising the available storage. The WALLRUS hydrographs again show a considerable amount of instability, illustrating its inability to adequately simulate unsteady conditions. In this case the apparent surge waves at upstream sections are inexplicably preceded by infeasible compensating 'troughs' (negative waves), possibly arising in an attempt to ensure mass balance in the computations, and reducing apparent flows below the DWF inputs. It is evident that the full dynamic equations need to be used to get a full appreciation of the true hydrodynamic behaviour of the system under such operation.

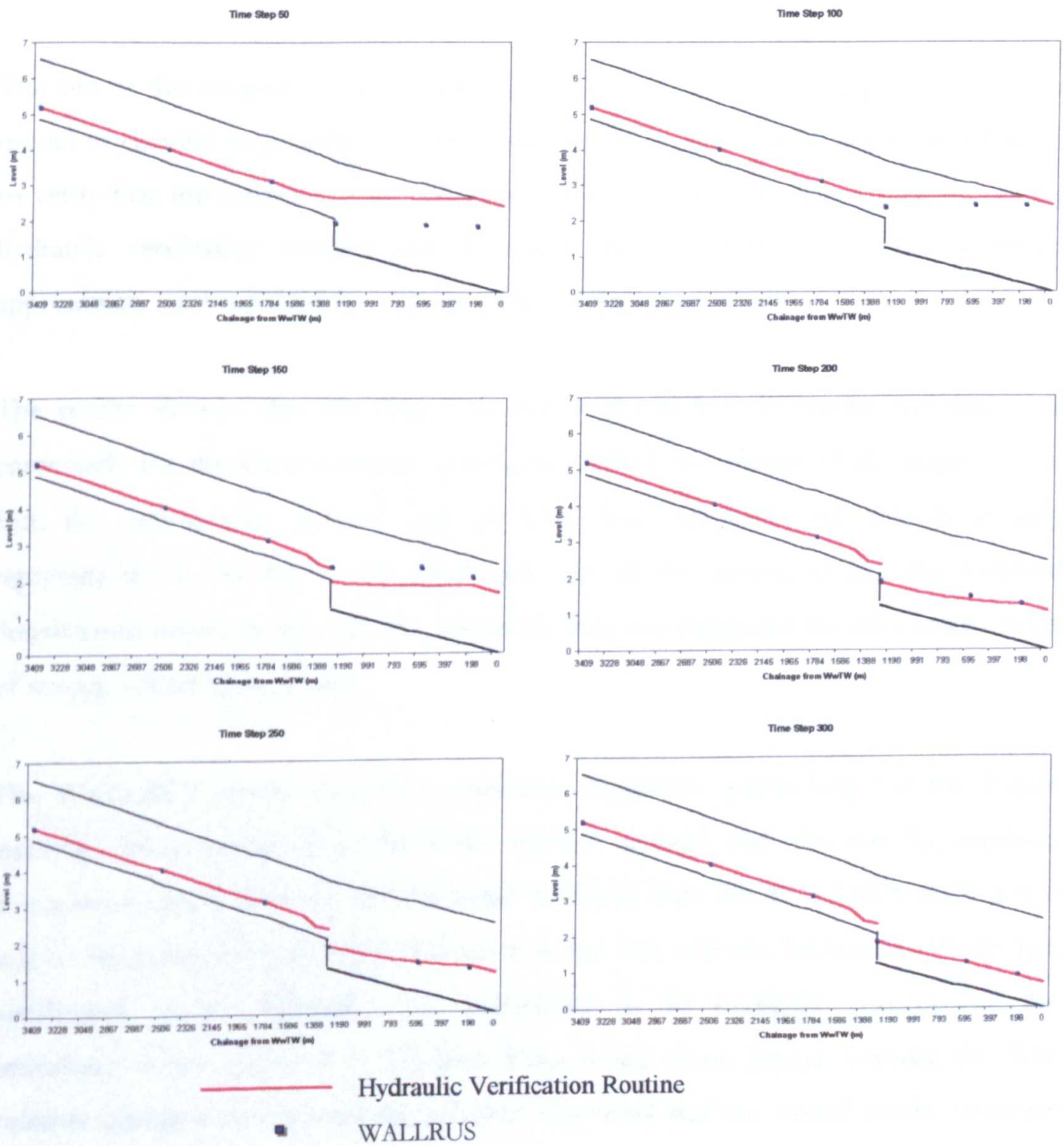


Figure 5.19: Sample Water Profiles for the Localised Peak Storm in the Downstream Intercept Point.

It must be conceded, therefore, that the slug flow approach cannot accurately represent the hydraulics in the interceptor sewer from the unusual situations of localised storms in the downstream sections only, when backwater volumes will be large.

The hydrographs were input into the hydraulic verification routine to obtain the sample water profiles in Figure 5.19. The sample water profiles in Figure 5.19 compare reasonably well with the water depths of WALLRUS. However, there are a few discrepancies that are explained by deviations in the sewer hydrographs.

5.6 CONCLUDING REMARKS

The aim of this chapter was to evaluate the sensitivity of the optimal pollution control model to changes in severity of storms and the effect of localised storms and, further to verify that the control actions generate physically feasible results. A post-processing hydraulic verification routine was developed for this purpose which determined approximate water profiles for each time step in the storm event.

The results showed that the slug flow and WALLRUS hydrographs correlate well, particularly for the lower intensity storms, illustrating the validity of the approach. In fact, the hydrographs showed that the slug flow approach was able to suitably represent the hydraulics in the interceptor for all the storms except the localised downstream event. In this case the approach does not represent the backwater effects of sewage advecting upstream.

The WALLRUS results showed considerable instability, particularly for the higher intensity storm (where there are more control actions) and also for the localised downstream storm. It must be concluded therefore that the WALLRUS code is not able to adequately simulate rapid changes in sewer flow and also backwater effects. The significance of the instability was considered to be negligible (except for the limitations of the synthesis of localised downstream storm inputs) because the flow volumes arising were minimal, the solution recovered and the overall results compare well.

The sample water profiles showed that the hydraulic verification routine determined depths of flow, which were validated against those calculated by WALLRUS. In fact, the water profiles and the WALLRUS depths were remarkably similar indicating that the slug flow approach was a sound formulation in the optimal control model. Most discrepancies between the water profiles and WALLRUS depths were conservative, with the exception of the high intensity and localised downstream storm events where the WALLRUS results suffered from the instability problems.

Overall, the results have validated the slug flow approach in the optimal pollution control model. The slug flow approach was able to simulate the hydraulics in the interceptor sewer from all storm events, apart from the localised downstream storm. It is concluded that the optimal pollution control model can be used confidently under most storm conditions. However, the model needs modifications to be used in storm conditions that are localised in the downstream sections of the interceptor sewer. These conditions would be likely to occur in large interceptor sewer systems where a large spatial distribution in rainfall might be expected. Some potential modifications to the approach to address this problem are discussed in the Recommendations for Further Work section.

CHAPTER 6

EXTENSIONS TO CONTROL MODELS

6.1 INTRODUCTION

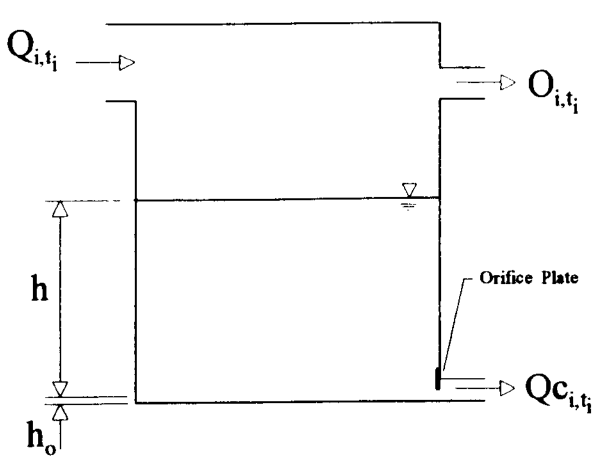
The optimal pollution control models have so far been developed for rather idealised interceptor sewer systems, which do not have realistic features such as combined sewer overflow (CSO) chambers. These models have been thoroughly described in Chapter 4 (see also Thomas *et al.,* 1998; and Thomas *et al.*, 2000) and verified using a post-processing hydraulic verification routine which was validated, in most cases, against WALLRUS (see also Thomas *et al.*, 1999a; and Thomas *et al.*, 2000), as shown in Chapter 5. Therefore, the viability of the slug flow modelling approach has been demonstrated and it is adopted for further model developments.

This chapter describes the extensions of the models to more realistic interceptor systems, which include combined sewer overflows (CSOs). The extensions are thoroughly described and then tested on a test case interceptor sewer system. A brief description of these extensions can be seen in Thomas *et al.* (1999b).

6.2 OVERFLOW CHAMBERS

The original control models determined optimum interceptor inflow rates based on the incoming pollutant concentrations that maximised pollutant load retention within the interceptor sewer. As described earlier, the model was formulated using a slug flow approach where the slugs were tracked through the interceptor and the control model determined the amount of sewage that should be added from the individual catchments based on the appropriate time delays and their respective pollution loadings. This optimisation problem was solved using two procedures, Linear Programming (LP) or Dynamic Programming (DP) (see Section 4.2). These models were applied successfully to an idealised interceptor sewer system (see Section 4.3) where the models significantly reduced the over-spill pollutant load discharged to the receiving waters, and outperformed all other control procedures used in the study. In these models, flows in excess of the interceptor inflows were assumed to be spilled without retention in storage chambers.

The control models are now extended to include storage chambers at the intercept points. A typical chamber arrangement with a moveable orifice plate (penstock gate) is shown in Figure 6.1.



where:

Q_{i,t_i} - Inflow from catchment i in time step t_i (cumecs).

O_{i,t_i} - Overflow from chamber i in time step t_i [if chamber level > spill level] (cumecs).

Qc_{i,t_i} - Continuation flow into interceptor from chamber i in time step t_i (cumecs).

h_o - Height of mid-point of orifice (m).

h - Head (m).

Figure 6.1: Typical Chamber Arrangement.

The continuation flow rate, Qc, into the interceptor sewer is now governed by the non-linear equation :

$$Qc = C_d a \sqrt{2gh} \tag{6.1}$$

where C_d is the coefficient of discharge of the orifice [dimensionless]; a is the area of the orifice [m^2]; g is the acceleration due to gravity [m/s^2]; and h is the head [m]. The area a of a rectangular orifice is given by:

$$a = p_w p_h \tag{6.2}$$

where p_w is the width of the orifice (constant)[m]; and p_h is the height of the orifice/penstock (variable)[m]. Therefore:

$$Qc = C_d p_w p_h \sqrt{2g\left(y - \frac{p_h}{2}\right)} \tag{6.3}$$

where y is the chamber water level above the invert of the chamber [m].

Equation (6.3) is valid for conditions when the chamber level y is greater than the orifice height p_h. Otherwise, the hydraulic characteristics of the continuation pipe govern the continuation flow rate Qc, the conditions becoming those of open channel flow. For example, an approximation may be given by an approach where the chamber level y forms the upstream level in the continuation pipe and the Manning equation is used to determine the flow rate assuming uniform flow conditions. However, as described in Chapter 5, the procedure for the iterative solution of the Manning equation is not computationally efficient and, therefore, this approach has not been used. Instead, in these conditions it is assumed that Qc is not governed by any hydraulic equations.

The idealised control models developed, as presented in Section 4.2, determine the desired Qc in each time step to maximise pollution load retention in the interceptor sewer. The problem now becomes that of determining how to adjust the penstock in each time step to achieve that value of Qc. Qc depends on various factors but it is primarily governed by the head in the chamber in each time step. Therefore equation (6.3) has to be solved in each time step to determine the movement of the penstocks. An explicit solution of equation (6.3) is not possible but it can be solved quickly using the numerical procedure described below.

Using $a = 0$ and $b = p_{top}$ as starting points, where p_{top} is the maximum level the orifice gate can open [m], and a and b are penstock height positions p_h [m], then the algorithm is:

$$f = C_d p_w p_h \sqrt{2g\left(y - \frac{p_h}{2}\right)} - Qc \tag{6.4}$$

$$c = \frac{af(b) - bf(a)}{f(b) - f(a)} \tag{6.5}$$

$$\textit{If } f(c) = 0 \textit{ then Solution Found} \tag{6.6}$$

$$\textit{If } f(c) < 0 \textit{ then } a = a - \frac{b-a}{f(b) - f(a)} f(a) \tag{6.7}$$

Else

$$\textit{If } f(c) > 0 \textit{ then } b = b - \frac{b-a}{f(b)-f(a)} f(b) \tag{6.8}$$

Repeat until (6.6) fulfilled.

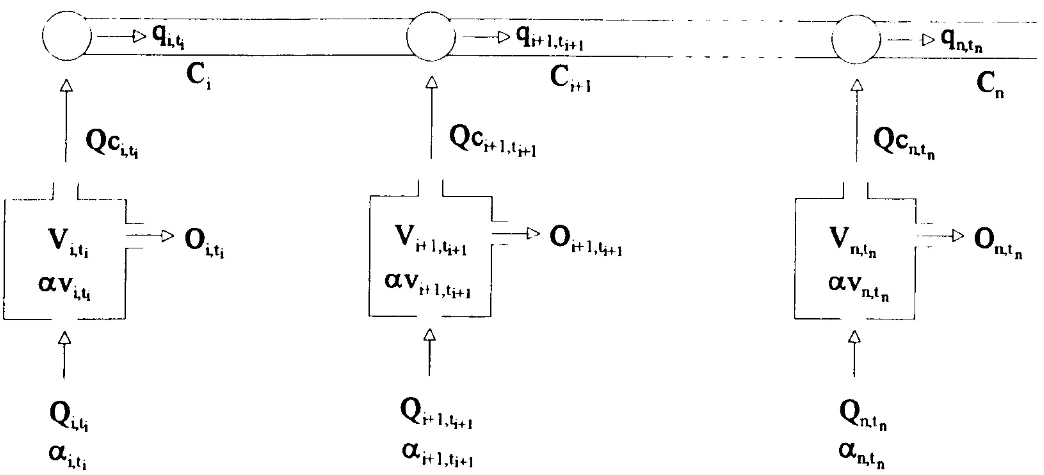
In application, incorporation of this numerical scheme enabled the solution to be achieved faster than real time and it does not significantly reduce the computational efficiency of the control models. However, other schemes may be more efficient but further evaluation of solution methods for this equation was beyond the scope of the study.

For conditions when the chamber level y is below the maximum level of the penstock p_{top}, the penstock is controlled using an alternative approach because of problems encountered when using equation (6.4). To illustrate this, consider the conditions when the control model decided to restrict flow into the interceptor sewer where the penstock moved down accordingly. A head would be generated and the continuation flow would be pressurised through an orifice. This generates a complex problem where the restricted continuation flow may be greater than the desired Qc from the control model because it was pressurised. This is obviously sensitive to the hydraulic characteristics of the continuation pipe, which are presently unknown. Therefore, an approach has been used where the penstock moves according to the change in volume of the chamber during the time step. This approach is not considered to have significant implications because the chambers are mostly above the penstock level in storm conditions where equation (6.4) is valid, and the problem forms a minor detail of the control of the chambers.

It should be noted that equation (6.3) is valid for steady flow conditions. However, the conditions in the overflow chambers are unsteady, i.e. conditions change through time. The continuation flow rate Qc calculated would only be realised in the actual system when the level of the chamber was constantly at the level used in the equation (6.3). Of course, the level would drop during the time step so the continuation flow rate Qc would also drop. Therefore, larger solution time steps in the control model will reduce the validity of this approach but it is considered acceptable for smaller time steps. It would be advisable that sensitivity studies be conducted to obtain the range of validity of this approach. However, one of the strengths of the successive solution approach, i.e. solving t=0, t=1, t=2, etc, is that it allows the inclusion of an updating procedure, where sensed data from the actual system can be used to periodically correct any errors in the control model. Obviously, this type of procedure could also be used to update the state of the interceptor where the sewer flow dynamics are modelled using the slug flow approach.

The optimal control model is formulated to determine the control strategies within discrete time steps, as shown in Figure 4.5, and equation (6.3) can be solved between the time step solutions of the objective function.

The extended optimal control model formulation is schematised in Figure 6.2.



where:

q_{i,t_i} - Interceptor flow rate below chamber i in time step t_i (cumecs).

O_{i,t_i} - Overflow from chamber i in time step t_i [if chamber level > spill level] (cumecs).

$\alpha v_{i,t_i}$ - Chamber pollutant concentration factor in chamber i in time step t_i (dimensionless).

Q_{i,t_i} - Inflow from catchment i in time step t_i (cumecs).

V_{i,t_i} - Chamber water volume in chamber i in time step t_i (m^3).

α_{i,t_i} - Polluntant concentration factor of inflow from catchment i in time step t_i (dimensionless).

Qc_{i,t_i} - Throughflow into interceptor from chamber i in time step t_i (cumecs).

Figure 6.2: Schematic of the Extended Optimal Control Model Formulation.

The chamber pollutant concentration factor $\alpha v_{i,t_i}$ in Figure 6.2 is determined by the mixing model:

$$\alpha v_{i,t_i} = \frac{\alpha v_{i,t_{i-1}} V_{i,t_{i-1}} + \alpha_{i,t_i} Q_{i,t_i} \Delta t}{Q_{i,t_i} \Delta t + V_{i,t_{i-1}}} \tag{6.9}$$

For computational convenience here the pollutant concentration factor is defined as a coefficient assigned to the inflow at each time step. For general illustration this coefficient can be considered to range from 0 to 1, i.e. absolutely 'clean' through to absolutely 'dirty' inflows, though more generally it might be the concentration (typically mg/l) of the chosen determinand.

Additionally, the control model maintains volumetric continuity within the chambers:

$$A_{stor}\frac{\Delta y}{\Delta t} = Q_{i,t_i} - Qc_{i,t_i} \tag{6.10}$$

where A_{stor} is the storage chamber area (m^2) and y is the chamber water level (m). However, equation (6.10) applies to conditions when the chamber level y is less than the spill level, which is normally the invert level of the overflow pipe. When the chamber level y is greater than this level:

$$A_{stor}\frac{\Delta y}{\Delta t} = Q_{i,t_i} - Qc_{i,t_i} - O_{i,t_i} \tag{6.11}$$

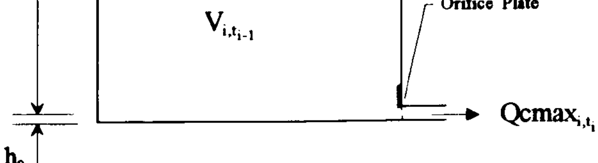


Figure 6.3: Procedure for the Calculation of the Maximum Continuation Flow Rate.

The overflow term also has to be included in the chamber pollutant concentration factor mixing model (6.9) under these conditions to maintain continuity.

In application, the initial state of the interceptor sewer system is known. During each time step, the control model adds the inflow volume $Q_{i,t_i}\Delta t$ to the known chamber volume $V_{i,t_{i-1}}$ to obtain the possible chamber retention volume $V\max_{i,t_i}$, as shown in Figure 6.3, assuming that the entire inflow volume is retained within the CSO chamber. The corresponding chamber level y for this volume is used in the calculation of (6.3) to determine the maximum possible outflow rate (from chamber) $Qc\max_{i,t_i}$ when the orifice is completely open, i.e. when a

is at a maximum in (6.1). If $Qc\max_{i,t_i}\Delta t$ (i.e. the volume allowed through into the interceptor in the solution time step) is greater than the chamber volume, then the continuation flow rate is reduced accordingly. Additionally, the chamber pollutant concentration factor $\alpha v_{i,t_i}$ is calculated from (6.9) for this volume $V\max_{i,t_i}$, again assuming that it is completely retained within the CSO chamber. These values are used within the optimisation routine where the objective function is solved within the appropriate capacity constraints to determine the optimum control strategy:

$$Max\sum_{i=1}^{n}\alpha v_{i,t_i}q_{i,t_i} \tag{6.12}$$

Subject to:

$$q_{i,t_i} \le Qc\max_{i,t_i} \qquad \forall i \tag{6.13}$$

$$\sum_{j=1}^{i} q_{j,t_j} \le C_i \qquad \forall i \tag{6.14}$$

and non-negativity constraints.

where n is the number of intercept points; $\alpha v_{i,t_i}$ is the pollutant concentration in chamber i in time step t_i; q_{i,t_i} is the interceptor inflow rate at intercept point i in time step t_i; $Qc\max_{i,t_i}$ is the maximum inflow into the interceptor from chamber i in time step t_i; and C_i is the interceptor sewer pipe full capacity below intercept point i..

The resource constraint (6.13) of the LP problem allows the decision variable q_{i,t_i} to range from zero to $Qc\max_{i,t_i}$, i.e $0 \le q_{i,t_i} \le Qc\max_{i,t_i}$. Therefore, the actual continuation flows Qc_{i,t_i} that satisfy the objective function (6.12) are calculated. From these values the penstock level p_b is calculated implicitly from (6.4) to determine the control action of the flow regulator (orifice gate or penstock). The default setting for the penstock gate is fully open. That is, if no restriction of throughflow is required, the penstock will move to the fully open position.

The chamber pollutant concentrations $\alpha v_{i,t_i}$ and chamber volumes V_{i,t_i}, as a consequence of the control strategy, are determined from (6.9) and (6.10) respectively.

These values now represent the state of the CSO before the next time step in the solution procedure.

In the next time step, the procedure again adds the subsequent inflow volume $Q_{i,t_{i+1}}\Delta t$ to the stored volume V_{i,t_i} to determine the maximum possible continuation flow $Qc\max_{i,t_{i+1}}$, as in Figure 6.3. The respective pollutant concentration factor is mixed with the chamber pollutant concentration in (6.9). These values are then used within the optimisation routine (6.12), (6.13) and (6.14). This continues until all discrete time steps solutions have been determined within the control time horizon.

In conditions when the chamber level is greater than the spill level it is assumed that the entire volume of sewage is initially retained and that the pollutant concentrations are completely mixed (6.8). This mixed pollutant concentration is used within the objective function (6.11) to determine the continuation flow rate. The volume of sewage above the spill level, after the continuation flow volume has been deducted from the total chamber volume in that time step, is then over-spilled. The spill load is calculated by multiplying the spill volume with the chamber pollutant concentration. The chamber level at the end of the time step therefore coincides with the spill level (invert of the overflow pipe). Obviously, this approach assumes that the hydraulic characteristics of the overflow pipe will allow the full spill discharge within the time step.

The objective function (6.12) represents decisions to be made that maximise the pollutant load received by a slug of sewage travelling through the interceptor incrementing inflows from the CSOs with the highest pollutant concentrations. However, equation (6.12) corresponds to only one time step chain t_i and the control strategies throughout the control time horizon are determined by altering the time step position (i.e t_i+1, t_i+2, ...*etc.*). Since, in this approach as previously, successive 'slugs' of water are assumed not to interact, then the sequence of optimal controls derived for each time step also represent the optimal control strategy for the entire event.

Overall, the optimisation is little changed from the original control model but the effects of storage (in the overflow chambers) on the pollutant concentration factors and inflow hydrographs are now accounted for. In this extended model formulation the control strategies are governed by the mixed pollutant concentrations in the storm chambers not the pollution concentration of the inflow hydrographs as in the original model.

6.3 TEST CASE

The northern leg of the Liverpool Interceptor Sewer has been simplified and used as a test case for the extended optimal pollution control model. A longitudinal section of the sewer can be seen in Figure 6.4 and the input data for this sewer is shown in Tables 6.1 and 6.2.

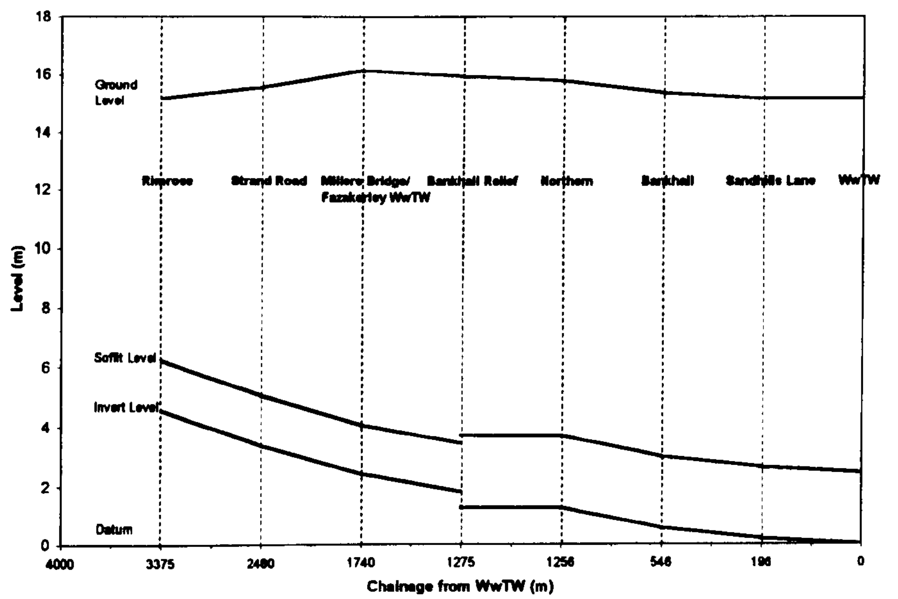


Figure 6.4: Longitudinal Section of the Simplified Northern Leg of the Liverpool Interceptor Sewer (not to scale).

Three control procedures were considered in this test case illustrated here:

Fixed Local Control (FLC) – Inflows up to the fixed inflow setting are passed forward to the interceptor regardless of conditions elsewhere in the sewer system (or in the interceptor for that matter). The procedure is therefore a volumetric local control system where no account is taken of the pollutant load of the flows.

Variable Local Control (VLC) – Inflows are permitted up to the interceptor sewer's capacity locally but no account is taken of the conditions elsewhere in the sewer system or the pollutant load of the flows. The procedure is therefore an extended version of local control.

Optimal Pollution Control (OPC) – The extended optimal pollution control model determines inflows using global information including pollutant concentrations within the interceptor and the overflow chambers. Therefore, the model maximises pollutant load retention within the entire interceptor sewer system. The linear programming (LP) solution procedure has been used in the control model rather than dynamic programming (DP) because of the improved computational efficiency.

All the control procedures use the slug flow approach within the interceptor sewer to convey the inflows along the interceptor and only the decision criteria differ.

Intercept Point (catchment)	*Sewer Diameter (m)*	*Sewer Gradient*	*Sewer Capacity (cumecs)*	*D.W.F. (cumecs)*	*Fixed Inflow Setting (cumecs)*
Rimrose	1.66	1/750	3.26	0.30	1.24
Strand Rd	1.66	1/750	3.26	0.09	0.25
Millers Bridge/Fazakerley	1.66	1/750	3.26	0.04	0.97
WwTW					
Bankhall Relief	2.44	1/1000	7.72	0.14	0.69
Northern	2.44	1/1000	7.72	0.50	2.13
Bankhall	2.44	1/1000	7.72	0.11	0.29
Sandhills Lane	2.44	1/1000	7.72	0.09	0.31

Table 6.1: Input Data for Interceptor Sewer.

Intercept Point (catchment)	*Chamber Area (m^2)*	*Spill Level [above invert level] (m)*	*Orifice Width (m)*	*Orifice Height (m)*
Rimrose	282.82	5.42	1.250	1.450
Strand Rd	136.03	6.91	1.700	0.625
Millers Bridge/ Fazakerley WwTW	50.31	7.95	1.500(E)	0.625(E)
Bankhal lRelief	169.78	8.04	2.075	0.625
Northern	328.24	8.18	2.650	1.450
Bankhall	167.06	8.47	1.800	0.625
Sandhills Lane	147.95	9.26	1.650	0.625

(E) – Estimated dimensions.

Table 6.2: Storm Chambers Input Data for Test Interceptor Sewer.

The extended control model was run with the hypothetical runoff hydrographs and pollutant concentrations factors presented in Figure 6.5, which are similar to those used in Chapter 5.5, and were loosely based on the catchment's response characteristics. For convenience the pollutant concentration factors were taken to be identical for each catchment.

A complete set of control strategies from the control model is shown in Figures 6.6 through to 6.12 for the fixed local control (FLC) and optimal pollution control (OPC) procedures. These figures show the results for each catchment along the test case interceptor. For example, Figure 6.6 shows the results for intercept point 1, the Rimrose catchment, and Figure 6.7 shows the results for intercept point 2, the Strand Road catchment, etc. The results presented are the flow rate control strategy, the penstock levels, the pollutant load spilled and the chamber levels.

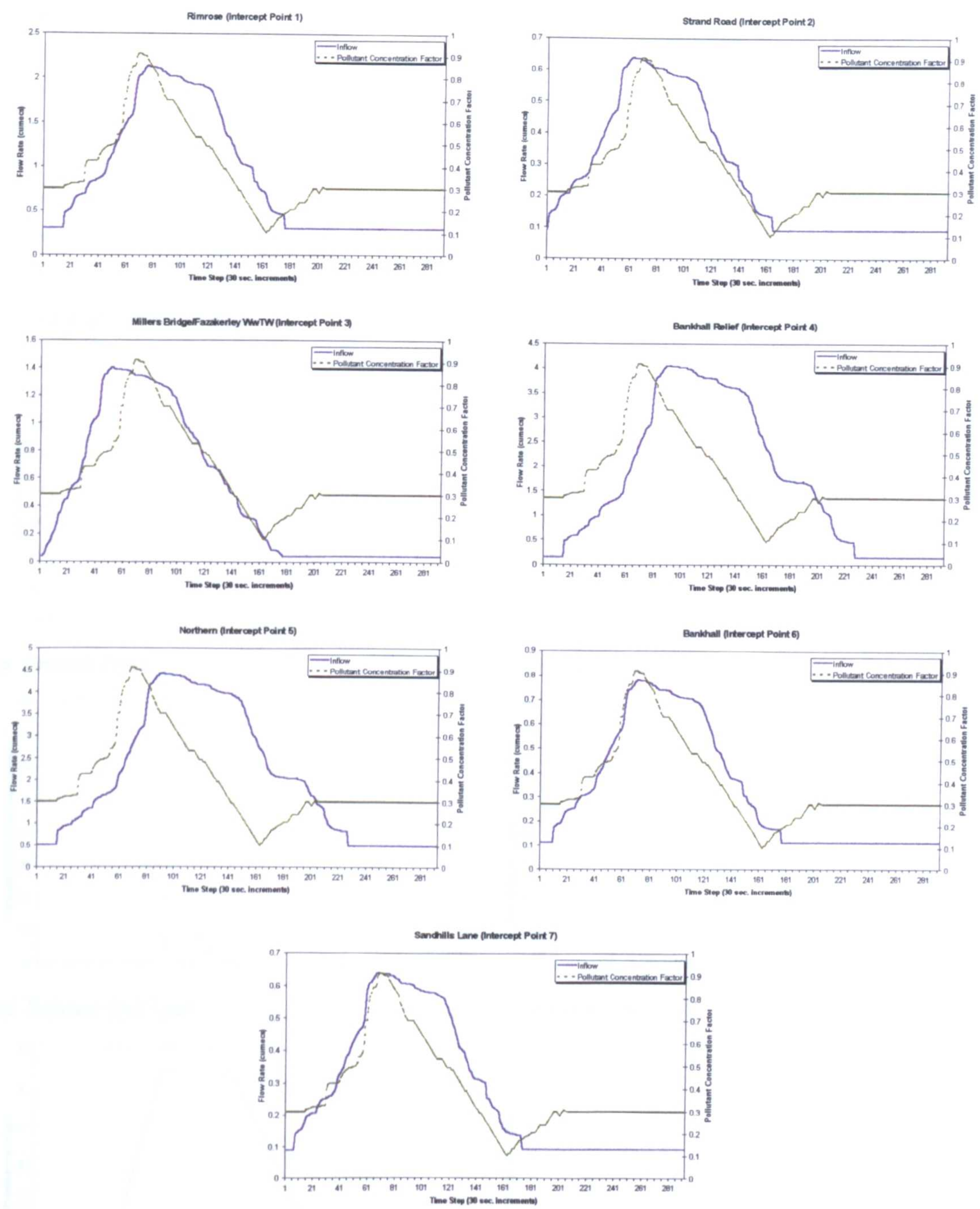
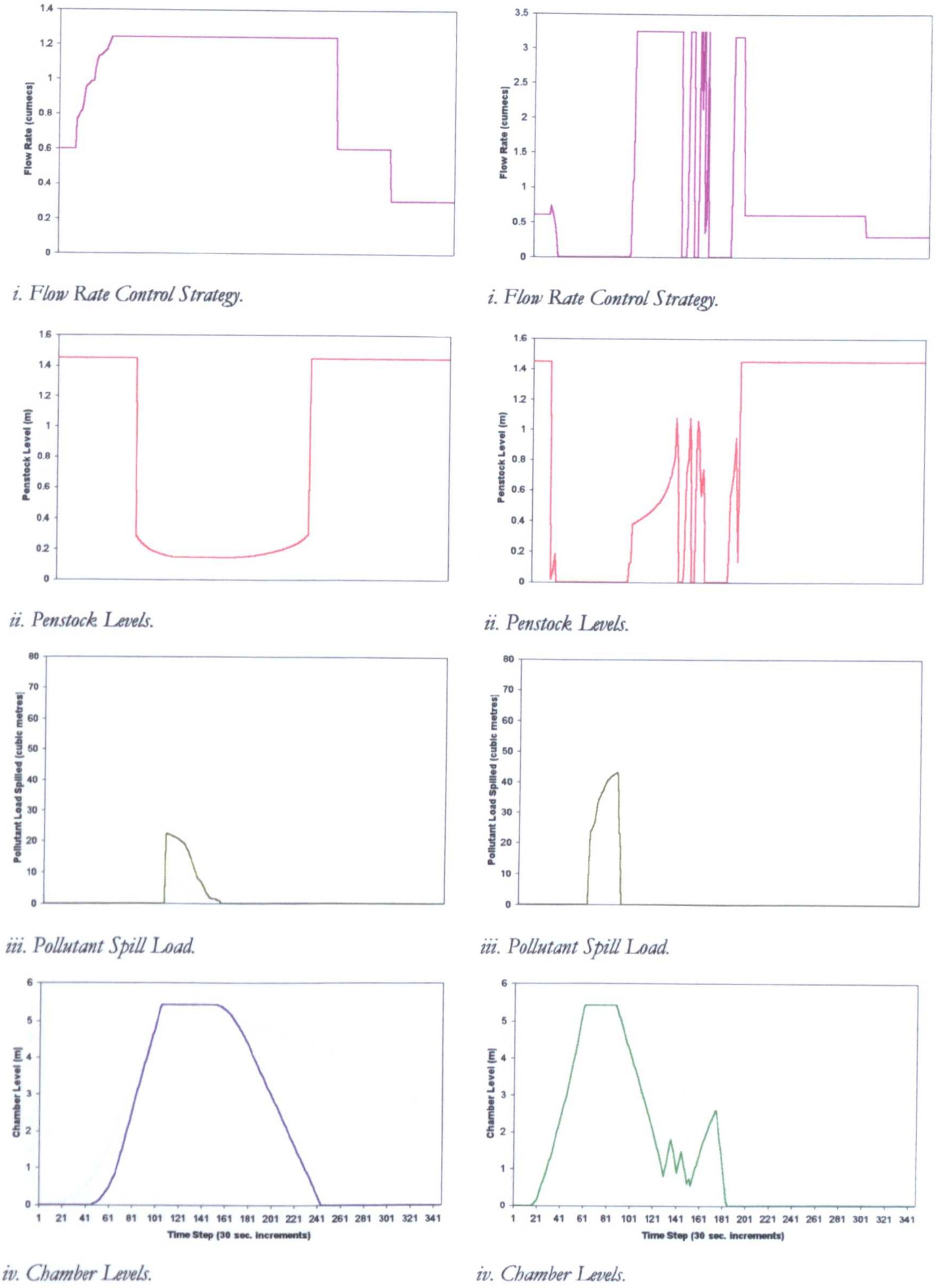


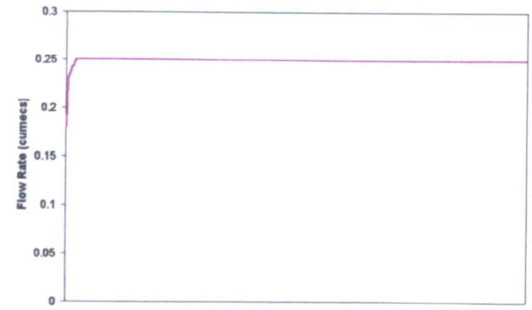
Figure 6.5: Inflow Hydrographs and Pollutant Concentration Factors for all Catchments.



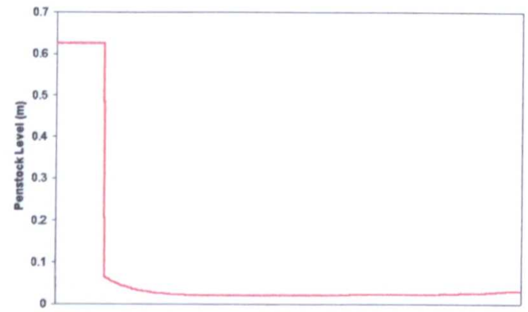
a. Fixed Local Control (FLC). **b.** Optimal Pollution Control (OPC).

Figure 6.6: Comparison between the Fixed Local Control (FLC) (**a**) and Optimal Pollution Control (OPC) (**b**) Strategies for the Rimrose Catchment (Intercept Point 1).

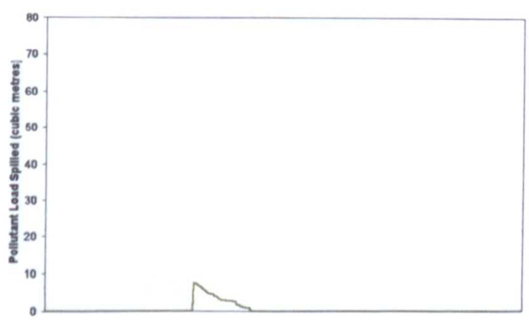
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



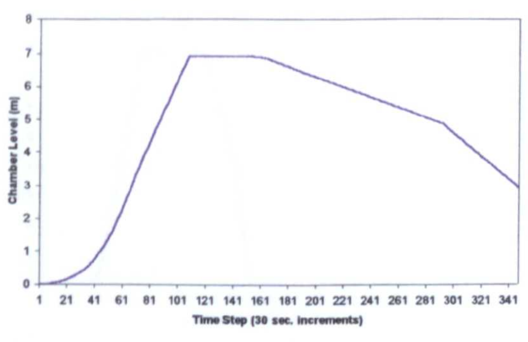
i. Flow Rate Control Strategy.



ii. Penstock Levels.

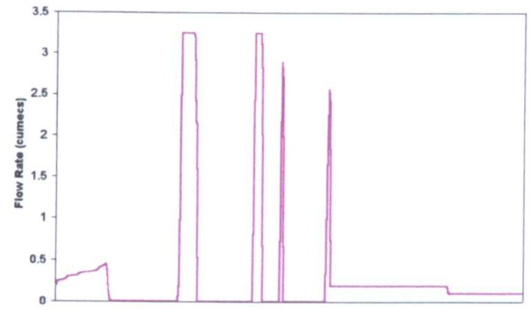


iii. Pollutant Spill Load.

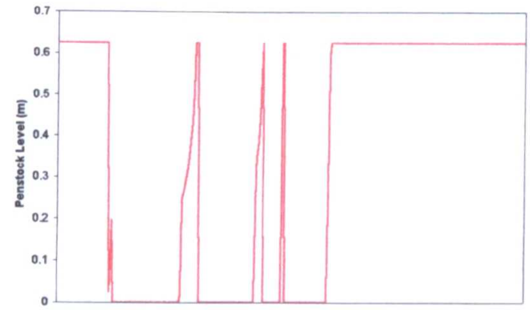


iv. Chamber Levels.

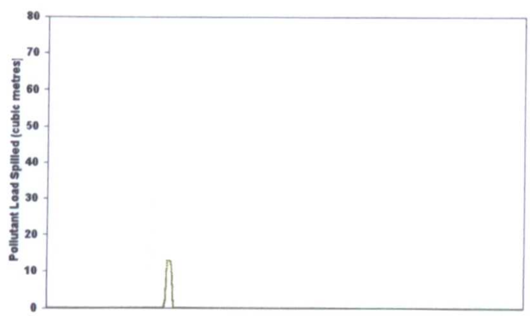
a. Fixed Local Control (FLC).



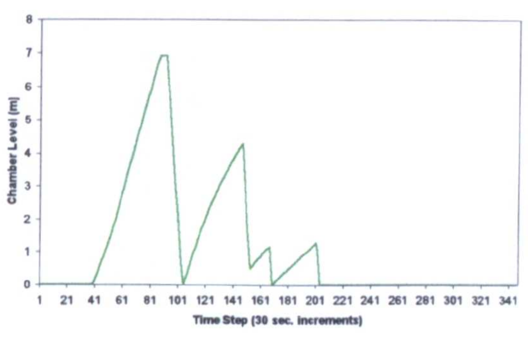
i. Flow Rate Control Strategy.



ii. Penstock Levels.



iii. Pollutant Spill Load.

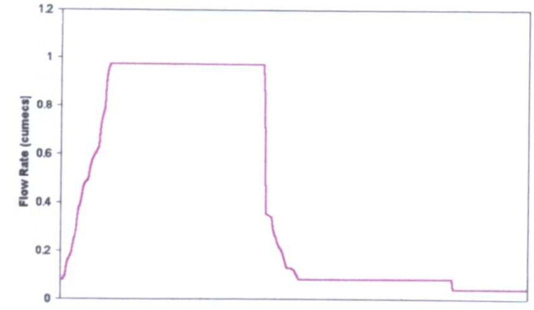


iv. Chamber Levels.

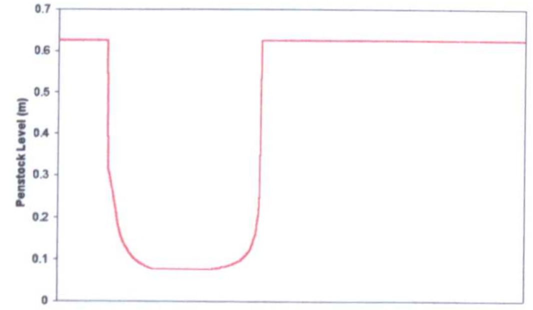
b. Optimal Pollution Control (OPC).

Figure 6.7: Comparison between the Fixed Local Control (FLC) (**a**) and Optimal Pollution Control (OPC) (**b**) Strategies for the Strand Road Catchment (Intercept Point 2).

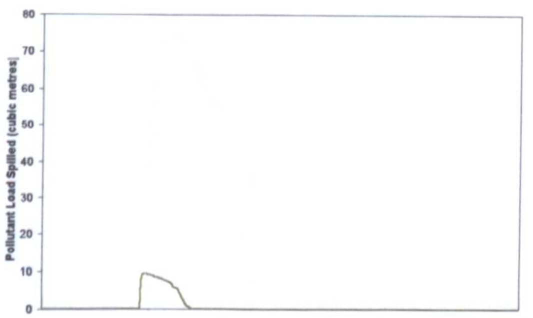
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



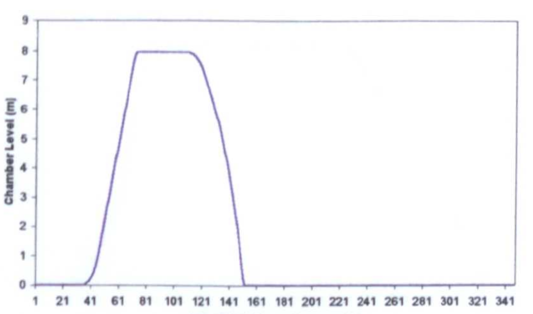
i. Flow Rate Control Strategy.



ii. Penstock Levels.

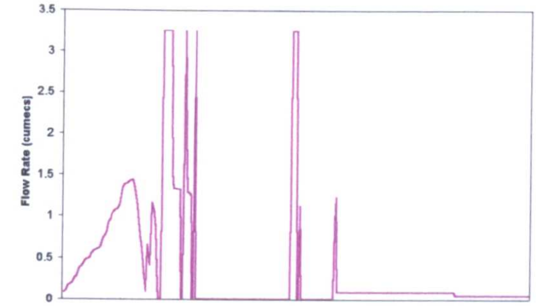


iii. Pollutant Spill Load.

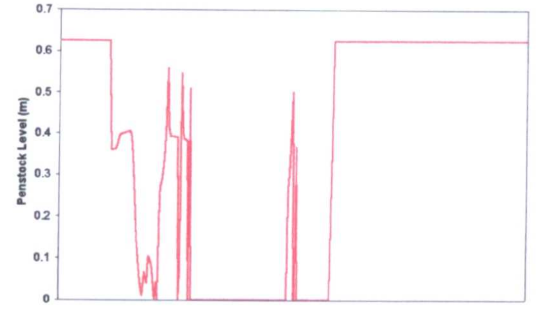


iv. Chamber Levels.

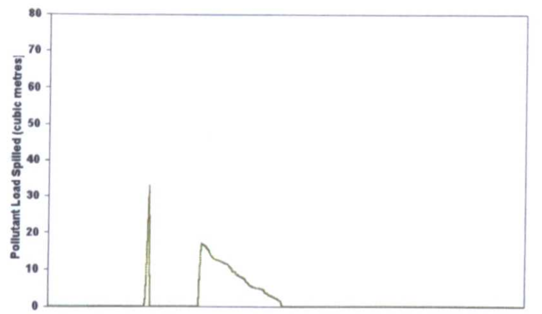
a. Fixed Local Control (FLC).



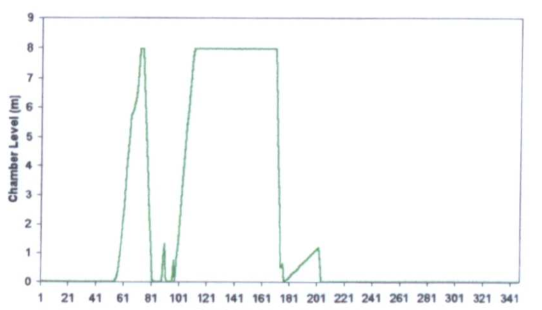
i. Flow Rate Control Strategy.



ii. Penstock Levels.



iii. Pollutant Spill Load.

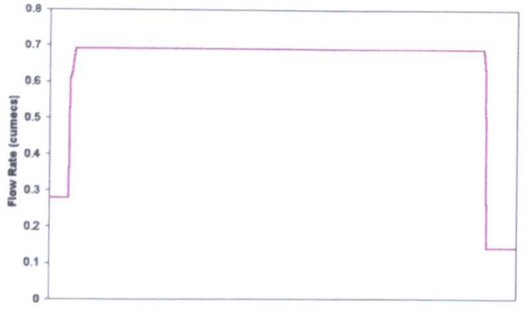


iv. Chamber Levels.

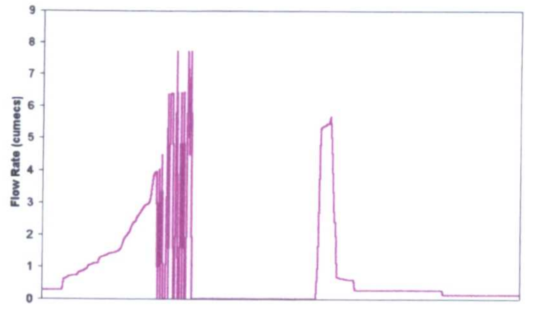
b. Optimal Pollution Control (OPC).

Figure 6.8: Comparison between the Fixed Local Control (FLC) (**a**) and Optimal Pollution Control (OPC) (**b**) Strategies for the Millers Bridge/Fazakerley WwTW Catchment (Intercept Point 3).

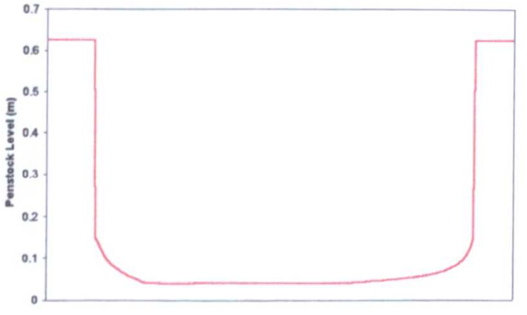
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



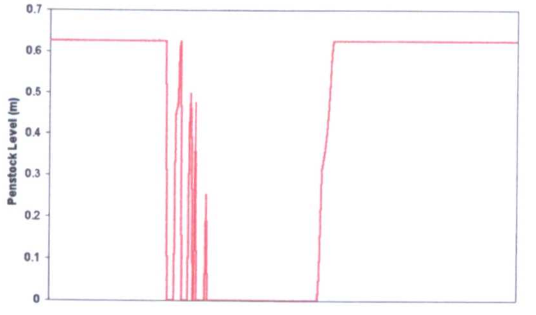
i. Flow Rate Control Strategy.



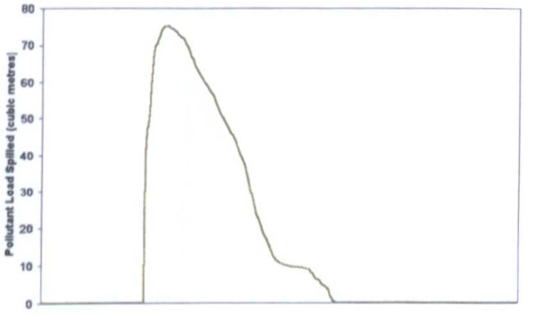
i. Flow Rate Control Strategy.



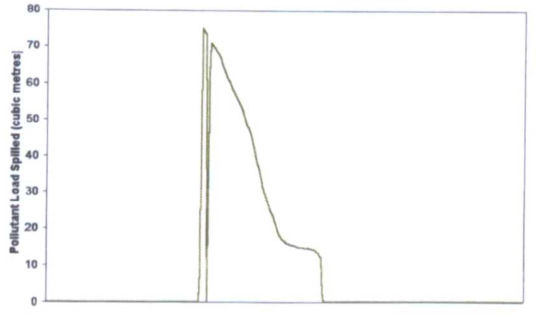
ii. Penstock Levels.



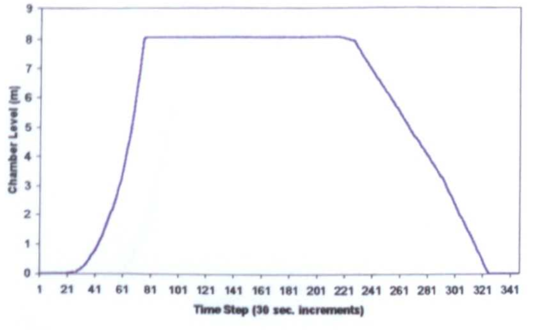
ii. Penstock Levels.



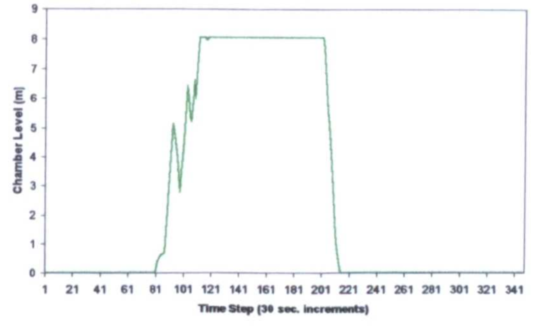
iii. Pollutant Spill Load.



iii. Pollutant Spill Load.



iv. Chamber Levels.

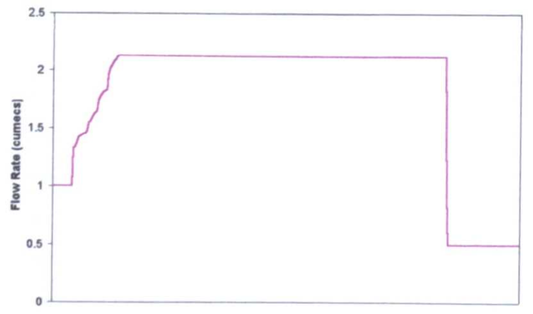


iv. Chamber Levels.

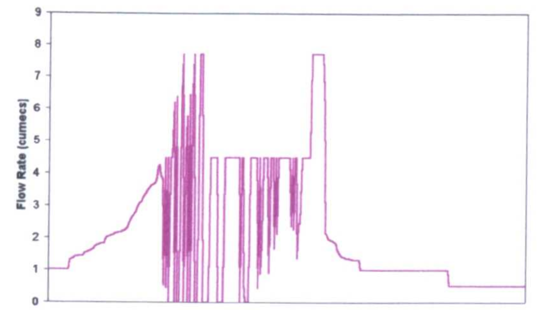
a. Fixed Local Control (FLC). **b.** Optimal Pollution Control (OPC).

Figure 6.9: Comparison between the Fixed Local Control (FLC) (**a**) and Optimal Pollution Control (OPC) (**b**) Strategies for the Bankhall Relief Catchment (Intercept Point 4).

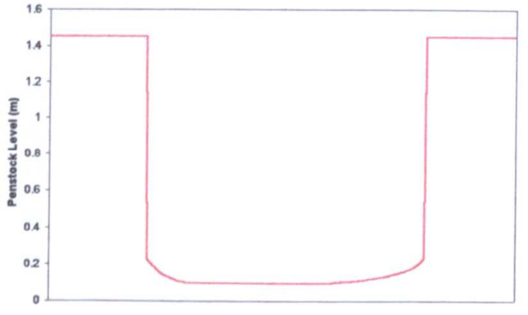
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



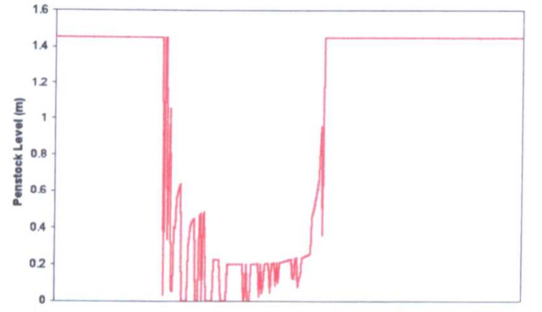
i. Flow Rate Control Strategy.



i. Flow Rate Control Strategy.



ii. Penstock Levels.



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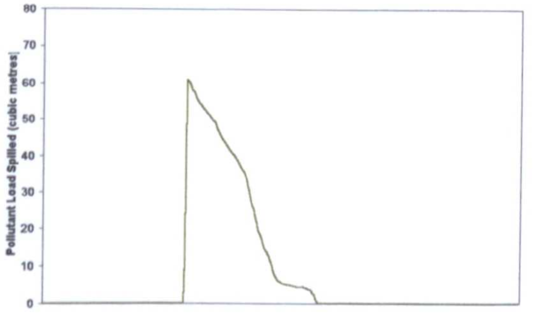
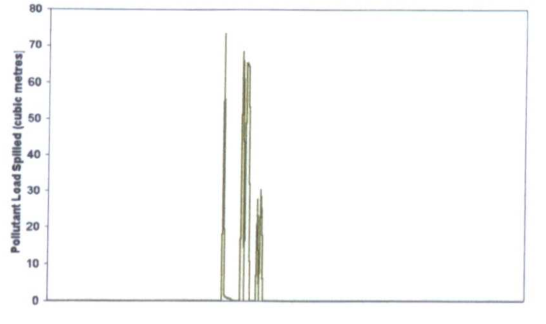
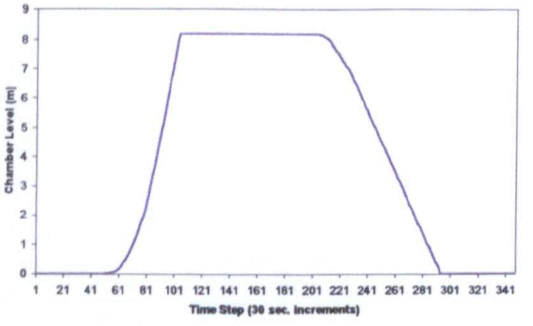


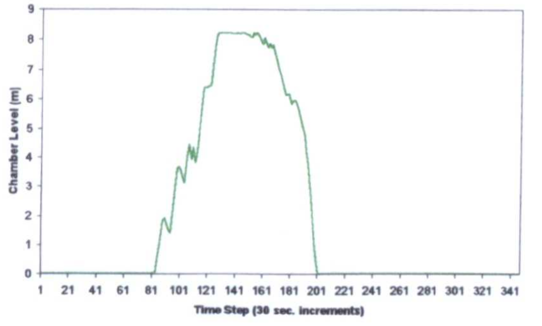
iii. Pollutant Spill Load.



iii. Pollutant Spill Load.



iv. Chamber Levels.

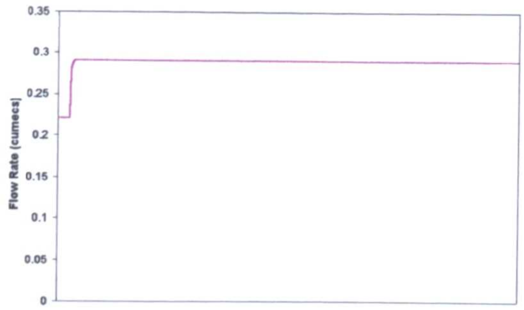


iv. Chamber Levels.

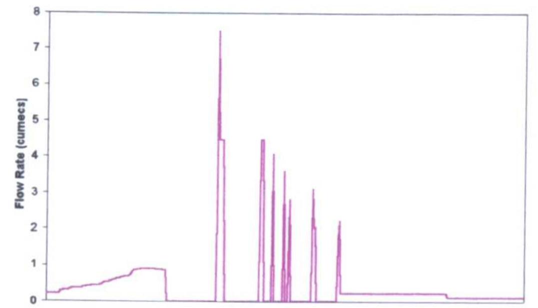
a. Fixed Local Control (FLC). **b.** Optimal Pollution Control (OPC).

Figure 6.10: Comparison between the Fixed Local Control (FLC) (**a**) and Optimal Pollution Control (OPC) (**b**) Strategies for the Northern Catchment (Intercept Point 5).

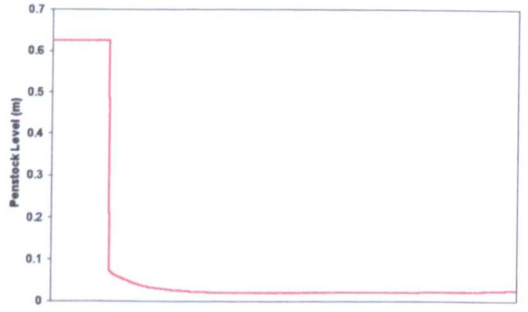
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



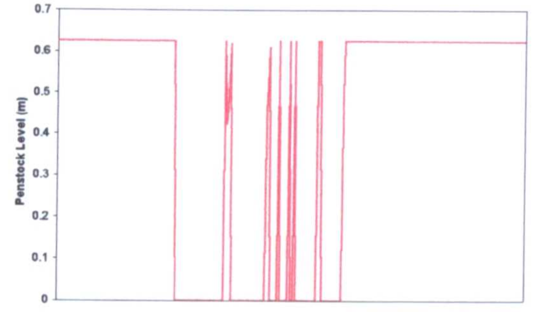
i. Flow Rate Control Strategy.



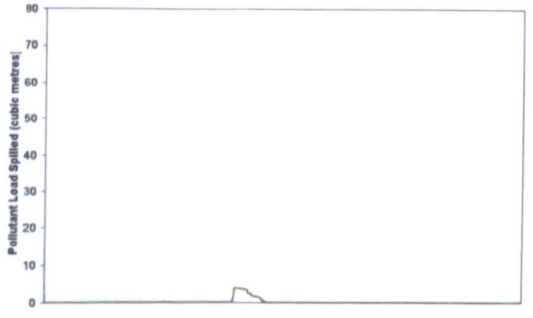
i. Flow Rate Control Strategy.



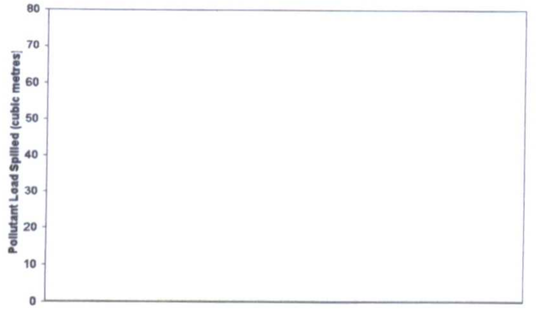
ii. Penstock Levels.



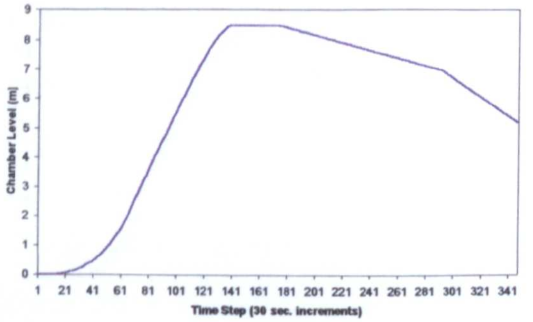
ii. Penstock Levels.



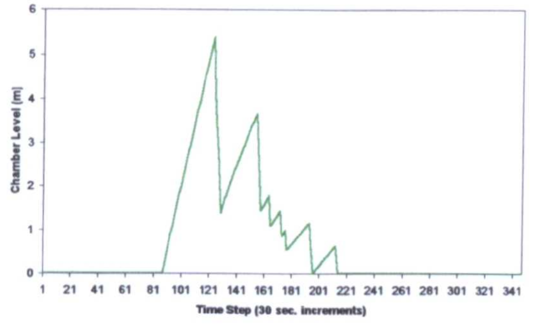
iii. Pollutant Spill Load.



iii. Pollutant Spill Load.



iv. Chamber Levels.

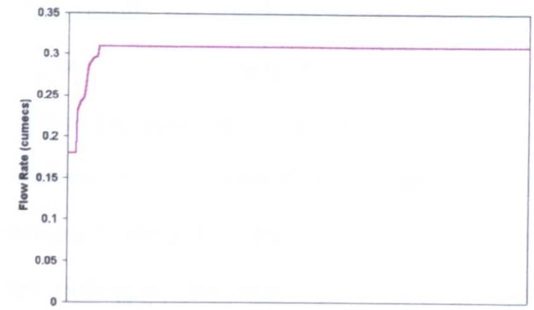


iv. Chamber Levels.

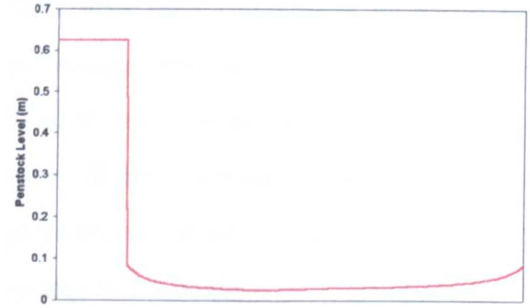
a. Fixed Local Control (FLC). **b.** Optimal Pollution Control (OPC).

Figure 6.11: Comparison between the Fixed Local Control (FLC) (**a**) and Optimal Pollution Control (OPC) (**b**) Strategies for the Bankhall Catchment (Intercept Point 6).

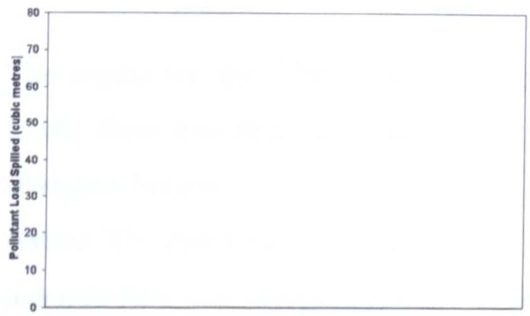
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



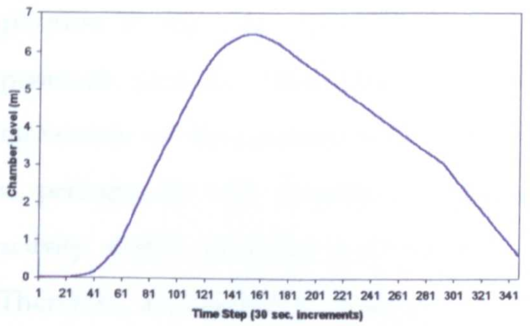
i. Flow Rate Control Strategy.



ii. Penstock Levels.

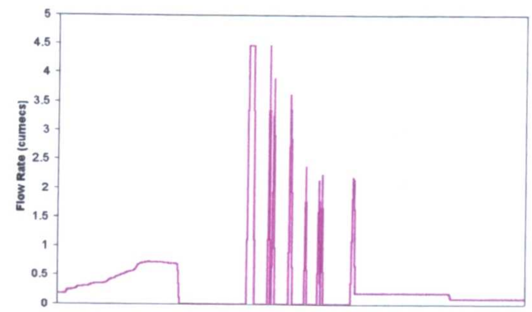


iii. Pollutant Spill Load.

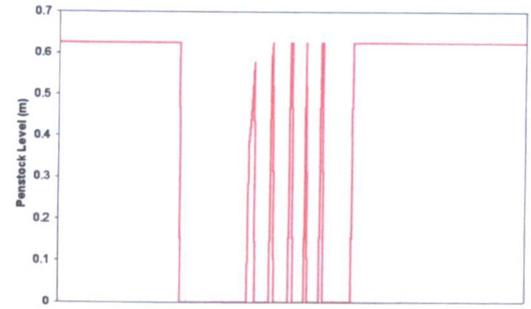


iv. Chamber Levels.

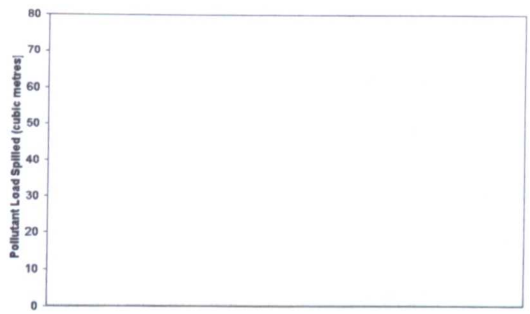
a. Fixed Local Control (FLC).



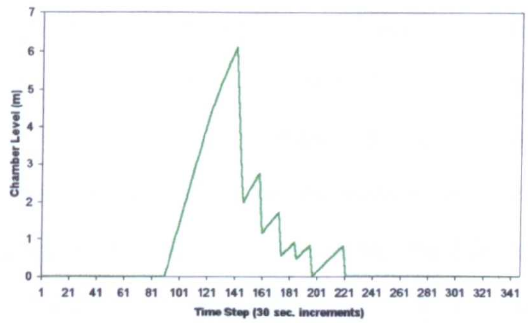
i. Flow Rate Control Strategy.



ii. Penstock Levels.



iii. Pollutant Spill Load.



iv. Chamber Levels.

b. Optimal Pollution Control (OPC).

Figure 6.12: Comparison between the Fixed Local Control (FLC) (**a**) and Optimal Pollution Control (OPC) (**b**) Strategies for the Sandhills Lane Catchment (Intercept Point 7).

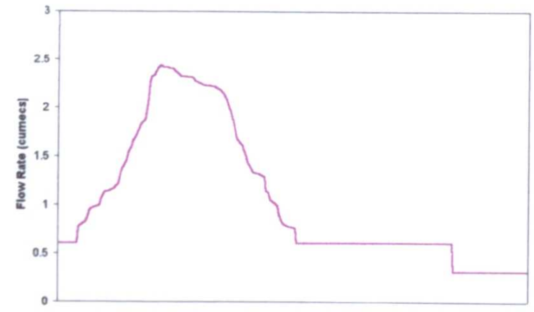
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)

The results for the FLC strategies (Figures 6.6a, 6.7a, 6.8a, 6.9a, 6.10a, 6.11a and 6.12a) show that the control procedure only responds to the level of the chamber. The penstock gate is controlled to allow up to the maximum inflow rate into the interceptor as set by the fixed inflow setting for each chamber. For example, the fixed inflow setting for the Rimrose catchment was 1.24 cumecs and the penstock moved, according to the chamber level, allowing up to this flow rate (when it was achievable). Therefore, the flow rate control strategies for the FLC procedure are fairly constant with very little movement in the penstock levels. The penstock levels lower with increasing head in the chambers to achieve the fixed inflows and then rise when the head in the chambers lowers. This is a common feature to all the FLC results where they all have similar 'filling' and 'emptying' characteristics. Obviously, this has a performance improvement over static control (static orifice) because the fixed inflow setting is achievable for longer periods with FLC. The fixed inflow setting can only be achieved with static control under design loads, i.e. when the chamber is full.

The results for the OPC strategies (Figures 6.6b, 6.7b, 6.8b, 6.9b, 6.10b, 6.11b and 6.12b) show that there are considerably more variations in the OPC flow rate control strategies because it used global information about the state of the interceptor sewer system. The penstock level responds to not only the level of the chamber but also the required flow rate setting from the control strategy. There are no restrictions on the flow rate control strategy within the control model so the penstock gate may be at any position in any time step. The model also assumes instantaneous changes in the penstock position. Illustrating the extreme, for example, the model allows the movement of the penstock from fully closed to fully open in a single time step. This is, perhaps, the only drawback of optimal active control since an increase in control activity would probably increase the frequency of penstock operating problems. Therefore, a compromise is needed where constraints are used within the optimisation routine to reduce the activity of the penstocks to within acceptable limits even though the resulting solution would then be sub-optimal. The inclusion of such additional constraints is a topic of further research.

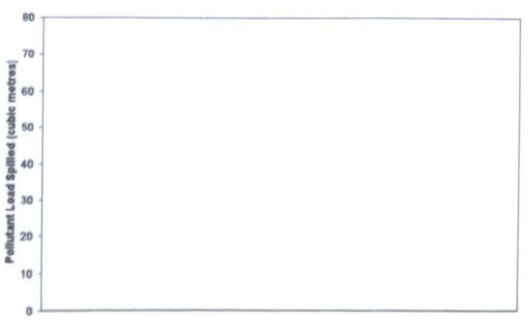
The results show that the OPC procedure improves the chamber recovery times compared to the FLC procedure. The chamber levels from the FLC procedure for Strand Road (Figure 6.7a), Bankhall (Figure 6.11a) and Sandhills Lane (Figure 6.12a) catchments do not fully recover from the storm event. The chamber levels from the OPC procedure fully recover by the end of the control time horizon. Therefore, the OPC procedure should be better able to control multiple-peak inflow hydrographs or pollutographs compared to the traditional FLC procedure.

Figures 6.13a through to 6.13g show the results for the VLC procedure for each catchment along the interceptor. The results are presented in a similar fashion to Figures 6.6 to 6.12, i.e. flow rate control strategy, penstock levels, pollutant load spilled and chamber levels.

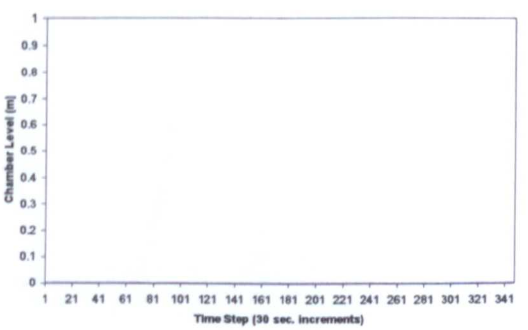


i. Flow Rate Control Strategy.

ii. Penstock Levels.



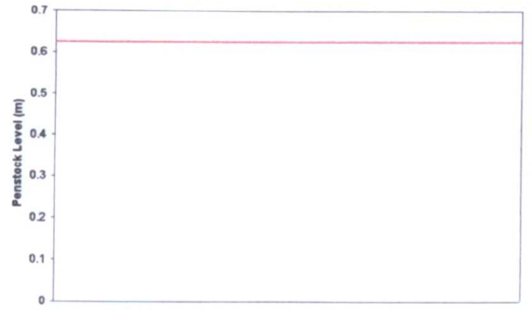
iii. Pollutant Spill Load.



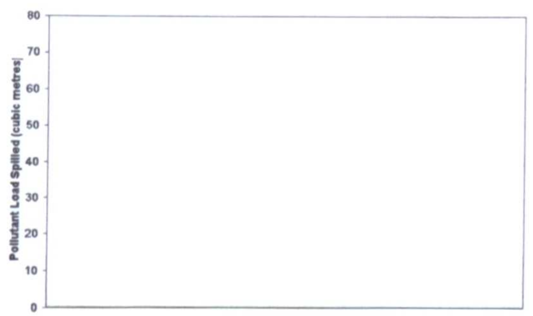
iv. Chamber Levels.

a. Rimrose Catchment (Intercept Point 1).

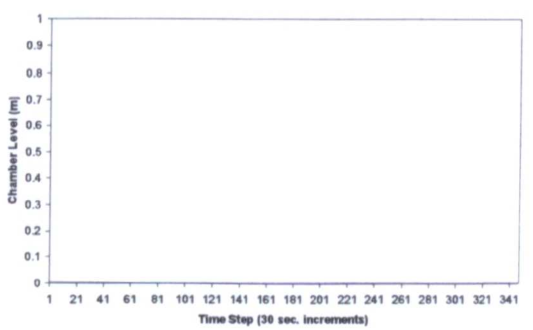
i. Flow Rate Control Strategy.



ii. Penstock Levels.



iii. Pollutant Spill Load.

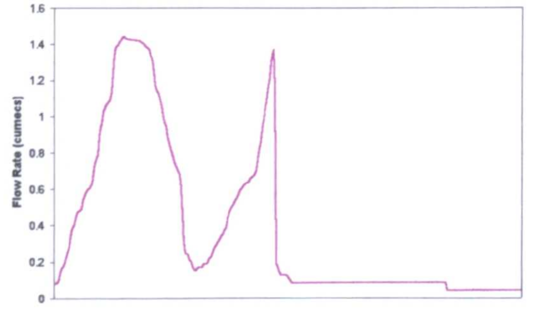


iv. Chamber Levels.

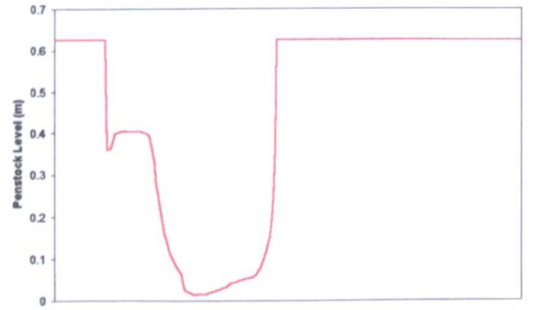
b. Strand Road Catchment (Intercept Point 2).

Figure 6.13: Variable Local Control (VLC) Strategies for the Rimrose (Intercept Point 1)(**a**) and Strand Road Catchments (Intercept Point 2)(**b**).

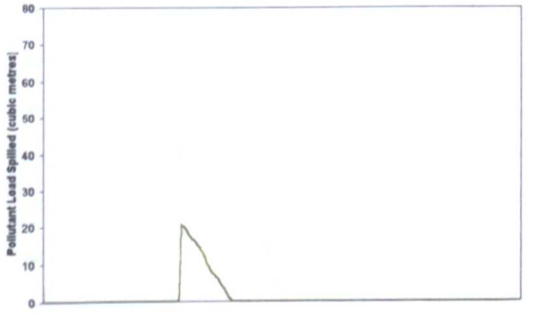
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



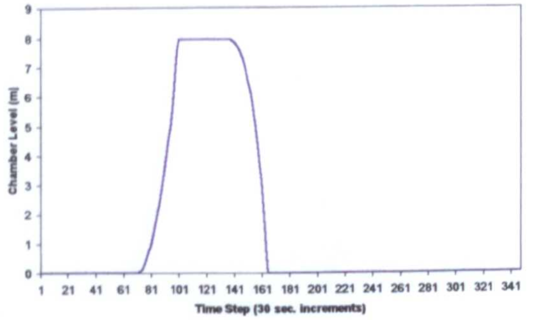
i. Flow Rate Control Strategy.



ii. Penstock Levels.

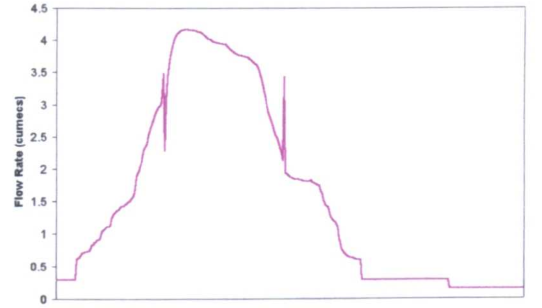


iii. Pollutant Spill Load.

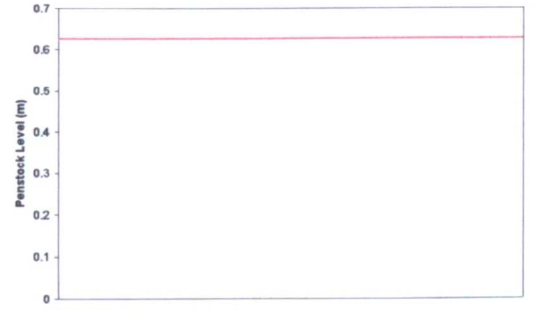


iv. Chamber Levels.

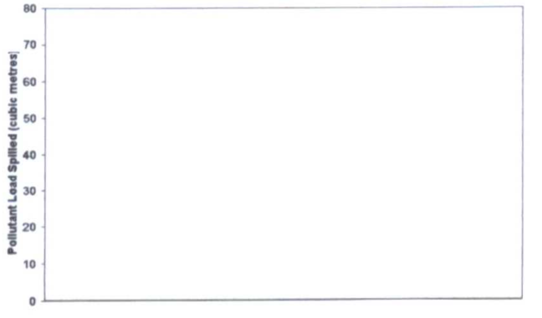
c. Millers Bridge/Fazakerley WwTW Catchment (Intercept Point 3).



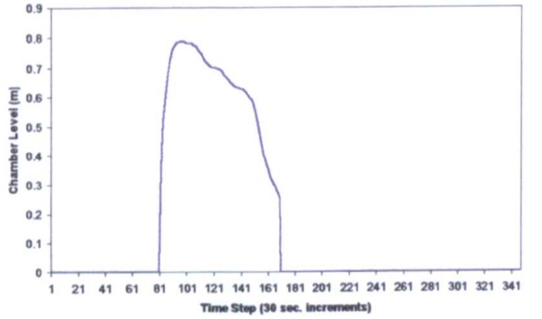
i. Flow Rate Control Strategy.



ii. Penstock Levels.



iii. Pollutant Spill Load.

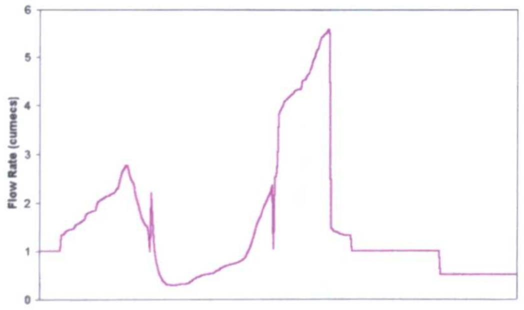


iv. Chamber Levels.

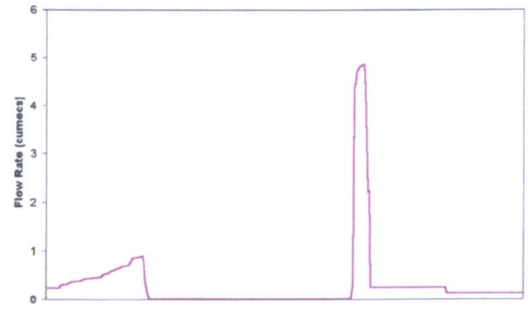
d. Bankhall Relief Catchment (Intercept Point 4).

Figure 6.13: Variable Local Control (VLC) Strategies for the Millers Bridge/Fazakerley WwTW (Intercept Point 3)(**c**) and Bankhall Relief Catchments (Intercept Point 4)(**d**).

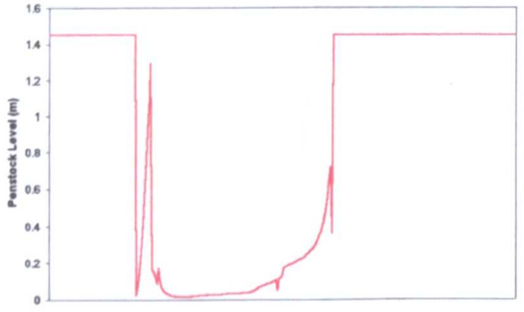
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



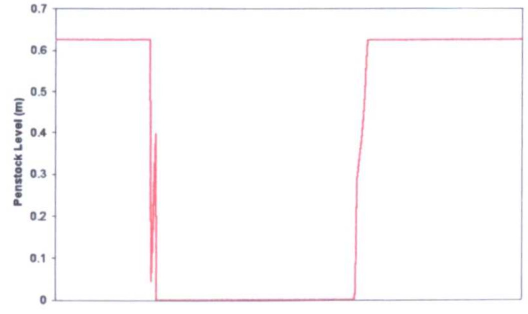
i. Flow Rate Control Strategy.



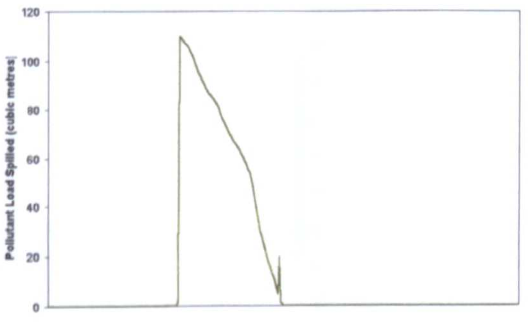
i. Flow Rate Control Strategy.



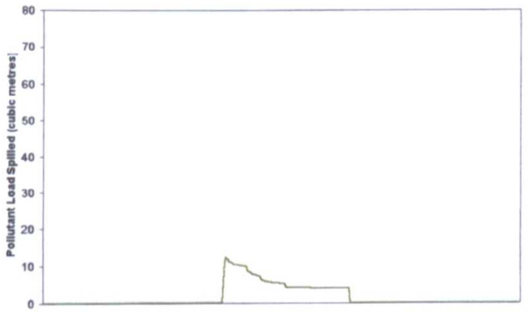
ii. Penstock Levels.



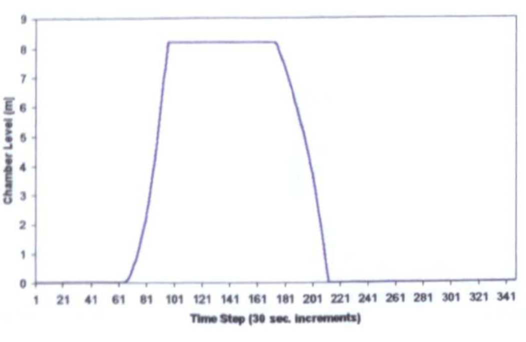
ii. Penstock Levels.



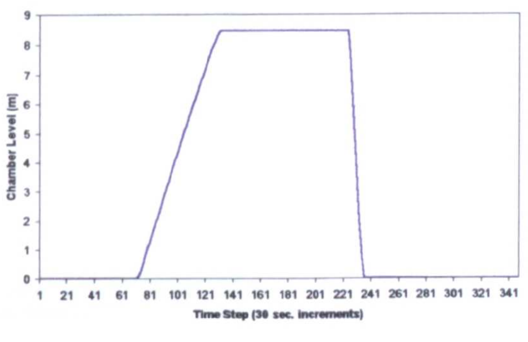
iii. Pollutant Spill Load.



iii. Pollutant Spill Load.



iv. Chamber Levels.

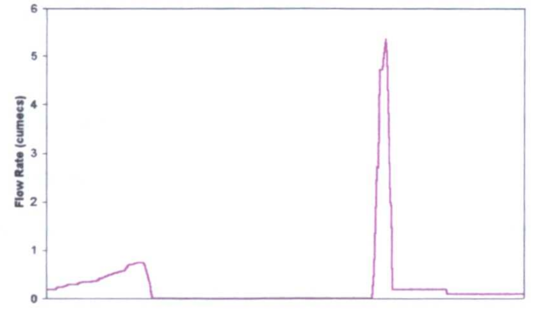


iv. Chamber Levels.

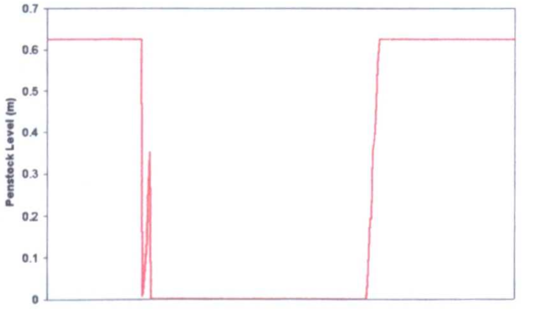
e. Northern Catchment (Intercept Point 5). **f.** Bankhall Catchment (Intercept Point 6).

Figure 6.13: Variable Local Control (VLC) Strategies for the Northern (Intercept Point 5)(**e**) and Bankhall Catchments (Intercept Point 6)(**f**).

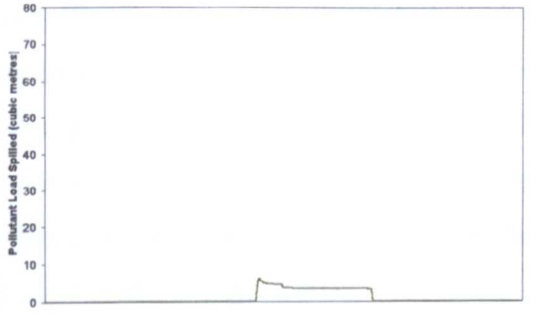
(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)



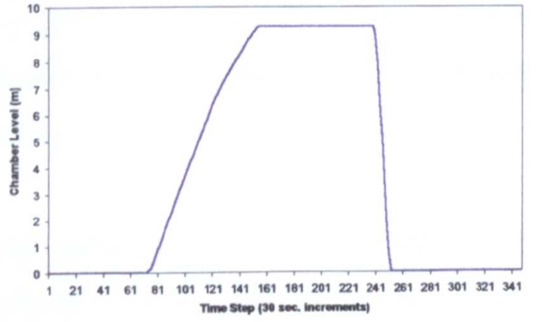
i. Flow Rate Control Strategy.



ii. Penstock Levels.



iii. Pollutant Spill Load.



iv. Chamber Levels.

g. Sandhills Lane Catchment (Intercept Point 7).

Figure 6.13: Variable Local Control (VLC) Strategies for the Sandhills Lane Catchment (Intercept Point 7)(**g**).

(Pollutant Load Spilled = Spill Volume × Pollutant Concentration)

The results from the VLC procedure in Figure 6.13 show that it always fully utilised the volumetric capacity of the interceptor pipe. Therefore, it has considerable improvements over the FLC procedure. However, the procedure does not take account of the pollutant concentrations in the CSO chambers. The results show that the control strategies from the VLC procedure filled the interceptor as soon as possible. This can be seen in Figures 6.13a and 6.6b where the chambers at the upstream section (Rimrose and Strand Road catchments) of the interceptor were not even used and, therefore, the penstocks were not activated. However, the chamber at the next intercept point (Millers Bridge/Fazakerley WwTW), in Figure 6.13c, was forced to over-spill because the interceptor was already full. The interceptor sewer pipe capacity then increases and the VLC strategy accepted all the flow from the Bankhall Relief catchment (Figure 6.13d). The interceptor was then full for the remainder of the downstream sections forcing over-spills at the Northern (intercept point 5)(Figure 6.13e), Bankhall (intercept point 6)(Figure 6.13f) and Sandhills (intercept point 7)(Figure 6.13g) chambers. This is an inherent problem of the VLC procedure; it does not account for the spatial aspects of sewerage systems. Therefore, the downstream sections of the interceptor sewer system spill more frequently. This could be improved with volumetric optimisation where coefficients could be introduced to, for example, spill from all the chambers evenly. Conversely, some penalty coefficients could be introduced to prevent over-spilling from certain chambers, perhaps into sensitive receiving waters. The VLC procedure does not allow for such control definitions but the performance improvements (in terms of spill load reduction) will be of a similar level to those of volumetric optimisation because both methods fully utilise the interceptor storage capacity.

There are some curious aspects in some of the results for the VLC procedure, particularly where the chamber starts to fill and empty. An example can be seen in Figure 6.13d where there are two rapid spikes in the flow rate control strategy. These occur because the chamber level fills beyond the maximum level of the orifice. The chamber level in the Bankhall Relief chamber (Figure 6.13d) was empty until time step 81. Therefore, the simplified method for calculating the maximum possible continuation flow was used, i.e. chamber volume divided by the time step, and the flow rate control strategy is identical to the inflow hydrograph. However, just beyond time step 81 the chamber level fills just above the maximum level of the orifice and here the maximum continuation flow rate is governed by equation (6.3). Consequently, there is a sudden drop in the control strategy where the model changes between methods. Equation (6.3) governs the flow rate control strategy until the chamber level reduces to the maximum level of the orifice around time step 150. From here the chamber level reduces and the simplified method was used to calculate the maximum possible continuation flow rate. It is possible for the VLC procedure to completely empty the chamber in one time step using this method. This explains the second spike in the flow rate control strategy in Figure 6.13d where the chamber was emptied around time step 170. The flow rates later in the control strategy follow the inflows from the catchment, which were allowed directly into the interceptor. There are a few of these apparent errors at some of the other intercept points, which are also explained as above. Obviously, this has implications on the decisions further downstream. This can be seen in Figure 6.13e, where there was a sudden drop at the start of the second peak in the flow rate control strategy. This is because of the second sudden spike in the flow rate control strategy for the intercept point upstream (Figure 6.13d). Therefore, the interceptor was suddenly fuller, so the flow rate in the control strategy in the next intercept point (intercept point 5) reduced accordingly.

Storm Condition	*Pollutant Load to Receiving Waters* *(Spill Volume × Pollutant Concentration)*			*Improvements*		
	Fixed Local Control (FLC)	Variable Local Control (VLC)	Optimal Pollution Control (OPC)	OPC v FLC %	OPC v VLC %	VLC v FLC %
Low Severity	8860.06	6214.36	5324.15	39.91	14.33	29.86
Medium Severity	33495.06	29160.08	27256.33	18.63	6.53	12.94
High Severity	109116.04	103520.90	100378.60	8.01	3.04	5.13

Table 6.3: Comparison of Control Procedures.

Table 6.3 shows comparisons between the control procedures. In the above application of a reasonably low intensity highly synchronised storm event (i.e. peak inflow to the system at approximately 2 times fixed inflow settings), the OPC procedure produced a

40% improvement in pollutant load spilled compared to the FLC procedure. The VLC procedure produced a 30% improvement compared to the FLC procedure. The OPC procedure also produced a 14% improvement compared to VLC. This improvement was only based on the introduction of pollutant concentrations into the control model, illustrating the significance of pollutant-based control. It should be noted that the hydraulic characteristics of the continuation pipes were not included in the OPC or VLC procedures and it was assumed that these pipes could convey flows up to the pipe full capacity of the interceptor. The continuation pipes generally have lower capacities than the interceptor and so the results presented may show these control procedures more favourably. This potential exaggeration is evaluated in Chapter 7.

Table 6.3 also shows the results from a medium and high severity storm. These storms were generated in a similar fashion to those used in Chapter 5, where the inflow rates from low intensity storm were multiplied by 2 to obtain the medium storm and 5 to obtain the high storm. The full results for these storms are not presented here but have been included in Table 6.3 to indicate the effects that storm severity has on the improvements. It can be seen that the improvements of using the OPC and VLC procedures reduce as the severity of the storm event increases. This is expected because spills are inevitable with larger inflows where the interceptor sewer and overflow chamber storage are quickly utilised. It is expected, therefore, that most improvements will be gained in the low to moderate rainfalls that occur more frequently.

The inclusion of overflow chambers has increased the computational demand of the optimal pollution control model because equation (6.3) is solved numerically between time step solutions to determine the actual control strategies on the penstocks. However, the effect is not found to be too prejudicial because the model runs considerably faster than real time.

6.4 CONCLUDING REMARKS

This chapter has described the extension of the optimal pollution control model to real interceptor sewer systems that include CSO chambers. The extended model continued to use the slug flow approach to convey the sewage through the interceptor sewer whilst retaining the computational efficiencies of the method.

The formulation of the optimal pollution control model was little changed from the original model. The model now responds to the pollutant concentrations within the overflow chambers along the interceptor not the pollutant concentrations of the inflows from the catchments. Therefore, the model allows flows from the most polluted CSO chambers into the interceptor and, consequently, over-spills the least polluted sewage into the receiving waters. The original control model solved each time step through the control time horizon successively, which allowed the inclusion of the non-linear equations that govern the continuation flow through the overflow chambers between time step solutions. These equations were numerically solved yet their inclusion has not significantly reduced the computational efficiency of the model.

The results from the application of the extended optimal pollution control model on a simplified version of the northern leg of the Liverpool Interceptor Sewer have shown considerable reductions in pollutant load spilled when compared to traditional fixed local control. The results also showed that reductions were achievable with the use of variable local control that fully utilised the storage in the interceptor sewer. However, further improvements were achievable with optimal pollution control. The results confirm basic intuition that the scale of reduction in pollutant load spilled decreases when the intensity of the rainfall increases.

CHAPTER 7

CASE STUDY – LIVERPOOL INTERCEPTOR SEWER SYSTEM

7.1 INTRODUCTION

The preceding chapters described the development of the optimal pollution control (OPC) model, its verification and extensions to include CSO chambers. Extensive results have been presented, which demonstrated the potential reductions in pollution over-spill loads on application of the OPC model. However, the storm events used hitherto were hypothetically generated. This chapter describes a case study where the OPC model is applied to the Liverpool Interceptor Sewer with historical rainfall events.

Initially, a brief description is given of the history of the Liverpool sewerage system and the Mersey Estuary Pollution Alleviation Scheme (MEPAS), from which the case study was generated. A typical year of rainfall is used as inputs into a Unit Hydrograph model to generate inflows into the Liverpool Interceptor. Additionally, the corresponding time-varying pollution concentrations are determined using a simplified method to determine the peak suspended solid concentration for each storm event in a typical year. These methods are thoroughly described. The inflows and pollutant concentrations are used on the case study system to determine the control actions within a typical year of rainfall in Liverpool. Obviously, the quantity of results is vast for such a range of events and, consequently, only summary results are presented giving overall comparisons sufficient to demonstrate the full implication of the findings.

7.2 DESCRIPTION OF THE LIVERPOOL SEWER SYSTEM

7.2.1 Historical Background[1]

Before any sewers were built in Liverpool, natural streams provided the only source of effluent disposal. The Pool, shown in Figure 7.1, was the main drain for Liverpool and pollution was a problem even in 1700 and fines were imposed on trades and industries that polluted it.

Figure 7.1: Liverpool in 1650 (Taken from Olsen, 1997).

Inadequate sanitation led to high levels of illness in Liverpool and periodic epidemics of fatal diseases such as cholera and typhoid occurred throughout the 19th Century. Liverpool's problems were particularly pressing as rapid population growth, especially after 1840, led to severe overcrowding. Cholera epidemics broke out in 1832, 1849, 1854 and 1866 and, in 1848, a series of other epidemics occurred. Between 1830 and 1840, 20 miles of sewers were built but by 1846, there were still only 56 miles of sewers

[1] Extracted from Moore (1998) and Olsen (1997).

in a town with a population of 251, 000. There were seven outfalls to the River Mersey and these drained only half of Liverpool's eight square miles.

The 'Sanitary Act' of 1846 provided the framework for improving sewers and drainage systems, the paving of roads and various sanitary improvements in Liverpool. The city also appointed its first Medical Officer of Health, Dr. Duncan. In 1847, when the act actually came into force, James Newland was appointed as City Engineer. His sewer construction programme began in 1848 and, during the next eleven years, 86 miles of sewers were built. Another 58 miles were built between 1856 and 1862 with the programme completing in 1869.

Liverpool absorbed outer boroughs or districts in 1895 and 1913, and in doing so inherited sewerage systems outfalling to treatment works at Walton and Fazakerley draining to the River Alt.

It soon became clear that the River Mersey could not cope with the sewerage from the towns and conurbations along its length. Not only did Liverpool, during the 1960s, discharge about 35 million gallons of untreated sewage into the lower Mersey each day, but the upper Mersey received sewage from the Manchester area and numerous towns and industries along its banks. By 1980, the dissolved oxygen levels at Widnes had fallen to zero. As the Mersey widens into the basin, the river water merged with the sea and the oxygen levels rose again. Raw sewage could frequently be seen in the River. New Brighton had been eclipsed as a resort and fishing was non-existent in many places.

7.2.2 Mersey Estuary Pollution Alleviation Scheme (MEPAS)

There were several options available for the alleviation of pollution in the River Mersey. The eventual choice and approval of a waterfront site at Sandon Dock enabled the design of the interceptor sewers, storm overflows and outfall penstock chambers as part of the Mersey Estuary Pollution Alleviation Scheme (MEPAS). An extensive description of this scheme is presented in Olsen *et al.* (1999), especially on the design and construction phases. Of particular relevance, Olsen *et al.* (1999) discuss the improvements on the River Mersey since the implementation of the interceptor sewer system and wastewater treatment works.

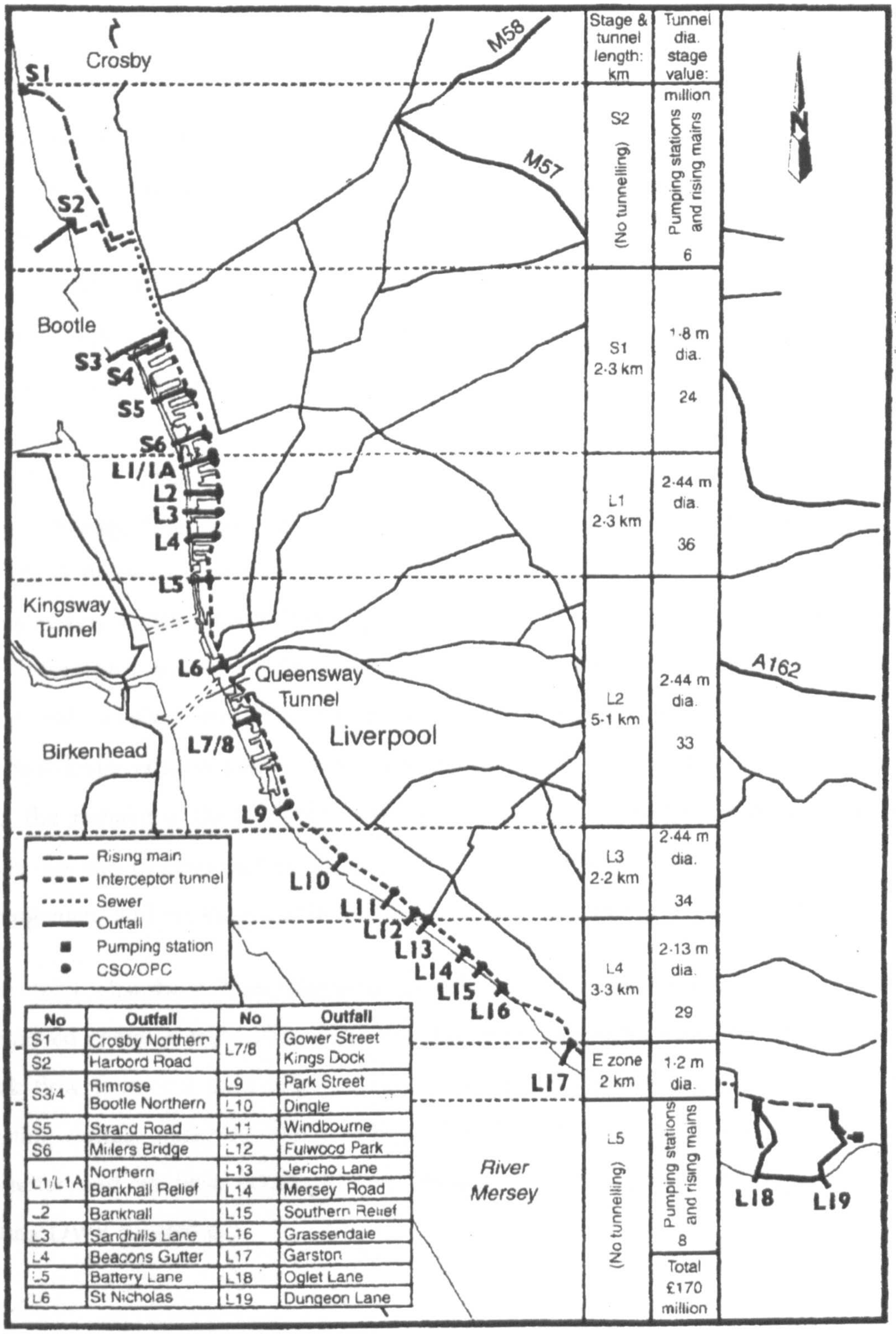


Figure 7.2: Liverpool Interceptor Sewer System and Outfalls (Taken from Olsen *et al.*, 1999).

Figure 7.2 shows the principal drainage areas and the route of the MEPAS interceptor sewer, extending some 29.3km with 26 major outfalls.

7.2.2.1 *Objectives of the MEPAS Interceptor Sewer System*

The objectives of the East Bank Interceptor Sewers project were (Leatherbarrow, 1993):

- to ensure that all dry weather flows and the "first foul flush" of storm flows were dispatched for treatment;
- to minimise the number, duration and volumes of overspills and reduce the polluting effects of any overspill events without increasing the risk of flooding; and
- to ensure that no more than a predetermined flow from each outfall entered the Interceptor Sewer in order to prevent surcharge or inundation of the treatment works and its pumping station.

7.2.2.2 *Details of the Interceptor Sewer System*

Storm Sewage Overflow (SSO) chambers were constructed on each outfall through which all flows pass into the lower level Interceptor Sewer or overspill into the River Mersey. Dry weather flows (DWFs) and some of the surface flows pass through into the Interceptor and on to the Sandon Dock WwTW via a control device (a penstock gate) and usually a vortex measurement device/dropshaft. During storm events the control device ensures that no more than the pre-set flow passes forward for treatment and the remaining flow initially is retained in the SSO before overspilling into the River via the original outfall. Figures 7.3 and 7.4 show a typical overflow chamber arrangement and the longitudinal section of the interceptor sewer respectively.

The SSO chambers were designed to maximise the retention of solids, and so contained chamfer and baffle walls. The inlet and outlet pipe sizes were based on the peak flow from a 1 in 2 year storm, which generally gives a capacity similar to the existing outfall pipes. The proportions of the SSO's were related to the inlet pipe diameter and all pipe gradients were designed to ensure self-cleansing conditions during DWF conditions.

At the River, another control device was installed in an Outfall Penstock Chamber to prevent the ingress of river water into the sewerage system whilst ensuring the free discharge of storm flows.

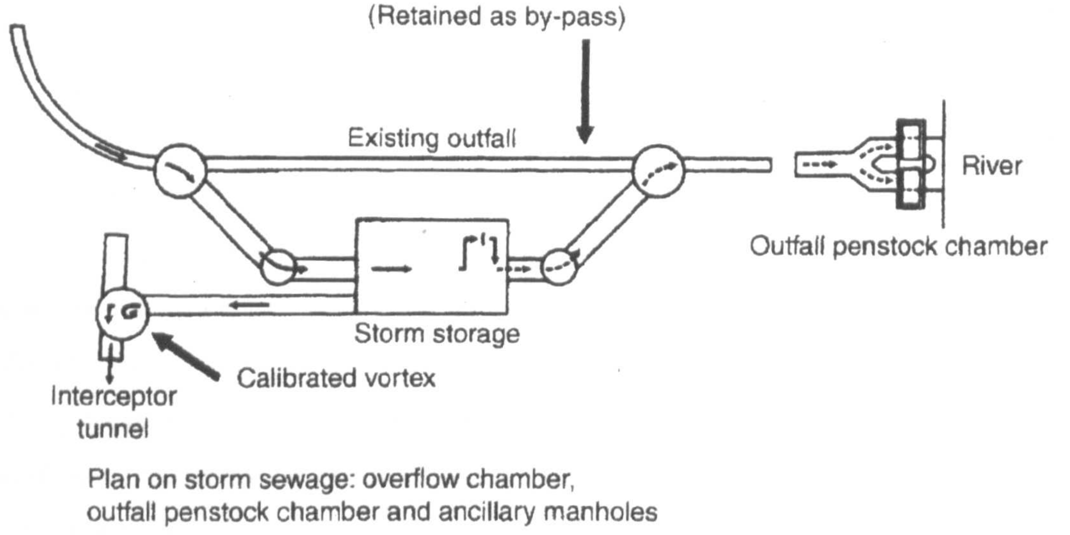


Figure 7.3: Typical Overflow Chamber Arrangement (Taken from Olsen *et al.*, 1999).

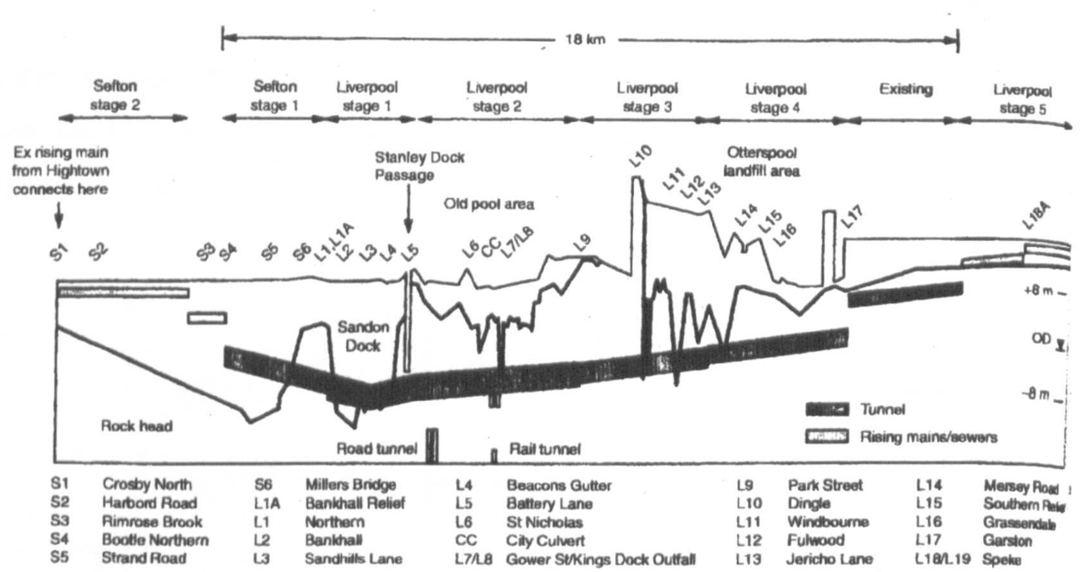


Figure 7.4: Longitudinal Section of the Liverpool Interceptor Sewer (Taken from Olsen *et al.*, 1999).

7.2.2.3 *The Control System*

The control system installed was a local reactive control system capable of being upgraded to a central system in the future. A modulating penstock gate was installed in each SSO with sensors, required to monitor levels and flows at various points in the system. These consisted of pressure and ultrasonic depth/level sensors, flow velocity sensors and position sensors on each penstock gate.

The flow management system limits "the flow to the Interceptor Sewer to a predetermined figure and to facilitate the discharge of excess flows to the River when the storage volume is exceeded whilst limiting such discharges wherever and whenever possible" (Leatherbarrow, 1993).

7.2.3 Case Study Interceptor System

The full Liverpool Interceptor System has not been used in this case study because of the lack of data. This, however, does not affect the validity of the conclusions of this chapter; it merely adds a scale factor. In fact, the improvements from this application of the optimal pollution control model are likely to be conservative when compared to its application on the full interceptor because of increased spatial distributions. Nevertheless, the model requires modifications to represent the full interceptor and more generic branched systems. These are described in the Recommendations for Further Work section.

In this case study the northern leg of the Liverpool Interceptor System (Figure 7.5) has been used to gain an understanding of the potential improvements with the application of the OPC model during historical rainfall events.

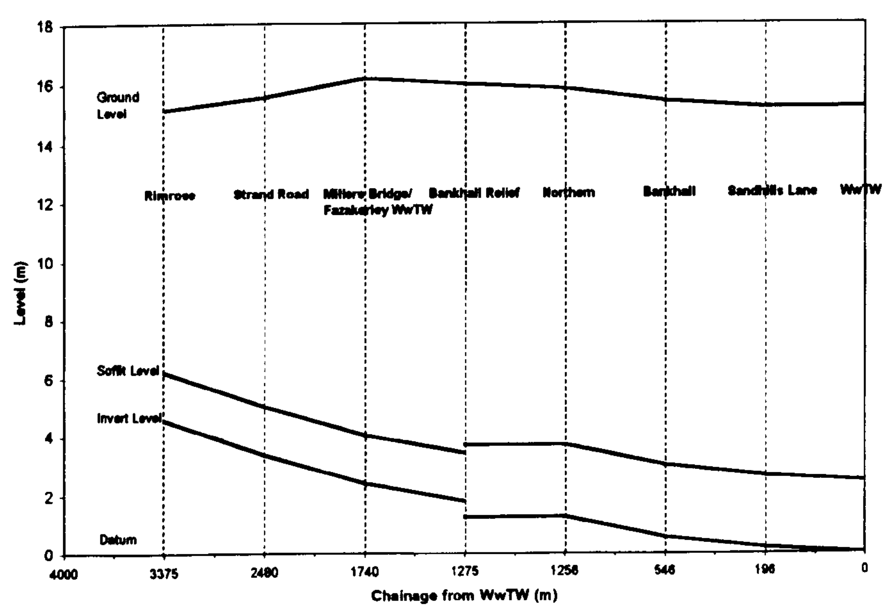


Figure 7.5: Longitudinal Section of the Northern Leg of the Liverpool Interceptor Sewer (not to scale).

Intercept Point (catchment)	*Sewer Diameter (m)*	*Sewer Gradient*	*Sewer Capacity (cumecs)*	*D.W.F. (cumecs)*	*Fixed Inflow Setting (cumecs)*
Rimrose	1.66	1/750	3.26	0.30	1.24
Strand Rd	1.66	1/750	3.26	0.09	0.25
Millers Bridge/Fazakerley WwTW	1.66	1/750	3.26	0.04	0.97
Bankhall Relief	2.44	1/1000	7.72	0.14	0.69
Northern	2.44	1/1000	7.72	0.50	2.13
Bankhall	2.44	1/1000	7.72	0.11	0.29
Sandhills Lane	2.44	1/1000	7.72	0.09	0.31

Table 7.1: Input Data for Interceptor Sewer.

Intercept Point (catchment)	*Chamber Area (m^2)*	*Spill Level [above invert level] (m)*	*Orifice Width (m)*	*Orifice Height (m)*
Rimrose	282.82	5.42	1.250	1.450
Strand Rd	136.03	6.91	1.700	0.625
Millers Bridge/ Fazakerley WwTW	50.31	7.95	1.500(E)	0.625(E)
Bankhal lRelief	169.78	8.04	2.075	0.625
Northern	328.24	8.18	2.650	1.450
Bankhall	167.06	8.47	1.800	0.625
Sandhills Lane	147.95	9.26	1.650	0.625

(E) – Estimated dimensions.

Table 7.2: Storm Chambers Input Data for Interceptor Sewer.

Tables 7.1 and 7.2 show the input data for the interceptor sewer and overflow chambers respectively.

Four control procedures were considered in this case study:

Fixed Local Control (FLC) – Inflows up to the fixed inflow setting (which is less than the interceptor sewer capacity locally) are passed forward to the interceptor regardless of conditions elsewhere in the sewer system, i.e. a volumetric control system where no account is taken of the pollutant load of the flows. This is the current control system in the Liverpool Interceptor Sewer.

Variable Local Control (VLC) – Inflows are permitted up to the interceptor sewer's capacity locally but no account is taken of the conditions elsewhere in the sewer system or the pollutant load of the flows.

Restricted Variable Local Control (Restricted VLC) – Inflows are permitted up to the capacity of the connection pipe between the overflow chamber and the interceptor sewer at each intercept point (which is slightly greater than the fixed inflow setting in this case study). No account is taken of conditions elsewhere in the sewer system or the pollutant load of the flows. This procedure has been included to evaluate the bias in the improvements made by the *VLC* and *OPC* procedure, as discussed in Section 6.3.

Optimal Pollution Control (OPC) – The (extended) optimal pollution control model determines inflows using global information including pollutant concentrations by the LP solution procedure.

7.3 RAINFALL INPUTS

A typical year of rainfall has been used in this application of the OPC model. One month's rainfall data is presented in Figure 7.6 as an example, which is the rainfall series for March in 1956. These events were taken from the South West Time Series Rainfall Data (WRC, 1986) and were not regionalised for any catchment. The data was used in condensed form as given (i.e. the periods of dry weather flow were not included) without favourably prejudicing the results since no spills arise in the omitted periods, because of data management problems. The data files utilised a considerable amount of computer memory and the inclusion of DWF data would have exacerbated this problem by an order of magnitude.

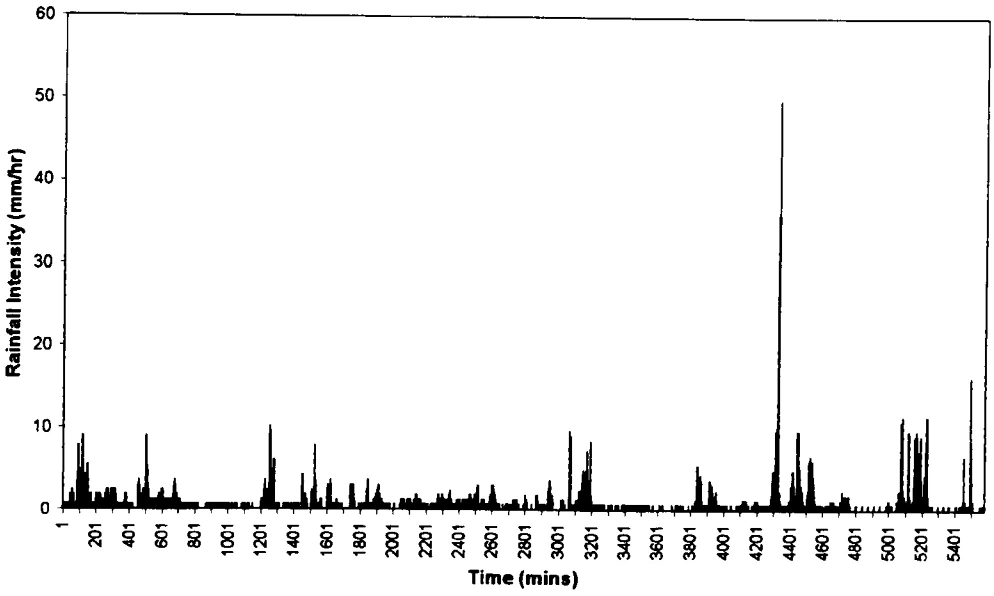


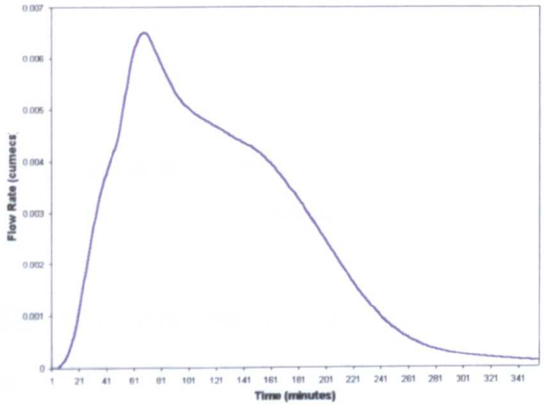
Figure 7.6: An Example of the Typical Year Rainfall Series for Liverpool (March).

7.4 DETERMINATION OF INFLOW HYDROGRAPHS

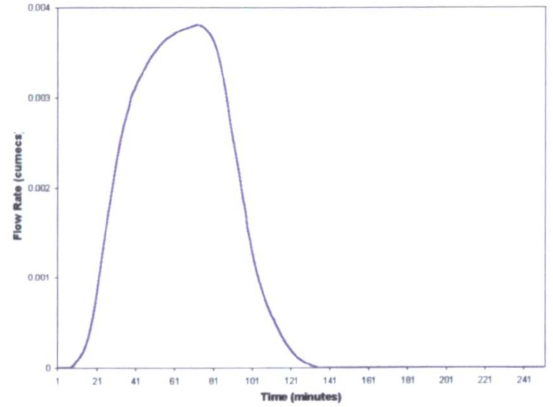
A Unit Hydrograph (UH) procedure has been used to determine the inflow hydrographs from the catchments to the interceptor sewer. The UH method has been described in Section 2.2.2.3, where the model COSSOM was introduced. This UH model was developed by Mehmood (1995) (see, also, Burrows *et al.*, 1995) for the long-term simulation of the operation of sewerage systems. The Unit Hydrographs were obtained from a preliminary application of an advanced hydraulic flow model. Mehmood (1995) advises that the maximum uniform rainfall intensity value in the rain hyetograph data be used with duration approximately equal to the time of concentration (Tc) of the catchment under consideration.

Mehmood (1995) gives a detailed description of the runoff synthesis using the UH approach. In essence, the UH procedure treats each catchment as a linear system whose input is a rainfall series with a specified time step of T minutes. Its output (i.e. outflow) is calculated as the convolution of the given rainfall series with the T-minute unit hydrograph.

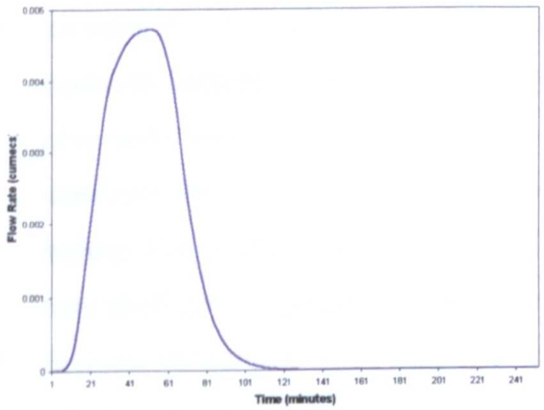
The Unit Hydrographs in Figure 7.7 were generated by applying the above method to verified HydroWorks models of the catchments. Unfortunately, at the time of this study, only four of the seven catchments contributing to the northern leg of the Liverpool Interceptor Sewer had been modelled by Liverpool City Engineers who supplied the relevant data files. Consequently, the three remaining Unit Hydrographs were estimated.



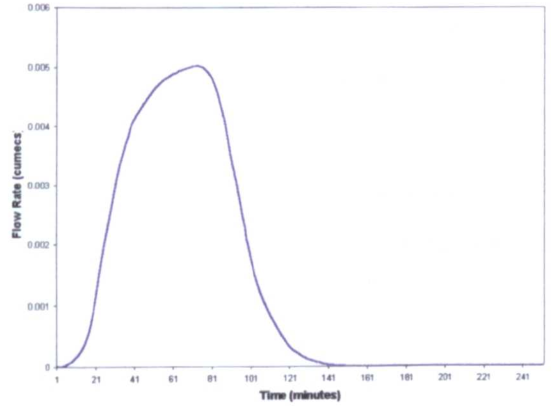
a. Rimrose Catchment (Estimated from Northern UH).



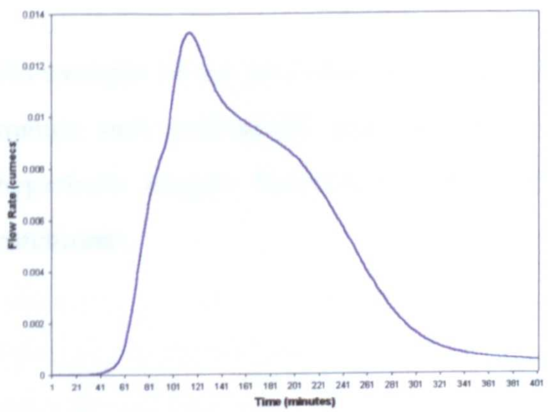
b. Strand Road Catchment (Estimated from Bankhall Relief UH).



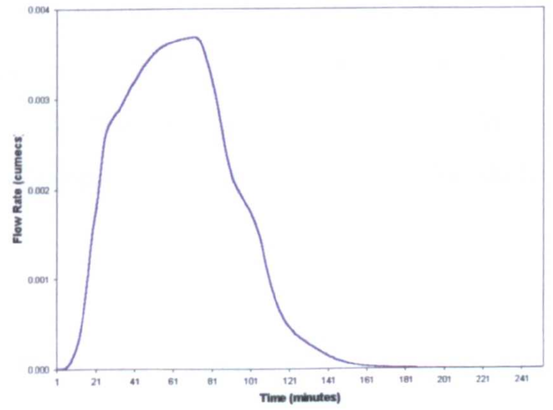
c. Millers Bridge/Fazakerley WwTW Catchment (Estimated from Sandhills Lane UH).



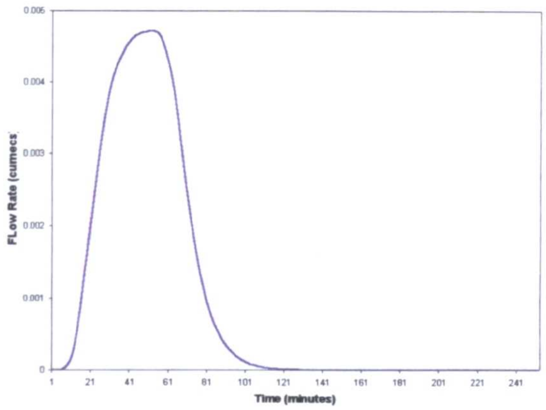
d. Bankhall Relief Catchment (derived from a rainfall intensity of 14mm/hr for 60mins).



e. Northern Catchment (derived from a rainfall intensity of 10mm/hr for 90mins).



f. Bankhall Catchment (derived from a rainfall intensity of 14mm/hr for 60mins).



g. Sandhills Lane Catchment (derived from a rainfall intensity of 17mm/hr for 40mins).

Figure 7.7: Unit Hydrographs for Each Catchment.

The UH model has been programmed in Matlab code and is described in Najafian *et al.* (1998). The UH model here allows the determination of a Tr-minute unit hydrograph, where Tr is the rainfall time step, which is derived from the Tc-minute unit hydrograph. Najafian *et al.* (1998) describe the procedure as:

- calculate the Tc-minute unit hydrograph from WALLRUS (or any other detailed hydraulic simulation model) for a rainfall intensity close to the maximum in the observed record;
- construct the S curve (S function) from the Tc-minute unit hydrograph by linear superposition. The S curve is the response of a linear system to a step function – here the S curve is simply the outflow due to a uniform rainfall;
- fit a smooth curve to the foregoing S curve; and
- calculate the Tr-minute unit hydrograph from the smooth theoretical S curve by employing a standard lagging technique.

An example of this procedure is shown in Figures 7.8 to 7.10. Figure 7.8 shows the Tc-minute unit hydrograph and theoretical unit hydrograph (in this case fitted by a hyperbolic tangent function as adopted by Najafian *et al.* (1998)) for the Bankhall catchment.

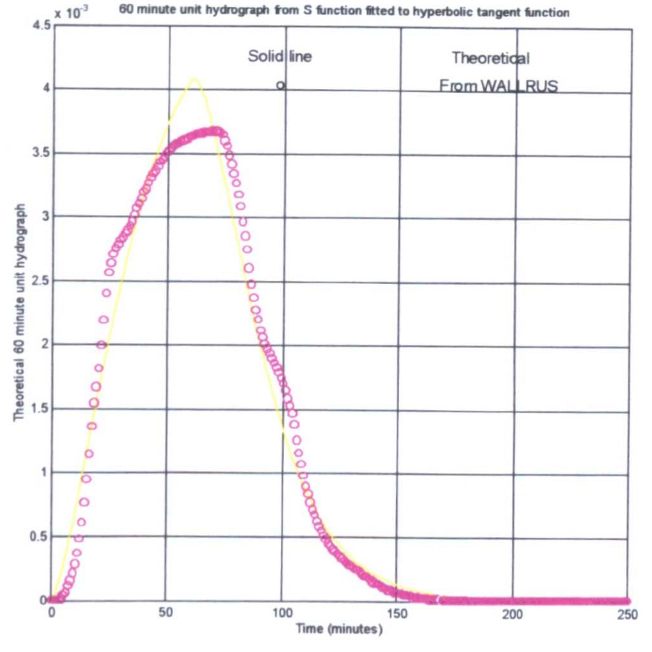


Figure 7.8 Sample Results from the Matlab UH Model showing the Theoretical UH and Hydroworks UH (labelled as WALLRUS) for the Bankhall Catchment.

Figure 7.9 shows the theoretical and Hydroworks S curves, which are expected to rise from zero to an equilibrium value at the time of concentration Tc. In practice, the resultant equilibrium value oscillates, as shown in Figure 7.9, about its theoretical value due to the periodic nature of the superposition used in the unit hydrograph procedure. This behaviour is unrealistic for a uniform rainfall. The problem can be resolved by by fitting a theoretical smooth curve to the S curve. This procedure is described in Najafian *et al.* (1998).

Figure 7.10 shows the resultant Tr-minute unit hydrograph, which in this case is the one-minute hydrograph for the Bankhall catchment, the recommended application being for computations to be made in 1 minute increments in time.

Once the Tr-minute unit hydrograph has been calculated, the outflow can be calculated as the convolution of the rainfall series with the Tr-minute unit hydrograph. Dry weather flow (DWF) is added as a constant value in this application although an alternative would be for proper (chronological) time records to be maintained so that diurnal variations in DWF could be added.

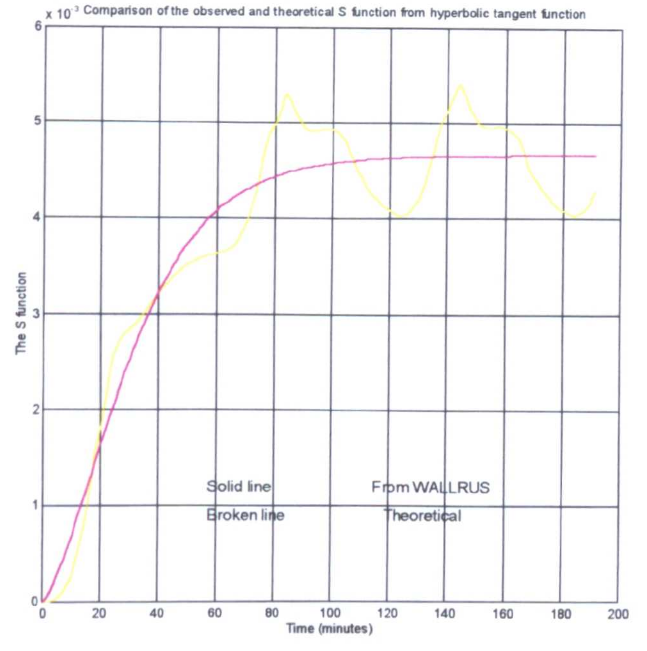


Figure 7.9: Sample Results from the Matlab UH Model showing the Theoretical and Actual S Curves for the Bankhall Catchment.

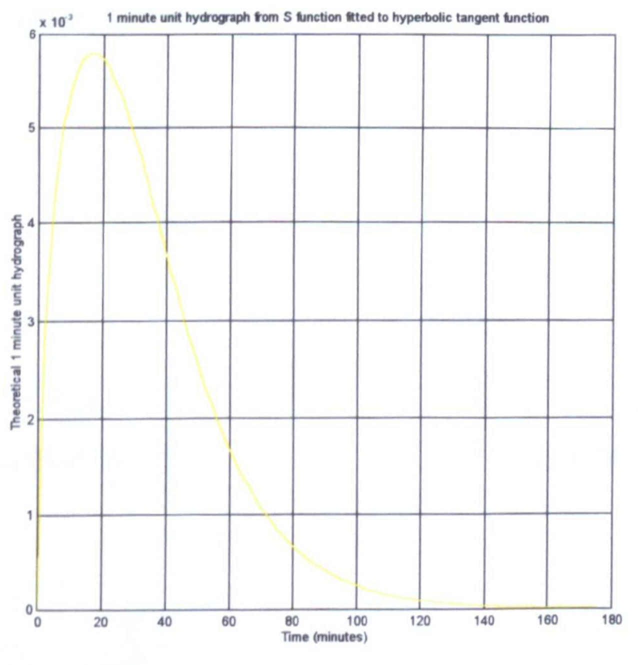
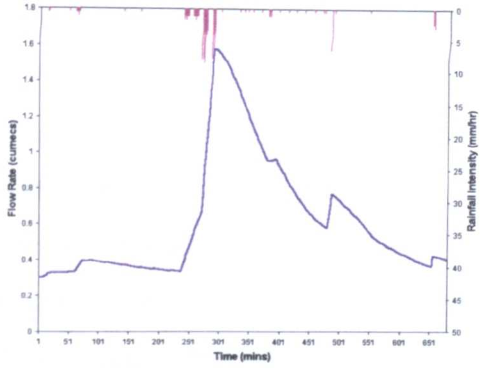
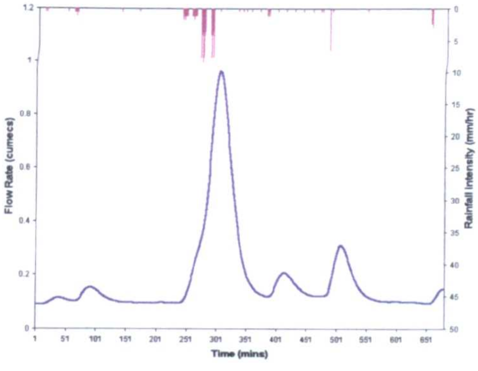


Figure 7.10: Sample Results of the One(Tr)-Minute Unit Hydrograph for the Bankhall Catchment.

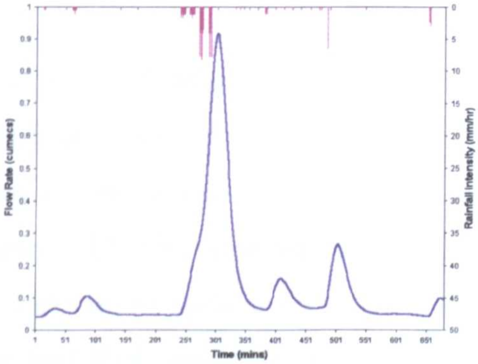
Figure 7.11 shows some sample hydrographs from the Matlab UH model for each catchment along the northern leg of the Liverpool Interceptor System. The examples shown in Figure 7.11 are from storm event one in the January rainfall data in the typical year.



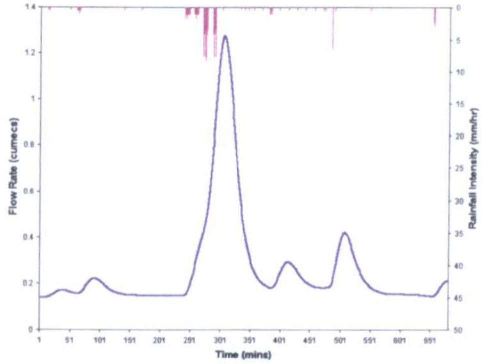
a. Rimrose Catchment.



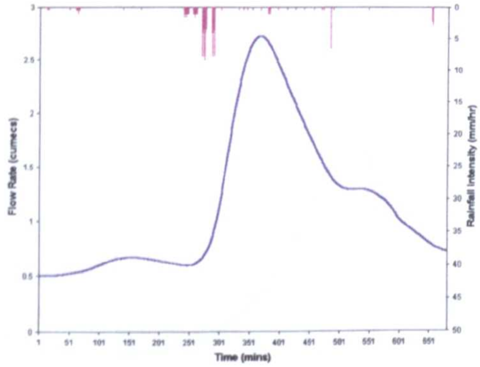
b. Strand Road Catchment.



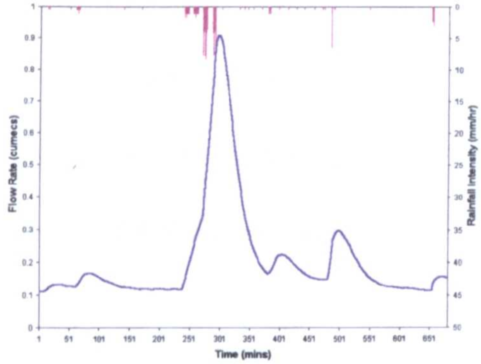
c. Millers Bridge/Fazakerley WwTW Catchment.



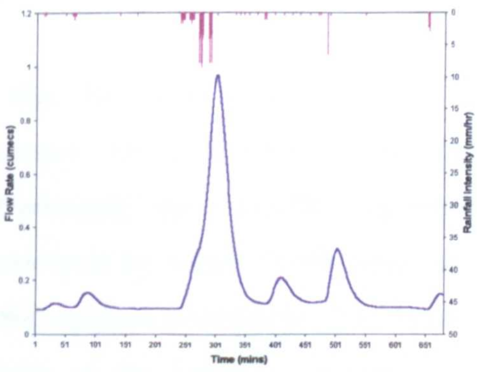
d. Bankhall Relief Catchment.



e. Northern Catchment.



f. Bankhall Catchment.



g. Sandhills Lane Catchment.

Figure 7.11: Sample Hydrographs from each Catchment for Storm Event One in the January Rainfall Data in a Typical Year.

7.5 DETERMINATION OF POLLUTANT CONCENTRATIONS

The aim of this study was to minimise total pollutant over-spill load from interceptor sewer systems, which requires not only the prediction of inflows but also the corresponding pollutant concentrations. A review of sewer water quality processes was given in Section 2.3, where various modelling techniques were described including the research undertaken by Gupta (1995). The pollutant concentrations are determined here using a simplification of the methodology developed by Gupta (1995).

The approach assumes that there is a base (DWF) level of pollutants, in this case suspended solids, throughout the storm event. The storm pollutant concentrations are then determined and added to the base pollutant level. This approach is shown in Figure 7.12. This approach does not fully represent the behaviour of storm pollutant concentrations because often the concentrations decrease to below DWF levels after the first flush period. However, the approach has been adopted because it gives a comparative evaluation between control procedures.

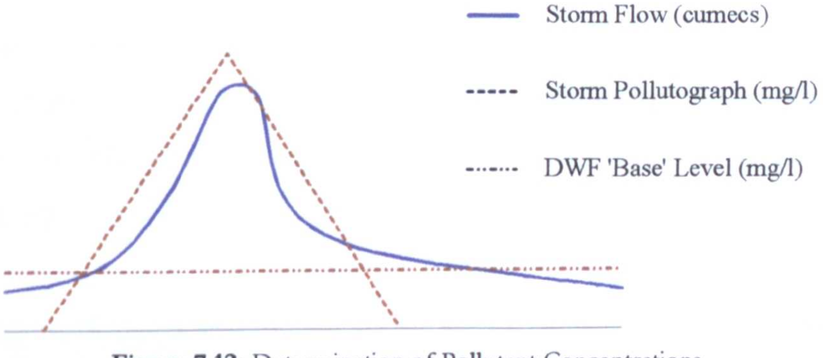


Figure 7.12: Determination of Pollutant Concentrations.

Gupta (1995) showed that the peak suspended solids concentration (TSSp) of pollutants in a storm event was a function of the peakedness (=peak rainfall intensity/average rainfall intensity) and antecedent dry weather period (ADWP), Eq. (2.20). The equations developed by Gupta (1995) were site specific with no generic method of determining the equation coefficients. The equations used in this study are therefore not representative of the Liverpool catchments and do not quantify the actual pollutant loads in the Liverpool sewer system. The important aspect, however, is that they allow for the comparative evaluation of the various control procedures used in this case study. The equation used (Gupta, 1995) to determine the peak suspended solids concentration (TSS_p)[mg/l] was:

$$TSS_p = 123.02(PEAKEDNESS)^{0.64}(ADWP)^{0.17} \tag{7.13}$$

where *PEAKEDNESS* = maximum rainfall intensity/average rainfall intensity; and *ADWP* is the antecedent dry weather period [hours].

The procedure recommended by Gupta (1995) to determine the location of the peak suspended solids concentration is quite tedious and a simplified method has been adopted. It is assumed here that the peak suspended solids concentration coincides with the peak rainfall intensity of the storm event. The location of the peak suspended solids concentration therefore precedes the peak flow rate in most cases, as would be expected in first foul flush conditions. The recession curves of the pollutographs were given by:

$$TSS(t) = TSS_p t^{-1.23} \tag{7.14}$$

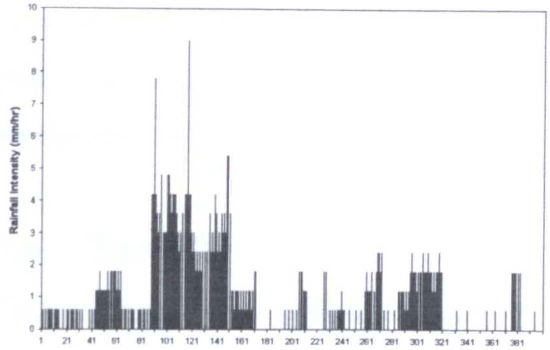
where $TSS(t)$ is the concentration of suspended solids [mg/l] at time t; and t is the time from TSS_p [mins]. This is modified from the equation suggested by Gupta (1995) because that equation generated results that were far too low (i.e. the recession was too fast) for the approach adopted in this study.

The build-up to the peak suspended solids concentration is linear from the base level at the start of the storm to the location of the peak concentration, as recommended by Gupta (1995).

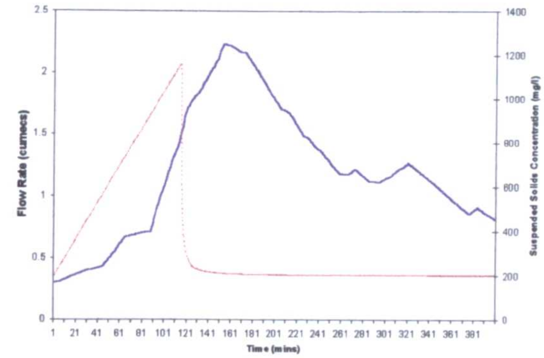
Table 7.3 shows sample calculations of TSS_p for each storm event in January of a typical year of rainfall in Liverpool. The remaining calculations are shown in Appendix 4.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
1	January	13.20	13.33	680	8.40	0.368	22.826	1414.5
2	January	22.50	22.17	530	11.40	0.675	16.889	1271.8
10	January	3.04	163.40	160	1.80	0.195	9.231	1213.3
13	January	5.23	71.65	240	4.80	0.460	10.435	1140.8
16	January	3.26	66.05	140	4.80	0.441	10.884	1155.9
16	January	8.09	2.38	60	3.00	0.660	4.545	375.8
17	January	5.01	19.87	90	2.40	0.467	5.139	582.9
17	January	13.40	7.15	140	14.40	0.433	33.256	1618.9
17	January	21.01	5.02	220	4.20	0.365	11.507	772.8
18	January	17.24	16.72	230	3.00	0.232	12.931	1021.8
19	January	6.26	9.20	90	17.40	1.427	12.193	889.1
20	January	3.30	19.57	620	16.20	1.395	11.613	979.7
20	January	14.10	0.33	50	3.60	0.816	4.412	263.9
22	January	5.00	38.00	640	2.40	0.373	6.434	751.6
23	January	0.30	8.63	740	10.80	0.814	13.268	932.0
23	January	14.33	1.72	500	15.00	2.192	6.843	461.8
24	January	16.04	17.18	450	1.80	0.080	22.500	1463.3
25	January	20.19	20.75	680	3.00	0.195	15.385	1184.7
26	January	9.37	1.97	100	9.60	1.662	5.776	424.0
28	January	18.33	55.27	100	9.60	0.420	22.857	1802.9
31	January	17.59	69.77	370	9.00	1.033	8.712	1011.8

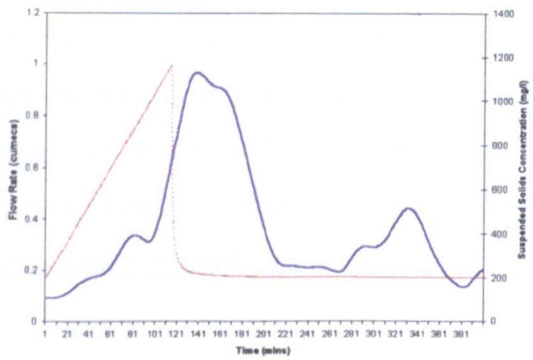
Table 7.3: Calculations of TSS_p for Typical Rainfall Events in January.



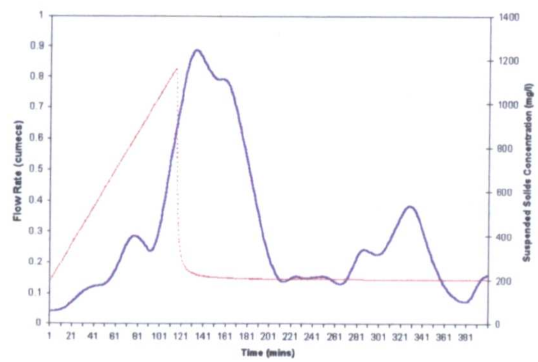
a. Rainfall Series for Event on 3rd March.



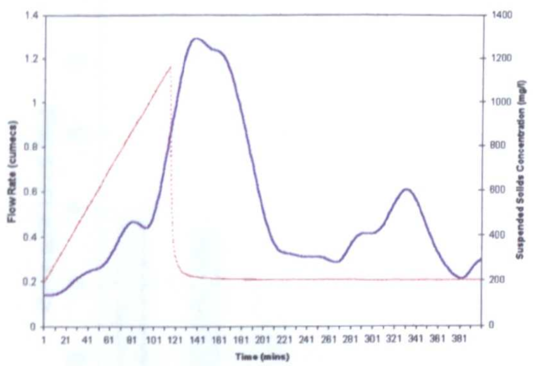
b. Rimrose Catchment.



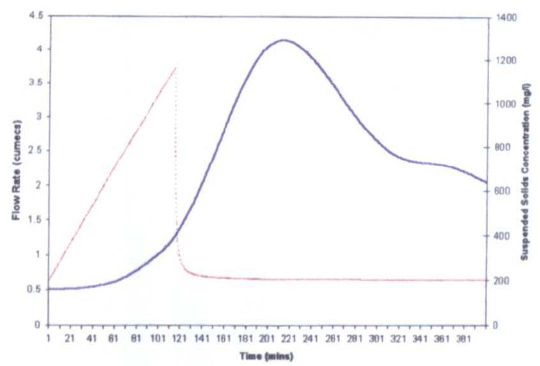
c. Strand Road.



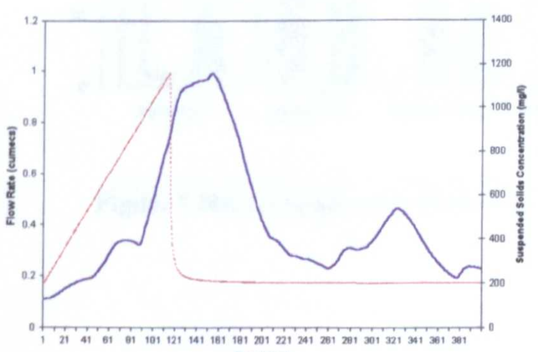
d. Millers Bridge/Fazakerley WwTW Catchment.



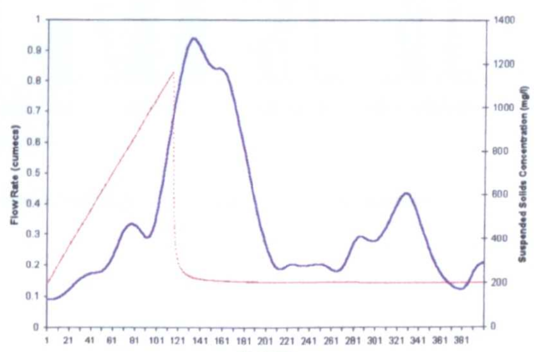
e. Bankhall Relief Catchment.



f. Northern Catchment.



g. Bankhall Catchment.



h. Sandhills Lane Catchment.

Figure 7.13: Runoff Hydrographs and Pollutographs for each Catchment (**b – h**) from Event One in March of a Typical Year (**a**).

The graphs in Figure 7.13 show samples of runoff hydrographs and pollutographs for each catchment from storm event one in March of a typical year. They show that the peak suspended solid concentration precedes the peak flow in the hydrograph, representing first foul flush effects.

7.6 RESULTS

7.6.1 Synchronised Typical Year Storms

The results presented in this section were generated from a typical year of rainfall which was synchronised over all of the catchments in the case study sewer system. The graphs in Figure 7.14 show comparisons between the different control strategies, i.e. fixed local control (FLC), variable local control (VLC), restricted VLC, and optimal pollution control (OPC), at each outfall along the northern leg of the Liverpool Interceptor Sewer for each month of the year.

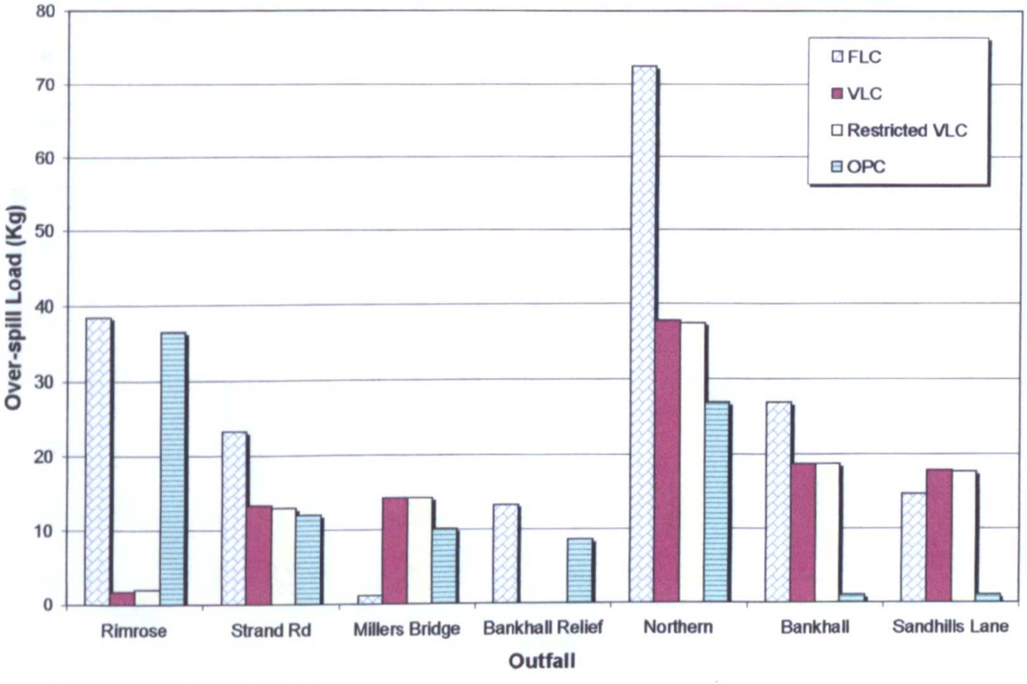


Figure 7.14a: Comparisons between the Control Strategy Over-spill Loads in January.

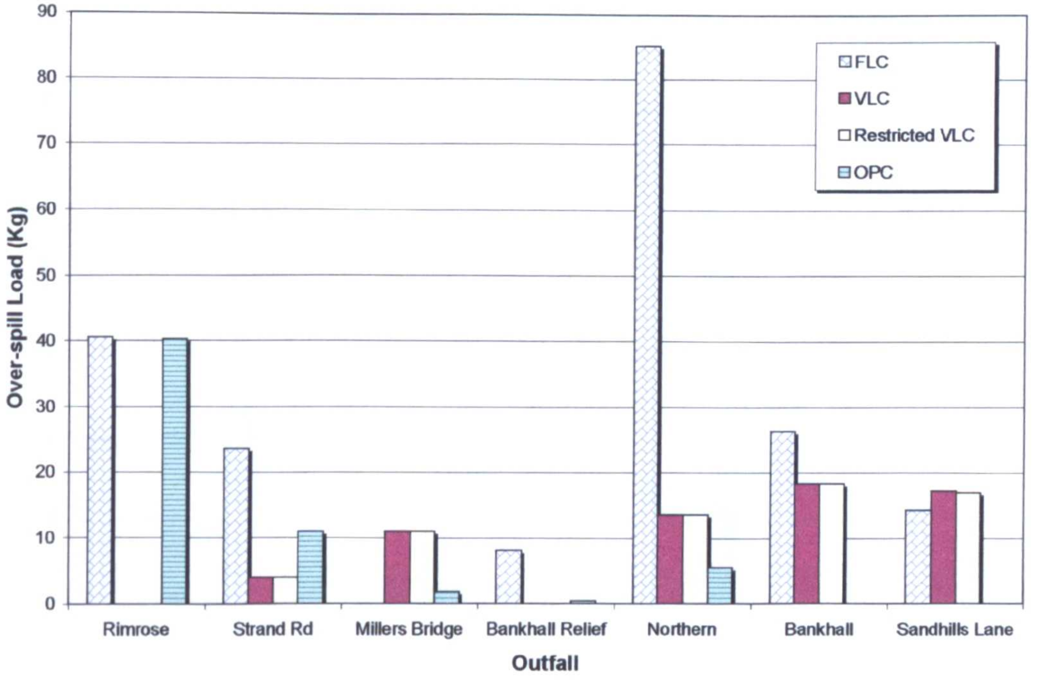


Figure 7.14b: Comparisons between the Control Strategy Over-spill Loads in February.

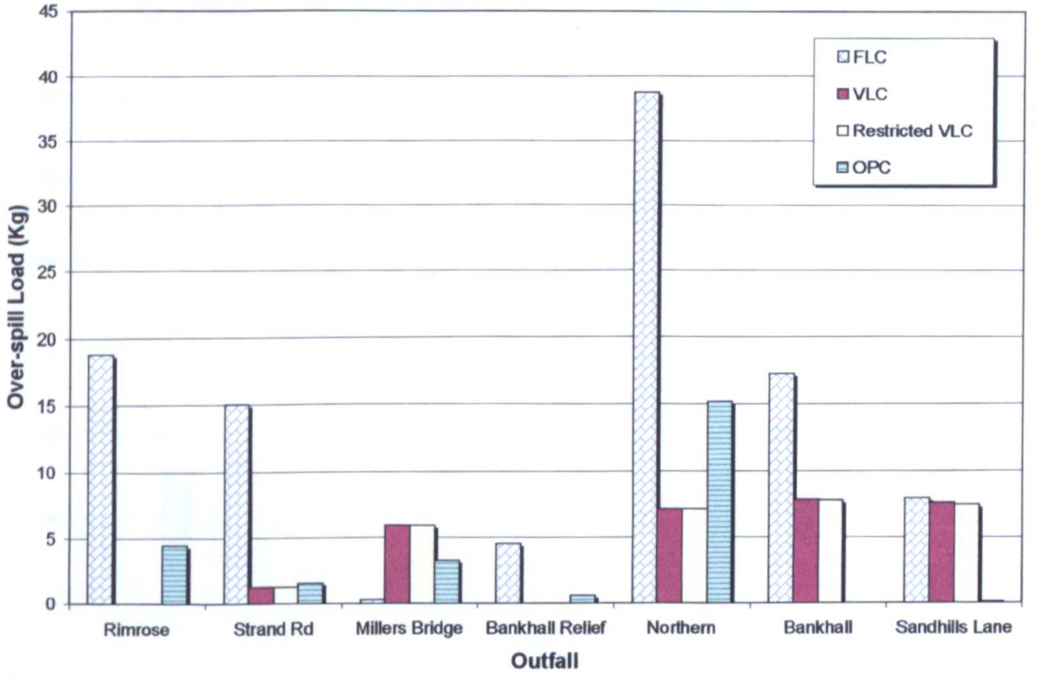


Figure 7.14c: Comparisons between the Control Strategy Over-spill Loads in March.

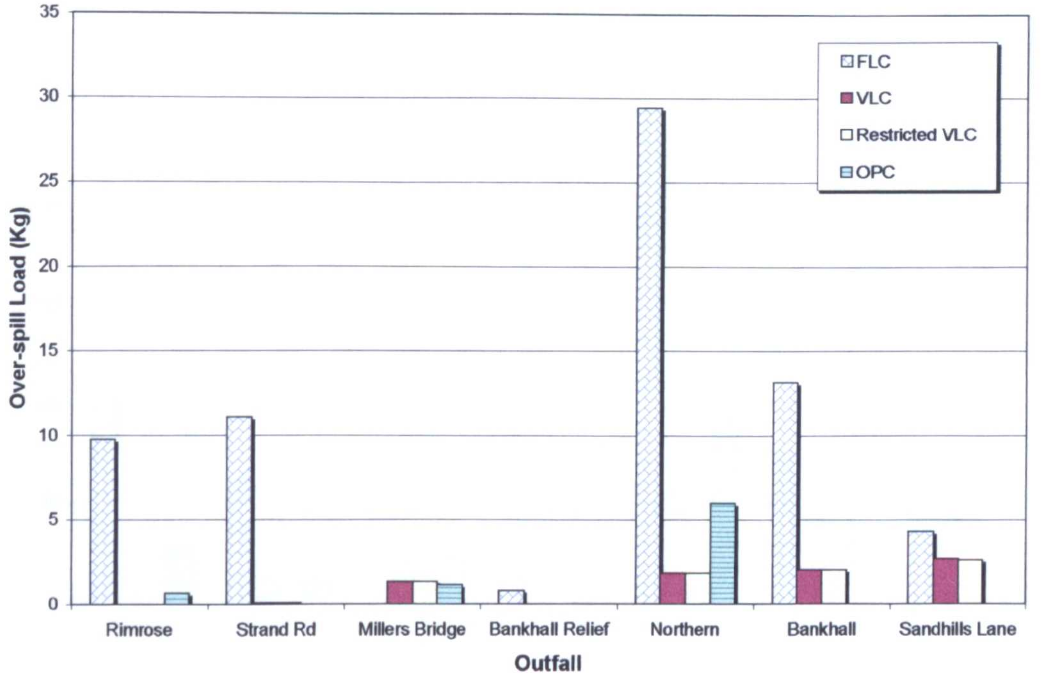


Figure 7.14d: Comparisons between the Control Strategy Over-spill Loads in April.

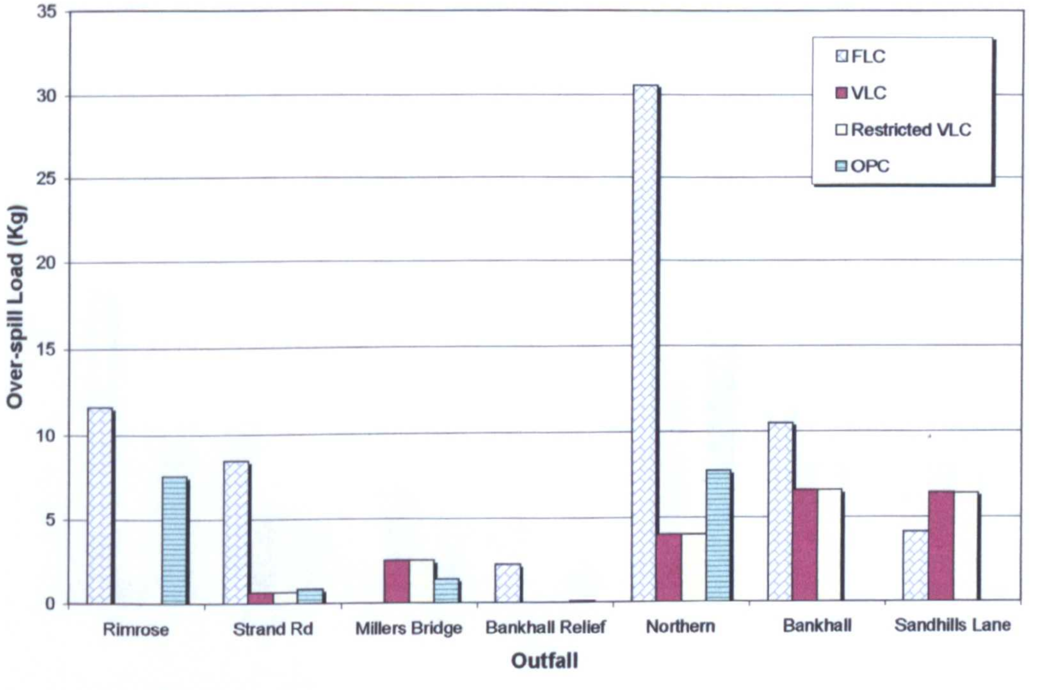


Figure 7.14e: Comparisons between the Control Strategy Over-spill Loads in May.

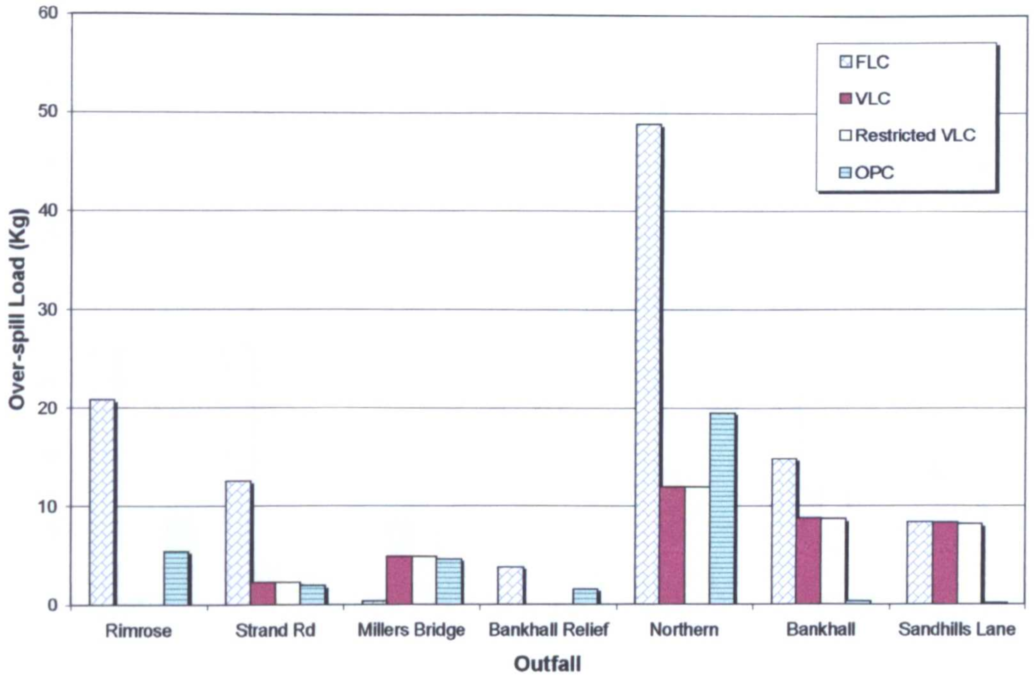


Figure 7.14f: Comparisons between the Control Strategy Over-spill Loads in June.

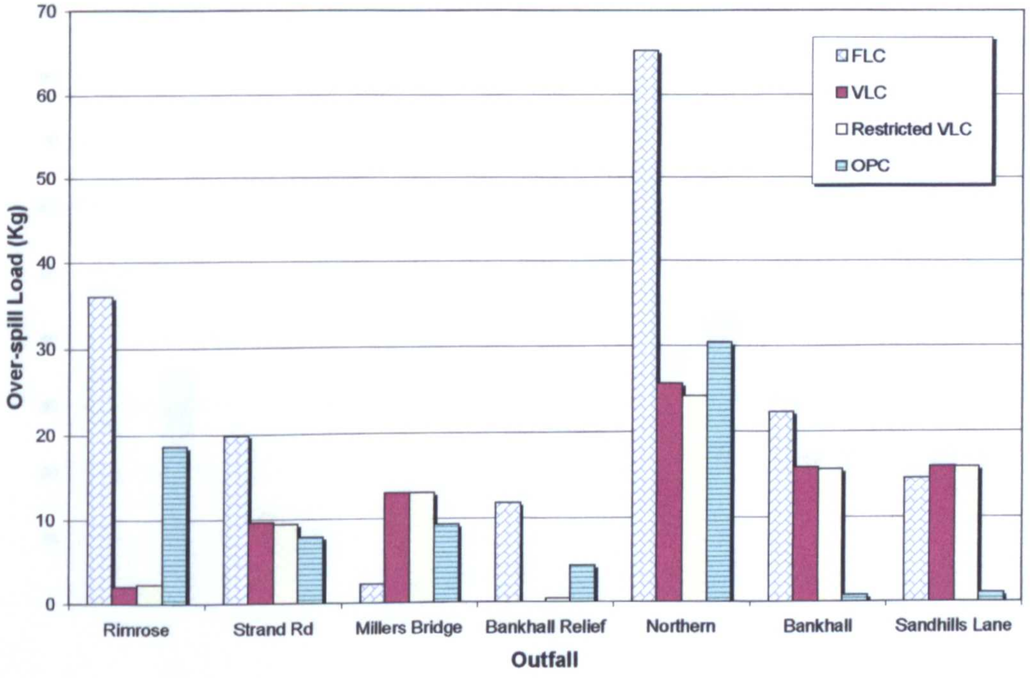


Figure 7.14g: Comparisons between the Control Strategy Over-spill Loads in July.

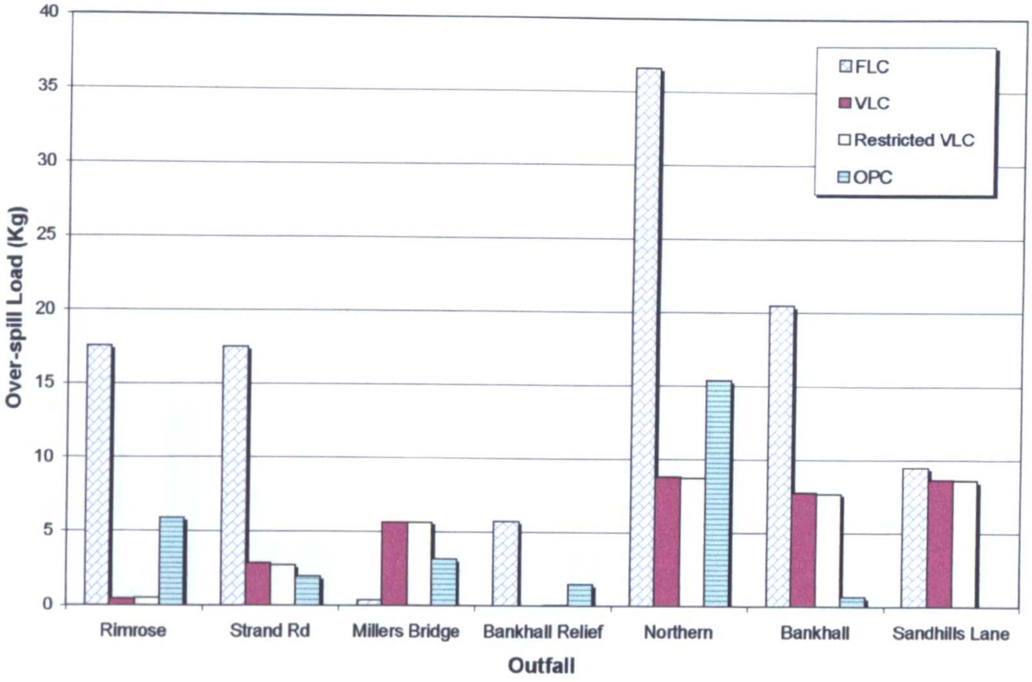


Figure 7.14h: Comparisons between the Control Strategy Over-spill Loads in August.

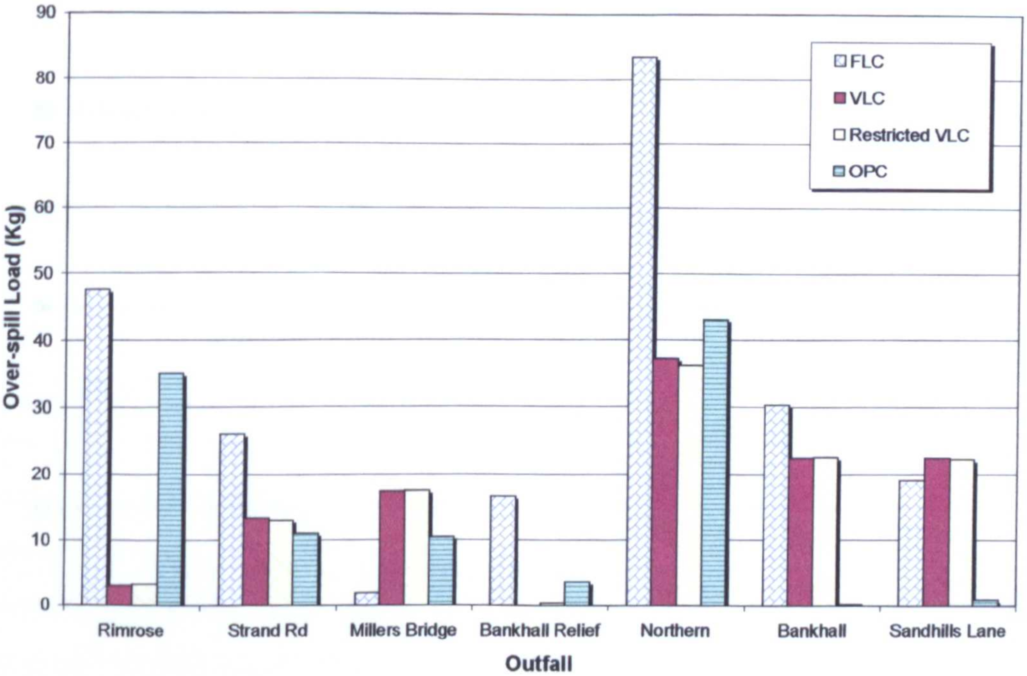


Figure 7.14i: Comparisons between the Control Strategy Over-spill Loads in September.

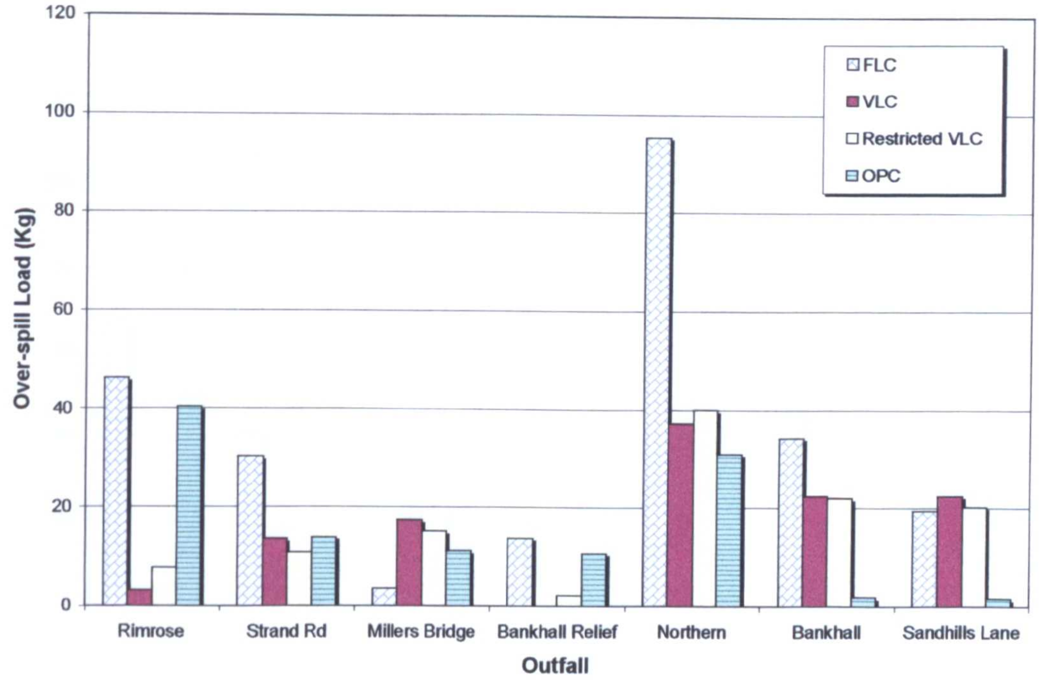


Figure 7.14j: Comparisons between the Control Strategy Over-spill Loads in October.

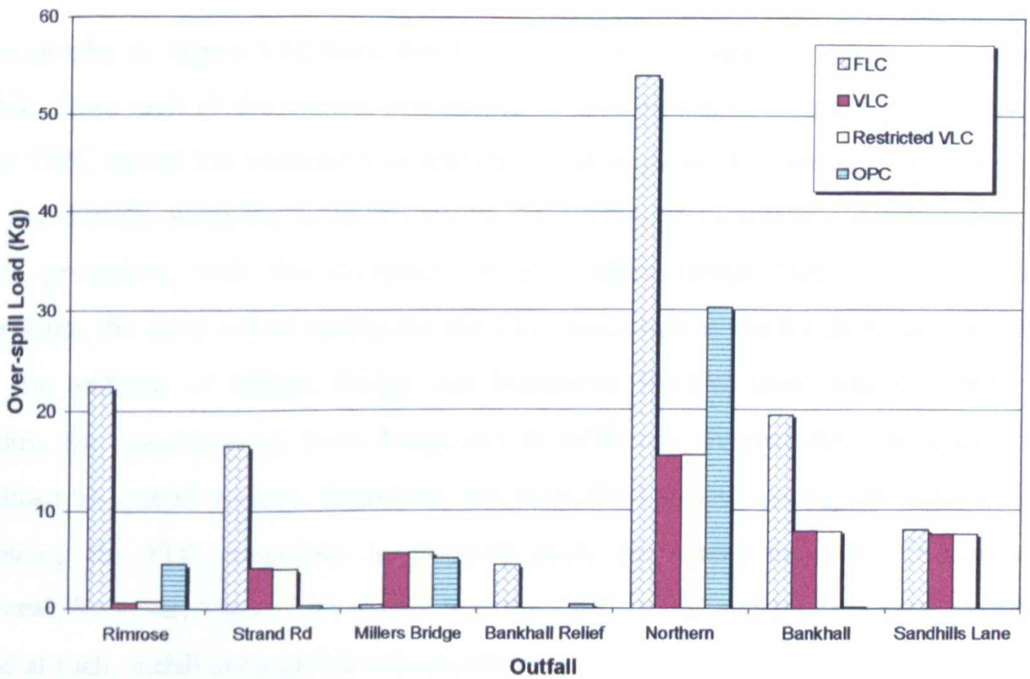


Figure 7.14k: Comparisons between the Control Strategy Over-spill Loads in November.

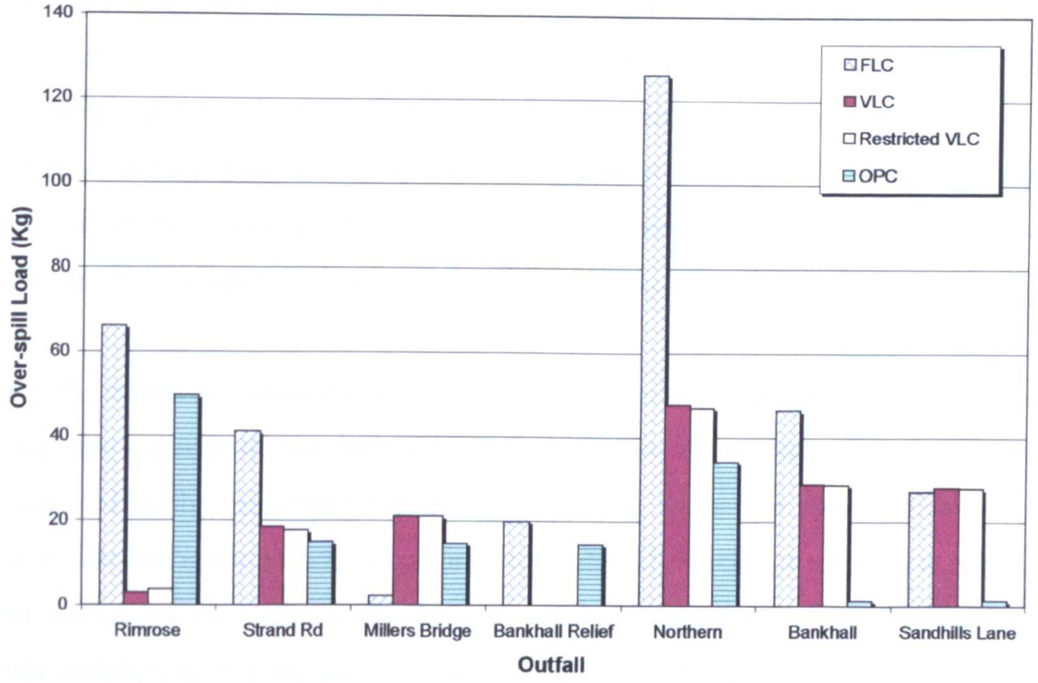


Figure 7.14l: Comparisons between the Control Strategy Over-spill Loads in December.

The graphs in Figure 7.14 show the resultant over-spill loads, in terms of suspended solids, from each of the control procedures in each month of a typical year of rainfall. The OPC model has reduced (considerably in most cases) the over-spill load on each of the outfalls along the northern leg of the Liverpool Interceptor compared to the FLC procedure, with the exception of the Millers Bridge outfall. For modelling purposes, the fixed inflow setting for the FLC procedure at this location was calculated by the addition of Millers Bridge and Fazakerley WwTW fixed inflow settings. In reality, the contributions from Fazakerley WwTW are diverted into the interceptor without an outfall system. Therefore, the joint fixed inflow setting was set too high allowing the FLC procedure to perform more favourably than the OPC model. Overall, however, Figure 7.14 shows that the OPC model reduced the local over-spill load at each outfall using global information.

There was a possibility that the results from the VLC and OPC procedures may be favourably biased because the hydraulic characteristics of the continuation pipe between the chamber and interceptor sewer was not included, as discussed in Chapter 6. Therefore, an additional control procedure was included in this section, Restricted VLC, to evaluate the extent of this bias. The results of this procedure are included on the graphs in Figure 7.14. These results deviate only slightly from the VLC procedure results indicating that the expected bias was negligible. This is explained by the fact that the chamber inlet and outlet pipe sizes were based on the peak flow from a 1 in 2 year storm, which generally gives a similar capacity to the original River outfall. Therefore, inflows from the catchments in a typical year would only exceed these pipe capacities on rare occasions. Nevertheless, the bias remains in the modelling approach and is sensitive to input data; larger inflows would have increased the bias.

The OPC model is formulated so that the inflows to the interceptor range from zero to the pipe capacity in any time step of the solution. An additional constraint could be included in the optimisation problem restricting the inflow to continuation pipe capacity alleviating the potential bias in the results. The inclusion of such a constraint is recommended for future work, although it is envisaged that its inclusion will have a similar influence to that shown in Figure 7.16 between VLC and restricted-VLC.

The graphs in Figure 7.14 show that the OPC model reduced the local over-spill loads at most outfalls when compared to the VLC procedure. However, the VLC procedure spilled less pollutant load than the OPC model at the Rimrose, Strand Road (occasionally), Bankhall Relief and Northern outfalls. The VLC procedure permits flows up to the local interceptor capacity and fills the sewer as soon as possible (i.e. in the upstream intercept points). Consequently, there is a bias in the VLC results where the upstream section of each sewer pipe (i.e. change in pipe size) would be utilised frequently. The outfalls located near the downstream sections would therefore spill more frequently because the interceptor sewer would already be full. Therefore, the improvement in over-spill load by the VLC procedure is fortuitous because it is volumetric-based and takes no account of pollutant concentrations.

The graphs in Figure 7.15 show comparisons between the overall control strategy over-spill loads for each month.

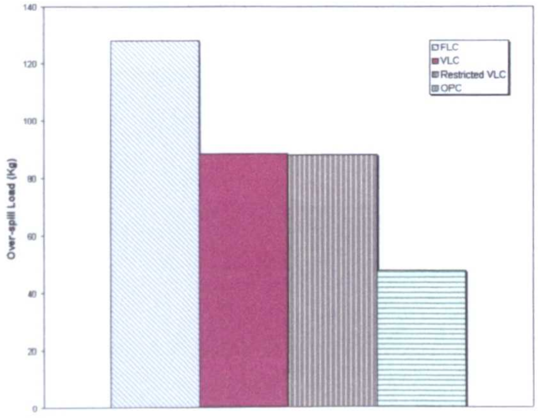


Figure 7.15a: Comparisons between Control Procedures for January.

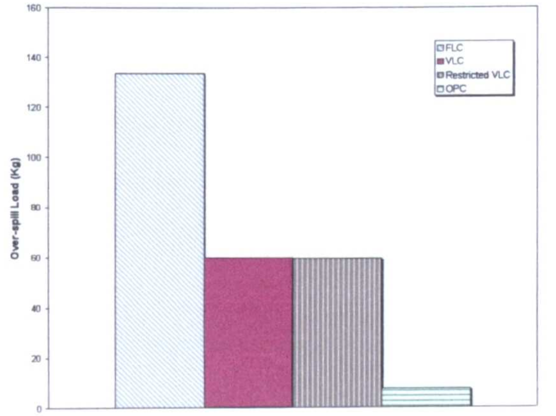


Figure 7.15b: Comparisons between Control Procedures for February.

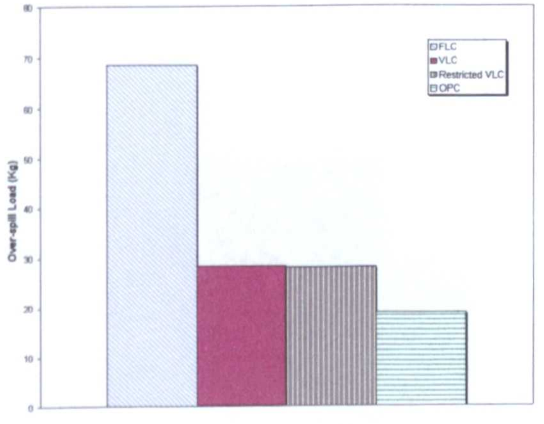


Figure 7.15c: Comparisons between Control Procedures for March.

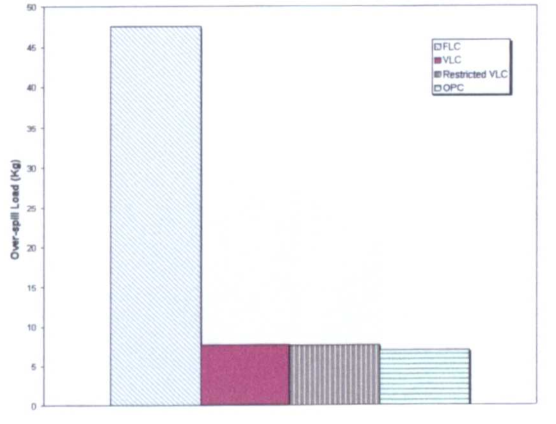


Figure 7.15d: Comparisons between Control Procedures for April.

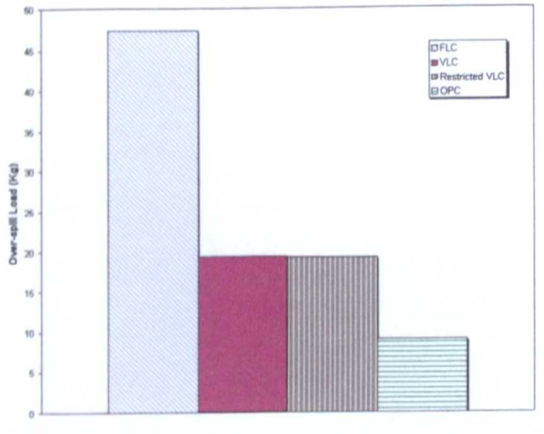


Figure 7.15e: Comparisons between Control Procedures for May.

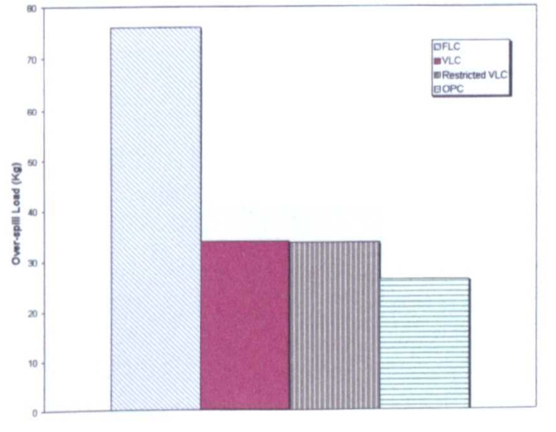


Figure 7.15f: Comparisons between Control Procedures for June.

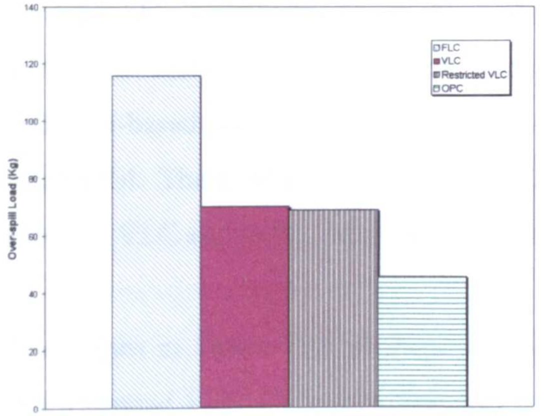


Figure 7.15g: Comparisons between Control Procedures for July.

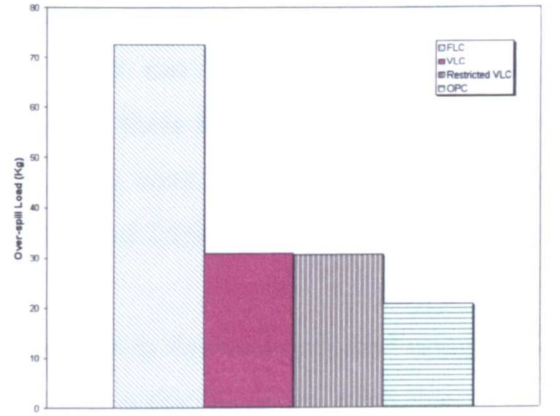


Figure 7.15h: Comparisons between Control Procedures for August.

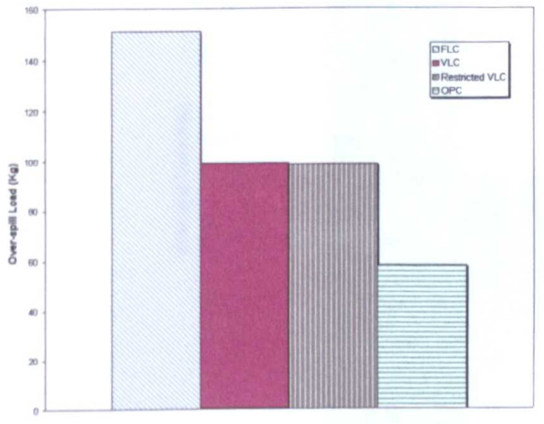


Figure 7.15i: Comparisons between Control Procedures for September.

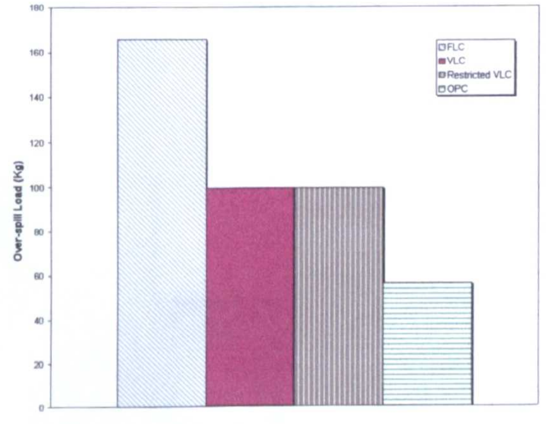


Figure 7.15j: Comparisons between Control Procedures for October.

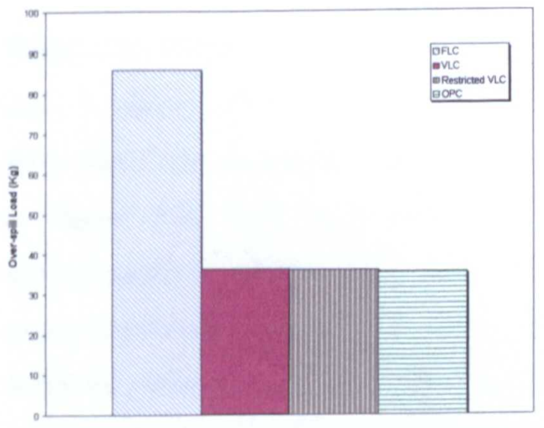


Figure 7.15k: Comparisons between Control Procedures for November.

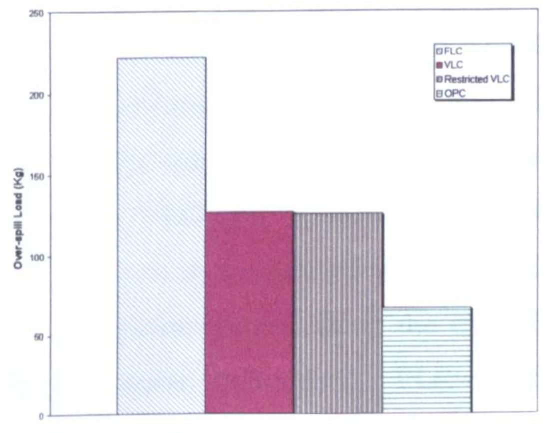


Figure 7.15l: Comparisons between Control Procedures for December.

The graphs in Figure 7.15 show that the OPC model considerably reduced the global over-spill load when compared to FLC, VLC and restricted VLC. The graphs show that considerable improvements can be achieved when applying volumetric control (i.e. VLC and restricted VLC). Further improvements can be achieved by applying pollution-based control (i.e. OPC). In most months, these further improvements were substantial. There were a few months where there were only slight improvements between VLC and OPC, for example Figures 7.15d and 7.15k.

The graphs in Figure 7.15 all show minor differences in the over-spill load using VLC and restricted VLC, illustrating that the potential bias in the results was negligible.

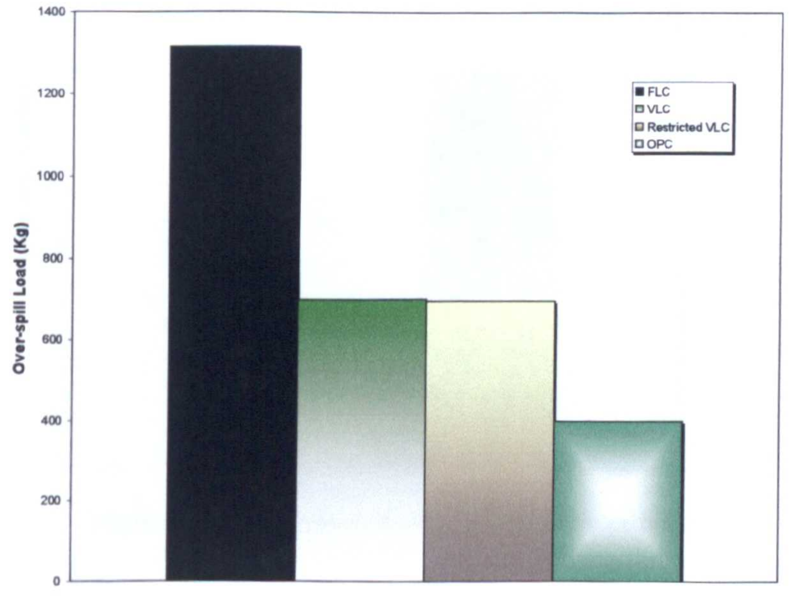


Figure 7.16: Overall Comparison between Control Procedures in a Typical Year.

Figure 7.16 shows the overall comparison between the control strategies in a typical year. It clearly shows that the OPC model out performed all the other control procedures. The percentage improvements between the control procedures are shown in Figure 7.17. This graph shows that both the VLC and restricted-VLC had approximately a 48% improvement over FLC. This is a measure of the potential improvements in over-spill load when utilising volumetric control where the control decisions utilise the full capacity of the interceptor sewer. Although the VLC is not strictly a global volumetric control technique it does ensure that the sewer remains full when achievable. A global volumetric control technique would also utilise the volumetric capacity of the interceptor sewer but may over-spill at different locations. Therefore, the local over-spill loads would deviate from the VLC loads but the global load would remain similar. Of course, this is sensitive to the hydraulic characteristics and loading of the sewer system.

The OPC model reduced the over-spill load by 70% when compared to the FLC procedure, as shown in Figure 7.17. The reduction is based on improved volumetric utilisation and pollution load retention. Additionally, the OPC model reduced the over-spill load by just over 40% when compared to VLC, which was only based on including pollution concentrations in the control model. Therefore, similar percentage improvements are achievable when moving from FLC to VLC and from VLC to OPC.

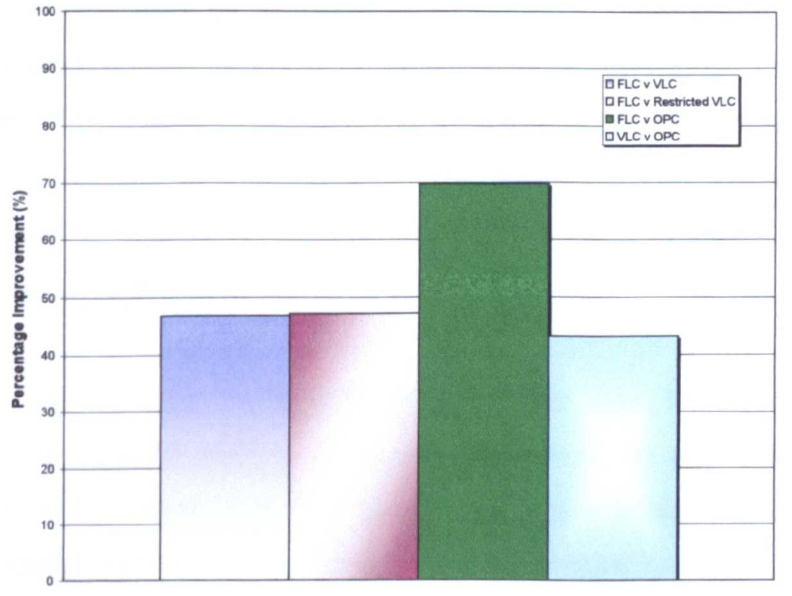


Figure 7.17: Improvements between Control Procedures.

The results presented in this section have been focused on comparisons between the control procedures in terms of pollutant over-spill load. However, other results could have been presented such as number of spills and spill volumes. Although these results have not been presented it is expected that there would be increased volumes of spill with OPC but with reduced pollution load. This is because pollution-based global control has to spill more frequently to accommodate 'dirtier' sewage elsewhere in the system.

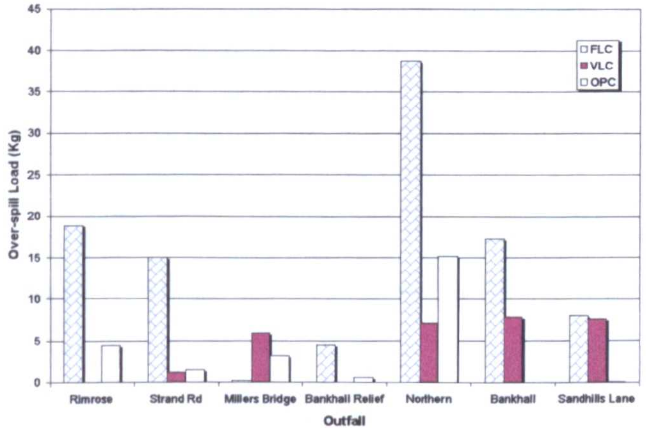
It must be stressed that the results presented in this section are for a synchronised storm where the rainfall falls uniformly across the interceptor sewer catchments. Each catchment responds differently, even under these conditions, but it is hypothesised that the improvements in such conditions are likely to be conservative. The greatest improvements between OPC and FLC are likely to be encountered in storm events that are spatially and temporally distributed.

7.6.2 Temporally Distributed Storms

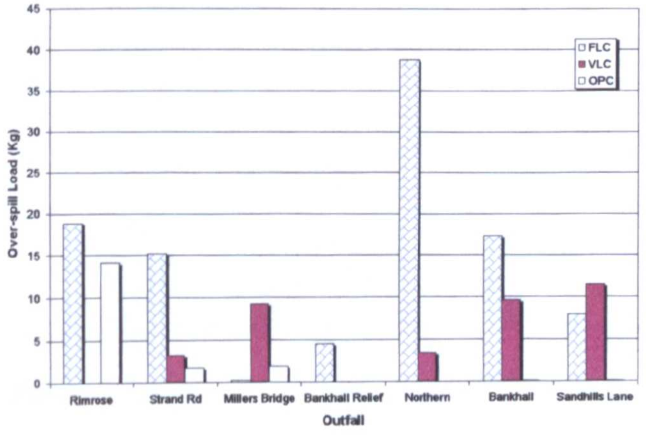
To investigate the significance of temporal variations in rainfall, the inflow hydrographs (and corresponding pollutographs) from the storms in March (see Figure 7.8) of the typical year were modified so that the storms moved downstream and upstream along the interceptor sewer. This was achieved by arbitrarily delaying the hydrographs by consecutive 20 minutes at each catchment. For example, to synthesise a storm that moved downstream along the interceptor the hydrograph for the second intercept point (Strand Road) was delayed by 20 minutes, the hydrograph for Millers Bridge was delayed by 40 minutes, etc. This procedure was also used on the corresponding pollutographs. The reverse process was used to synthesise a storm that moved upstream along the interceptor.

A comparison of the local over-spill loads from each outfall from a synchronised storm (March of a typical year), a storm moving downstream and a storm moving upstream is shown in Figure 7.18. The graphs in Figure 7.18 show that FLC is not sensitive to storm type as the spill loads remain identical under all three storm conditions. However, the VLC and OPC procedures are sensitive to storm conditions. The VLC strategy over-spilled a greater amount of suspended solids when the storm moved in the downstream (Figure 7.18b) direction and less when the storm moved upstream (Figure 7.18c). The OPC strategy spilled less pollutant load under the temporally distributed storm conditions (Figures 7.18b and 7.18c) than the synchronised storm (Figure 7.18a).

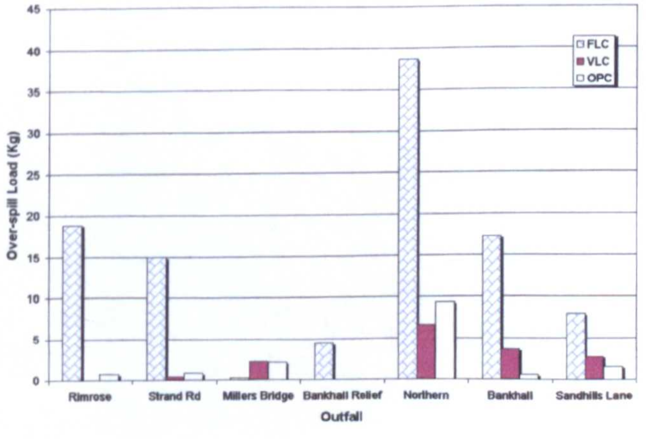
Figure 7.19 shows comparisons between the over-spill load in each control procedure under the storm conditions. Figure 7.19a shows comparisons under each of the storm events using FLC and clearly shows that the over-spill load remained identical. Figure 7.19b shows that the VLC over-spilled differing amounts of suspended solids depending on the storm condition. The VLC procedure spill load increased with the storm that moved in the downstream direction and decreased with the storm that moved upstream, when compared to the synchronised storm event. The VLC procedure fills the interceptor as quickly as possible (i.e. in the upstream sections) and under the downstream moving storm event the peaks in the hydrograph (and pollutograph) were delayed. Therefore, the sewer was full when the downstream hydrograph peaks entered the interceptor chambers requiring the over-spill of this peak pollutant load. The converse occurs in storms that move in the upstream direction. Figure 7.19c shows that under the temporally distributed storms the OPC strategy spilled less suspended solids. This reduction was particularly significant under the upstream moving storm.



a. Synchronised Storm.



b. Storm Moving Downstream.



c. Storm Moving Upstream.

Figure 7.18: Comparison between Over-spill Loads at each Outfall in Different Spatial Storm Events.

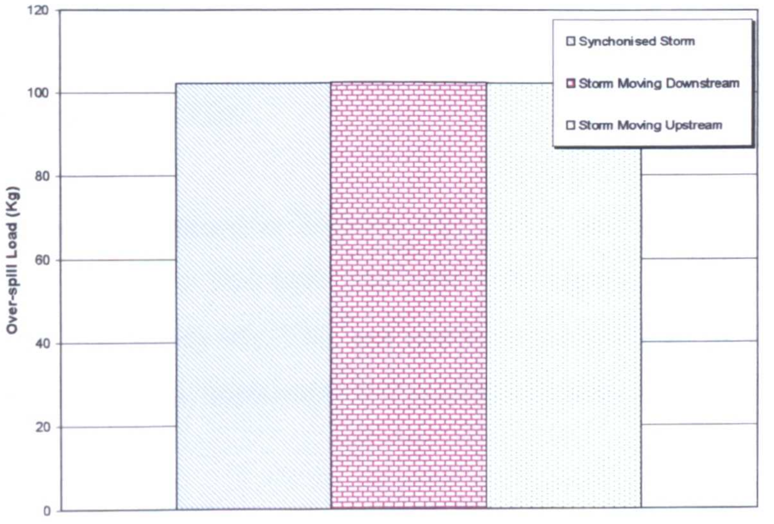


Figure 7.19a: Effect of Moving Storms on FLC in March.

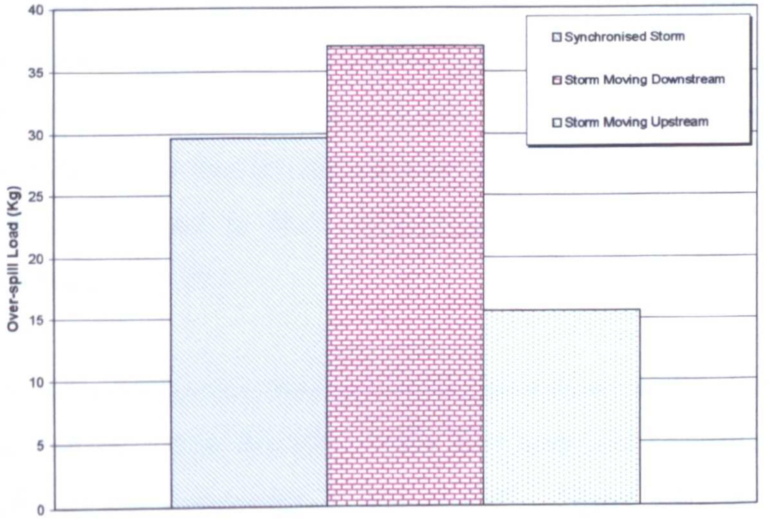


Figure 7.19b: Effect of Moving Storms on VLC in March.

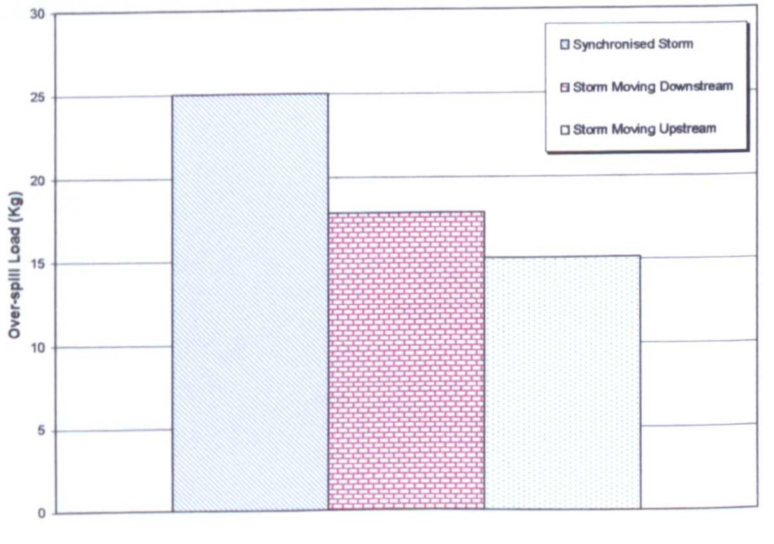


Figure 7.19c: Effect of Moving Storms on OPC in March.

Figure 7.20 shows comparisons between the control procedures in each of the storm conditions. The percentage improvements between the control procedures for these storms are shown in Figure 7.21.

Figure 7.20a shows a comparison between FLC, VLC and OPC for synchronised storms in March and shows that both VLC and OPC significantly reduced the over-spill load. Figure 7.21a shows that VLC reduced the over-spill load by 71% compared to FLC but a further 15% improvement was achieved with OPC, a 75% reduction compared to FLC.

Figure 7.20b shows comparisons in over-spill load from each control procedure in the downstream moving storms. In this case, VLC over-spilled more suspended solids than in the synchronised storms. Conversely, OPC spilled significantly less pollutant load than FLC and VLC under this loading and also less than OPC in the synchronised storms. Figure 7.21b shows that the reduction in suspended solids over-spilled reduced to 63% for VLC compared to FLC. Additionally, the OPC strategy made a 50% improvement compared to VLC.

Figure 7.20c shows a comparison between control procedures in the upstream moving storms. In these storm events, the VLC results have improved dramatically where the VLC over-spill load was comparable to that of OPC. Figure 7.21c shows that VLC made a 85% reduction in over-spill load compared to FLC, which was only slightly improved, by approximately 5%, with OPC.

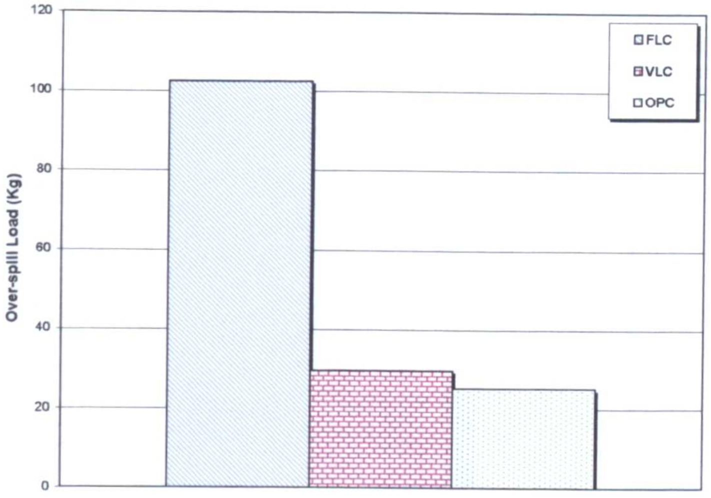


Figure 7.20a: Comparison between Control Procedures in Synchronised Storms in March.

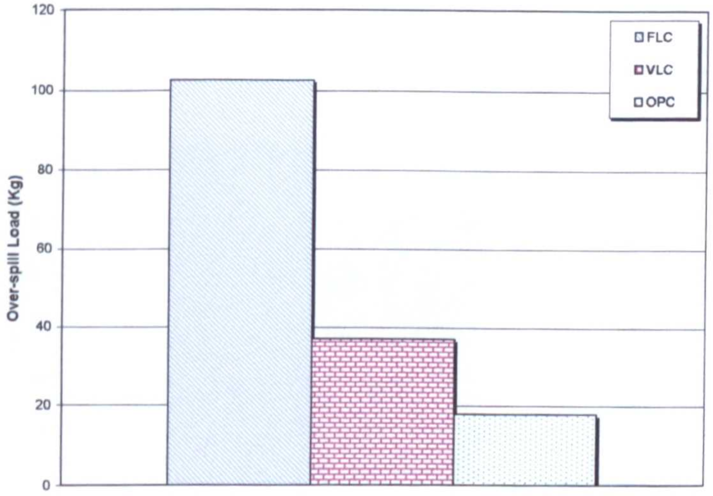


Figure 7.20b: Comparison between Control Procedures in Storms Moving Downstream in March.

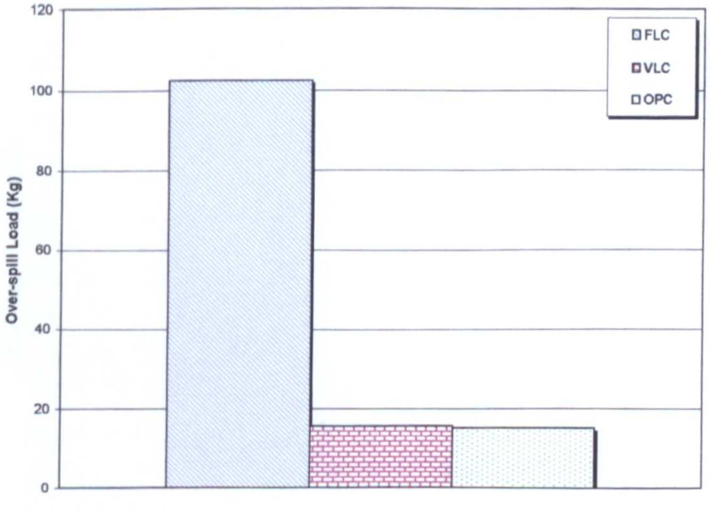
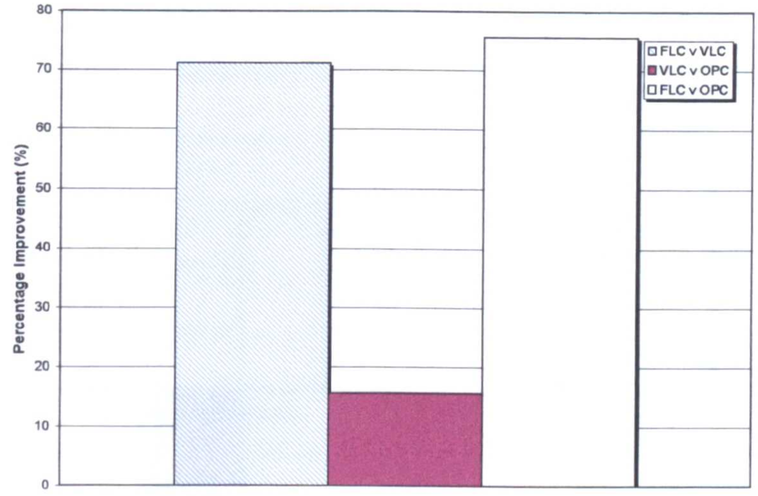
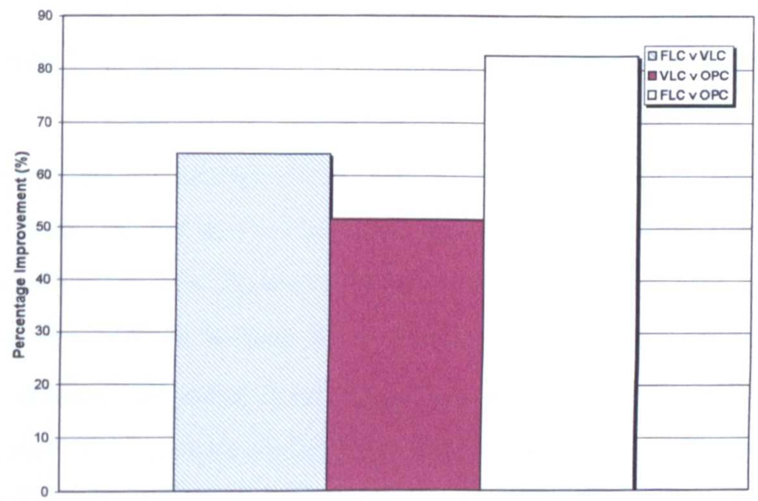


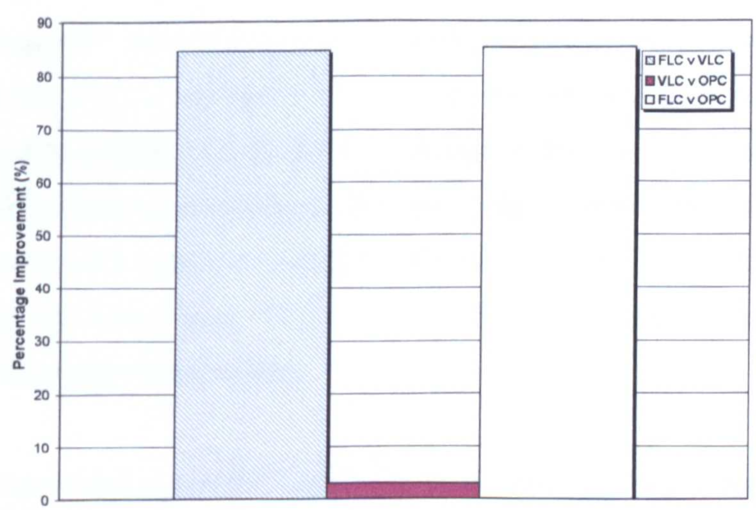
Figure 7.20c: Comparison between Control Procedures in Storms Moving Upstream in March.



a. Synchronised Storms



b. Storms Moving Downstream.



c. Storms Moving Upstream.

Figure 7.21: Improvements between Control Procedure in the Storm Conditions.

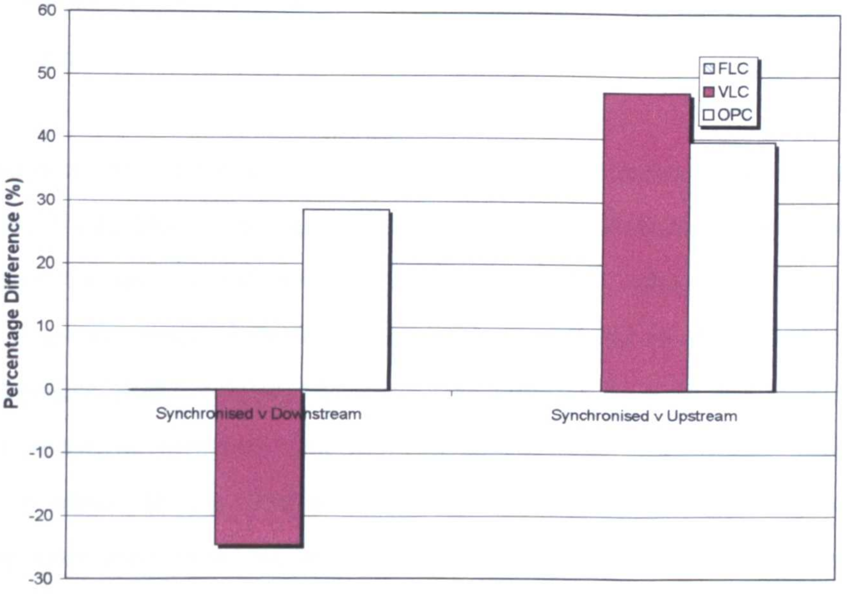


Figure 7.22: Improvements between Over-spill Loads in Each Storm Event.

Figure 7.22 shows the percentage differences for each control procedure between the synchronised storm and the downstream moving storm, and between the synchronised and upstream moving storms. As seen in Figure 7.22 there were no differences in over-spill load from FLC. This is because the control procedure uses a pre-set flow setting, which is not affected by actual flow conditions or pollutant concentrations. In the comparison between the synchronised storm and downstream moving storm, the VLC procedure performed worse for the moving storm. The problem of the downstream outfalls spilling more frequently with VLC (as described earlier) was exacerbated. The peaks in hydrographs entered the upstream intercept points and were advected down the sewer. In this storm event where the hydrographs were delayed in the downstream direction, the slugs would more frequently coincide in the lower sections of the sewer with the peak inflows (and pollutographs) from the downstream intercept points, therefore causing spills of greater pollution load. The converse is true for the upstream moving storm and here Figure 7.22 shows that the VLC results improved by 48% compared to the synchronised storm.

Figure 7.22 shows that the OPC results for the temporally distributed storms were considerably improved over the synchronised storm results. In fact, the improvement was particularly significant for the upstream moving storm, a 40% improvement over the synchronised storm. The improvement was reduced to 30% for the downstream moving storm.

The differences in improvements between the storm conditions are explained by two factors, the travel times in the interceptor sewer and the times of concentrations in the catchments. In the hypothetical worst case scenario where the catchments have identical response times, if the hydrographs were delayed by the travel times between intercept points in the interceptor sewer (see Figure 4.5), then the peaks would enter the interceptor at identical times, losing the spatial distribution benefits of the interceptor sewer. In this situation, there would not be the flexibility in the system loading to over-spill 'clean' flows to accommodate the 'dirty' inflows elsewhere in the system. Therefore, there would be a significant increase in pollutant over-spill load. However, in reality this would rarely occur because the catchments would have different response times so there would still be spatial distribution benefits. Nevertheless, a downstream moving storm will move the peaks from the catchments closer together and worsen the improvements in pollutant over-spill load if there was insufficient variability in the times of concentration.

Table 7.4 shows the effect the delays in hydrographs have on the times to peak flow in the first storm in the rainfall series for each catchment. It shows for the case study example there is a greater variability in the response times for both moving storms compared to the synchronised storm.

Catchment	*Flow Times between Intercept Points (minutes)*	*Synchronised Storm Response Time (minutes)*	*Downstream Moving Storm Response Time (minutes)*	*Upstream Moving Storm Response Time (minutes)*
Rimrose	8.5	90	90 (90+0)	210 (90+120)
Strand Rd	6.0	60	80 (60+20)	160 (60+100)
Millers Bridge/ Fazakerley WwTW	4.0	40	80 (40+40)	120 (40+80)
Bankhall Relief	0.5	60	120 (60+60)	120 (60+60)
Northern	6.0	90	170 (90+80)	130 (90+40)
Bankhall	3.0	60	160 (60+100)	80 (60+20)
Sandhills Lane	1.5	40	160 (40+120)	40 (40+0)

N.B. Numbers in brackets show calculations for moving storm response times.

Table 7.4: Effects the Delays in Hydrographs have on Catchment Response Times.

In the downstream moving storm here the delay times did not coincide with the travel times between the intercept points and there was considerable variability in the times of concentration as shown in Table 7.4. Table 7.4 shows that for this example there was a greater variability in the response times for the downstream moving storm than for the synchronised storm. Therefore, a reduction in over-spill load, compared to the synchronised storm, would be expected.

In an upstream moving storm a greater improvement would be expected because the peaks in hydrographs and suspended solid concentrations were temporally distributed in the opposite direction to the times of concentration, adding to the benefits of the sewer systems spatial distribution. Therefore, the upstream moving storm would separate the times to peak along the interceptor sewer and so add to the spatial distribution benefits in the interceptor sewer and catchments. Therefore, a greater improvement in pollutant over-spill load would be expected.

The application of the OPC model in the case study has illustrated the computational efficiency of the model. For example, the OPC model determined the pollutant-based control strategies throughout the case study sewer system within minutes for an entire month's rainfall data (using a Sun Solaris Unix workstation).

7.7 CONCLUDING REMARKS

This chapter has described the application of the optimal pollution control model to a case study Liverpool Interceptor Sewer system using historical rainfall events. A typical year of rainfall data was used in a Unit Hydrograph model to generate typical inflows into the interceptor sewer. Additionally, a simplified approach was used to determine time-varying pollutant concentrations. Four control procedures were considered: fixed local control (FLC), variable local control (VLC), restricted VLC, and optimal pollution control (OPC).

The results from the case study have shown that the OPC model considerably reduces pollutant over-spill load in a typical year, compared to the current FLC system. Furthermore, the OPC model not only reduces global pollutant over-spill load but also individual outfall pollutant over-spill loads. In this application, a 70% global reduction in pollutant over-spill load was achieved in synchronised storms, compared to fixed local control.

An alternative volumetric-based control procedure, variable local control, reduced global pollutant over-spill load by some 48%. This control procedure (although an extension of a local control procedure) ensures full utilisation of the sewer capacity and so indicates the performance improvement of volumetric-based global control. The results from the restricted VLC illustrated that there was a negligible bias, arising from the omission of the continuation pipe capacity constraint on the throughflow, in the VLC and OPC procedures.

The overall results in Figure 7.17 illustrated that similar percentage improvements were achievable when moving from FLC to VLC and from VLC to OPC. This is sensitive to model inputs requiring further study but it indicates that there is significant scope for improving current control systems beyond the obvious volumetric-based global control.

It was hypothesised that the improvements between OPC and FLC in synchronised storms were conservative and that most improvements would be in spatially and temporally distributed storms. On application to moving storms, the results confirmed (as expected) that the FLC procedure was not sensitive to storm movements unlike the VLC and OPC procedures. In fact, for a storm of the same magnitude but moving the OPC model reduced pollutant over-spill loads even further, ranging from 30% to 40% depending on the storm direction. This indicates that in large sewer systems, where there will be larger spatial and temporal distributions, there is a potential of greater reductions in pollutant over-spill loads. It must be stressed that the influence of moving storms on the systems performance will be dependent upon the differences in the times of concentration of the various catchments.

Overall, this chapter has illustrated that the OPC model determines control strategies that significantly reduce pollutant over-spill loads, particularly in temporally distributed storms, compared to all other control procedures used in the study.

CHAPTER 8

CONCLUSIONS AND FUTURE WORK

8.1 SUMMARY

The primary objective of this study, as stated in Chapter 1, was to develop novel methods of controlling large interceptor sewer systems to minimise total pollutant over-spill load to the receiving waters. A unique 'slug flow' approach was adopted in the development of the control models allowing the determination of optimum pollutant-based control decisions very quickly.

Some preliminary conclusions have already been made in the main chapters of this thesis and are summarised in the paragraphs below.

The development of the idealised control models using linear programming (LP) and dynamic programming (DP) was described in Chapter 4, where the 'slug flow' approach was introduced. The results from the application of these optimal pollution control models on an idealised interceptor system (see Figure 4.12), using hypothetically generated hydrographs and pollutant concentrations, illustrated that significant improvements in pollutant over-spill load could be achieved when compared to fixed local control (the traditional control procedure) and variable local control.

The validity of the slug flow approach was confirmed in Chapter 5 where a post-processing hydraulic verification routine, which was validated to WALLRUS, was developed to determine approximate water profiles for each time step in the storm event. The results illustrated that the slug flow and WALLRUS hydrographs correlated well, particularly for the lower intensity storms (see, for example, Figure 5.8). The water profiles from the hydraulic verification routine and the WALLRUS depths were remarkably similar, offering further validation of the slug flow approach (see, for example, Figure 5.17). However, the results illustrated that the slug flow approach was unable to fully represent the hydraulics in the interceptor sewer in localised downstream storms (see Figure 5.18), where backwater effects were shown to be important. The formulation of the control models needs modifications in such conditions, which actually rarely occur in reality, to allow interactions between 'slugs' in the interceptor sewer and are included as a recommendation for further work.

The formulation of the idealised control models allowed the inclusion of non-linear equations that govern the flow through overflow chambers, illustrating that the modelling approach was sufficiently robust to incorporate additional hydraulic constraints. The extended optimal pollution control model was described in Chapter 6 and responded to the pollutant concentrations in the overflow chambers, over-spilling from the chambers with the least polluted sewage. The results from the application of the extended optimal pollution control model on a simplified version of the northern leg of the Liverpool Interceptor Sewer (see Table 6.3) showed considerable reductions in pollutant over-spill load when compared to traditional fixed local control.

The application of the extended optimal pollution control model to a case study Liverpool Interceptor Sewer using historical rainfall events in Chapter 7 reduced pollutant over-spill load considerably (see Figure 7.17) when compared to fixed local control and variable local control. Significantly, the OPC model not only reduced global pollutant over-spill load but also individual outfall pollutant over-spill loads. The results showed that similar improvements were achievable when moving from fixed local control to variable local control and from variable local control to optimal pollution control. This indicates that there is significant scope for improving current control systems beyond the obvious volumetric-based global control.

Investigations on temporally moving storms in Section 7.6.2 showed that fixed local control was not sensitive to storm movements, unlike variable local control and optimal pollution control. There were greater reductions in pollutant over-spill load with OPC for storms that were temporally distributed, particularly for storms that move in the upstream direction (see Figure 7.22). This indicates that in large systems with larger spatial and temporal distributions, there is a potential of greater reductions in pollutant over-spill loads.

Overall, the results in Chapter 7 illustrated that the OPC model out-performed all other control procedures, in terms of pollutant over-spill load reduction. Importantly, the full OPC model was computationally efficient and is therefore entirely suitable for application in real time. For example, the OPC model determined the pollution-based control strategies throughout the case study sewer system within minutes for an entire month's rainfall data.

8.2 LIMITATIONS OF THE WORK

The main limitation of the OPC model is that the slug flow approach is unable to fully represent the hydraulics of backwater effects in the interceptor sewer. However, verification studies in Chapter 5 have shown that these effects are only important in storm events that are localised in the downstream section of the sewer. In such conditions there is significant upstream storage available, which is utilised before a sufficient hydraulic gradient is achieved for forward motion of the water. The formulation of the model assumes that there are no interactions between the 'slugs' advecting through the interceptor sewer. However, if the slug flow approach were to represent backwater effects, the 'slugs' would have to interact. A possible modification to this modelling approach is described as a recommendation for future work.

The optimal pollution control models have been verified in this study to a post-processing hydraulic verification routine, which was validated to WALLRUS. The results in Chapter 5 showed that the numerical codes in WALLRUS were unable to fully represent the rapid changes in sewer dynamics, illustrated by instability in the results. Therefore, the control strategies have not been fully validated against a full hydrodynamic sewer simulation model. This further validation would add confidence to the feasibility of the control strategies from the optimal pollution control model. Clearly, the ultimate validation would be achieved on-line in a real interceptor system comparing sensed data with the modelled data.

The optimal pollution control model is intended to be the 'decision-maker' requiring other applications to generate inflow hydrographs and time-varying pollutant concentrations. This is perhaps a limitation of the model because there are few commercial software packages available, particularly for the pollutant concentration modelling, that are appropriate for this application, i.e. computationally efficient. On the other hand, this may be considered a strength of the control model, allowing greater flexibility for the user to apply the most convenient software package. Nevertheless, the optimal pollution control model relies on the prediction of hydrographs and pollutant concentrations before control decisions can be made.

It must be emphasised that the results presented illustrate the potential of the application of the OPC model. The OPC model has not been applied in real time and there are additional consequences of not having exact deterministic inputs for the hydrographs and pollutographs (for example, uncertainty in data, incomplete data, etc.). Additional problems would be encountered during equipment malfunctions.

The inflow hydrographs and pollutant concentrations modelled in this study were not fully representative of the Liverpool Sewer system but they did allow for a comparative evaluation of control procedures. It would be interesting to quantify the real-life improvements and this is included as a recommendation for future work.

The OPC model is currently programmed in FORTRAN 90 code, which is only intended to form a prototype software package. The model really needs re-programming to improve the user 'friendliness', perhaps by the inclusion of a graphical user interface (GUI). As previously stated, the OPC model is the decision-maker requiring the data from other software applications. However, the OPC model is not coded to interface with other software applications and these outputs have to be edited manually by the user to form input files in the OPC model.

Other minor limitations are listed below:

- the overflow chambers were currently modelled as circular and vertically sided (so unable to cope with benching without re-coding);
- the penstocks were modelled as rectangular;

- the computational efficiency of the model may be capable of improvement by using alternative numerical schemes for the numerical solution of non-linear equations;
- there were simplifications in overflow chamber modelling; and
- there were simplifications in pollutant concentration modelling as represented in the case study investigations;

8.3 CONCLUSIONS

The current operating procedures in real-life interceptor sewers that have active control devices have been shown to be relatively crude, in terms of maximum utilisation of sewer infrastructure. Improved utilisation is achieved through volumetric-based global control where conditions throughout the sewer system are used within the control procedure. Such control procedures significantly reduce the pollutant over-spill load to the receiving waters. However, this study has shown that further improvements can be achieved with pollution-based global control where the pollution load retention in the system is maximised. Extensive results from a typical year of rainfall in Liverpool have shown that this further improvement is significant and should be considered as a goal for sewer performance. It is impossible to achieve lower pollutant over-spill loads with other control procedures and it therefore represents the ultimate in performance.

The regulatory authorities give little flexibility in their standards to allow the application of global control procedures. The current standards in the UK for example impose discharge consents for each individual outfall. For the application of global control (volumetric or pollution-based) a global consent for the entire sewer system would be required. However, there is little scope for these consents in the current standards. It is envisaged that the results in this study will increase the water industry confidence in pollution-based real time control and therefore influence sewer operators and regulators alike, demonstrating that this should be considered as an ultimate goal in interceptor sewer operation.

Overall, a robust, computationally efficient optimal pollution control model has been developed using a novel 'slug flow' approach. The optimal pollution control model procedure is shown in Figure 8.1 and a user guide is given in Appendix 5 together with sample input files.

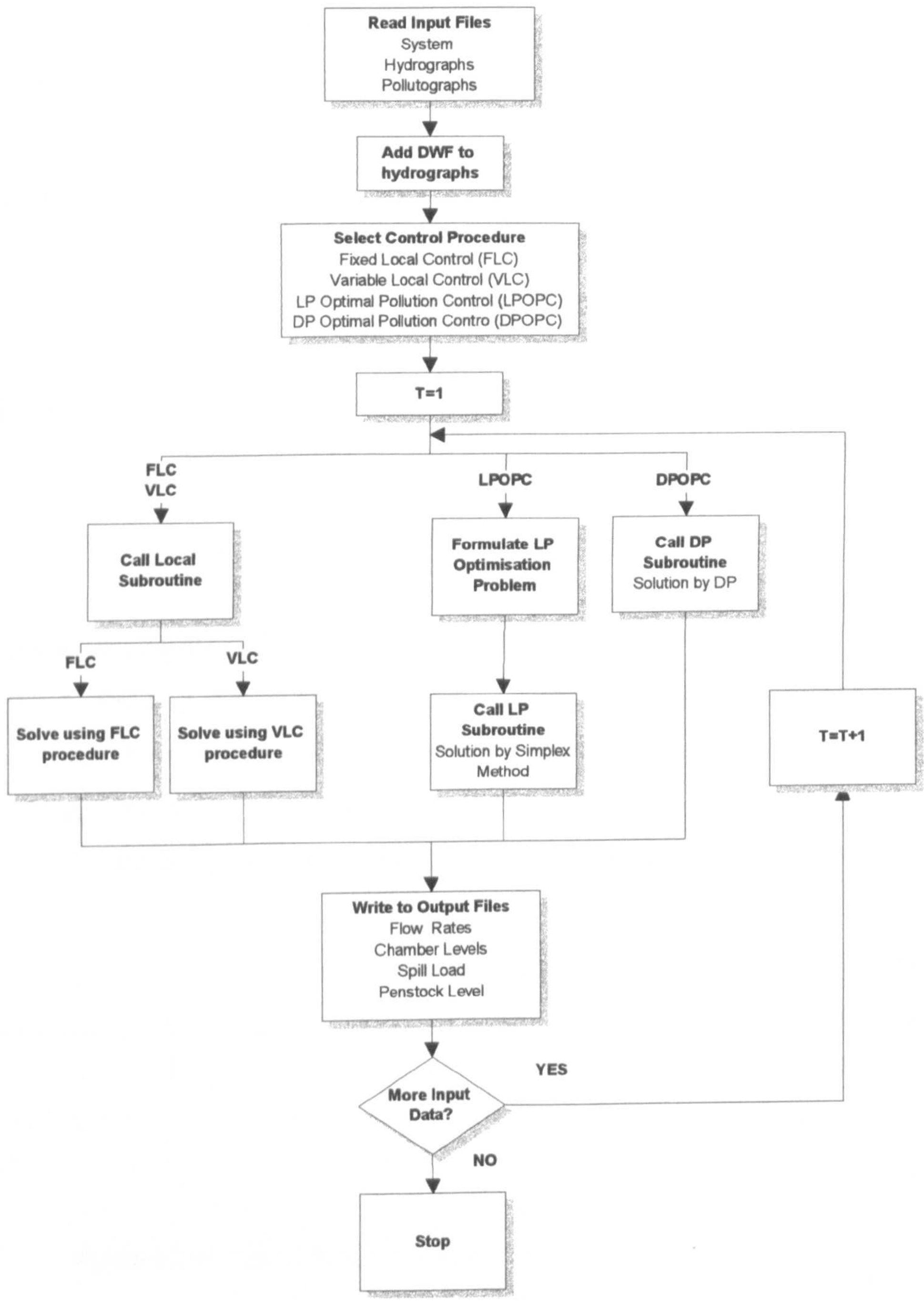


Figure 8.1: Optimal Pollution Control (OPC) Model Procedure.

8.4 RECOMMENDATIONS FOR FUTURE WORK

This section outlines possible areas of future work in the enhancement of the optimal pollution control models to overcome the limitations of the work as outlined in section 8.2. They are listed below in no particular order.

The optimal pollution control model is currently coded to determine control actions in a single pipe (see Figures 4.4 and 4.5), where the 'slugs' advect in the downstream direction. The linear programming (LP) model for this is:

$$Max\sum_{i=1}^{n}\alpha_{i,t_i}q_{i,t_i} \tag{8.1}$$

Subject to:

$$q_{i,t_i} \le Q_{i,t_i} \qquad \forall i \tag{8.2}$$

$$\sum_{j=1}^{i} q_{j,t_j} \le C_i \qquad \forall i \tag{8.3}$$

$$q_{i,t_i} \ge 0 \qquad \forall i \tag{8.4}$$

where n - the number of intercept points;

t_i - the time step position within the interceptor of intercept point i;

$\alpha_{i,t}$ - the pollutant concentration factor at intercept point i in time step t_i;

$q_{i,t}$ - the interceptor sewer flow rate after intercept point i in time step t; and

C_i – the interceptor sewer pipe capacity just downstream of intercept point i.

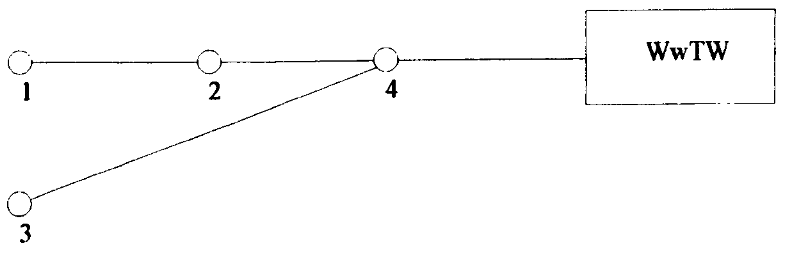


Figure 8.2: A Generic Branched Interceptor System.

A more generic case is given in Figure 8.2, where the interceptor system is branched. The LP model for this system remains as above (with the same notations):

$$Max\sum_{i=1}^{4}\alpha_{i,t_i}q_{i,t_i} \tag{8.5}$$

Subject to:

$$q_{i,t_i} \le Q_{i,t_i} \qquad i=1,2,3,4 \tag{8.6}$$

$$\sum_{j=1}^{i} q_{j,t_j} \le C_i \qquad i=1,2,3,4 \qquad (8.7)$$

$$q_{i,t_i} \ge 0 \qquad i=1,2,3,4 \qquad (8.8)$$

However, the slugs would not accumulate in the same manner as in the currently coded sequential interceptor sewer system and care needs to be taken in the numbering of the sewer system. Problems may be encountered in constraint (8.7) if the system is numbered incorrectly. In the current OPC model, the interceptor system is numbered from the most upstream intercept point to the most downstream intercept point. If this system is not adhered to then constraint (8.7) will not represent the actual system.

An alternative system is shown in Figure 8.3, which has several legs to the wastewater treatment works (WwTW).

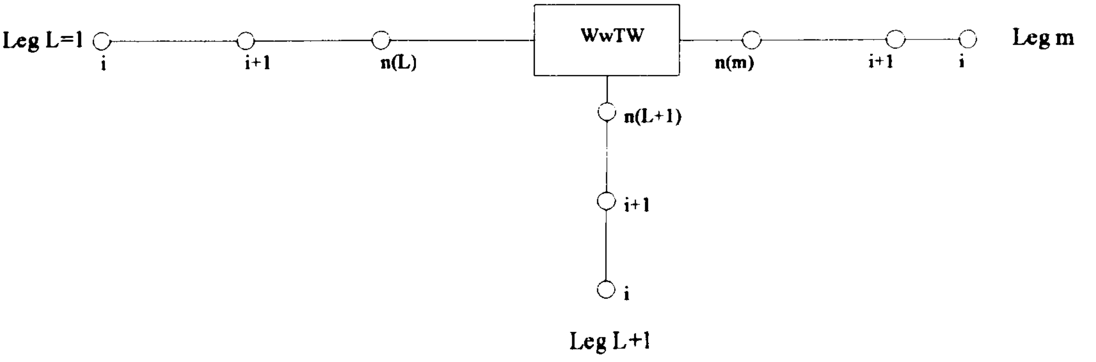


Figure 8.3: A Generic System with Several Legs.

The LP model for such a system is slightly modified to those presented above:

$$Max \sum_{L=1}^{m} \sum_{i=1}^{n(L)} \alpha_{i,t_i} q_{i,t_i} \qquad (8.9)$$

Subject to:

$$q_{i,t_i} \le Q_{i,t_i} \qquad \forall i \qquad (8.10)$$

$$\sum_{j=1}^{i} q_{j,t_j} \le C_i \qquad \forall i \qquad (8.11)$$

$$q_{i,t_i} \ge 0 \qquad \forall i \qquad (8.12)$$

where L is the interceptor leg number; and m is the total number of interceptor legs.

This modified LP model is conceptually the same as the one developed in this study using the slug flow approach to advect sewage through the sewer system at pipe full

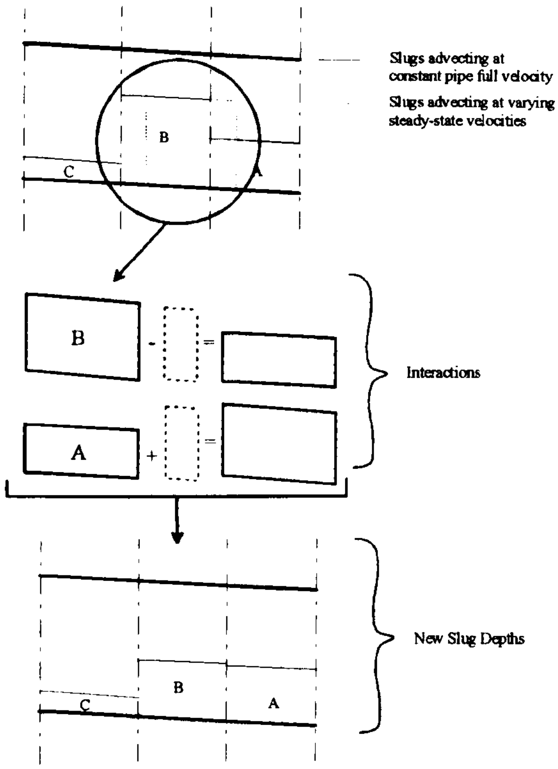


Figure 8.4: Interactions between Slugs.

velocities and the difference between the models lies in the model representation of the sewer system. This study has shown the validity of the slug flow concept indicating that it is a sound approach, which could be utilised in other sewerage systems.

The main limitation of the OPC model has been shown to be that it is unable to fully represent backwater effects, shown to be important in localised downstream storms in this study, and frontal wave advection. The effects may be included into the model by allowing interactions between the 'slugs' in the interceptor sewer system. Such modifications would allow the slug flow approach to better represent backwater effects and frontal wave advection in the interceptor sewer.

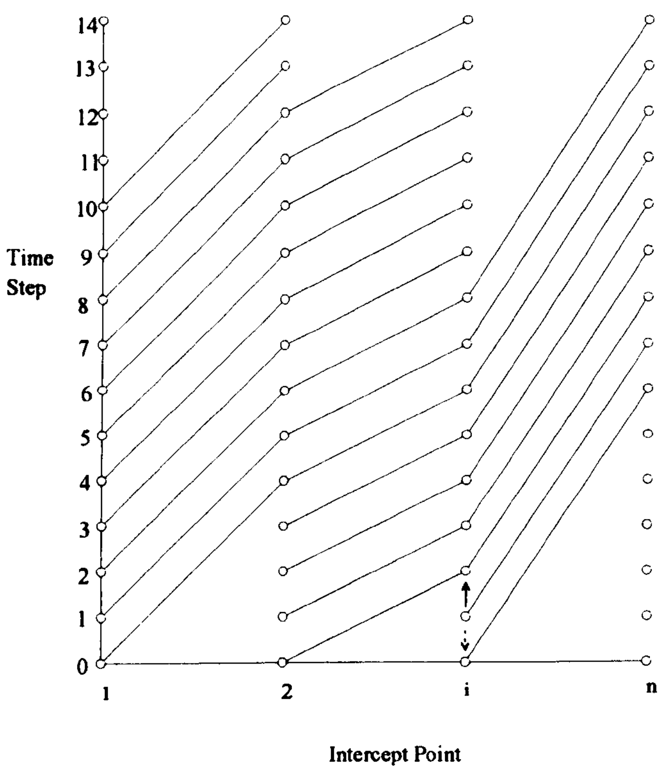


Figure 8.5: Effects Interactions between Slugs has on Solution Procedure.

Figure 8.4 shows an example of the interactions between slugs. Here, slug B is travelling faster than slug A because of its greater depth. Therefore, a proportion of slug B will 'catch-up' with slug A in the solution time step and the volume of sewage in slug A will increase, decreasing the volume in slug B and therefore its velocity.

These effects are shown in Figure 8.4, where the depth of slug B lowers as part of its sewage surges into slug A and fills the gap between slug B and slug C (so acting to diminish discontinuities in the 'slugs').

The effects that these interactions have on the model solution procedure are shown in Figure 8.5. If these interactions occur at intercept point *i* at time step 1 in Figure 8.5, then there would be interactions in the vertical direction as indicated by the arrows. The solid arrow indicates the interactions shown in Figure 8.4, i.e. slug B moving faster than slug A. The dashed arrow indicates the opposite interactions if slug B moves slower than slug C.

The inclusion of such interactions requires a degree of innovative thinking but a potential solution could be the formulation of the operational problem so that several 'slugs' can be represented. Figure 8.5 shows that in the current solution procedure, each slug is solved sequentially, where the circles represent decision points and the lines represent times of travel between intercept points. In order to model the interactions, at least three slugs (three lines in Figure 8.5) would have to included in the operational problem to allow for interactions in the upstream and downstream directions. The main problem, however, is how to include the varying slug velocities. The average steady-state velocities can easily be determined from proportional pipeflow relationships, but only after the control strategy has been determined. Therefore, an iterative procedure is required as follows: -

- Determine the control strategy using assumed velocities;
- Calculate the actual velocities of the slugs using their depths in the pipeflow relationship;
- Determine the modified control strategy using the actual velocities allowing interactions;
- Calculate the actual velocities from the new control strategy; and
- Continue until the control strategy does not change.

This procedure may be too computationally demanding to use in real time but it would be interesting to evaluate the effect varying velocities has on the control strategy.

It should be noted that (as previously stated) the strength of the solution procedure where each solution time step is solved sequentially, is that an update procedure can be implemented. This would allow for sensed data from the actual system to update the OPC model system state, i.e. chamber levels, interceptor levels, etc., from time to time. This would improve the accuracy of the OPC model.

Other recommendations to overcome the simplifications used in the present study might include: -

- Extend the model to allow it to represent various overflow chamber configurations; and
- Evaluate the alternative numerical schemes for the implicit solution of the non-linear equations, to improve the OPC model's computational efficiency;

Future studies could evaluate how pollution-based control strategies affect the effectiveness of the wastewater treatment works (WwTW). The pollutant load of treated discharge from the WwTW may be a function of the pollutant load of the flows from the interceptor sewer, i.e. the greater the pollutant load of the sewage to the WwTW, the greater the discharge from the WwTW. These studies would ensure that total pollutant load discharges from the sewer system, including WwTW discharges, were minimised. Additionally, it would be useful to investigate the effects the different control procedures have on the receiving waters. This investigation is not strictly integrated control as defined by Schütze (1998a) (see Section 3.3.1.5), but it would evaulate the receiving water's response to over-spills using the OPC model. These studies would highlight areas of sensitivity in the receiving waters and the OPC model may be re-formulated to restrict over-spills in these 'critical' locations. Furthermore, these investigations would allow further comparisons between the various control procedures used in this study.

Finally, it would be advantageous to extend the OPC model so it would be able to make control decisions in abnormal circumstances such as equipment malfunctions, incomplete and uncertain data, and tidal effects. This would increase the reliability and effectiveness of the model. Additionally, the results in Section 6.3 have shown that the OPC model control strategies cause very rapid changes in the penstock positions, which would be impractical in a real-life interceptor sewer system. It is likely that these rapid changes would increase the frequency of operating problems. Therefore, these penstock movements could be reduced by the inclusion of a constraint in the LP model. Currently, the OPC model allows the interceptor inflows to be $0 \leq Qc \leq Qcmax$, i.e. a complete range is permissible. However, the constraints of the model could be modified to give boundaries to the permissible inflows depending on the allowable movement of the penstocks. The inclusion of such a constraint would reduce the rapid changes in interceptor sewer flows, such as surge waves.

On the software perspective, more attention could be devoted to the coding of the OPC model to improve user friendliness and to allow for the interface with other applications to allow the automatic generation of input data. Of course, should the model be implemented on a real-life interceptor sewer then the model would have to be set-up on-line to receive sensed data from the sewer system and, possibly, weather radar to allow for the predictive real time control of interceptor sewer systems.

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APPENDIX 1

OVERVIEW OF EXISTING SOFTWARE

This appendix briefly describes some of the existing software for the simulation of flows, pollutants in a sewer system and for the control of sewers. A detailed description is not given and the reader is advised to consult other publications on the software packages. The packages are presented in alphabetical order.

A.1 COSSOM

A detailed description of this model is given in Section 7.4.

A.2 HYDROWORKS

The HYDROWORKS package was developed by Wallingford Software in the UK and is the successor to the WALLRUS package. According to the promotional material, it is designed as an integrated package. The package has various components: the main HYDROWORKS PM+ package and optional upgrades HYDROWORKS QM, HYDROWORKS RTC, and HYDROWORKS DESIGNER.

The hydraulic model in the HYDROWORKS PM+ package is based on the full Saint Venant equations, enabling the modelling of backwater effects and reverse flow. Surcharged flows are modelled using the Preismann slot concept. A wastewater generator calculates dry weather flow using populations, catchment area and per capita flow.

The HYDROWORKS QM optional upgrade is an advanced water quality simulator based on the development of MOSQUITO. The pollutants modelled include total suspended solids, BOD, COD ammonia, total nitrogen and total phosphorous. It also allows for user-defined pollutants, bed-load sediment fractions, and the modelling of bed-load movement separately from the suspended sediment movement. Physical process models include a surface pollutant build-up model, surface pollutant wash off model, gully pot model, wastewater generator, sediment transport model and an in-pipe water quality model. The transport of suspended sediment and dissolved pollutants is modelled using a mass-conservation approach where dispersion is assumed to be negligible. No physical or biochemical degradation is modelled in the package.

HYDROWORKS RTC is the real time control module of the software package.

HYDROWORKS DESIGNER is the wastewater systems design module, which is based on the modified rational design method.

A.3 KOSIM

KOSIM is a model for the long term, continuous simulation of flow and water quality variables in combined sewer systems developed at the Institute of Wasserwirtschaft of the University of Hanover, Germany.

A sewer network is represented by sub-catchments, which are interconnected by pipes. On-line and off-line reservoirs, pumps and overflows can also be defined.

For each sub-catchment, a conceptual hydrologic model calculates the net rainfall. For impervious areas, the model accounts for initial losses, an exponential decay of the depression losses and a final loss rate. Horton's infiltration equation is used to calculate the losses from pervious areas. The outflow hydrograph of a sub-catchment is obtained through the unit hydrograph or Nash cascade. The routing of the inlet hydrographs in the interconnecting pipes is described by a linear translation. Therefore, this modelling approach cannot model backwater effects.

Pollutants are assumed to originate from two soucres: domestic wastewater and rainfall-runoff. The pollutants are routed trough the system, where they are assumed to mix completely and without any interactions. Optionally, sedimentation and resuspension on the surface and in the sewers can be modelled in KOSIM. Another option allows the sedimentation of pollutants in storage tanks. If this option is not set, then the pollutants are assumed to be completely mixed.

Demuynck et al. (1996) presents the application of KOSIM to a sewer network in Brussels. Other researchers have also used KOSIM, including Bauwens et al. (1996), Schütze (1998a).

A.4 MICRO DRAINAGE

Micro Drainage has developed two software packages: WinDes and WinDap. WinDes was developed primarily for the design of new sewer systems although there is also a facility for the analysis of systems. This package is not discussed further here.

WinDap is a drainage area planning suite and, therefore, was developed for the analysis of sewer systems. According to the promotional material, this package is an integrated suite of analysis, design and simulation resources that automates every stage of the drainage area planning process including CSO analysis and design. The package has several components:

- *QuAM* – This module searches through the system survey data for errors (e.g. negative backfalls, pipes above ground level, etc.) before any analysis begins. The errors are ranked according to the probability of the listed error actually being incorrect.
- *VeriData* – This module assists the calibration and validation of the model with the real installation.
- *Simulation* – This module provides analysis of system overloads, storage, reverse flow characteristics, surcharges and backwater effects.
- *CASDeF* – The WinDap package has been developed with an expert system, CASDeF, which determines potential solutions for drainage problems. The user sets the constraints of the problem and this module determines a solution. This may not be an optimal solution but the user has the opportunity to change the constraints of the problem to determine a new solution. The CASDeF module also has the facility to use RTC, allowing for simple RTC strategies in the sewer system.

- *Rainfall Workshop* – This module provides the opportunity of inspecting the rainfall files with the facility to decompress and create hyetographs from IDF data. Also, a "Super-Storm" can be created that, according to the developers, is a single rainfall file that comprises all the significant events from many rainfall files and delivers a result equivalent to the analysis of the full set.

At present, there are no facilities within the WinDap package that model the water quality processes in the sewer system.

A.5 MOSQUITO (now subsumed within Hydroworks)

The detailed deterministic sewer flow quality model MOSQUITO was developed by Wallingford Software, U.K. as an add-on package to the flow simulation model WALLRUS. The objective of the MOSQUITO package was to simulate the behaviour of pollutants and sediments in sewer systems for different rainfall and foul inputs and to produce pollutograph outputs.

MOSQUITO simulates surface runoff, pollutant transport (dispersion is neglected), sedimentation, wash off and sediment transport. Pollutants are modelled in two forms (dissolved and suspended) and include BOD, COD, ammonia and suspended solids. Three sub-models simulate the behaviour of the sediments and pollutants in the sewer. Dissolved and suspended pollutants are routed by advection whilst sediment transport, deposition and erosion are based on the Ackers-White equation (Ackers and White, 1973). Complete mixing is assumed to occur at manholes but sediment settlement is modelled at CSOs and tanks. MOSQUITO does not simulate any biochemical interactions between pollutants nor are any degradation processes considered.

A.6 MOUSE

The MOUSE software package is an integrated modelling package of urban drainage and sewer systems (according to the promotional publications) developed by the Danish Hydraulic Institute. The package contains several standard modules and a number of add-on modules, which are described below.

A.6.1 MOUSE Standard – Surface Runoff Models

Three levels of sophistication are provided within this module:

- a time area curve model;
- a more detailed hydrological description, including a non-linear reservoir routing of hydrographs; and
- a linear reservoir model.

The outputs from the runoff models are the discharges from each catchment exposed to rainfall.

A.6.2 MOUSE Standard – Pipe Flow Model

The pipe flow model carries out computation of unsteady flows in pipe networks. The computation is founded on an implicit, finite difference numerical solution of the St. Venant equations. Therefore, backwater effects and surcharges are simulated within the model. Pressurised flows are also computed by the inclusion of a narrow slot as a vertical extension of the closed pipe cross-section.

A.6.3 MOUSE Add-on – MOUSE NAM – Continuous Models of Runoff Processes

MOUSE NAM is a tool for detailed modelling of the complete land phase of the hydrological cycle. The prediction is routed through four different types of storage (snow, surface, root zone and ground water) resulting in accurate hydrographs.

The module also transforms hydraulic and pollution load analysis of the sewer system to a continuous process covering both wet and dry periods. This generates a more realistic picture of the actual loads on treatment plants and combined sewer overflows.

A.6.4 MOUSE Add-on – MOUSE RTC – Analysis Tool for Real-Time Control Applications

MOUSE RTC is a model for the analysis of potentials for RTC application in sewer systems. The model can be used for long-term simulations of pipe flows and calculates expected statistical effects of various applied control strategies. The model includes a wide selection of controllable devices with user-configurable control rules.

A.6.5 MOUSE Add-on – MOUSE TRAP – Sediment Transport and Water Quality

MOUSE TRAP is a package of modules for the simulation of sediment transport on the catchment surfaces and within the sewer systems. The package consists of the following modules:

- The *Surface Runoff Quality Module (SQM)* models the build-up and wash off of surface sediments and of dissolved pollutants in gully pots.
- The *Pipe Sediment Transport Module (ST)* simulates in-pipe sediment transport including deposition and erosion processes for graded sediments. The module runs in parallel with the Pipe Flow Model, simulating the dynamic development of sediment deposits, providing a feedback to the hydrodynamics from the changed resistance in the pipes due to the sediment.
- The *Pipe Advection-Dispersion Module (AD)* simulates the transport of dissolved substances or suspended fine sediments in the sewer flow. The solution of the advection-dispersion equation is obtained using an implicit, finite-difference scheme.
- The *Pipe Water Quality Module (WQ)* is an add-on module for the AD Module that describes the reaction processes of multicompound systems, including the degradation of organic matter, bacterial fate, exchange of oxygen with the atmosphere and oxygen demand from eroded sediment sewer sediments.

MOUSE also has the facility to link with the Geographic Information System (GIS) Arcview.

MOUSE ONLINE is a module for the model-based real-time control of urban sewer systems. The module is run in forecast mode and uses the MOUSE model and knowledge-based modules within the control loop to determine control strategies. MOUSE ONLINE is implemented as a further level superimposed on a standard SCADA (System Control and Data Acquisition) package. Two other RTC tools are MOUSE PILOT and MOUSE SIMULATOR. MOUSE PILOT enables the designer of a real-time control system to test the long-term effects of various specific control strategies. MOUSE SIMULATOR was established for the testing of the on-line system, taking on the role of the instrumented sewer system. It enables the designer to test all sorts of standard and non-standard events, including sensor malfunctions and communication breakdowns. These models are described and applied to case studies in so called model based control (Lindberg *et al.*, 1993; Nielsen *et al.*, 1993; Amdisen *et al.*, 1994).

A.7 SIMPOL

It has been identified that the use of an advanced water quality model is an onerous process (Dempsey et al., 1997 and UPM Manual; FWR, 1994), and this can form constraints for planners investigating potential upgrading or operational options (N.B. The UPM Manual referenced gives a thorough description of the SIMPOL model but a summary is given in Dempsey et al. (1997)). An alternative simplified approach has been developed in SIMPOL as part of the UPM Procedure. Accuracy is lost in this approach, although calibration against detailed models (e.g. MOSQUITO) ensures adequate accuracy, but a greater range of event simulations are possible because of shorter execution times.

SIMPOL represents the elements of the sewer system as a series of tanks. These are:

- a *surface tank* that models the rainfall-runoff process using a standard percentage relationship without any storage. The runoff BOD concentration is assumed to be constant;
- a *sewer tank* that attenuates the flows within the sewer system and the tank outflow is governed by a non-linear relationship of the volume in the tank. BOD is deposited and eroded (based on the runoff quantity) during storm events. These attenuation and sediment parameters are used to calibrate the model to detailed models.
- a *CSO tank* that represents a simple on-line tank with a maximum pass forward capacity; and
- a *storm tank* that represents an off-line tank, which includes an algorithm for the partitioning of suspended BOD.

The model only considers one pollutant during simulation, normally BOD, which is assumed to originate from three sources: dry weather flows, surface runoff and sewer sediments. Each rainfall event (in the STORMPAC format) is assumed to be independent and a time step of one hour is applied through the simulation.

Examples of the application of SIMPOL are given by Wotherspoon et al. (1996)

A.8 SWMM

The Storm Water Management Model (SWMM) was originally developed for the U.S. Environmental Protection Agency (EPA).

Single-event and continuous simulations may be performed for the prediction of flows and pollutant concentrations. The EXTRAN block solves complete dynamic flow routing equations for accurate simulation of backwater, looped connections, surcharging, and pressure flow.

Pollutant transport is modelled by advection and mixing in conduits and complete mixing in the storage tanks. Sedimentation, resuspension and decay processes are also included.

A.9 WALLRUS (now superseded by Hydroworks and Infoworks)

A detailed description of this model is given in Section 5.4.

APPENDIX 2

EXAMPLE SOLUTIONS

A2.1 EXAMPLE OF SOLUTION OF LP PROBLEM USING THE SIMPLEX METHOD

The following example of the use of the simplex method was extracted from Templeman (1982):

$$\text{Minimise} \qquad f = 6x_1 + 4x_2 \tag{A2.1}$$

$$\text{subject to} \qquad 2x_1 + 2x_2 \geq 60 \tag{A2.2}$$

$$2x_1 + 4x_2 \geq 80 \tag{A2.3}$$

$$4x_1 \geq 60 \tag{A2.4}$$

$$4x_2 \geq 20 \tag{A2.5}$$

$$3x_1 + 2x_2 \leq 120 \tag{A2.6}$$

$$x_1, x_2 \geq 0 \tag{A2.7}$$

Introducing the slack variables:

$$2x_1 + 2x_2 - x_3 = 60 \tag{A2.8}$$

$$2x_1 + 4x_2 - x_4 = 80 \tag{A2.9}$$

$$4x_1 - x_5 = 60 \tag{A2.10}$$

$$4x_2 - x_6 = 20 \tag{A2.11}$$

$$3x_1 + 4x_2 + x_7 = 120 \tag{A2.12}$$

$$x_1, \ldots, x_7 \geq 0 \tag{A2.13}$$

Rearranging and introducing objective function (A2.1):

$$x_3 = -60 + 2x_1 + 2x_2 \tag{A2.14}$$

$$x_4 = -80 + 2x_1 + 4x_2 \tag{A2.15}$$

$$x_5 = -60 + 4x_1 \tag{A2.16}$$

$$x_6 = -20 \quad + 4x_2 \tag{A2.17}$$

$$x_7 = 120 - 3x_1 - 2x_2 \tag{A2.18}$$

$$f(\min) = 6x_1 + 4x_2 \tag{A2.19}$$

Tabulating the simplex table:

		x_1	x_2		
x_3	-60	2	2	20	0
x_4	-80	2	4	40	0
x_5	-60	4		20	20
x_6	-20		4	60	20
x_7	120	-3	-2	20	40
f		6	4	200	160
		20	20		
		20	10		

Pivoting:

$x_3 = -60 + 2x_1 + 2x_2 \quad \therefore$

$$x_2 = 30 - x_1 + \frac{1}{2}x_3 \tag{A2.20}$$

$$x_4 = 40 - 2x_1 + 2x_3 \tag{A2.21}$$

$$x_5 = -60 + 4x_1 \tag{A2.22}$$

$$x_6 = 100 - 4x_1 + 2x_3 \tag{A2.23}$$

$$x_7 = 60 - x_1 - x_3 \tag{A2.24}$$

$$f = 120 + 2x_1 + 2x_3 \tag{A2.25}$$

		x_1	x_3		
x_2	30	-1	½	10	15
x_4	40	-2	2	0	10
x_5	-60	4		20	0
x_6	100	-4	2	20	40
x_7	60	-1	-1	40	45
f	120	2	2	160	150
		20	0		
		15	0		

Pivoting:

$x_5 = -60 + 4x_1$

$\therefore$

$$x_1 = 15 + \frac{1}{4}x_5 \tag{A2.26}$$

$$x_2 = 15 + \frac{1}{2}x_3 - \frac{1}{4}x_5 \tag{A2.27}$$

$$x_4 = 10 + 2x_3 + \frac{1}{2}x_5 \tag{A2.28}$$

$$x_6 = 40 + 2x_3 - x_5 \tag{A2.29}$$

$$x_7 = 45 - x_3 - \frac{1}{4}x_5 \tag{A2.30}$$

$$f = 150 + 2x_3 + \frac{1}{2}x_5 \tag{A2.31}$$

		x_3	x_5	
x_1	15		¼	15
x_2	15	½	-¼	15
x_4	10	2	½	10
x_6	40	2	-1	40
x_7	45	-1	-¼	45
f	150	2	½	150
		0	0	

Optimal solution found: $x_1 = 15$

$x_2 = 15$

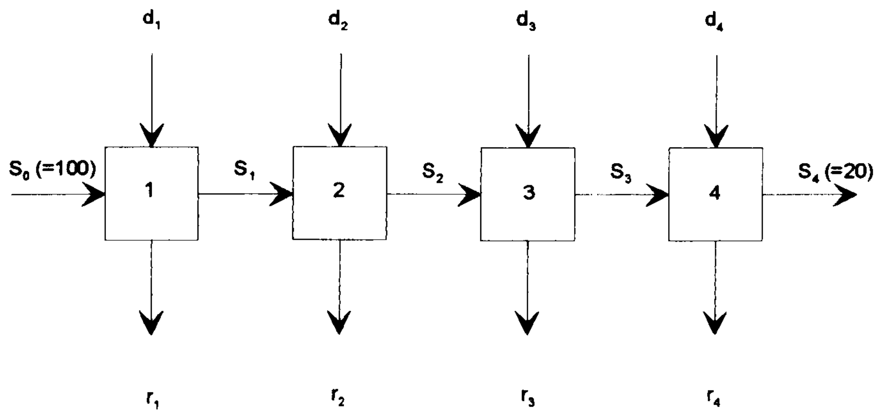
$f = 150$

A2.2 EXAMPLE OF DP SOLUTION PROCEDURE – A WASTEWATER TREATMENT PLANT

This example is somewhat theoretical but it does illustrate the DP solution procedure well. A wastewater treatment plant has four stages where the sewage can be purified by up to 30 ppm of the chosen determinand at each stage. Initially, the sewage is polluted to 100 ppm and the determinand has to be reduced to 20 ppm by the end of the process to meet regulations. In order to make the solution easier to view, each stage can only remove 0, 10, 20 or 30 ppm and the costs of removing these in each stage is given below:

	Costs (£)			
Process Stage	**1**	**2**	**3**	**4**
ppm Removed				
0	5	5	5	5
10	100	90	80	110
20	190	185	180	200
30	260	270	280	270

The wastewater treatment plant has to be operated to achieve the objective of a final pollution level of 20 ppm in the sewage but the cost has to be minimised. DP is an ideal procedure for calculating this.



The above DP serial diagram for the wastewater treatment plant operation. The DP solution procedure is shown below.

N.B.: The lines beneath some of the numbers represent points of the optimal solution. This is described at the end of the procedure.

Stage 1: $S_0 = 100$;

$d_1 = 0, 10, 20, 30$;

$S_1 = S_0 - d_1 = 100, 90, 80, 70$;

$r_1 = 5, 100, 190, 260$

Costs	$Cs_1(100)$	$= 0 + 5$	$= 5$
	$Cs_1(90)$	$= 0 + 100$	$= 100$
	$Cs_1(80)$	$= 0 + 190$	$= 190$
	$Cs_1(70)$	$= 0 + 260$	$= \underline{260}$

Stage 2: $S_1 = 100, 90, 80, 70$;

$d_2 = 0, 10, 20, 30$;

S_2 = ~~100, 90~~, 80, 70, 60, 50, 40 (100 and 90 not needed because would not achieve final objective of 20 ppm);

$r_2 = 5, 90, 185, 270$.

$$Cs_2(80) = Min\begin{bmatrix}5+185\\100+90\\190+5\end{bmatrix} = \begin{bmatrix}190\\190\\195\end{bmatrix} = 190$$

$$Cs_2(70) = Min\begin{bmatrix}5+270\\100+185\\190+90\\260+5\end{bmatrix} = \begin{bmatrix}275\\285\\280\\265\end{bmatrix} = 265$$

$$Cs_2(60) = Min\begin{bmatrix}100+270\\190+185\\\underline{260+90}\end{bmatrix} = \begin{bmatrix}370\\375\\350\end{bmatrix} = \underline{350}$$

$$Cs_2(50) = Min\begin{bmatrix}190+270\\260+185\end{bmatrix} = \begin{bmatrix}460\\445\end{bmatrix} = 445$$

$$Cs_2(40) = [260+270] = 530$$

Stage 3: $S_2 = 80, 70, 60, 50, 40;$

$d_3 = 0, 10, 20, 30;$

$S_3 = \cancel{80, 70, 60,}\ 50, 40, 30, 20;$

$r_3 = 5, 80, 180, 280.$

$$Cs_3(50) = Min\begin{bmatrix}190+280\\265+180\\\underline{350+80}\\445+5\end{bmatrix} = \begin{bmatrix}470\\445\\430\\450\end{bmatrix} = \underline{430}$$

$$Cs_3(40) = Min\begin{bmatrix}265+280\\350+180\\445+80\\530+5\end{bmatrix} = \begin{bmatrix}545\\530\\525\\535\end{bmatrix} = 525$$

$$Cs_3(30) = Min\begin{bmatrix}350+280\\445+180\\530+80\end{bmatrix} = \begin{bmatrix}630\\625\\610\end{bmatrix} = 610$$

$$Cs_3(20) \quad = Min\begin{bmatrix}445+280\\530+180\end{bmatrix} \quad = \begin{bmatrix}725\\710\end{bmatrix} \quad = 710$$

<u>Stage 4:</u> $S_3 = 50, 40, 30, 20;$

$d_3 = 0, 10, 20, 30;$

$S_4 = 20;$

$r_4 = 5, 110, 200, 270.$

$$Cs_4(20) \quad = Min\begin{bmatrix}\underline{430+270}\\525+200\\610+110\\710+5\end{bmatrix} \quad = \begin{bmatrix}700\\725\\720\\715\end{bmatrix} \quad = \underline{700}$$

The cheapest cost of operating the wastewater treatment plant and still meet the regulations of 20 ppm is £700. The solution is obtained by using a traceback procedure and is shown where the numbers are underlined. For example, the £700 final cost was achieved from the £430 cost in stage 3, i.e $Cs_3(50)$. In other words, the operation strategy is indexed during the DP procedure and a traceback procedure identifies the strategy that generated the cheapest cumulative final cost.

The operation strategy for this problem is:

Decision 4 (d_4)	Remove 30 ppm
Decision 3 (d_3)	Remove 10 ppm
Decision 2 (d_2)	Remove 10 ppm
Decision 1 (d_1)	Remove 30 ppm.

APPENDIX 3

SAMPLE SOLUTIONS FROM OPC MODELS

This appendix shows a full set of results from the application of the extended optimal control model to a test case idealised interceptor sewer system in section 4.3. The results have a common legend:

Pollutant Concentration Factors

Inflows

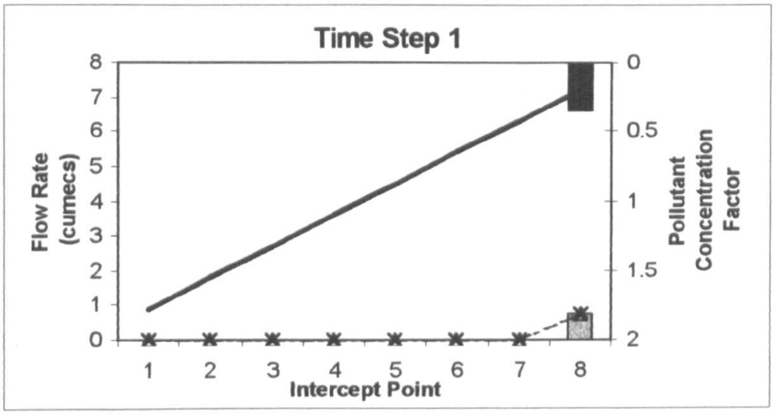
Fixed Local Control

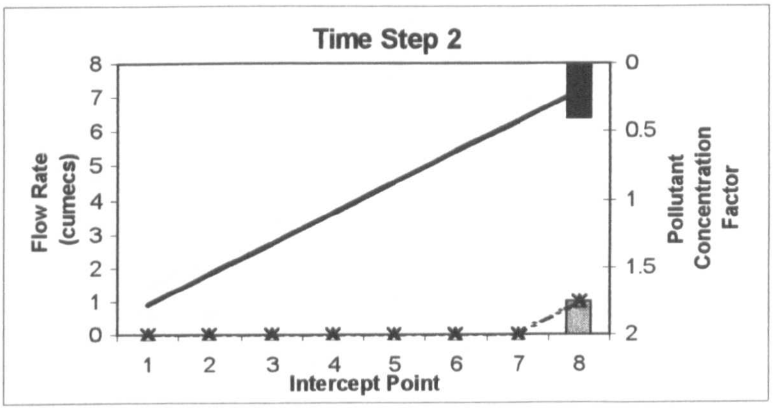
LP Global Control

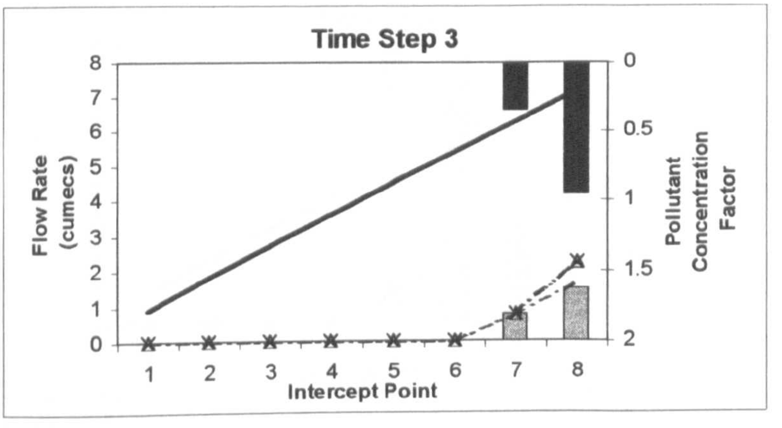
Variable Local Control

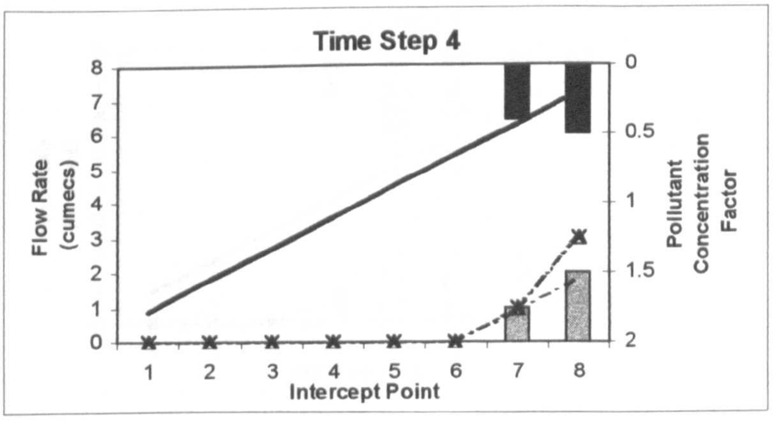
DP Global Control

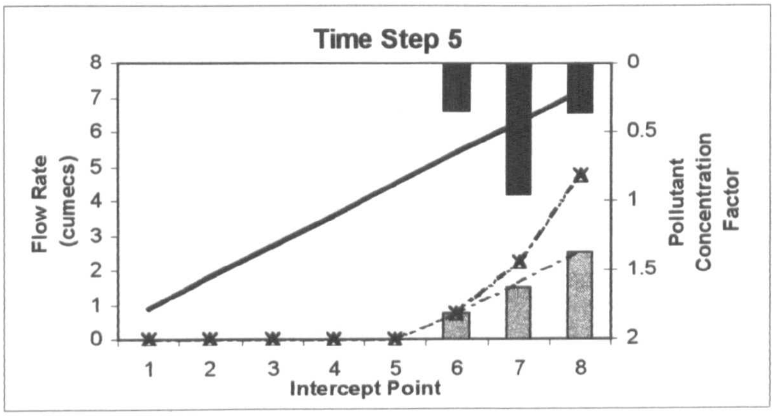
Pipe Full Capacity

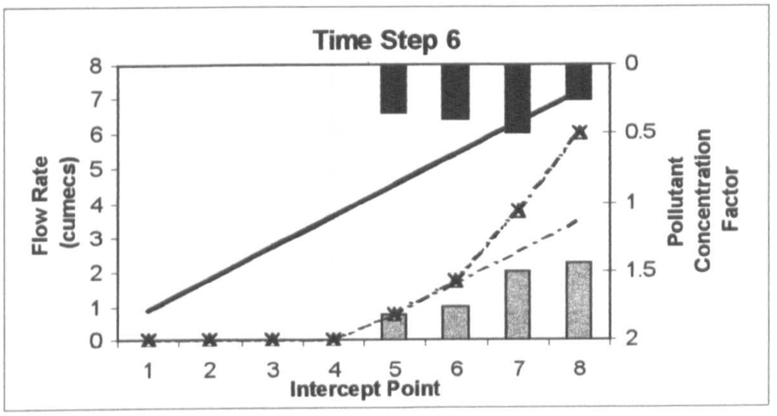

Time Step 1
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

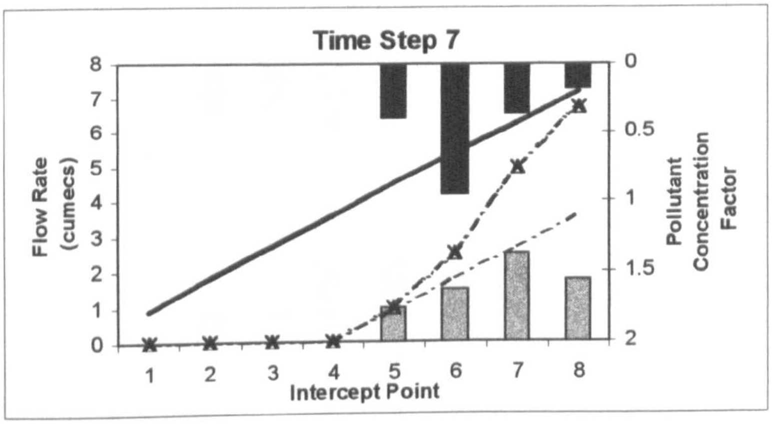

Time Step 2
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

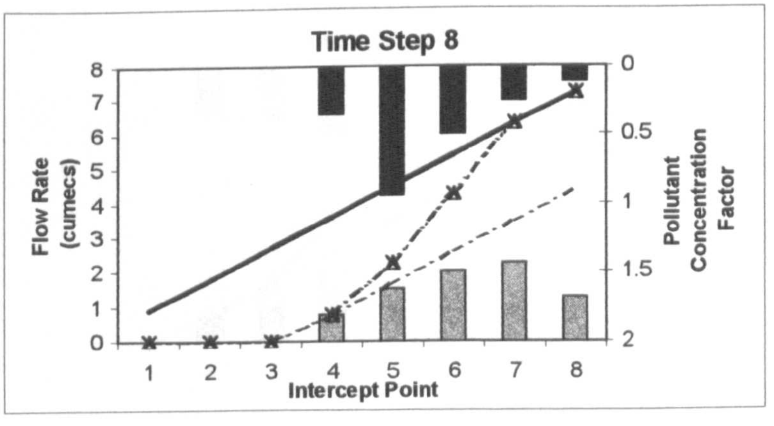

Time Step 3
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

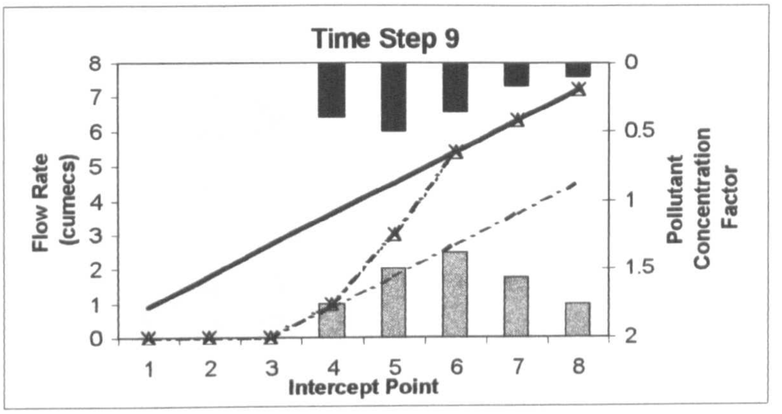

Time Step 4
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

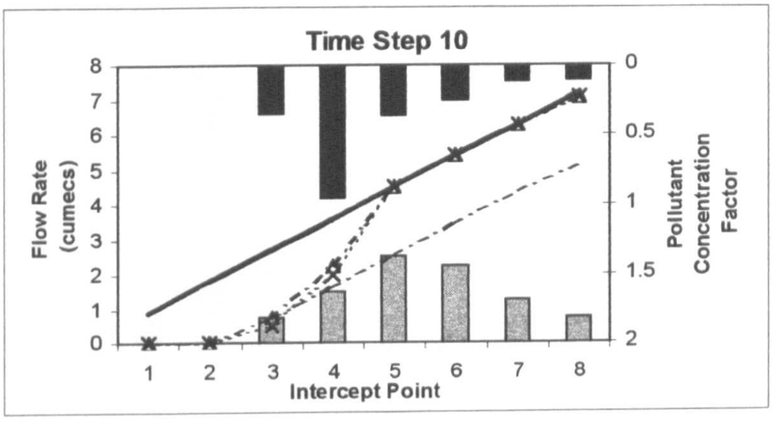

Time Step 5
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

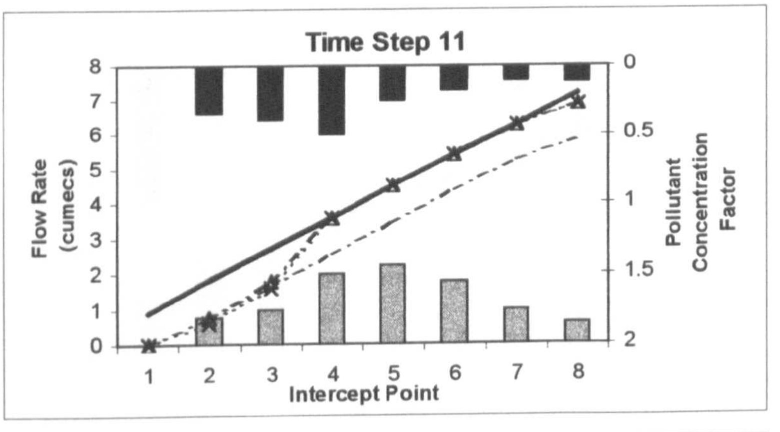

Time Step 6
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

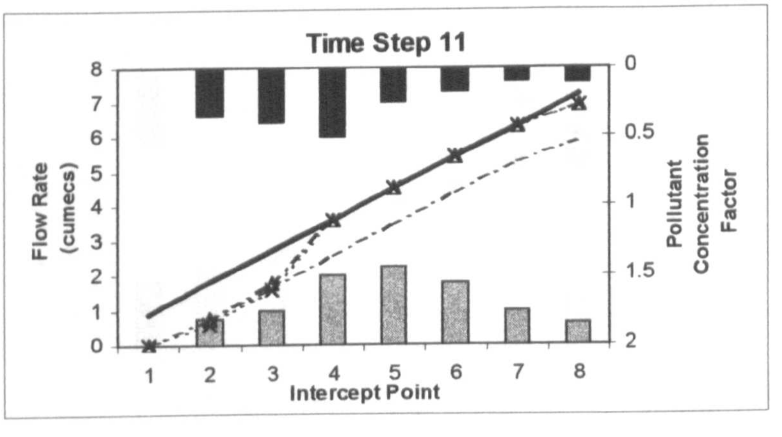

Time Step 7
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

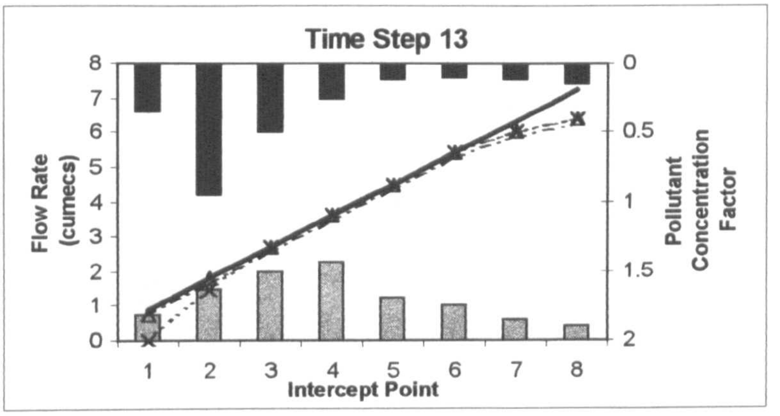

Time Step 8
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

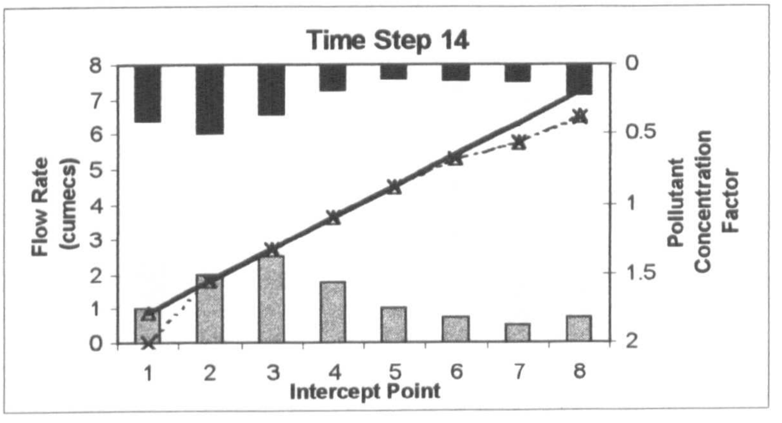

Time Step 9
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

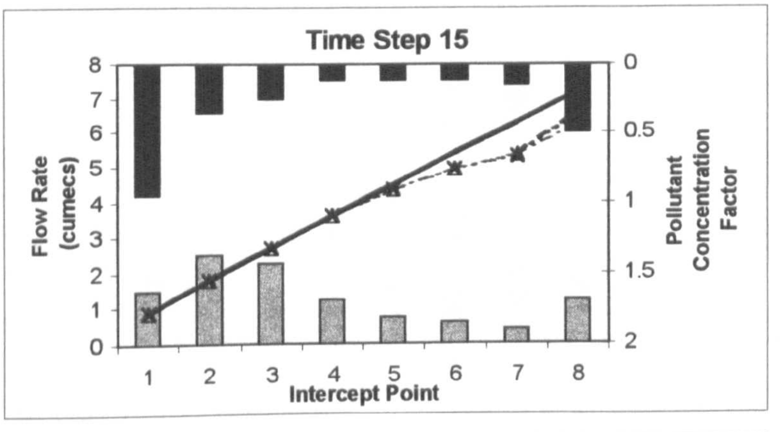

Time Step 10
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

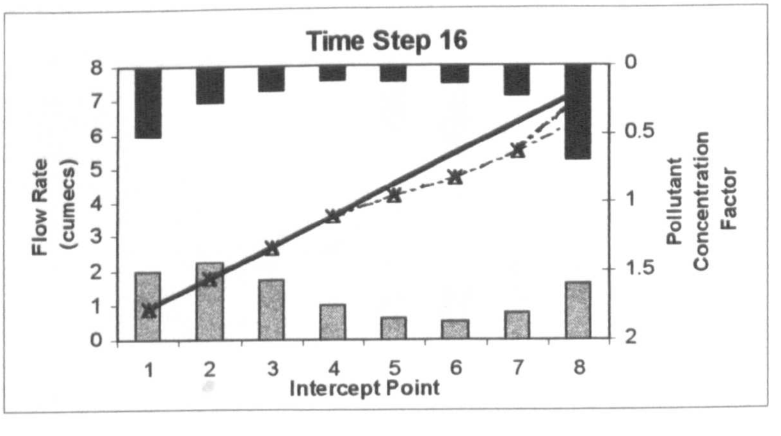

Time Step 11
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

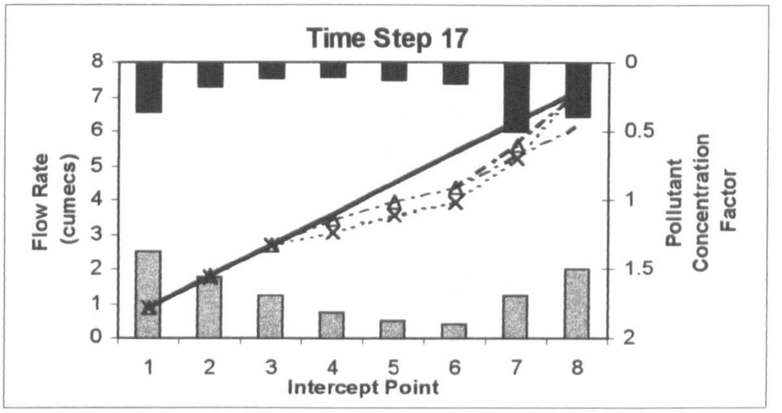

Time Step 11
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

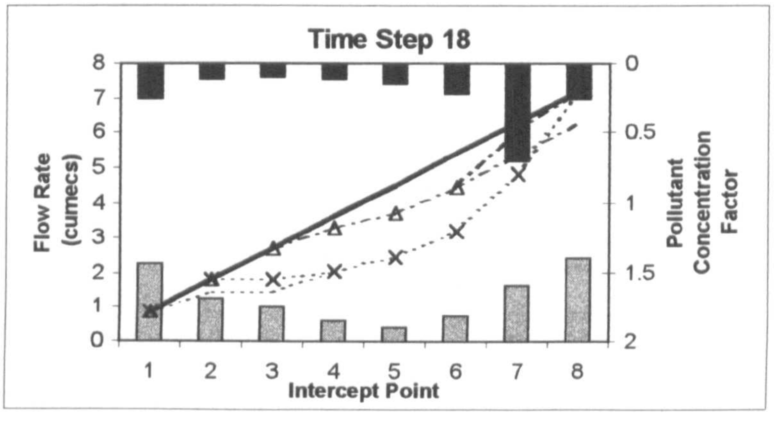

Time Step 13
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0 1 2 3 4 5 6 7 8
0 0.5 1 1.5 2
1 2 3 4 5 6 7 8

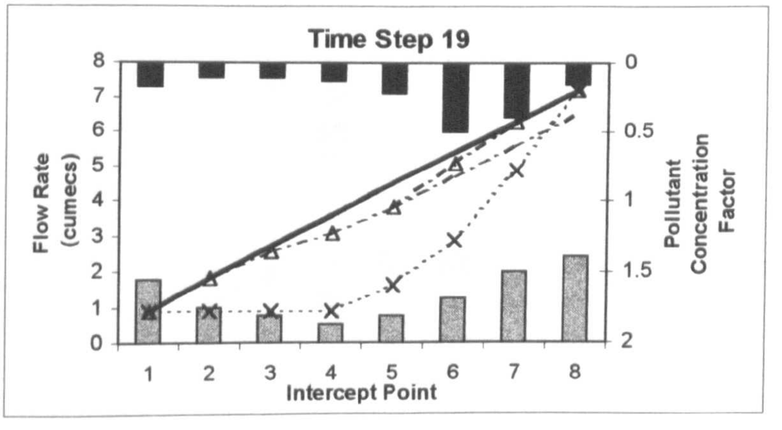

Time Step 14
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0 1 2 3 4 5 6 7 8
0 0.5 1 1.5 2
1 2 3 4 5 6 7 8

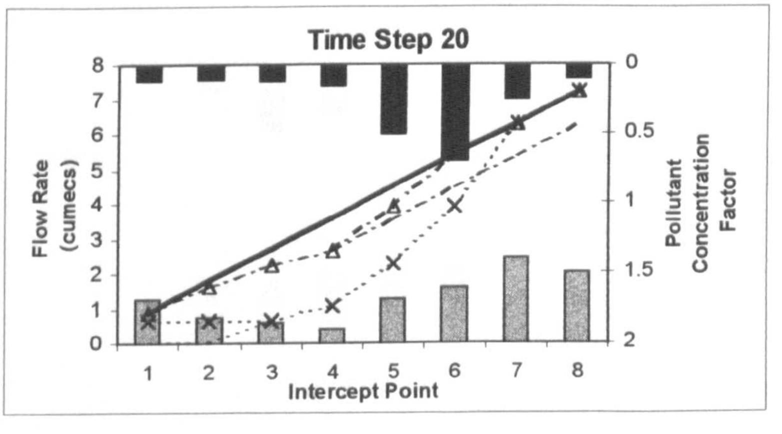

Time Step 15
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0 1 2 3 4 5 6 7 8
0 0.5 1 1.5 2
1 2 3 4 5 6 7 8

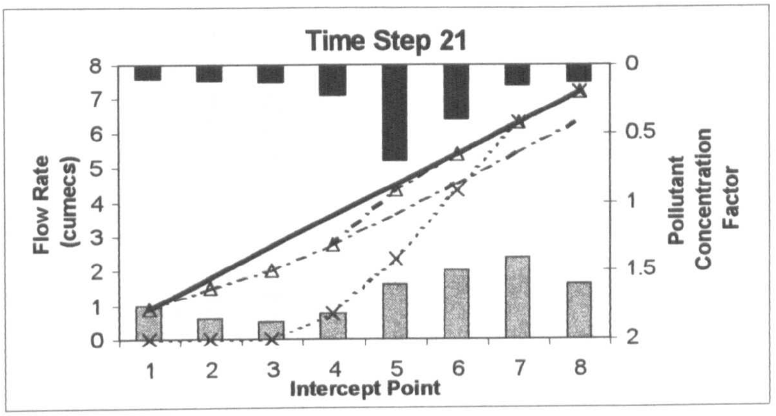

Time Step 16
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0 1 2 3 4 5 6 7 8
0 0.5 1 1.5 2
1 2 3 4 5 6 7 8

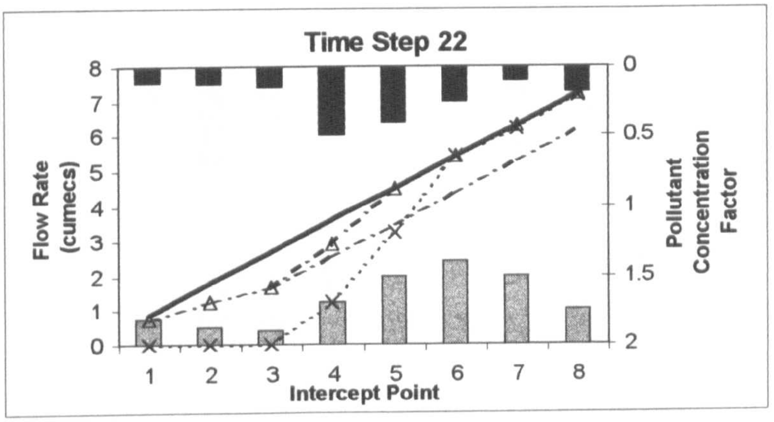

Time Step 17
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

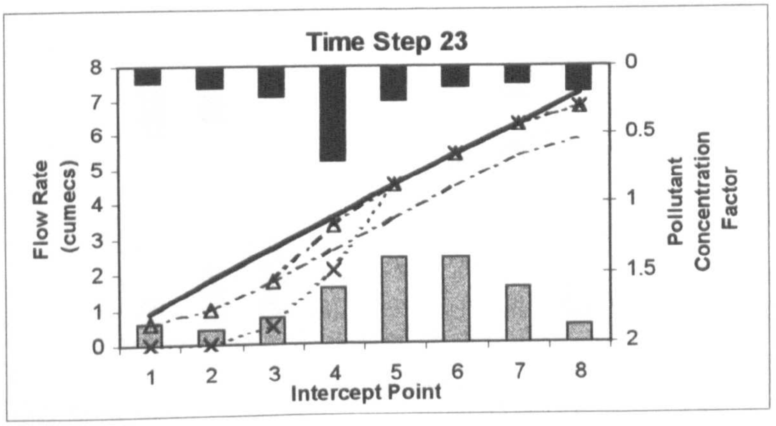

Time Step 18
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

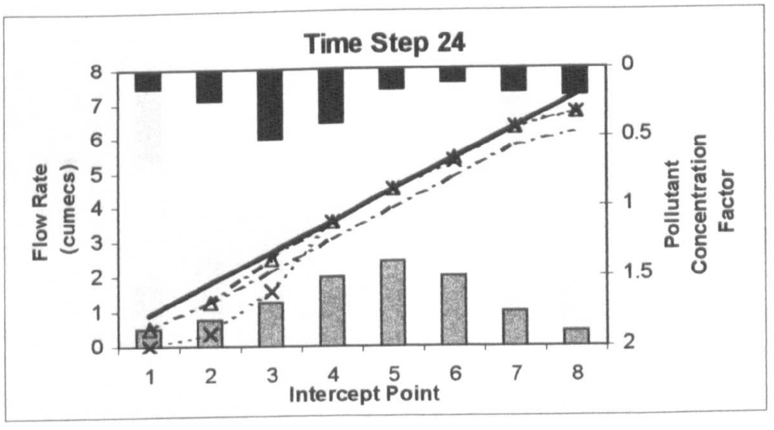

Time Step 19
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

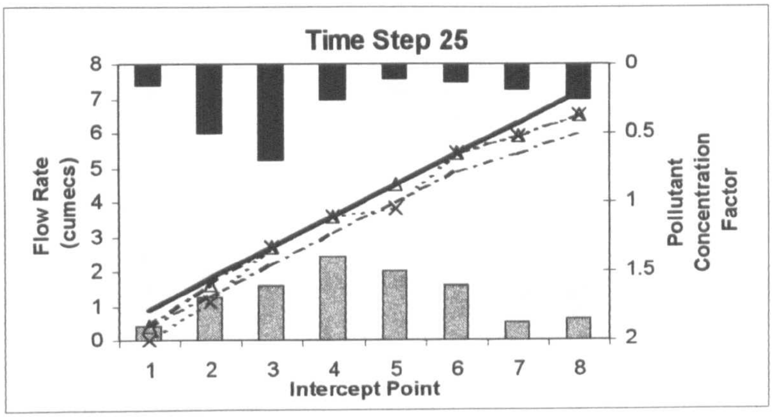

Time Step 20
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

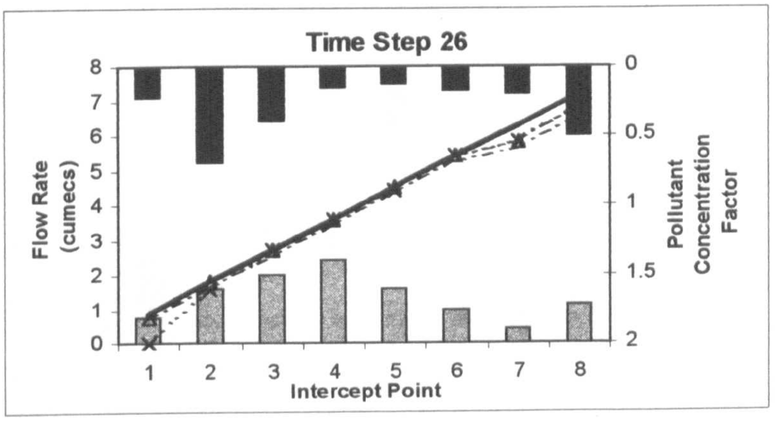

Time Step 21
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

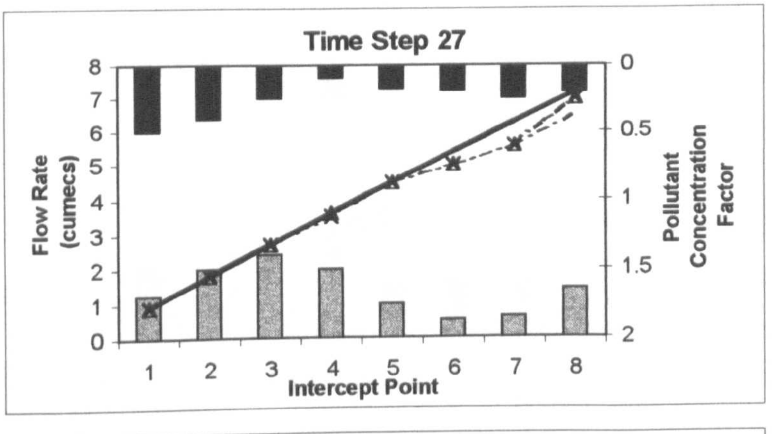

Time Step 22
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

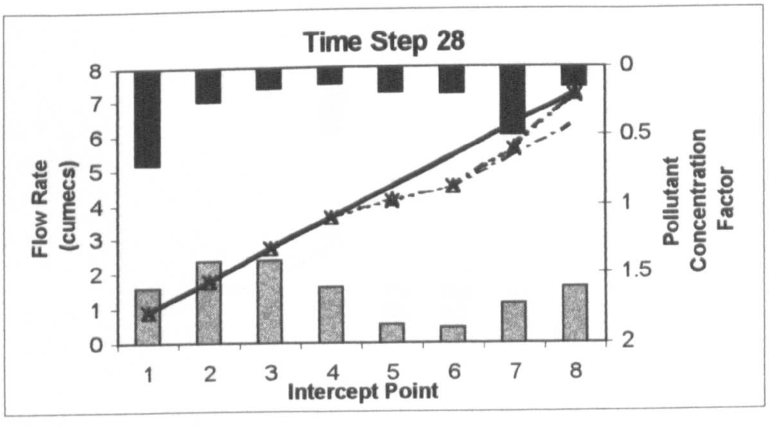

Time Step 23
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

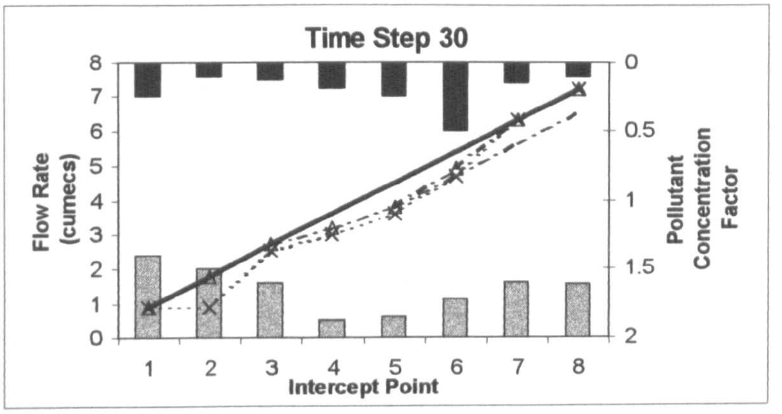

Time Step 24
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

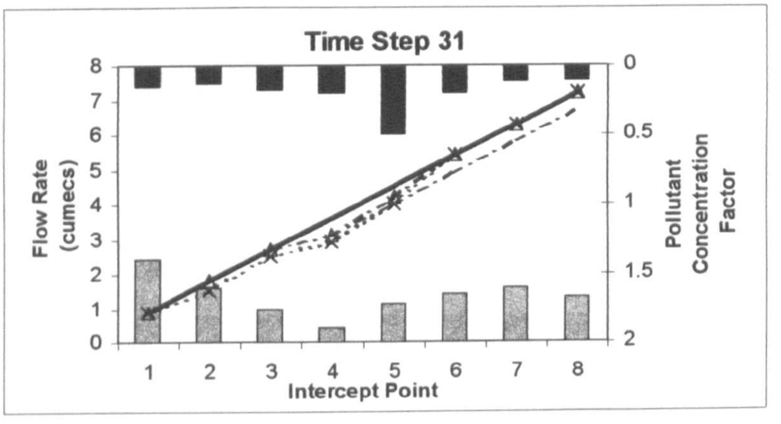

Time Step 25
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

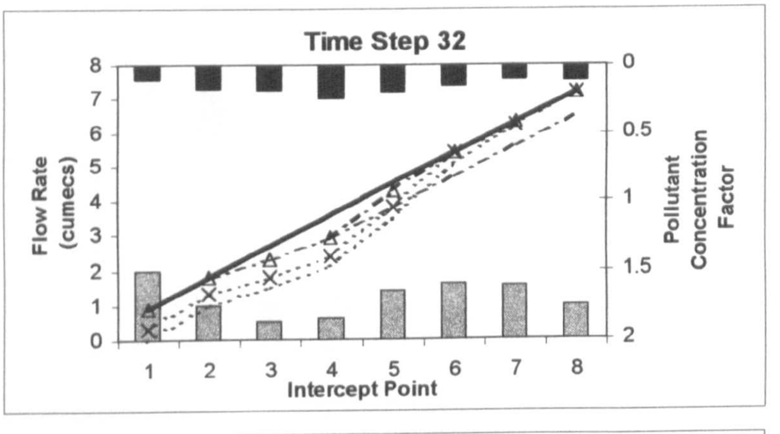

Time Step 26
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

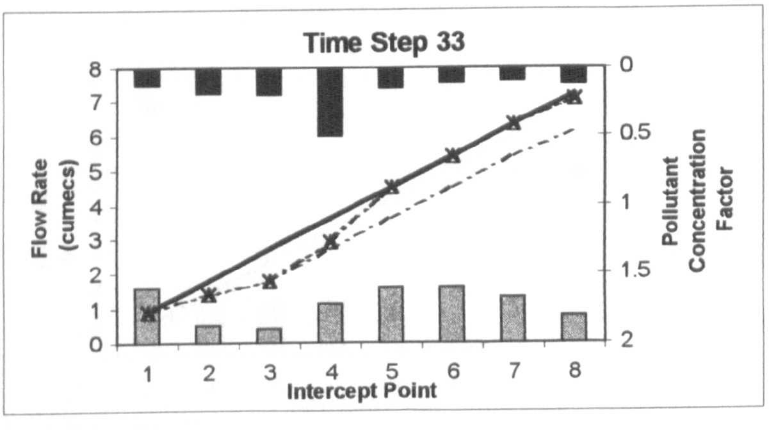

Time Step 27
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

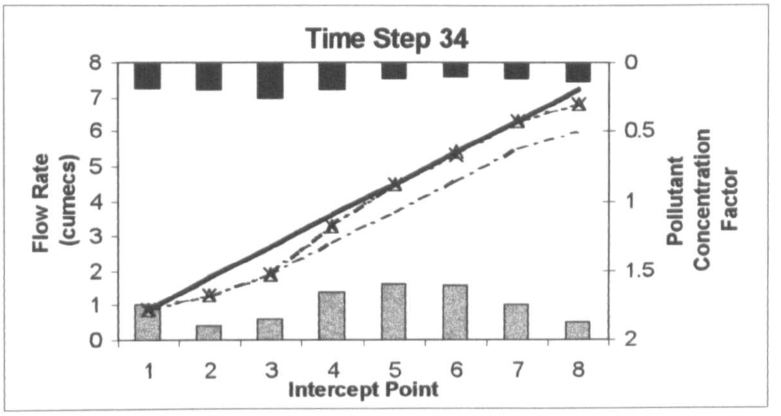

Time Step 28
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

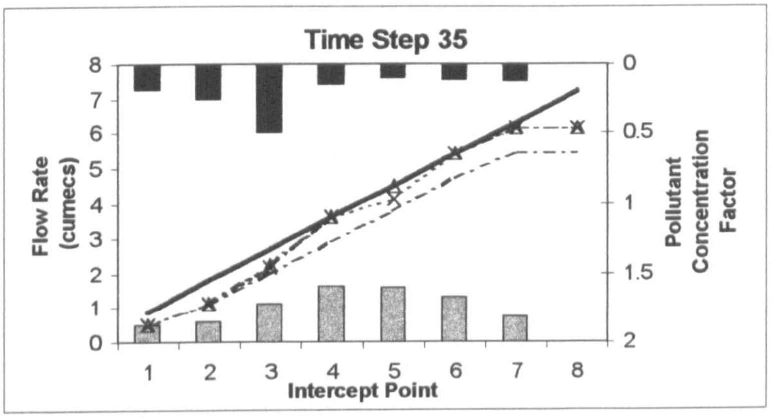

Time Step 30
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0 1 2 3 4 5 6 7 8
0 0.5 1 1.5 2
1 2 3 4 5 6 7 8

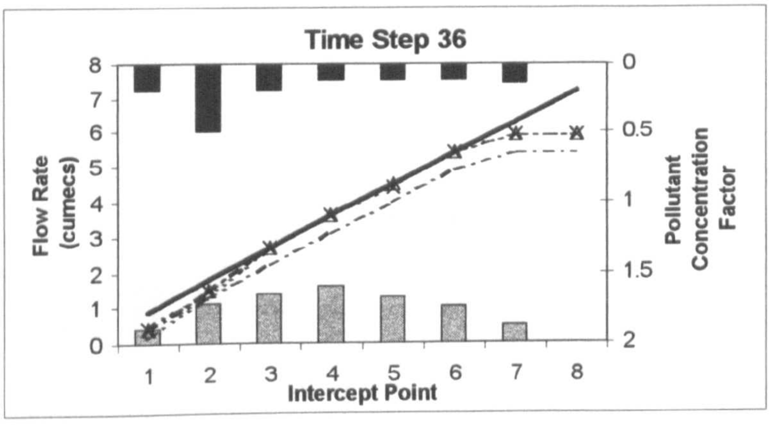

Time Step 31
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0 1 2 3 4 5 6 7 8
0 0.5 1 1.5 2
1 2 3 4 5 6 7 8

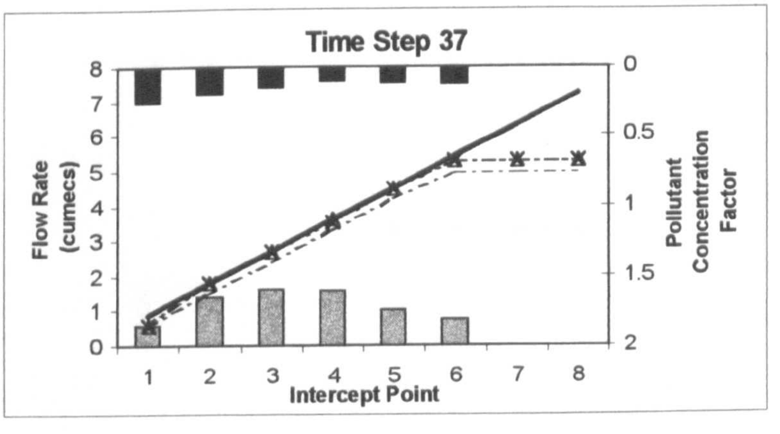

Time Step 32
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0 1 2 3 4 5 6 7 8
0 0.5 1 1.5 2
1 2 3 4 5 6 7 8

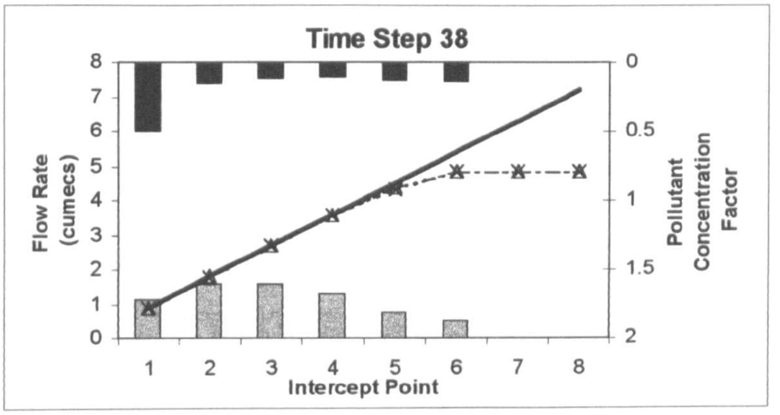

Time Step 33
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0 1 2 3 4 5 6 7 8
0 0.5 1 1.5 2
1 2 3 4 5 6 7 8

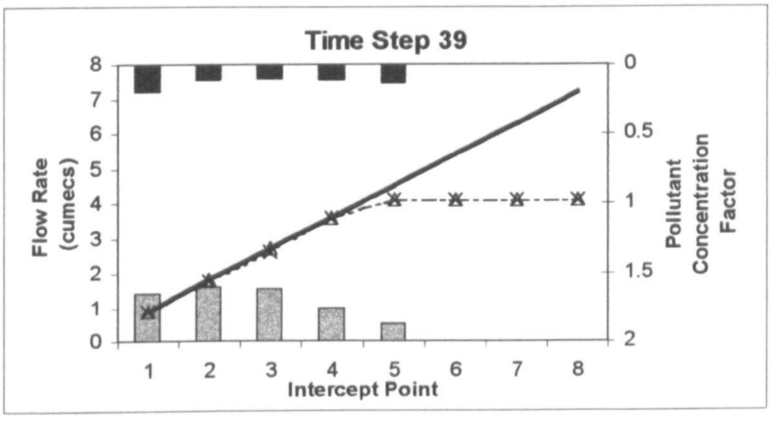

Time Step 34
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

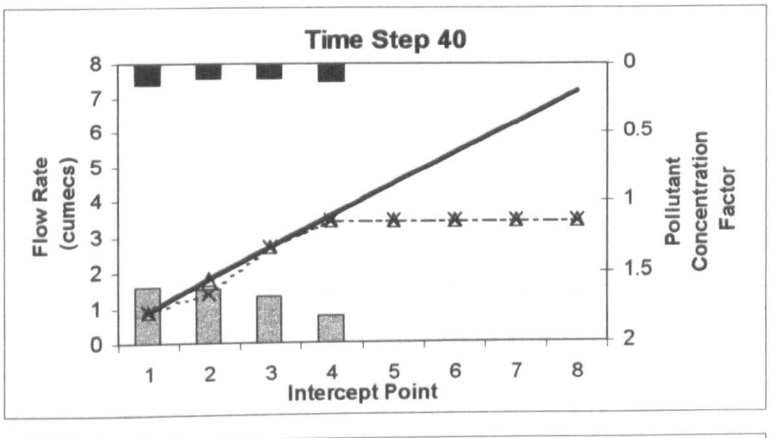

Time Step 35
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

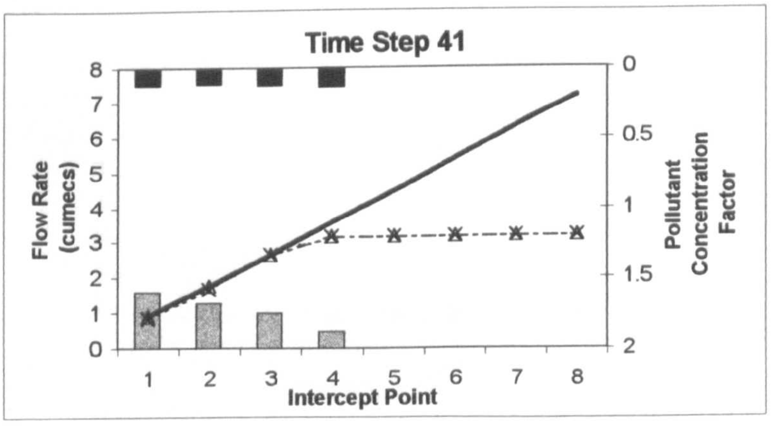

Time Step 36
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

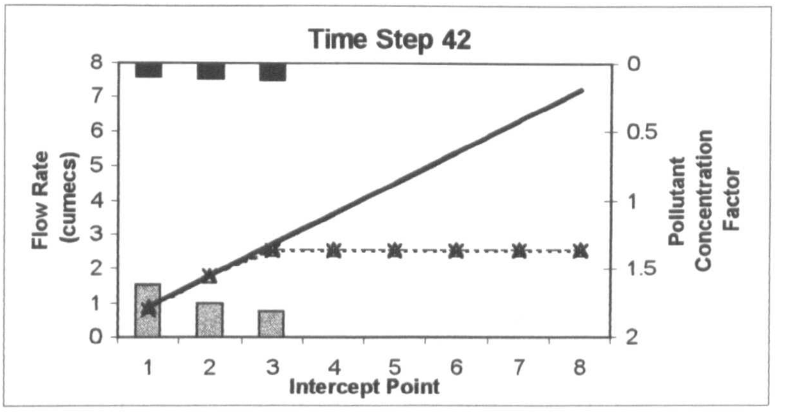

Time Step 37
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point

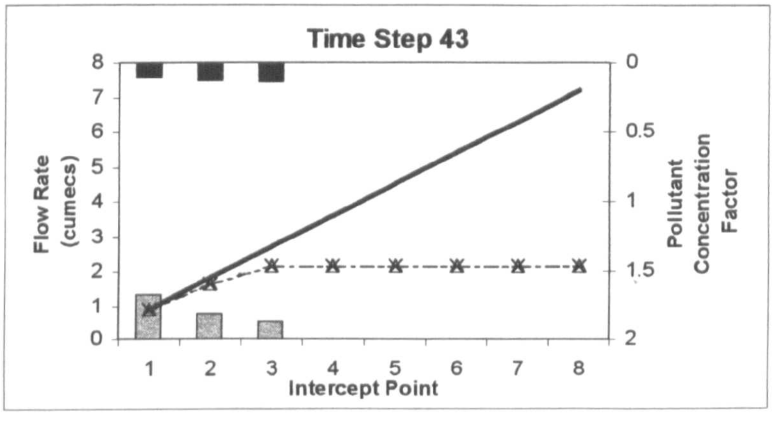

Time Step 38
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

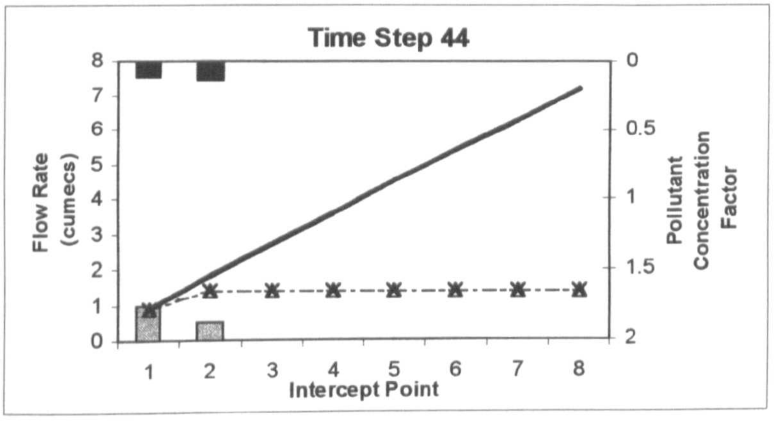

Time Step 39
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

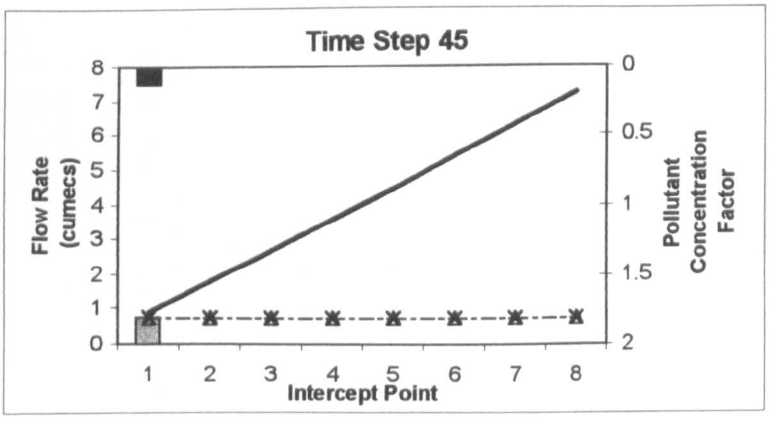

Time Step 40
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5

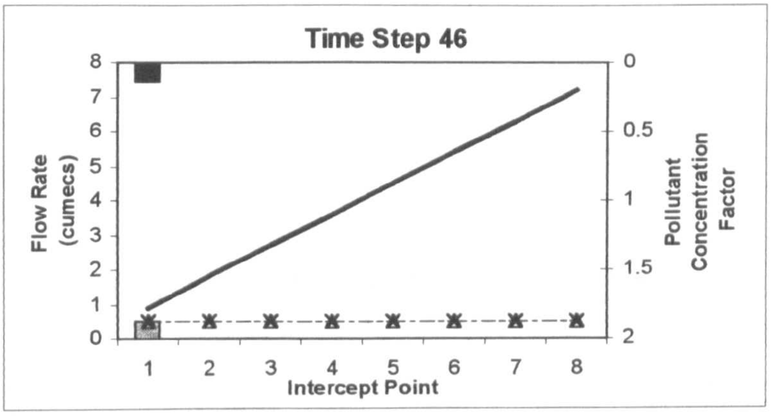

Time Step 41
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point
0
1
2
3
4
5
6
7
8
0.5
1.5


Time Step 42
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point


Time Step 43
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point


Time Step 44
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point


Time Step 45
Flow Rate (cumecs)
Pollutant Concentration Factor
Intercept Point


Time Step 46
Flow Rate (cumecs)
8
7
6
5
4
3
2
1
0
Pollutant Concentration Factor
0
0.5
1
1.5
2
1
2
3
4
5
6
7
8
Intercept Point

APPENDIX 4

POLLUTOGRAPH CALCULATIONS

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
8	February	1.26	169.28	40	2.40	0.555	4.324	751.3
8	February	5.47	3.68	560	8.40	0.601	13.977	830.3
8	February	17.43	2.60	380	4.80	0.249	19.277	961.5
11	February	13.40	61.62	890	36.00	1.385	25.993	1994.0
13	February	15.42	35.20	320	2.40	0.373	6.434	741.9
13	February	22.27	1.42	150	1.20	0.184	6.522	433.6
14	February	5.48	4.85	70	3.00	0.531	5.650	487.4
21	February	0.13	161.25	320	3.00	0.818	3.667	670.6
23	February	9.58	52.42	210	4.80	0.317	15.142	1372.8
25	February	1.42	36.23	360	1.80	0.697	2.582	415.7
27	February	19.06	59.40	230	1.20	0.167	7.186	870.3
28	February	6.21	7.42	1040	3.00	0.520	5.769	530.9

Table A4.1: Calculations of TSS_p for Typical Rainfall Events in February.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
3	March	16.15	64.57	400	9.00	1.107	8.130	955.3
10	March	3.52	148.95	130	9.00	1.214	7.414	1038.0
10	March	12.58	9.93	670	3.60	0.414	8.696	725.5
11	March	3.03	2.92	20	1.20	0.390	3.077	303.0
11	March	4.31	1.13	210	10.20	1.009	10.109	552.1
11	March	15.02	7.02	410	7.80	0.544	14.338	941.9
12	March	8.03	10.18	40	3.60	0.465	7.742	676.4
12	March	10.29	1.77	170	3.00	0.494	6.073	430.0
16	March	23.00	81.68	40	1.20	0.495	2.424	458.3
17	March	0.44	1.07	180	1.80	0.437	4.119	307.8
17	March	8.13	4.48	400	3.00	0.839	3.576	358.8
17	March	22.18	7.42	430	9.60	0.412	23.301	1297.4
19	March	14.56	33.47	210	8.40	1.223	6.868	766.9
20	March	9.23	14.95	780	5.40	0.260	20.769	1357.7
21	March	6.12	7.82	180	1.20	0.320	3.750	406.6
22	March	15.22	30.17	440	49.80	1.698	29.329	1907.9
29	March	18.32	163.83	310	2.40	0.170	14.118	1593.2
30	March	9.26	9.73	120	11.40	1.425	8.000	685.4
30	March	14.22	2.93	300	11.40	1.052	10.837	678.7
31	March	0.00	28.63	30	6.60	0.840	7.857	814.0
31	March	3.53	3.38	110	16.20	0.655	24.733	1179.2

Table A4.2: Calculations of TSS_p for Typical Rainfall Events in March.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
1	April	16.13	36.33	130	4.80	1.048	4.580	600.0
5	April	0.15	77.86	160	4.20	0.495	8.485	1013.5
5	April	18.43	15.80	310	4.80	0.515	9.320	820.7
8	April	22.03	70.16	190	1.80	0.483	3.727	588.1
10	April	20.05	42.86	260	0.60	0.115	5.217	670.8
13	April	19.44	67.32	380	2.40	0.161	14.907	1418.2
14	April	4.15	2.19	610	4.80	0.825	5.818	433.7
15	April	13.48	23.38	180	1.20	0.260	4.615	559.5
16	April	19.47	26.98	520	3.00	0.824	3.641	492.5
17	April	16.13	11.76	110	10.80	0.960	11.250	880.4
18	April	1.38	7.59	100	10.20	1.032	9.884	752.2
19	April	18.02	38.73	440	3.60	0.393	9.160	945.3
20	April	5.25	4.05	130	1.20	0.254	4.724	421.5
20	April	10.11	2.60	360	18.00	0.798	22.556	1063.4
21	April	7.49	15.63	30	1.80	0.620	2.903	388.3
23	April	19.31	59.20	100	20.40	1.470	13.878	1325.4
28	April	14.44	113.55	480	5.40	0.560	9.643	1172.9
30	April	12.54	38.17	670	10.80	1.293	8.353	888.9

Table A4.3: Calculations of TSS_p for Typical Rainfall Events in April.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
4	May	4.53	87.98	100	1.80	0.684	2.632	489.2
4	May	17.51	11.30	540	8.40	1.479	5.680	564.7
5	May	4.21	1.50	1170	4.80	0.257	18.677	858.1
6	May	7.26	7.58	150	1.20	0.224	5.357	508.2
7	May	13.16	27.33	170	14.40	0.664	21.687	1546.6
8	May	8.08	16.04	190	8.40	1.159	7.248	700.4
8	May	15.12	3.90	480	16.20	0.269	60.223	2135.8
10	May	8.24	33.20	40	12.00	4.155	2.888	439.9
10	May	11.32	2.46	10	4.20	2.280	1.842	212.0
15	May	0.00	108.30	400	4.80	0.243	19.753	1841.1
16	May	17.52	35.20	210	7.80	0.297	26.263	1825.0
17	May	14.35	17.22	40	1.20	0.555	2.162	326.9
17	May	17.48	2.55	180	4.80	0.523	9.178	596.1
18	May	6.36	9.80	160	4.20	1.054	3.985	439.3
18	May	10.11	0.91	700	8.40	0.362	23.204	906.2
18	May	20.51	0.17	60	4.20	0.790	5.316	264.3
19	May	19.25	21.40	250	1.80	0.062	29.032	1788.1
20	May	3.47	4.20	420	9.60	1.079	8.897	636.1
20	May	16.52	6.08	240	4.80	0.327	14.679	933.1
27	May	12.44	159.87	50	1.80	0.480	3.750	679.2

Table A4.4: Calculations of TSS_p for Typical Rainfall Events in May.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
1	June	2.46	110.03	300	1.80	0.474	3.797	642.6
3	June	0.32	40.77	150	2.40	0.448	5.357	676.5
3	June	15.55	12.88	310	1.20	0.217	5.530	567.5
5	June	0.00	26.91	610	36.00	1.435	25.087	1693.2
5	June	10.20	0.16	40	5.40	1.155	4.675	242.6
5	June	20.09	9.15	220	5.40	1.486	3.634	409.3
6	June	3.01	3.20	30	4.20	1.220	3.443	330.8
6	June	14.42	11.18	630	9.60	1.071	8.964	754.7
11	June	5.45	100.55	90	9.00	0.607	14.827	1513.1
12	June	7.19	24.07	380	19.20	1.494	12.851	1082.8
12	June	14.55	1.27	50	9.60	1.260	7.619	469.7
13	June	5.47	14.04	300	4.20	0.514	8.171	739.4
14	June	11.57	25.17	610	2.40	0.206	11.650	1024.7

Table A4.5: Calculations of TSS_p for Typical Rainfall Events in June.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
5	July	19.34	271.62	10	1.20	0.660	1.818	467.6
13	July	15.11	187.45	360	17.40	0.913	19.058	1975.2
13	July	23.24	2.22	320	1.80	0.360	5.000	394.6
14	July	8.03	3.32	880	16.20	0.874	18.535	977.3
15	July	3.15	4.53	160	11.40	2.389	4.772	432.4
16	July	20.14	38.31	220	2.40	0.635	3.780	535.4
18	July	15.39	39.75	370	11.40	1.479	7.708	850.2
19	July	10.03	12.23	120	9.00	0.605	14.876	1059.9
19	July	16.45	4.70	90	17.40	1.640	10.610	725.6
19	July	20.17	2.03	200	8.40	1.260	6.667	467.3
20	July	2.08	2.52	340	23.40	2.656	8.810	579.3
20	July	17.15	9.45	130	2.40	0.162	14.815	1011.7
22	July	18.11	46.76	290	9.60	0.230	41.739	2576.4
25	July	2.26	51.42	100	51.00	6.324	8.065	914.3
25	July	9.12	5.10	20	2.40	0.300	8.000	614.2

Table A4.6: Calculations of TSS_p for Typical Rainfall Events in July.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
1	August	0.00	158.80	300	7.20	1.620	4.444	756.4
4	August	7.35	74.58	70	2.40	0.686	3.499	570.7
4	August	12.43	3.96	210	9.60	0.594	16.162	922.7
4	August	16.27	1.23	110	2.40	0.431	5.568	382.4
5	August	4.05	8.80	40	3.60	1.245	2.892	351.3
12	August	3.36	166.85	540	9.00	0.657	13.699	1567.6
16	August	0.33	83.95	220	5.40	0.218	24.771	2037.9
16	August	13.16	9.05	130	1.80	0.263	6.844	612.7
16	August	16.39	1.21	320	3.00	0.174	17.241	786.4
17	August	7.20	9.35	40	1.80	0.525	3.429	395.8
17	August	11.42	3.70	450	4.80	0.528	9.091	631.2
18	August	1.50	6.63	130	9.60	0.448	21.429	1206.3
18	August	5.13	1.21	110	1.80	0.218	8.257	490.9
19	August	15.39	32.60	70	10.80	0.763	14.155	1212.8
19	August	18.13	1.40	300	61.80	0.542	114.022	2700.6
21	August	3.28	28.25	130	19.80	0.983	20.142	1483.5
21	August	10.37	4.98	450	4.80	0.777	6.178	518.4
23	August	10.39	40.53	120	3.00	0.420	7.143	812.4
23	August	13.50	1.18	70	13.20	1.397	9.449	532.6
24	August	10.37	19.61	120	2.40	0.455	5.275	591.4
24	August	13.46	1.15	130	12.00	0.849	14.134	686.2
25	August	2.37	10.68	190	11.40	0.249	45.783	2126.7
25	August	10.07	4.33	200	23.40	1.599	14.634	879.2
25	August	14.38	1.19	80	1.80	0.278	6.475	418.6
25	August	21.12	5.24	90	19.80	0.733	27.012	1344.0
26	August	3.59	5.28	130	3.00	0.305	9.836	705.1
26	August	8.41	2.53	110	26.40	1.015	26.010	1159.6
26	August	12.48	2.29	790	39.00	0.992	39.315	1484.4
27	August	8.40	6.70	70	18.00	1.431	12.579	859.4
28	August	19.09	33.31	50	16.80	1.644	10.219	988.2
29	August	18.10	22.19	360	15.00	0.978	15.337	1195.9

Table A4.7: Calculations of TSS_p for Typical Rainfall Events in August.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
1	September	5.04	58.90	250	6.00	0.854	7.026	856.6
2	September	7.58	22.73	70	3.60	0.411	8.759	839.0
3	September	13.21	29.05	910	19.80	1.060	18.679	1420.3
5	September	18.04	37.55	90	17.40	0.427	40.749	2444.2
6	September	12.29	16.92	610	4.80	0.356	13.483	1051.6
8	September	14.49	40.16	260	38.40	1.603	23.955	1759.7
8	September	21.02	1.89	320	20.40	0.294	69.388	2066.6
10	September	0.00	21.64	110	4.20	0.682	6.158	664.1
10	September	4.40	2.92	60	2.40	0.490	4.898	408.0
10	September	9.50	4.08	520	73.20	0.885	82.712	2636.5
23	September	11.23	304.88	40	1.80	0.315	5.714	992.5
23	September	20.15	8.20	490	34.80	1.978	17.594	1102.5
24	September	7.38	3.21	140	9.60	0.287	33.449	1418.3
24	September	11.55	1.95	280	23.40	1.513	15.466	795.0
25	September	11.11	18.60	320	28.80	2.739	10.515	911.5
29	September	3.24	82.89	100	1.20	0.282	4.255	658.6

Table A4.8: Calculations of TSS_p for Typical Rainfall Events in September.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
5	October	1.11	141.78	440	25.80	1.062	24.294	2200.2
6	October	16.59	32.47	320	15.00	2.002	7.493	806.6
7	October	9.07	10.80	680	45.00	0.708	63.559	2628.3
7	October	20.47	0.34	90	1.20	0.220	5.455	302.8
8	October	18.49	7.03	520	34.20	2.476	13.813	919.9
9	October	9.58	6.48	80	33.00	1.125	29.333	1469.2
9	October	12.54	1.60	80	1.80	0.443	4.063	326.7
9	October	16.37	2.39	450	13.20	0.679	19.440	952.7
10	October	3.49	5.30	70	6.60	0.420	15.714	952.2
10	October	9.27	4.46	60	1.20	0.460	2.609	293.0
11	October	13.49	27.37	740	42.00	0.503	83.499	3666.0
14	October	0.12	45.85	70	1.80	0.154	11.688	1137.0
14	October	10.54	9.53	50	28.20	1.464	19.262	1198.6
14	October	21.04	9.34	170	20.40	1.179	17.303	1115.1
15	October	1.38	1.74	20	1.80	0.840	2.143	220.1
15	October	3.12	1.24	210	25.80	1.157	22.299	930.2
15	October	8.40	1.97	70	1.80	0.163	11.043	642.1
15	October	10.55	1.08	410	9.60	0.401	23.940	951.8
15	October	20.07	2.37	150	10.20	0.416	24.519	1103.7
22	October	10.50	156.22	150	4.20	0.400	10.500	1307.6
22	October	20.40	7.33	130	1.80	0.337	5.341	504.4
23	October	6.40	7.83	140	9.60	0.669	14.350	960.1
23	October	10.17	1.29	100	11.40	0.972	11.728	620.7
23	October	14.11	2.23	90	10.20	0.793	12.863	723.2
24	October	12.38	20.95	140	4.20	0.733	5.730	630.7

Table A4.9: Calculations of TSS_p for Typical Rainfall Events in October.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
2	November	5.44	209.10	610	4.20	0.238	17.647	1915.6
2	November	19.59	4.08	130	11.40	1.551	7.350	560.1
3	November	0.22	2.21	420	4.20	0.093	45.161	1613.2
3	November	8.23	1.02	240	9.00	0.230	39.130	1290.2
6	November	12.35	72.20	420	2.40	0.344	6.977	882.8
8	November	1.28	29.88	400	1.80	0.192	9.375	918.0
9	November	3.21	19.21	150	3.00	0.684	4.386	523.7
10	November	3.15	21.40	160	2.40	0.229	10.480	931.5
10	November	18.06	12.18	630	42.60	1.432	29.749	1650.3
11	November	8.47	4.18	390	39.00	0.812	48.030	1869.6
11	November	16.14	0.95	400	18.60	0.713	26.087	983.3
12	November	19.22	20.46	870	4.80	0.288	16.667	1244.0
17	November	11.20	97.47	310	1.80	0.283	6.360	875.6
19	November	10.35	42.08	680	10.80	1.181	9.145	957.7
20	November	4.55	7.00	320	3.60	0.529	6.805	584.3
20	November	12.34	2.32	430	22.80	2.180	10.459	637.5
51	November	12.04	16.33	290	1.80	0.445	4.045	483.8
23	November	4.34	35.67	300	2.40	0.164	14.634	1258.0
23	November	11.50	2.27	270	2.40	0.287	8.362	550.5
26	November	17.58	73.63	230	5.40	0.548	9.854	1104.8
27	November	5.38	7.84	220	21.00	0.646	32.508	1620.5
29	November	10.27	49.15	760	3.00	0.612	4.902	659.7

Table A4.10: Calculations of TSS_p for Typical Rainfall Events in November.

Day	Month	Time	ADWP (Hours)	Duration (Mins)	Max. Inten. (mm/hr)	Ave. Inten. (mm/hr)	Peakedness (Max Inten/Ave Inten)	TSSp (mg/l)
1	December	0.07	37.67	800	18.60	0.910	20.440	1572.5
2	December	14.17	24.84	90	2.40	0.707	3.395	464.4
3	December	0.50	9.05	570	3.00	0.648	4.630	477.0
3	December	13.08	2.80	710	9.60	0.406	23.645	1109.7
4	December	11.46	10.80	70	1.80	0.463	3.888	439.6
4	December	20.10	7.23	130	9.60	1.385	6.931	594.6
6	December	2.15	27.91	120	4.80	0.820	5.854	671.3
7	December	17.05	36.83	980	10.20	0.787	12.961	1170.3
8	December	14.52	5.45	260	7.20	0.224	32.143	1512.3
9	December	14.09	19.55	560	28.20	1.571	17.950	1294.4
10	December	6.13	6.73	330	9.60	1.195	8.033	645.4
11	December	6.06	18.38	870	32.40	1.263	25.653	1609.8
12	December	12.29	15.88	300	18.60	1.296	14.352	1082.8
12	December	19.18	1.82	90	13.80	1.167	11.825	661.9
12	December	22.00	1.20	100	9.60	0.768	12.500	638.9
13	December	1.24	1.73	240	53.40	1.190	44.874	1541.2
28	December	23.13	377.82	710	6.00	0.480	12.500	1698.8
29	December	12.40	1.62	650	4.80	0.733	6.548	444.4

Table A4.11: Calculations of TSS_p for Typical Rainfall Events in December.

APPENDIX 5

OPC MODEL USER GUIDE

The optimal pollution control model determines control strategies using one of four procedures (fixed local control, variable local control, optimal pollution control using linear programming and optimal pollution control using dynamic programming), where one has to be selected by keyboard input. The OPC model determines control actions throughout the storm duration and prints results to output files for the desired control procedure. The model needs re-running with new output filenames if comparisons are to be made between control procedures.

The OPC model is written mainly in FORTRAN 90 code, although there are a few FORTRAN 77 subroutines and is text orientated.

Four input files are required and are listed below.

The first system input file includes data for the number of intercept points [-], pipe full capacity just downstream of each intercept point [cumecs], DWF from each intercept point [cumecs], fixed inflow setting for each intercept point [cumecs], solution time step [minutes], pipe full capacity of intercept point's continuation pipe [cumecs], chainage from upstream intercept point [m], and proportions of inflow permissible at each intercept point for dynamic programming solution [-]. A sample data file is shown below:

sys1.dat (sample)

```
7                                     ! No. of intercept points
3.25 3.25 3.25 7.72 7.72 7.72 7.72    ! Pipe full capacities
0.30 0.09 0.04 0.14 0.50 0.11 0.09    ! DWFs
1.24 0.25 0.97 0.69 2.13 0.29 0.31    ! Fixed inflow settings
1.0                                   ! Solution time step
```

```
3.20 1.70 0.97 2.00 5.48 1.66 1.52  ! Cont. pipe full cap.
0.0 895 1635 2100 2119 2829 3179    ! Chainage
1000 1000 1000 1000 1000 1000 1000  !  Inflow proportions for DP
```

The second system input file includes data for invert levels of interceptor sewer at intercept points [m], chainage of WwTW [m], interceptor pipe roughness height [mm], level of downstream invert at WwTW [m], interceptor pipe radii [m], chamber areas [m], chamber heights [m], penstock heights [m], and penstock widths [m].

sys2.dat compulsory filename (sample)

```
4.855  3.662  2.675  1.275 1.256 0.546  0.196    ! Invert levels

 3375 1.5 0.000 ! WwTW chainage, roughness height, WwTW invert level

  0.83   0.83   0.83   1.22 1.22 1.22    1.22    ! Radii
282.82 136.03  50.31 169.78 328.24 167.06 147.95 ! Chamber areas
  5.42   6.91   7.95   8.04   8.18   8.47   9.26 ! Chamber heights
  1.25   1.70   1.50  2.075   2.65   1.80   1.65 ! Penstock heights
  1.45  0.625  0.625 0.625   1.45  0.625  0.625 ! Penstock widths
```

The OPC model requires hydrographs and pollutographs, which have to be determined before the model can run. These are included in tabular form and sample files can be seen below:

inflow.dat (sample)

```
0.3   0.09   0.04   0.14   0.5   0.11    0.09
0.3   0.09   0.04   0.14   0.5   0.11    0.09
0.3   0.09   0.04   0.14   0.5   0.11 0.09002
0.3   0.09   0.04   0.14   0.5   0.11 0.09011
0.3   0.09   0.04   0.14   0.5   0.11  0.0903
0.3   0.09   0.04   0.14   0.5   0.11 0.09062
0.3   0.09   0.04   0.14   0.5   0.11 0.09107
0.3   0.09   0.04   0.14   0.5   0.11 0.09172
0.3   0.09   0.04   0.14   0.5   0.11 0.09261
0.3   0.09   0.04   0.14   0.5   0.11 0.09377
0.3   0.09   0.04   0.14   0.5   0.11 0.09522
0.3   0.09   0.04   0.14   0.5   0.11 0.09692
0.3   0.09   0.04   0.14   0.5   0.11 0.09888
0.3   0.09   0.04   0.14   0.5   0.11  0.1011
0.3   0.09   0.04   0.14   0.5   0.11  0.1036
0.3   0.09   0.04   0.14   0.5   0.11 0.10636
0.3   0.09   0.04   0.14   0.5   0.11 0.10935
0.3   0.09   0.04   0.14   0.5   0.11  0.1125
0.3   0.09   0.04   0.14   0.5   0.11  0.1158
0.3   0.09   0.04   0.14   0.5   0.11 0.11922
0.3   0.09   0.04   0.14   0.5   0.11 0.12275
0.3   0.09   0.04   0.14   0.5   0.11 0.12629
0.3   0.09   0.04   0.14   0.5   0.11 0.12984
```

0.3	0.09	0.04	0.14	0.5	0.11	0.13338
0.3	0.09	0.04	0.14	0.5	0.11	0.13683
0.3	0.09	0.04	0.14	0.5	0.11	0.1402
0.3	0.09	0.04	0.14	0.5	0.11	0.14349
0.3	0.09	0.04	0.14	0.5	0.11	0.14669
0.3	0.09	0.04	0.14	0.5	0.11	0.14974
0.3	0.09	0.04	0.14	0.5	0.11	0.15266
0.3	0.09	0.04	0.14	0.5	0.11112	0.15544
0.3	0.09	0.04	0.14	0.5	0.11175	0.15805
0.3	0.09	0.04	0.14	0.5	0.11322	0.16048
0.3	0.09	0.04	0.14	0.5	0.11412	0.16271
0.3	0.09	0.04	0.14	0.5	0.11581	0.16474
0.3	0.09	0.04	0.14	0.5	0.118	0.16655
0.3	0.09	0.04	0.14	0.5	0.12047	0.16816
0.3	0.09	0.04	0.14	0.5	0.12314	0.16956
0.3	0.09	0.04	0.14	0.5	0.12487	0.17075
0.3	0.09	0.04	0.14	0.5	0.1272	0.17174
0.3	0.09	0.04	0.14	0.5	0.12994	0.17255
0.3	0.09	0.04	0.14	0.5	0.13285	0.17317
0.3	0.09	0.04	0.14	0.5	0.1359	0.17364
0.3	0.09	0.04	0.14	0.5	0.13904	0.17395
0.3	0.09	0.04	0.14	0.5	0.14115	0.17417
0.3	0.09	0.04	0.14	0.5	0.1438	0.17442
0.3	0.09	0.04	0.14	0.5	0.14677	0.17483
0.3	0.09	0.04	0.14	0.5	0.14987	0.17552
0.3	0.09	0.04	0.14	0.5	0.15192	0.17662
0.3	0.09	0.04	0.14	0.5	0.1545	0.1782
0.3	0.09	0.04	0.14	0.5	0.15739	0.18028
0.3	0.09	0.04	0.14	0.5	0.15927	0.18291
0.3	0.09	0.04	0.14	0.5	0.16169	0.18607
0.3	0.09	0.04	0.14	0.5	0.16441	0.18978
0.3	0.09	0.04	0.14	0.5	0.16725	0.19408
0.3	0.09	0.04	0.14	0.5	0.16902	0.19891
0.3	0.09	0.04	0.14	0.5	0.1713	0.20425
0.3	0.09	0.04	0.14	0.50001	0.17387	0.21012
0.3	0.09	0.04	0.14	0.50002	0.17542	0.21658
0.3	0.09	0.04	0.14	0.50005	0.17749	0.22362
0.3	0.09	0.04	0.14	0.50009	0.17874	0.23121

pollut.dat (sample)

200	200	200	200	200	200	200
200	200	200	200	200	200	208.454
200	200	200	200	200	200	216.908
200	200	200	200	200	200	225.362
200	200	200	200	200	200	233.816
200	200	200	200	200	200	242.27
200	200	200	200	200	200	250.724
200	200	200	200	200	200	259.178
200	200	200	200	200	200	267.632
200	200	200	200	200	200	276.086
200	200	200	200	200	200	284.539
200	200	200	200	200	200	292.993
200	200	200	200	200	200	301.447
200	200	200	200	200	200	309.901
200	200	200	200	200	200	318.355
200	200	200	200	200	200	326.809
200	200	200	200	200	200	335.263
200	200	200	200	200	200	343.717
200	200	200	200	200	200	352.171
200	200	200	200	200	200	360.625
200	200	200	200	200	200	369.079
200	200	200	200	200	200	377.533

200	200	200	200	200	200	385.987
200	200	200	200	200	200	394.441
200	200	200	200	200	200	402.895
200	200	200	200	200	200	411.349
200	200	200	200	200	200	419.803
200	200	200	200	200	200	428.257
200	200	200	200	200	200	436.71
200	200	200	200	200	208.454	445.164
200	200	200	200	200	216.908	453.618
200	200	200	200	200	225.362	462.072
200	200	200	200	200	233.816	470.526
200	200	200	200	200	242.27	478.98
200	200	200	200	200	250.724	487.434
200	200	200	200	200	259.178	495.888
200	200	200	200	200	267.632	504.342
200	200	200	200	200	276.086	512.796
200	200	200	200	200	284.539	521.25
200	200	200	200	200	292.993	529.704
200	200	200	200	200	301.447	538.158
200	200	200	200	200	309.901	546.612
200	200	200	200	200	318.355	555.066
200	200	200	200	200	326.809	563.52
200	200	200	200	200	335.263	571.974
200	200	200	200	200	343.717	580.428
200	200	200	200	200	352.171	588.881
200	200	200	200	200	360.625	597.335
200	200	200	200	200	369.079	605.789
200	200	200	200	200	377.533	614.243
200	200	200	200	200	385.987	622.697
200	200	200	200	200	394.441	631.151
200	200	200	200	200	402.895	639.605
200	200	200	200	200	411.349	648.059
200	200	200	200	200	419.803	656.513
200	200	200	200	208.454	428.257	664.967
200	200	200	200	216.908	436.71	673.421
200	200	200	200	225.362	445.164	681.875
200	200	200	200	233.816	453.618	690.329
200	200	200	200	242.27	462.072	698.783
200	200	200	200	250.724	470.526	707.237

An additional input file has been created to replace keyboard inputs and includes input and output filenames, and control procedure.

run.dat

```
flowmr21.prn ! Hydrograph input file
pollmr21.prn ! Pollutograph input file
globsys.dat  ! System input file
lp           ! Control Procedure - lc = local control, lp = linear
             programming, DP = dynamic programming
vl           ! Local control procedure - fl = fixed local control,
             vl = variable local control
r21chgc.out  ! Output file for chamber levels
r21spgc.out  ! Output file for spill loads
r21pgc.out   ! Output file for penstock levels
r21qgc.out   ! Output file for flow rate control strategies
```

The model is currently coded not to implement the post-processing hydraulic verification but the code is listed at the end of the program should it be required in future developments.

The model listing is not included in this thesis but can be obtained from the Department of Civil Engineering, if required.

Appendix 6

Supporting Papers

Published in the Proceedings of the 4th International Conference on Developments in Urban Storm Drainage Modelling, London, 1998, pp 691 – 698

OPTIMAL CONTROL MODELS FOR INTERCEPTOR SEWER SYSTEMS

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ABSTRACT

It is well recognised that the discharge of raw sewage from urban areas causes significant impacts on receiving waters. Interceptor sewer systems were originally conceived to reduce these impacts by diverting combined sewer flow to treatment at a Wastewater Treatment Works (WwTW). However, remaining combined sewer overflow (CSO) structures still cause intermittent impacts on receiving waters by their spills during storm events. These impacts can be minimised by the use of an optimal control model using global information about the system.

This paper describes the development and testing of optimal control models for a generalised interceptor sewer system. The formulations of two optimisation models are presented: a Linear Programming (LP) model; and a Dynamic Programming (DP) model.

The models are tested using a simple hypothetical interceptor sewer system and some initial results are presented. These results show that there is significant scope for environmental and operational improvements with the use of these optimal control models.

KEYWORDS

Interceptor sewer systems; optimisation; mathematical modelling.

INTRODUCTION

Historically, urban drainage systems have been designed to provide hydraulically and economically effective transport of surface runoff and foul sewage from urban areas into receiving waters. It is well recognised that this discharge from urban areas causes significant impacts on receiving waters. The interceptor sewer was originally conceived as a solution to these water pollution problems by intercepting the original outfalls and diverting the flow to a wastewater treatment works (WwTW). Interceptor sewer systems, generally, consist of long pipe runs, storage tanks and pumping stations and often have long travel times. They have limited flow capacities that are periodically exceeded by sewer flows generated during heavy storms. The sewerage systems are often designed to spill from overflow structures, at pre-set flow settings to mitigate surcharge of the interceptor. Currently in the UK, most interceptor sewers are operated locally

with static operating rules. It is proposed that significant environmental and operational improvements can be gained by the use of an optimal control model using global information about the system.
Significant research has been carried out in the control of urban drainage systems in general, notably Schilling (1989, 1996). Other researchers include Nielson (1993), Fuchs (1995) and Nelen (1993). However, to the authors' knowledge there is little research in the unique case of a sequential interceptor system. Gómez (1991) undertook some research into the automatic control of sewer interceptors but this was not based around optimisation techniques.

The models described in this paper are based on a simple sequential interceptor system and use global information to determine control variables (the throughflow rates from CSO's into the interceptor). Furthermore, pollutant concentration factors are also introduced into the models in order to compute optimum flow rates using optimisation techniques on grounds of pollution load as well as the simple minimisation of spill volume. Both a Linear Programming (LP) model and a Dynamic Programming (DP) model are presented in this paper. Various assumptions have had to be made in the formulation of the models. These include the requirements that: a) all inflows and their respective pollutant concentrations are known; b) hydrographs are piecewise constant; c) unintercepted flows are spilled to the river; d) all pipe flows are in the downstream direction; e) there is complete control over the proportion of flow diverted into the interceptor from the CSOs.

LINEAR PROGRAMMING (LP) MODEL

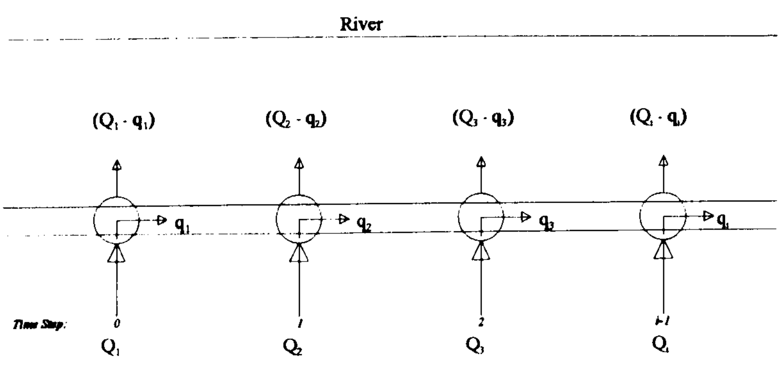


Figure 1: Theoretical Basis of the Model

The Linear Programming (LP) model described in this section determines optimum flow rates for an interceptor sewer system based on estimated pollutant factors. Figure 1 shows the fundamentals of the LP model.

Figure 1 represents a decision to be made at each intercept point where q_i is the control variable. Q_i are hydrographs so Q_i and q_i are functions of time. Therefore, a second subscript is included in the notation to allow for this - j corresponding to a particular time step.

Initial Base Assumption. In this most idealised case of equi-spaced interception points along the interceptor pipe the time step is chosen equal to the time of flow in the interceptor sewer between any two intercept points.

As a consequence of this assumption, the model can be described as a chain of water travelling down the interceptor system as shown in Figure 2. The 'slugs' of water are therefore treated as being separate in time and space.

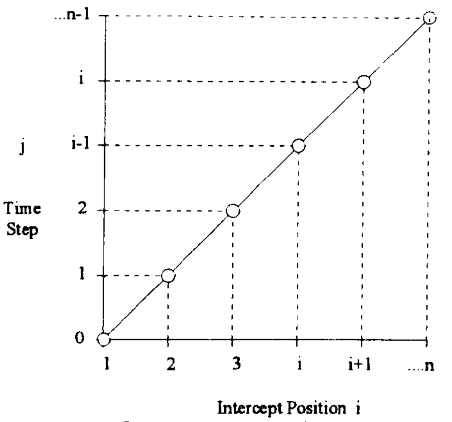


Figure 2: Chain of Water Commencing at Time Step 0

It is important to note that the volumetric optimisation leads to an obvious control strategy - fill the interceptor sewer up as soon as possible and keep it at capacity. This strategy would minimise total overflow volume discharged to the receiving water. However, if the system is to be optimised against water quality consents then there is significant scope for operational improvements. To facilitate this a pollutant concentration factor is introduced into the model to translate volumes into measures of total pollutant load. Thus, $\alpha_{i,j}$ represents the pollutant concentration factor of the flow at intercept point i in time step j. Typically, $\alpha = 0$ represents absolutely clean water and, conversely, $\alpha = 1$ represents absolutely polluted water. Generally, $0 \le \alpha_{i,j} \le 1$ and all $\alpha_{i,j}$ are assumed to be known. A standard LP optimisation model can now be set up.

Objective Function: Minimise pollutant load to the receiving water over variables $q_{i,j}$.

Thus,

$$Min\sum_{i=1}^{n} \alpha_{i,i-1}(Q_{i,i-1} - q_{i,i-1})$$

where n is the number of intercept points.

or,

$$Max\sum_{i=1}^{n} \alpha_{i,i-1}q_{i,i-1}$$

subject to capacity constraints.

Constraints:

$$q_{i,i-1} \le Q_{i,i-1} \text{ for all i}$$

$$\sum_{j=1}^{i} q_{j,j-1} \le C_i \quad \text{for all i}$$

where C_i = Interceptor capacity at point i.

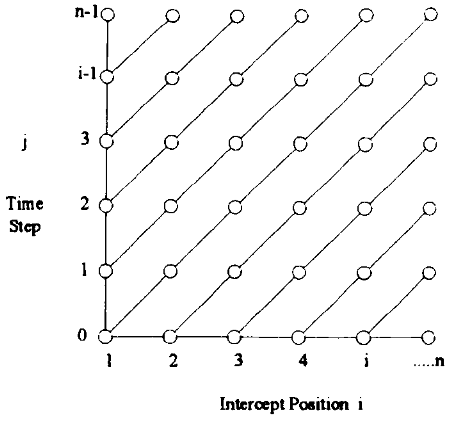


Figure 3: Complete Model with Chains from All Time Steps

This is the LP model for the chain of water shown in Figure 2. This LP problem can be solved by any standard LP solver, in this case a computer program written in FORTRAN code using the Simplex Method. However this is just one chain of water running through the interceptor and there are other chains at different time steps. The complete formulation to the LP model is shown in Figure 3. This can be solved by the same method as described above but each chain is solved in sequence.

Relaxation of Initial Base Assumption. A new version of the LP model relaxes the above assumption. The

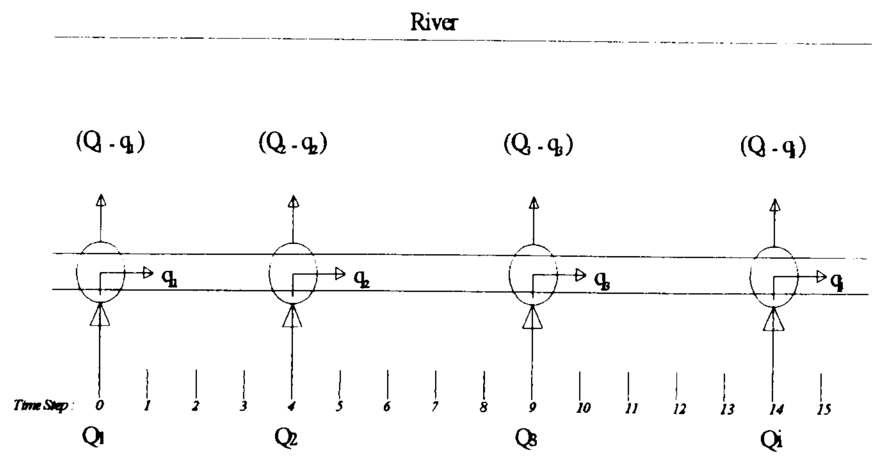
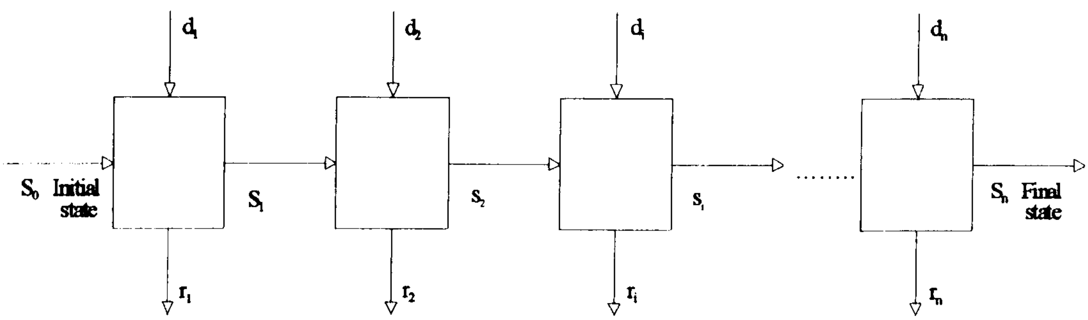


Figure 4: New Version of Model

interceptor pipe is divided into time steps and there are intercept points on only some of these steps. This is shown graphically in Figure 4 and is better able to represent a realistic system. The model can now control interceptor systems where the intercept points are at irregular intervals and times of flow between intercept points are not constant. The only difference between this version of the model and the original version of the model is in the selection of the correct inflow rates and pollutant factors at the appropriate time steps. Therefore, the



where S_i - State Variable at Intercept Point i (i.e. Intercept flow);
d_i - Decision Variable at Intercept Point i (i.e. How much flow to intercept?);
r_i - Cost of Decision at Intercept Point i (i.e Pollutant load to river).

Figure 5: Fundamentals of the DP Solution Method

model formulation is unchanged. It is clear, however, that the data files for the model will become extremely large.

DYNAMIC PROGRAMMING (DP) MODEL

Dynamic Programming is a very useful optimisation technique used in sequential decision making. It is an extremely fast and efficient method of solving multi-stage sequential optimisation problems. DP models are relatively difficult to formulate and, unlike Linear Programming, there is no standard solver. Each DP problem is formulated from first principles and is solved using a computer program unique to that problem. It is set up in a similar fashion to the LP model, in that the model determines the optimum intercepted flow rates for the chains in Figure 3. However, the difference between the models lies in the solution method. LP solves for the optimum directly whereas the DP model determines all the possible permutations. Figure 5 shows the fundamentals of the DP model which determines how much flow

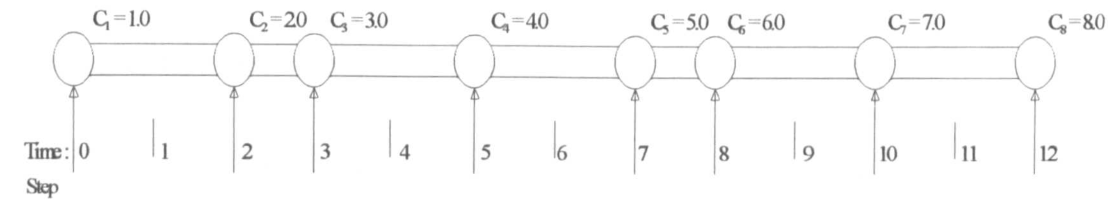


Figure 6: Simple Interceptor System

to add to the interceptor flow at each intercept point. The cost of a particular decision would be the resulting spillage to the river, in terms of pollutant load.

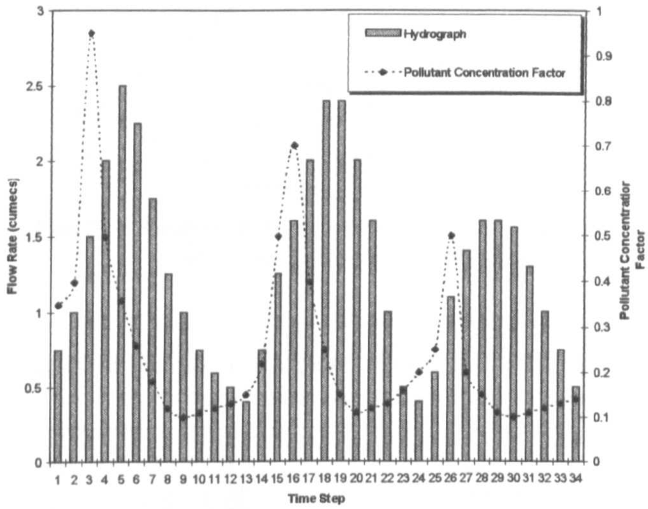


Figure 7: Inflow Hydrograph and Pollutant Concentration Factors

RESULTS

A simple hypothetical interceptor sewer system has been used to test the models. This system is presented in Figure 6. This has eight intercept points each with intercepted dry-weather flows (DWF) of 0.1 cumecs. The interceptor pipe capacity C_i increases at each intercept point with a final capacity of eight cumecs. This is also the WwTW (at intercept point 8) treatment capacity. The DWF's are deducted from the absolute interceptor capacities to determine the effective interceptor capacities. Each catchment is identical in layout and hydraulic design. The fixed pre-set maximum intercepted flow rate, for the fixed local control strategy calculations, is one cumec at each interception point. One thousand possible proportions (settings) of the control gate have been considered for the DP method. A rainstorm hits all the catchments at the same time and the resulting inflow hydrograph and pollutant concentration factors are shown in Figure 7. The model was run using the simple interceptor system data and inflow hydrograph with pollutant concentration factors.

Control Strategies

Four different control strategies were considered:

Local Control (Fixed or pre-set). This is the standard method of control in existing systems. Intercepted flows are determined from local conditions at the interception point. Flows to the interceptor are governed by the use of flow restrictors (e.g. vortex devices) which restrict the inflow to a pre-set maximum. In this system the maximum flow rate was set at one cumec at every interception point. No account is made of the conditions in the interceptor system or conditions at other interception points. The method is volumetric based and no account is made of the pollutant load of the flows.
Local Control (Variable). This method determines intercepted flows using information about the conditions locally in the interceptor system. That is, if there is spare

capacity locally in the interceptor, intercept flows up to this capacity can be permitted. The method is volumetric based and no account is made of the quality of the flows.

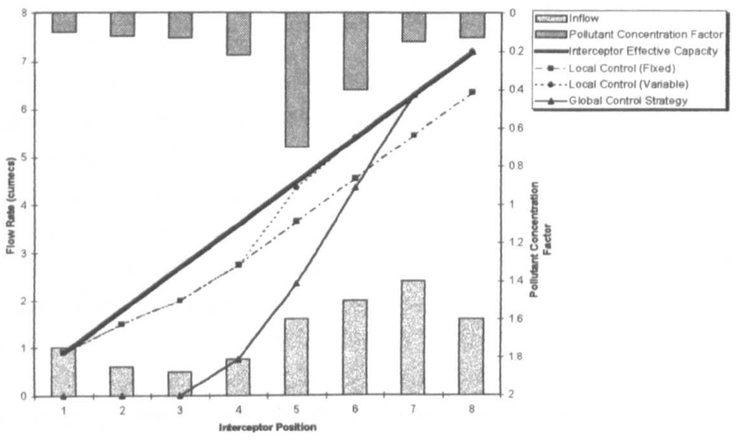


Figure 8: Graphical Interpretation of a Solution to a Chain of Water in Time Step 21 (LP Model)

Global Control (LP). This method uses global information, including pollutant concentrations, to determine optimum strategies using the LP model.

Global Control (DP). This method uses global information, including pollutant concentrations, to determine optimum strategies using the DP model.

LP Model Results

Figure 8 shows a solution of one chain of water in time step 21 by the LP method. It shows the inflows, their pollutant concentrations and the control strategies. The local control strategies were obtained on volumetric criteria (i.e. pre-set throughflow for fixed local control and enhanced input up to local interceptor reach capacity in the case of variable local control) and no account was taken of the pollutant concentrations.

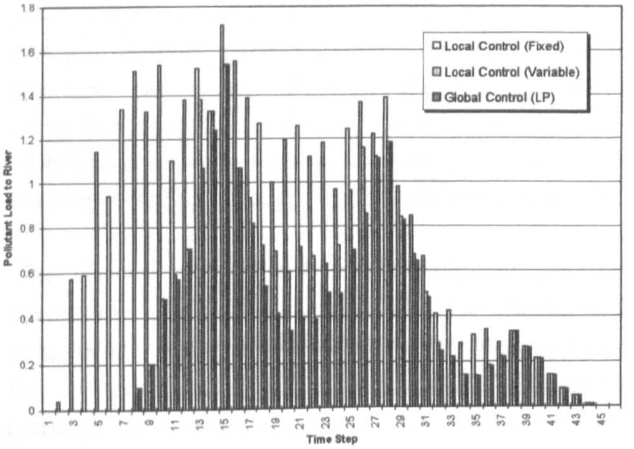


Figure 9: Comparison of Local and Global Strategies within the Control Horizon (LP Model)

DP Model Results

The DP results are almost identical to the LP results, as would be expected because only the solution method differs between the two models. However, it is apparent that in some time steps the global control strategy spills more pollutant load than the other solution methods. This is explained by small rounding errors inherent in the DP solution where the resolution is to 0.008 cumecs. This means that in certain circumstances the global control strategy will appear to spill 0.01 cumecs more than the other control strategies.

DISCUSSION

Figures 8 shows a graphical representation of the solution to the chain of water in time step 21. This illustrates a situation possible with the fixed local control strategy. Inflows at certain interception points were less than the pre-set maximum flow rate. Therefore, there was spare capacity within the interceptor system. The strategy could not make allowances for this and, consequently, there were unnecessary overflows at the downstream interception points. The variable local strategy improved on this strategy because it uses information about the interceptor system state. Therefore, this strategy would always fully utilise the capacity available locally within the interceptor system. However, there is a danger of a bias, as Figure 8 illustrates, within the strategy of downstream interceptor points throttling back (i.e. closing of control gates). This would cause the downstream intercept points to spill more frequently than the upstream points. This is because the interceptor may already be at capacity because of decisions made upstream. Inspection of these graphs illustrates the potential of the optimal control models. There are two particularly polluted inflows at intercept positions 5 and 6. The global control strategy did not intercept the flows at points 1 and 2 (relatively clean flow) in order to intercept these 'dirty' flows. The local control strategies did not make such allowances and as a consequence overflowed at intercept point 6. It was by pure chance that there was sufficient spare capacity within the system that the variable local control strategy did not spill at intercept point 5. In this time step the global control strategies had a 69% improvement over the fixed local control strategy, and a 44% improvement over the variable local control strategy, measured in terms of pollutant load reduction to the river.

Figure 9 shows comparisons of the solutions in each time step. They show that both the variable local control strategy and the global control strategies are significant improvements over the fixed local control strategy. The global control strategies have most improvement between time steps thirteen and twenty-six. At worst the global control strategy spills the same amount of pollutant load as the variable local control strategy. The results illustrate the environmental improvements possible with the use of global control models. It must be stressed that the results presented here are for a storm that commences at the same time all over the catchments. The greatest potential for improvement for global control would likely be with storms that vary spatially.

The choice of method that should be developed further for real-world interceptor systems is presently unclear. The LP model runs considerably faster and yields exact solutions. However, the DP model may become more efficient as the complexity of the systems increase. In LP the computational time increases considerably as the number of variables increase. The computational time for DP solutions increases at a slower rate because more variables (intercept points) merely add stages to the method. At some point the relative computational demands for each model may reverse.

It is clear that the models are heavily reliant on the ability to accurately predict the inflows and their respective pollutant concentration factors. However, the sensitivity of the models to these inputs remains a topic of further research. Moreover, the robustness of the models has to be enhanced to account for possible uncertainties within these inputs. In further developments inflows will be predicted using a suitable long-term sewer simulation model (Mehmood 1995) and the pollutant concentration factors will be obtained using a suitable methodology (Gupta 1995).

CONCLUSIONS

The results presented in this paper clearly illustrate the potential of optimal control models in interceptor sewer operation. Local control strategies that use information about the interceptor system state can significantly improve the performance of systems (in terms of pollutant load spills reduction) that use pre-set local control strategies. However, the results show that a further improvement can be achieved with the use of optimal control models using global information. Also, the global control models make more efficient use of the available storage within the interceptor system. There is an inherent danger of downstream interception points spilling, and possibly flooding, more frequently with variable local control strategies. This may prove unsatisfactory in many cases.

A significant amount of further research needs to be carried out, the models (in their present state) being highly idealised and simplistic. They were constructed to test the viability of the methods. In the next phase of the study they will be expanded to incorporate real-world considerations including the incorporation overflow chambers and system arrangements. Efficient and practicable methodologies for the prediction of the inflows and the pollutant concentrations will also be sourced for the later developments.

ACKNOWLEDGEMENTS

This paper presents the findings from an ongoing PhD research programme at the Department of Civil Engineering, University of Liverpool. The authors would like to thank North West Water Limited for their assistance in the supply of data for the Liverpool Interceptor Sewer System upon which later phases of model development will be based.

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OPTIMAL POLLUTION CONTROL OF LARGE INTERCEPTOR SEWER SYSTEMS

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ABSTRACT

This paper describes the hydraulic verification of optimal control models that have been developed for large interceptor sewer systems. The models - both Linear Programming and Dynamic Programming alternatives - each based initially on a slug flow approach, were tested using idealised interceptor sewers. The models have now been extended to include a simple hydraulic verification routine in which a quasi-steady approach is used to estimate interceptor sewer water profiles at each time step in the solution.

The results illustrate that the slug flow approach is a sound formulation for the optimal control model and that the control strategies give physically feasible solutions. The water profiles from the hydraulic verification routine show that the control strategies from the control models are marginally conservative, WALLRUS verification showing the pipes to become full and surcharged in some limited instances. In general, however, WALLRUS has validated the interceptor sewer hydrographs and 'snapshot' water profiles determined by the hydraulic verification routine. Furthermore, the results have also illustrated the suitability of using pipe full velocity to advect the slug flows within the optimisation routine.

KEYWORDS

Interceptor sewer systems; optimisation; mathematical modelling; hydraulic verification.

INTRODUCTION

Interceptor sewer systems were conceived to reduce environmental impacts on receiving waters by diverting combined sewer flow to a Wastewater Treatment Works (WwTW). However, spills from combined sewer overflows (CSO's) during storm events cause intermittent impacts that can be significant. These impacts can be minimised by the use of optimal control strategies that draw from global information about the state of the system, including incoming pollutant concentration levels. It should be noted that whilst volumetric global control of interceptor sewer systems leads to an obvious result, fill the sewer as quickly as possible and keep it full, this will not generally be the best strategy for minimising of total pollutant overspill.

Interceptor sewer systems, generally, consist of pipes, storage tanks and pumping stations. Such a system has been installed in the Liverpool area as part of the Mersey

Estuary Pollution Alleviation Scheme (MEPAS). This scheme has been implemented in phases over the last two decades leading to significant improvement in the River Mersey's status, which has seen a return of fish species to the estuary. The Liverpool Interceptor Sewer runs parallel and close to the banks of the river where it intercepts the original combined sewer outfalls. The system is very complex being twenty-nine kilometres long with twenty-six intercept points where storm overflow structures form the junction between existing outfall sewers and the lower level interceptor sewer. The flow times within the sewer are therefore very large necessitating the forecasting of imminent rains for optimal control actions. The system, with electro-mechanical penstocks at each overflow structure, potentially has millions of possible control states governing the entry of sewage into the interceptor during storm conditions. For example, if there were only two operational possibilities at each intercept point (a gate being fully open or closed) then there would be over sixty-seven million possible control strategies at every time step in the solution.

This paper describes the hydraulic verification of optimal pollution control models that have been developed specifically for large interceptor sewer systems. To the authors' knowledge there has been little research on optimal pollution control modelling of these systems. Weinreich et al (1997) researched pollution based real time control of combined sewer systems and applied it to an interceptor sewer system. However, their linear programming formulation differs from the optimal pollution control model formulation discussed here. Reference should be made to Nelen (1993) and Schilling (1989, 1996) for initiating study of optimal control in urban drainage systems.

The present models - both Linear Programming and Dynamic Programming alternatives - each based on a slug flow approach, were originally tested using idealised interceptor sewers. Early results (Thomas et al, 1998) showed that considerable environmental improvements, in terms of pollutant load reduction to the receiving waters, could be achieved when using the optimal control models compared with traditional local (fixed) control strategies. The models have now been extended to include a hydraulic verification routine in which a quasi-steady approach is used to develop approximate interceptor sewer water profiles, for each time step in the solution.

OPTIMAL CONTROL MODEL FORMULATION

The control model determines optimum interceptor inflow rates based on incoming pollutant concentrations to maximise pollutant load within the interceptor sewer. The model is formulated using a slug flow approach where the 'slugs' are tracked through the interceptor and the control model determines the amount of sewage that should be added from the individual catchment inflows based on the appropriate time delays. This optimisation problem can be solved using two procedures, Linear Programming and Dynamic Programming. A detailed description of the model formulation is presented in Thomas et al (1998) but is briefly summarised below:-

The Linear Programming objective function in one time step is:

$$Max \sum_{i=1}^{n} \alpha_{i,t_i} q_{i,t_i}$$

where n - number of intercept points, t_i - time step position within the interceptor of intercept point i, $\alpha_{i,ti}$ - pollutant concentration factor at intercept point i in time step t_i and $q_{i,ti}$ - interceptor sewer flow rate after intercept point i in time step t_i. For computational convenience here the pollutant concentration factor is defined as a coefficient assigned to each time step inflow at each time step. For general illustration this coefficient can be considered to range from 0 to 1, i.e. absolutely 'clean' through to absolutely 'dirty' inflows.

The objective function is solved using the Simplex Method for Linear Programming or using Dynamic Programming, within the appropriate capacity constraints. Flows in excess of the interceptor inflows are assumed to be spilled but the solution by maximising the pollutant load within the interceptor ensures minimisation of spillage load to the receiving waters. The objective function represents decisions to be made in one time step and the control strategies are determined throughout the control time horizon by altering the time step position. Since, in the approach, successive 'slugs' of water are assumed not to interact, then the sequence of optimal controls derived each time step also represent the optimal control strategy for the entire event.

The control model determines the intercept point time step positions within the interceptor sewer using the pipe full velocities. Generally, this is hydraulically acceptable but time is the critical parameter within the slug flow approach in the model. It is essential to be able to track the 'slugs' accurately through the interceptor sewer. Since the actual velocity of travel will deviate from the assumed pipe full velocity the approach will have a deficiency (and 'slugs' of fluid will of course interact to some degree). Therefore, a post-processing hydraulic verification routine was introduced into the model to verify that the control strategies from the optimisation algorithms provide a physically feasible solution.

POST-PROCESSING HYDRAULIC VERIFICATION

The post-processing hydraulic verification routine determines the water profiles within the interceptor sewer in each time step solution. The procedure is shown in Figure 1 and may be interpreted as 'snapshot' water profiles from each time step throughout the control time horizon.

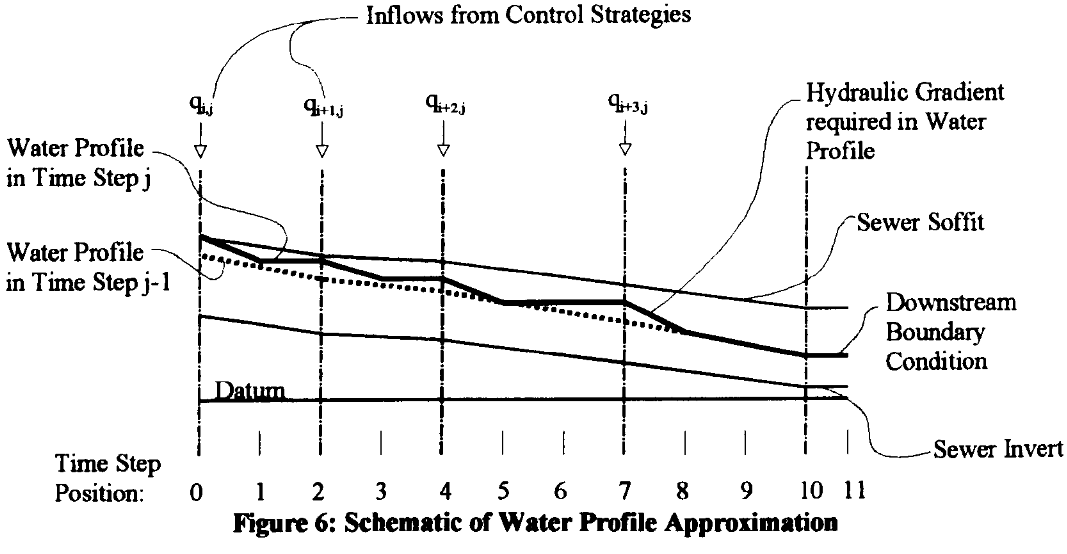

Figure 6: Schematic of Water Profile Approximation

The water profiles are determined by a quasi-steady approach using the Manning equation. It is assumed that inflow q_{ij} will have reached one time step position downstream in the interceptor at the end of the time step. Therefore, the position (based on pipe full velocity) and size (from the optimisation module) of the 'slugs' are known. The Manning equation is then used to determine the hydraulic gradient required to transport the flow through the reach. Any hydraulic inconsistencies and positions of surcharging are then illustrated.

The procedure commences at the downstream boundary point and determines the water level at the next time step position upstream so that there is sufficient hydraulic gradient for that flow rate. The routine then determines the water level for the next time step position and so on. The critical depth is calculated at positions where pipe invert is discontinuous such as where the diameter of the sewer alters. If the water surface level from the Manning equation is lower than this depth, then the critical depth is used in the subsequent calculation of the upstream water profile. Transitions in water profile can therefore be determined. The verification routine continues this procedure for all the control strategies throughout the control time horizon.

TEST CASE – SIMPLIFIED NORTHERN LEG OF LIVERPOOL INTERCEPTOR SYSTEM

The northern leg of the Liverpool Interceptor Sewer has been simplified and used as a test case for the hydraulic verification routine. A longitudinal section of the sewer can be seen in Figure 2. The input data for the interceptor sewer is shown in Table 1. The model was run with several runoff hydrographs and respective pollutant concentrations, an example of which is shown in Figure 3. The dry weather flows from each catchment were added to the runoff hydrographs to obtain the total combined sewer flow. The runoff hydrographs consisted of three hypothetical storm events, of varying severity and loosely based on the catchment's response characteristics: a low storm event (~1.5-2 times fixed inflow setting, i.e. to 'Formula A' criteria), a medium storm event (~3-4 times fixed inflow setting), and a high storm event (~7-10 times fixed inflow setting).

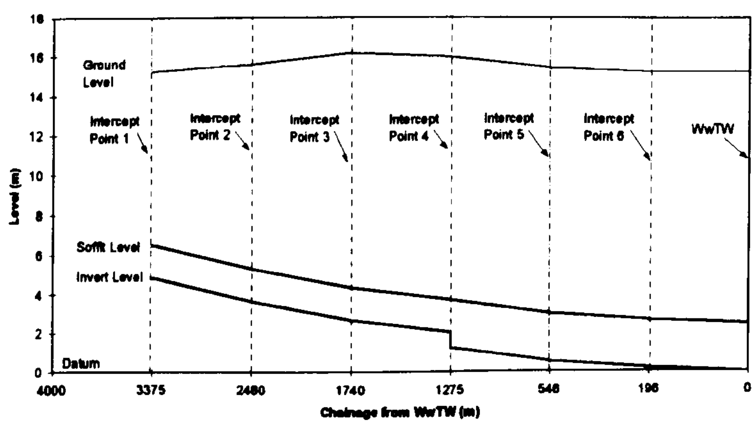

Figure 2: Longitudinal Section of Test Sewer

Three control procedures were applied to the test case:- fixed local control (where inflows up to the fixed inflow setting can be permitted and no account is taken of the interceptor sewer state or pollutant concentrations); variable local control taken inflows up to the local interceptor sewer capacity can be permitted (but no account is made of the pollutant concentrations); and optimal pollution control where the optimal control model uses global information on the interceptor sewer state (including pollutant concentrations). All the control procedures use the slug flow approach within the interceptor sewer and only the decision criteria differ. Volumetric optimisation has not been included explicitly because the variable local control objective also fully utilises the available storage within the sewer (though spills will occur at different locations).

Intercept Point	Sewer Diameter (m)	Sewer Gradient	Sewer Capacity (cumecs)	D.W.F. (cumecs)	Fixed Inflow Setting (cumecs)
1	1.66	1/750	3.26	0.30	1.24
2	1.66	1/750	3.26	0.09	0.25
3	1.66	1/750	3.26	0.04	0.97
4	2.44	1/1000	7.72	0.64	2.82
5	2.44	1/1000	7.72	0.02	0.29
6	2.44	1/1000	7.72	0.09	0.31

Table 1: Input Data for Test Interceptor Sewer

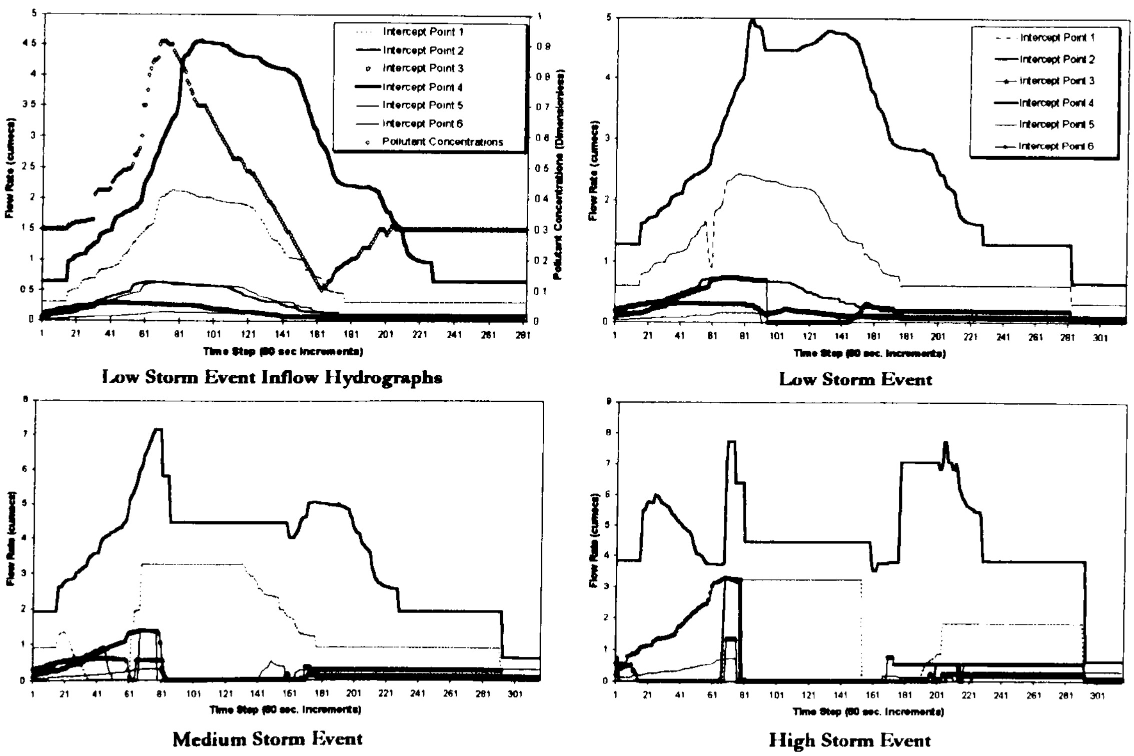

Figure 3: Example of Inflow Hydrographs for Low Storm Event and Pollutant Concentrations (Top Left) **and Control Strategies from Optimal Pollution Control Model for Three Storm Events** (Note - all control strategies use the same legend)

The control strategies from the optimal pollution control model, illustrated in Figure 3, were determined using the Linear Programming routine. A comparison between the control procedures can be seen in Table 2 and the results show that the variable local control procedure significantly reduced the pollutant load discharged to the receiving waters compared to the fixed local control but the optimal pollution control model offered further enhancement. The results show that the environmental improvements decrease, compared to the fixed local control, as the severity of the storm increases. This is expected because spills are inevitable with larger inflows where the interceptor sewer storage is largely utilised. It must be stressed, therefore, that these results are likely to be a poor illustration of the potential of the optimal pollution control because the inflow hydrographs and pollutant concentrations were highly synchronised. It is likely that most improvements would be encountered when the hydrographs and corresponding pollutant concentrations varied spatially and temporally and that overall performance will be heavily weighted to the more moderate rainfalls which occur more frequently.

Storm Condition	Pollutant Load to Receiving Waters (Spill Volume × Pollutant Concentration)			Improvement (OPC v FLC) %	Improvement (OPC v VLC) %
	Fixed Local Control (FLC)	Variable Local Control (VLC)	Optimal Pollution Control (OPC)		
Low Storm (~1.5-2 × Fixed Inflow Setting)	192.44	34.47	34.34	82.16	0.38
Medium Storm (~3-4 × Fixed Inflow Setting)	639.59	433.46	399.35	37.56	7.87
High Storm (~7-10 × Fixed Inflow Setting)	2146.86	1897.32	1823.37	15.07	3.40

Table 2 : Comparison between Control Procedures

The control strategies from the optimal pollution control model were verified in the post-processing hydraulic verification routine and these, in turn, were validated using WALLRUS. Figure 4 shows sample results from both the hydraulic verification routine and WALLRUS (including backwater effects).

The sewer hydrographs in Figure 4 from the hydraulic verification routine and WALLRUS correlate well illustrating the validity of the slug flow approach in the optimal control model. This application of WALLRUS has shown some instability in the results particularly where rapid changes in inflow are imposed when the pipes are running close to full. This is clearly seen for the medium and high storms around time step 90. This instability probably occurs because of the numerical methods used in the WALLRUS code. However, the significance of the instability is considered negligible because the flow volumes within the instability are minimal, the solution recovers and the overall results compare well.

The water profiles in Figure 4, taken at various time steps and determined by the post-processing hydraulic routine, show that the control strategies from the optimisation module were conservative and imply that there was additional volume available. These profiles illustrate the optimisation module's idealisation of the interceptor sewer system state, i.e. all 'slugs' of flow travel through the interceptor sewer at the pipe full velocity irrespective of the water depth and surface slope. The optimisation module permits all flows from zero to pipe full capacity at any point within the interceptor sewer. Therefore, the control strategies should not contain surcharging, as this would invalidate the assumptions made in the optimisation formulation. The water profiles confirm these assumptions because the profiles do not even reach the interceptor sewer soffit.

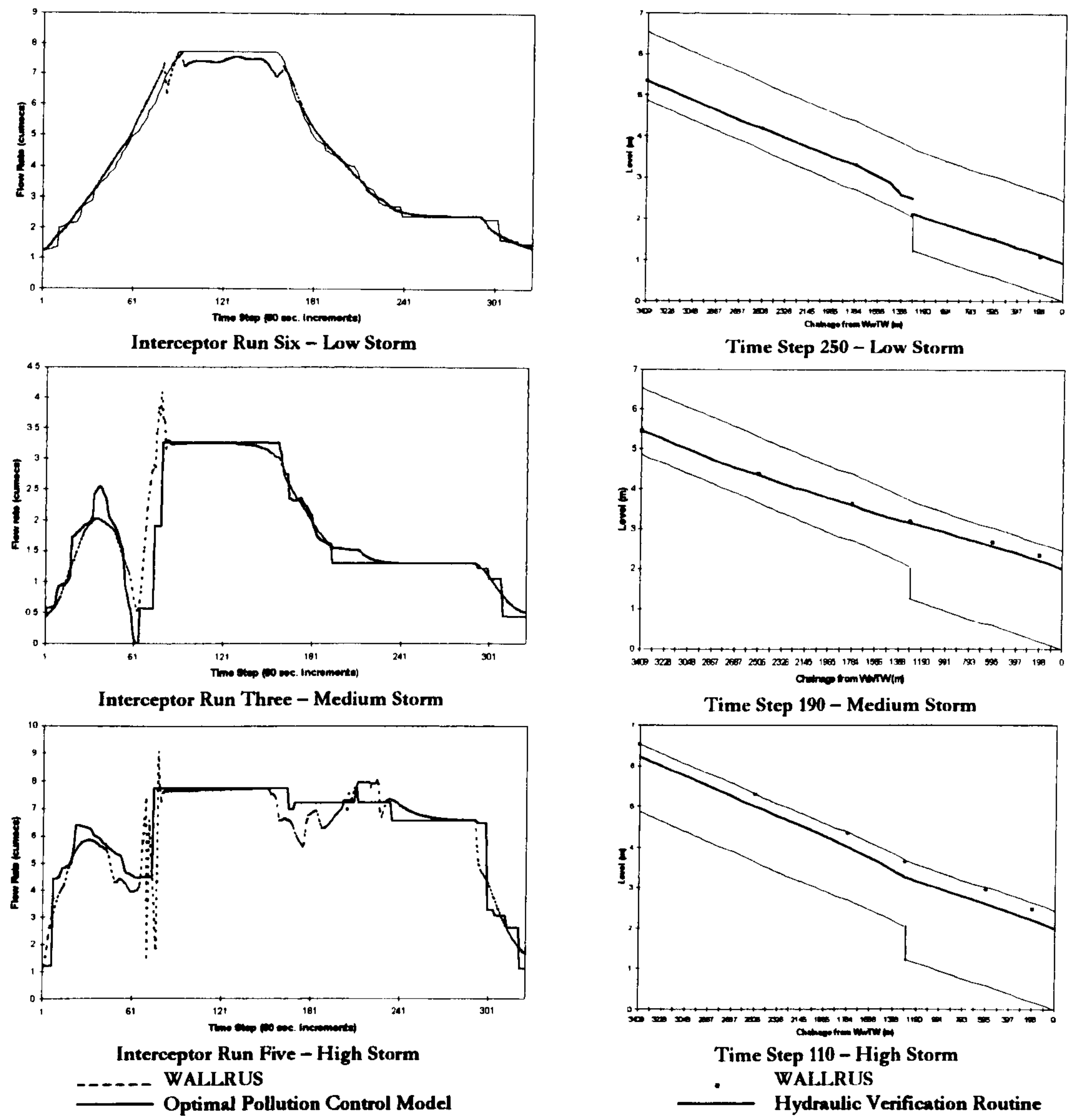

Figure 4: Sample Interceptor Sewer Hydrographs and Water Profiles

The points in Figure 4 show depths of flow, calculated by WALLRUS, in each section of the interceptor and provide validation of the hydraulic verification routine's water profiles. The apparent discrepancy at the pipe full condition arises because the hydraulic gradient routine, as presently coded, seeks the lower theoretical depth for the given discharge from proportional pipeflow relationships. The optimal control model has identified such conditions as being pipe full, so that this has no adverse effect upon the validity of the spill load prediction.

CONCLUSIONS

The results presented in this paper illustrate that the slug flow approach is a computationally efficient and sound formulation for the optimal control model and gives physically feasible interceptor sewer hydrographs and water profiles. More significantly, the approach is highly efficient computationally and offers promise for practical implementation of optimal real time control. Additionally, the results illustrate that the use of pipe full velocity in the optimisation module was acceptable within engineering limits.

Further research will include the extension of the control models to include secondary storage structures such as overflow chambers. As a final step towards practical implementation, efficient methodologies will later be used for the simulation of inflows from rainfall (Mehmood, 1996) and for specification of time-varying pollutant concentrations (Gupta, 1995).

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Optimal Pollution Control Models for Interceptor Sewers and Overflow Chambers

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1. Abstract

A robust method has been described for the optimal pollution control of interceptor sewer systems possessing overflow chambers. The hydraulics of the interceptor sewer has been idealised as a slug flow, which enables a computationally efficient solution of global control actions. Overflow chambers have been included into the control model allowing the determination of control strategies for realistic interceptor sewer systems.

The results from the application of the optimal pollution control model on the northern leg of the Liverpool Interceptor Sewer have shown considerable reductions in pollutant load spilled when compared to traditional fixed local control.

The formulation of the optimal pollution control model enabled the inclusion of non-linear equations that governed the continuation flow through the overflow chambers between time step control solutions, which did not significantly reduce the computational efficiency of the model. Therefore, the approach offers promise for practical implementation of optimal real time control (RTC).

2. INTRODUCTION

Interceptor sewer systems are designed to reduce environmental impacts from urban drainage systems (UDS) by diverting the flows from existing combined sewer outfalls to Wastewater Treatment Works (WwTW). Most combined sewer overflows (CSOs), at the junction between interceptor sewer and sewer system outfalls, are operated locally with static operating rules, commonly to 'Formula A' consents in the UK. These systems have, in general, considerably improved the quality of the receiving waters, especially aesthetically since the overflow structures are often designed to maximise gross solid retention. However, spills from CSOs during storm events can still be significant and now form the dominant source of pollution in many watercourses. In fact, it has been estimated that the CSOs contribute about one third of the pollutant load to urban streams (Andoh, 1994).

Many interceptor sewers have the facility for flexible active and remote control but these facilities are only used to regulate local control actions (where only the local measurements are used by the controller). This type of operating procedure has deficiencies since the loading of the entire interceptor sewer varies temporally and spatially. This is due to the heterogeneity of rainfall, the variations in response characteristics of the sub-catchments and the temporal and spatial variations in dry weather flow (DWF). Therefore by adopting only local control, it is likely that there will be needless overspills when storage is available elsewhere in the system. Global control, where measurements taken across the system are used to operate the flow regulators throughout the system, enhances the efficiency of the control actions. Here, the control procedure would reduce the frequency of spills by allowing overflow (spill) only when the entire interceptor sewer storage is fully utilised (if this is physically achievable). This has been an active area of research for the last decade (notably Schilling, 1989, 1994 and 1996).

An improved control procedure (for environmental improvements) of interceptor sewers, and UDS in general, is active pollution control. The development of these control models has been hindered by the complexities of synthesising the pollutant concentrations temporally within the sewer flow. However, this control procedure determines control actions (or control strategies) that not only maximise the sewer volume utilisation but also maximise the pollutant load retention, i.e. only the least polluting sewage would be spilled and this only when the sewer was completely full (if this was physically achievable). This has become a topical area of research in the last few years, especially with the increasing pressure of imposed targets of frequency of spills in regulations often arising from EC Directives. An example of this type of research is Weinreich et al (1997) who investigated pollution-based real time control of combined sewer systems and applied it to an interceptor sewer system. Their linear programming formulation differs from the model now discussed here.

An optimal pollution control model has recently been developed at the University of Liverpool for interceptor sewer systems where linear (LP) or dynamic programming (DP) can be used to maximise pollutant load retention within the sewer. The formulation of the control model for idealised interceptor sewers using a slug flow approach has been presented in Thomas et al (1998 and 1999b). The results illustrated the viability of using LP or DP and that significant environmental improvements could be achieved, in terms of pollutant load reduction to the receiving waters, when compared to fixed local control procedures. The model assigned a varying pollutant concentration factor to each inflow to synthesise pollution load from the flow hydrographs. The control model objective therefore minimised pollutant load in the spills from the sewer system. In this model sub-catchment inflows over and above the controlled interceptor inflows were assumed to spill without retention in overflow chambers.

The control models have been verified and validated against hydraulic criteria (Thomas et al, 1999a) utilising the WALLRUS (HRS, 1991) flow simulation package. For this a post-processing hydraulic verification routine was included into the model where a quasi-steady approach was used to develop approximate interceptor sewer water profiles using the Manning equation. Comparison against WALLRUS solutions demonstrated the adequacy of the slug flow approach in the control models.

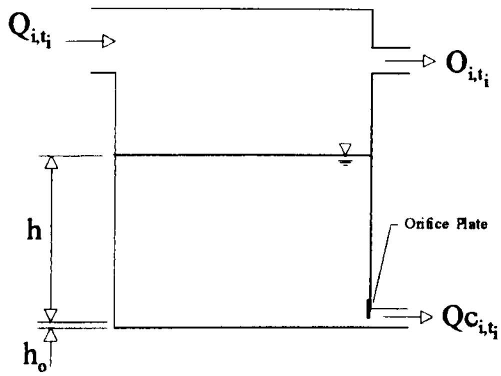
This paper presents the extension of the original control models to include the secondary storage effects of CSO volume. The formulation of this extension is described and applied to a simplified version of the northern leg of the Liverpool Interceptor Sewer System. Both fixed and variable local control procedures are included for the purpose of comparison.

3. OPTIMAL CONTROL MODEL EXTENSION TO STORAGE OVERFLOW CHAMBERS

The original control models determine optimum interceptor inflow rates based on incoming pollutant concentrations to maximise pollutant load retention within the interceptor sewer. The model is formulated using a slug flow approach where the 'slugs' are tracked through the interceptor and the control model determines the amount of sewage that should be added from the individual catchments based on the appropriate time delays and their respective pollution loadings. This optimisation problem is solved using two procedures, Linear Programming (LP) or Dynamic

Programming (DP). A detailed description of the model formulation and validation is presented in Thomas et al (1999b). In these models, flows in excess of the interceptor inflows were assumed to be spilled.

The control models have now been extended to include storage chambers at the intercept points. A typical chamber arrangement is shown in Figure 2.1.



where:

Q_{i,t_i} - Inflow from catchment i in time step t_i (cumecs).

O_{i,t_i} - Overflow from chamber i in time step t_i [if chamber level > spill level] (cumecs).

Qc_{i,t_i} - Continuation flow into interceptor from chamber i in time step t_i (cumecs).

h_o - Height of mid-point of orifice (m).

h - Head (m).

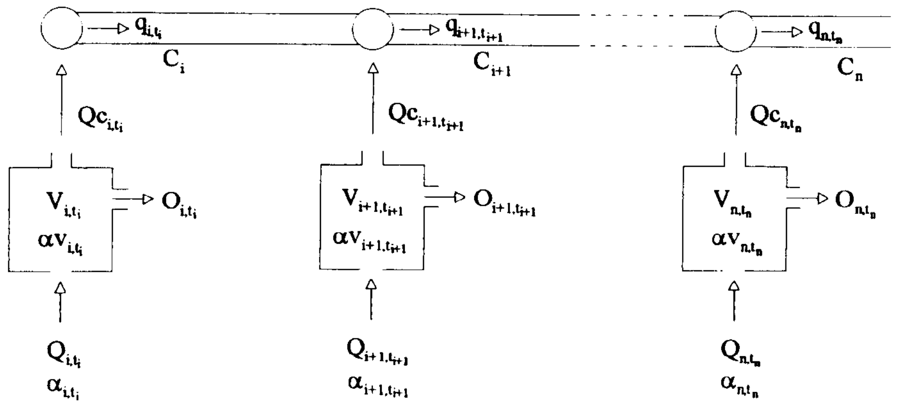
Fig. 2.1 Typical chamber arrangement.

The continuation flow rate, *Qc*, into the interceptor is governed by the non-linear equation :

$$Qc = C_d a \sqrt{2gh} \qquad \text{(i)}$$

where C_d is the coefficient of discharge of the orifice (dimensionless), a is the area of the orifice (m^2), g is the acceleration due to gravity (m/s^2) and h is the head (m). However, the optimal control model is formulated to determine the control strategies within discrete time steps and equation (i) can be solved between the time step solutions of the objective function.

The fundamentals of the extended optimal control model formulation are shown in Figure 2.2.



where:

q_{i,t_i} - Interceptor flow rate below chamber i in time step t_i (cumecs).

O_{i,t_i} - Overflow from chamber i in time step t_i [if chamber level > spill level] (cumecs).

$\alpha v_{i,t_i}$ - Chamber pollutant concentration factor in chamber i in time step t_i (dimensionless).

Q_{i,t_i} - Inflow from catchment i in time step t_i (cumecs).

V_{i,t_i} - Chamber volume in chamber i in time step t_i (m³).

α_{i,t_i} - Pollutant concentration factor of inflow from catchment i in time step t_i (dimensionless).

Qc_{i,t_i} - Throughflow into interceptor from chamber i in time step t_i (cumecs).

Fig. 2.2 Fundamentals of the extended optimal control model formulation.

The chamber pollutant concentration factor, $\alpha v_{i,t_i}$, is determined by the mixing model:

$$\alpha v_{i,t_i} = \frac{\alpha v_{i,t_{i-1}} V_{i,t_{i-1}} + \alpha_{i,t_i} Q_{i,t_i} \Delta t}{Q_{i,t_i} \Delta t + V_{i,t_{i-1}}} \tag{ii}$$

For computational convenience here the pollutant concentration factor is defined as a coefficient assigned to the inflow at each time step. For general illustration this coefficient can be considered to range from 0 to 1, i.e. absolutely 'clean' through to absolutely 'dirty' inflows, though more generally it might be the concentration (typically mg/l) of the chosen determinand.

Additionally, the control model maintains volumetric continuity within the chambers:

$$A\frac{\Delta h}{\Delta t} = Q_{i,t_i} - Qc_{i,t_i} \tag{iii}$$

where A is the storage chamber area (m^2) and h is the chamber level (m). However, equation (iii) applies to conditions when the chamber level h is less than the spill level, which is normally the invert level of the overflow pipe. When the chamber level h is greater than this level then:

$$A\frac{\Delta h}{\Delta t} = Q_{i,t_i} - Qc_{i,t_i} - O_{i,t_i} \tag{iv}$$

The overflow term also has to be included in the chamber pollutant concentration factor mixing model (ii) under these conditions to maintain continuity.

In application the initial state of the interceptor sewer system is assumed to be known. During each time step, therefore, the control model adds the inflow volume $Q_{i,t_i}\Delta t$ to the known chamber volume $V_{i,t_{i-1}}$ to obtain the possible chamber retention volume $V\max_{i,t_i}$, assuming that the entire inflow volume is retained within the CSO chamber. The corresponding chamber level h for this volume is used in the calculation of (i) to determine the maximum possible inflow rate $Qc\max_{i,t_i}$ when the orifice is completely open, i.e. when a is at a maximum in (i). Additionally, the chamber pollutant concentration factor $\alpha v_{i,t_i}$ is calculated from (ii) for this volume $V\max_{i,t_i}$, again assuming that it is completely retained within the CSO chamber. These values are used within the optimisation routine where the objective function is solved within the appropriate capacity constraints to determine the optimum control strategy:

$$Max\sum_{i=1}^{n}\alpha v_{i,t_i}q_{i,t_i} \tag{v}$$

subject to:

$$q_{i,t_i} \leq Qc\max_{i,t_i} \qquad \forall i \tag{vi}$$

$$\sum_{j=1}^{i} q_{j,t_j} \leq C_i \qquad \forall i \tag{vii}$$

where:-

n - number of intercept points;
t_i - time step position within the interceptor of intercept point i;
$\alpha v_{i,t_i}$ - pollutant concentration in chamber i in time step t_i;
q_{i,t_i} - interceptor sewer flow rate below intercept point i in time step t_i;
$Qc\max_{i,t_i}$ - maximum inflow into interceptor from chamber i in time step t_i; and
C_i - interceptor sewer pipe full capacity below intercept point i.

The actual continuation flows Qc_{i,t_i} that satisfy the objective function (v) are then calculated. From these values the orifice area a is calculated implicitly from (i) to determine the control action of the flow regulator (orifice gate or penstock). The chamber pollutant concentrations $\alpha v_{i,t_i}$ and chamber volumes V_{i,t_i}, as a consequence of the control strategy, are determined from (ii) and (iii) respectively. These values now represent the state of the CSO before the next time step in the solution procedure.

In the next time step, the procedure again adds the subsequent inflow volume $Q_{i,t_{i+1}}\Delta t$ to the stored volume V_{i,t_i} to determine the maximum possible continuation flow $Qc\max_{i,t_{i+1}}$ in this time step. The respective pollutant concentration factor is mixed with the chamber pollutant concentration in (ii). These values are then used within the optimisation routine, for LP (v), (vi) and (vii). This continues until all discrete time steps solutions have been determined within the control time horizon.

The objective function (v) represents decisions to be made that maximise the pollutant load received by a slug of sewage travelling through the interceptor incrementing inflows from the CSOs with the highest pollutant concentrations.

However, equation (v) corresponds to *only one time step t_i and the control strategies* throughout the control time horizon are determined by altering the time step position (i.e t_i+1, t_i+2, ...*etc*.). Since, in this approach, successive 'slugs' of water are assumed not to interact, then the sequence of optimal controls derived for each time step also represent the optimal control strategy for the entire event.

Overall, the optimisation is little changed from the original control model but the effects of storage (in the overflow chambers) on the pollutant concentration factors and inflow hydrographs are now accounted for. In this extended model formulation the control strategies are governed by the mixed pollutant concentrations in the storm chambers not the pollution concentration of the inflow hydrographs as in the original model.

4. TEST CASE – NORTHERN LEG OF LIVERPOOL INTERCEPTOR SEWER SYSTEM

The northern leg of the Liverpool Interceptor Sewer has been simplified and used as a test case for the extended optimal pollution control model. A longitudinal section of the sewer can be seen in Figure 3.1 and the input data for this sewer is shown in Table 3.1. The input data for the overflow chambers at the intercept points is shown in Table 3.2.

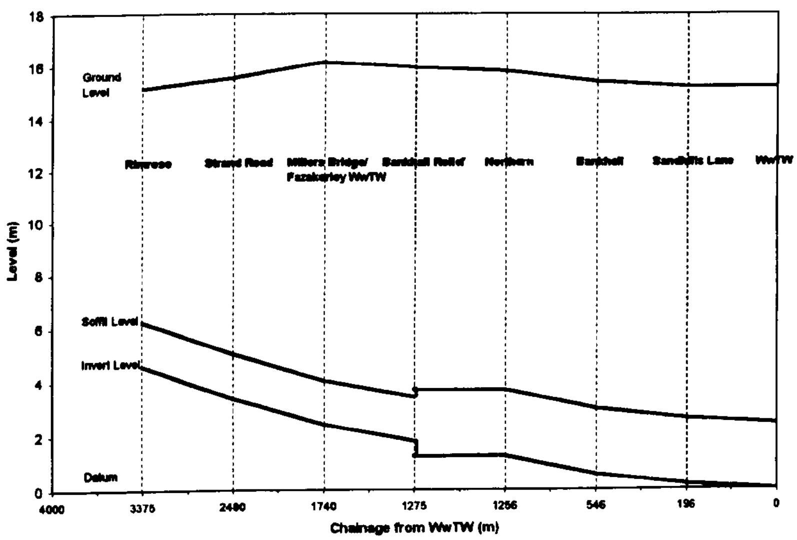


Fig. 3.1 Longitudinal section of the simplified northern leg of the Liverpool Interceptor Sewer (not to scale).

Intercept Point (catchment)	Sewer Diameter (m)	Sewer Gradient	Sewer Capacity (cumecs)	D.W.F. (cumecs)	Fixed Inflow Setting (cumecs)
RIMROSE	1.66	1/750	3.26	0.30	1.24
STRAND RD	1.66	1/750	3.26	0.09	0.25
MILLERS BRIDGE/ FAZAKERLEY WWTW	1.66	1/750	3.26	0.04	0.97
BANKHALL RELIEF	2.44	1/1000	7.72	0.14	0.69
NORTHERN	2.44	1/1000	7.72	0.50	2.13
BANKHALL	2.44	1/1000	7.72	0.11	0.29
SANDHILLS LANE	2.44	1/1000	7.72	0.09	0.31

Table 3.1 Input data for test interceptor sewer.

INTERCEPT POINT (CATCHMENT)	CHAMBER AREA (M^2)	SPILL LEVEL [ABOVE INVERT LEVEL] (M)	ORIFICE WIDTH (M)	ORIFICE HEIGHT (M)
RIMROSE	282.82	5.42	1.250	1.450
STRAND RD	136.03	6.91	1.700	0.625
MILLERS BRIDGE/ FAZAKERLEY WWTW	50.31	7.95	0.354(E)	0.354(E)
BANKHALL RELIEF	169.78	8.04	2.075	0.625
NORTHERN	328.24	8.18	2.650	1.450
BANKHALL	167.06	8.47	1.800	0.625
SANDHILLS LANE	147.95	9.26	1.650	0.625

(E) – Equivalent dimensions.

Table 3.2 Storm chambers input data for test interceptor sewer.

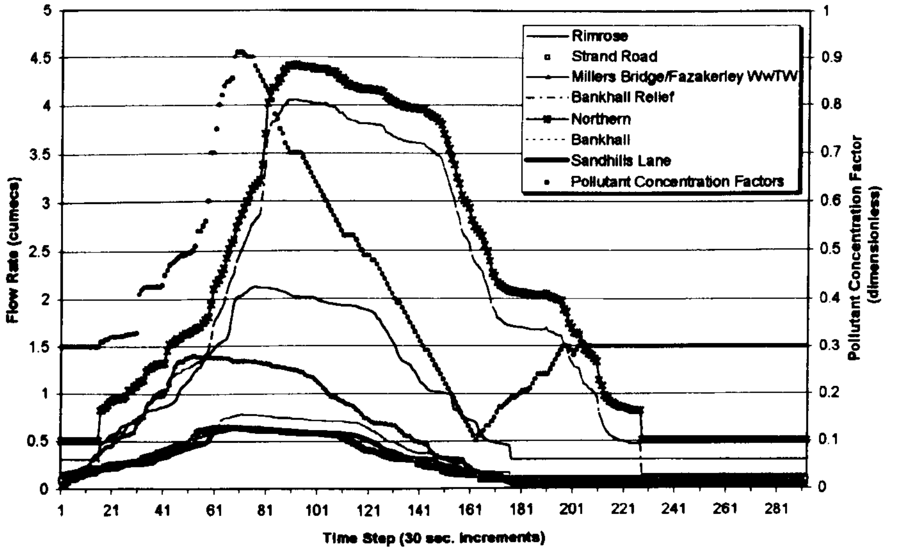


Fig. 3.2 Inflow hydrographs and pollutant concentration factors for all catchments.

The control model was run with hypothetical runoff hydrographs (Figure 3.2), which were loosely based on the catchment's response characteristics, and the respective pollutant concentration factors (Figure 3.2). For computational convenience the pollutant concentration factors were taken to be identical for each catchment. Two control procedures were considered in the test case illustrated here: fixed local control (FLC), where inflows up to the fixed inflow setting are passed forward and no account is taken of the interceptor sewer's pollutant concentrations; and optimal pollution control (OPC), where the optimal control model uses global information including pollutant concentrations within the interceptor and the storage chambers in making its decisions. Both control procedures use the slug flow approach within the interceptor sewer to convey the inflows along the interceptor and only the decision criteria differ.

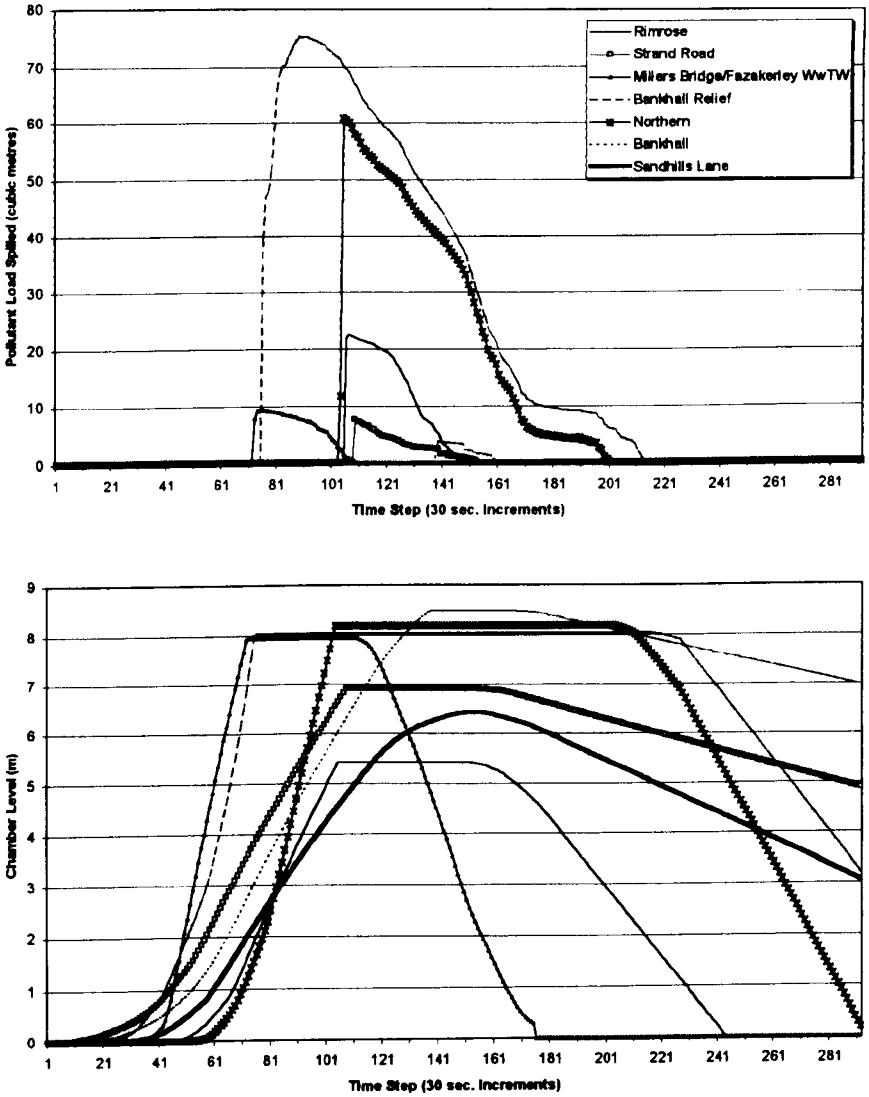


FIG. 3.3 Fixed local control (FLC) chamber levels (bottom) and pollutant load spilled (top) – common legend. (Pollutant load spilled = spill volume × pollutant concentration)

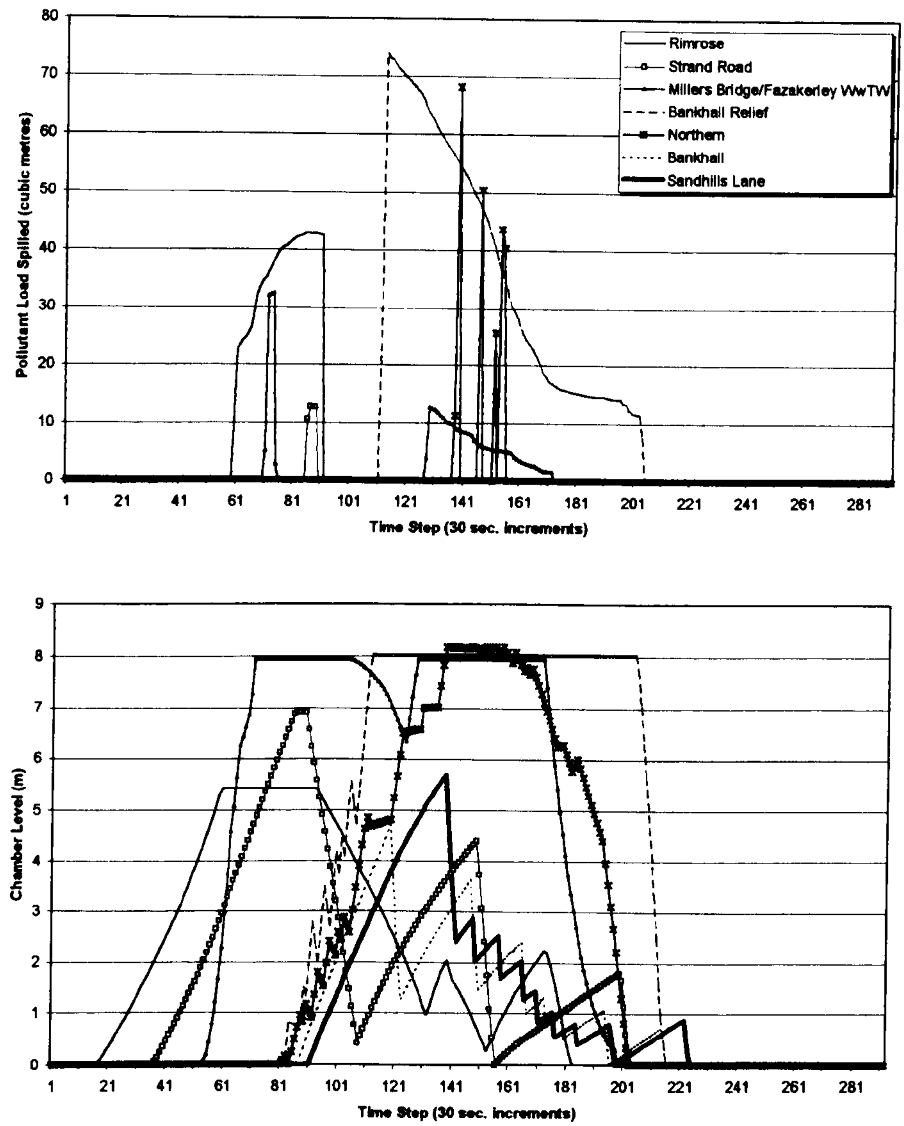


Fig. 3.4 Optimal pollution control (OPC) chamber levels (bottom) and pollutant load spilled (top) – common legend. (Pollutant load spilled = spill volume × pollutant concentration)

POLLUTANT LOAD TO RECEIVING WATERS (Spill Volume × Pollutant Concentration)			IMPROVEMENT (OPC v FLC) %	IMPROVEMENT (OPC v VLC) %
Fixed Local Control (FLC)	Variable Local Control (VLC)	Optimal Pollution Control (OPC)		
8860.06	6203.25	5189.48	41.43	19.54

Table 3.3 Comparison of control procedures.

Figures 3.3 and 3.4 show the variations of chamber levels and pollutant load spilled from each overflow chamber along the northern leg of the Liverpool Interceptor using the FLC procedure and the OPC procedure. They show that the OPC procedure reduced the pollutant load spilled compared to the FLC procedure. Under moderate rainfall where the interceptor capacity is only slightly insufficient to deal with flows, the OPC procedure should only spill at CSOs that have the highest pollutant concentrations so a reduction in spill occurrences may be expected. However, it is evident in this case that the same chambers have spilled as in the FLC procedure, but at reduced volumes. This arises because of the magnitude of the run-off event and the fact that the inflow hydrographs and pollutant concentrations were highly synchronised. A spatially and temporally varied storm would generate localised peaks in pollutant load that would more clearly show the potential benefits of the optimal pollution control model.

Figures 3.3 and 3.4 illustrate the effect of using optimal active control in that there are considerably more fluctuations in storage conditions with this type of control procedure. This is evident in the results for both the pollutant load spilled and chamber levels. This is because the control procedure responds to variations in the operative state of the sewer system through time rather than using a fixed operating procedure. Figure 3.3 does not show the fluctuations in conditions within the CSO chambers and all the chambers follow similar 'filling' and 'emptying' characteristics. This is, perhaps, the only drawback of optimal active control since an increase in control actions will have a corresponding increase in flow regulator activity. It is likely that an increase in regulator activity would increase the frequency of operating problems. Therefore, a compromise is needed where constraints are used within the optimisation to reduce the activity of the flow regulators to within acceptable limits even though the resulting control solution would then be sub-optimal. The inclusion of such additional constraints is a topic of current research.

Figure 3.4 shows that the OPC procedure improves the chamber recovery times compared to the FLC procedure. In fact, Figure 3.3 shows that the control time horizon selected for the calculations was not of sufficient length in this case since some of the chambers had not fully recovered from the storm event. This shows an additional benefit of using optimal active control. The OPC procedure should be better able to control multiple-peak inflow hydrographs or pollutographs compared to the traditional FLC procedure.

Table 3.3 shows a comparison between the FLC and OPC procedures. In this application of a reasonably low intensity highly synchronised storm event (i.e. peak inflow to the system at approximately 2 times fixed inflow settings), the OPC procedure produced a 41% improvement in pollutant load spilled compared to the FLC procedure. An additional result has been included in Table 3.3, arising from adoption of variable local control VLC, where interceptor inflows are permitted up to the interceptor sewer's capacity locally but no account is taken of pollutant concentrations. In this case, the interceptor sewer volume is fully utilised, if physically possible. The OPC procedure produced a 20% improvement compared to VLC. The results from the VLC procedure (not presented) show that only the downstream CSO chambers spill in each of the two sewer sections. This is because the VLC procedure fills the interceptor as soon as possible in the upstream section of the sewer. Other results (which have not been presented) show that, generally, the improvements of using the OPC procedure reduce as the severity of the storm event increases. This is expected because spills are inevitable with larger inflows where the interceptor sewer and overflow chamber storage are quickly utilised. It is likely that most improvements would be encountered when the hydrographs and corresponding pollutant concentrations vary spatially and temporally in low to moderate rainfalls that occur more frequently.

The inclusion of overflow chambers has increased the computational demand of the optimal pollution control model. This is because equation (i) is solved implicitly between time step solutions to determine the actual control strategies on the penstocks. This is obviously sensitive to the numerical method used and an increase in computationally efficiency may be achievable with the use of other numerical methods and this is also a topic of ongoing research. The effect is not, however, considered to be too prejudicial since the model runs considerably faster than real time.

5. CONCLUSIONS

A robust method has been described for the optimal pollution control of interceptor sewer systems that include overflow chambers. The hydraulics of the interceptor sewer has been idealised by a slug flow approach, which allows for a computationally efficient solution of global control actions. Overflow chambers have now been included into the control model to allow control strategies to be determined for more realistic interceptor sewer systems.

The results from the application of the optimal pollution control model on the northern leg of the Liverpool Interceptor Sewer have shown considerable reductions in pollutant load spilled when compared to traditional fixed local control.

Overall, the formulation of the optimal pollution control model enabled the inclusion of non-linear equations that governed the continuation flow through the overflow chambers between time step control solutions, which did not significantly reduce the computational efficiency of the model. Therefore, the approach offers promise for practical implementation of optimal real time control (RTC). As a final step towards practical implementation, efficient methodologies will later be used for the simulation of inflows from rainfall (Mehmood, 1996) and for specification of time-varying pollutant concentrations (Gupta, 1995).

6. Acknowledgements

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POLLUTANT LOAD OVERSPILL MINIMIZATION OF INTERCEPTOR SEWER SYSTEMS

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This paper describes the development of an optimal pollution control model for interceptor sewer systems from conceptualization to detailed validation. The formulation of two optimization alternatives using a slug flow approach is presented. The models are tested on an idealized interceptor sewer system and a representative storm event to confirm the viability of the optimization techniques. A hydraulic verification routine is included and the WALLRUS hydrodynamic sewer flow simulation software package is used to validate the results from the hydraulic verification routine. Application of the model to an existing interceptor sewer is presented. The results from the idealised test case show the viability of using Linear Programming and Dynamic Programming within the optimal pollution control models. The models give physically feasible solutions that can be validated using WALLRUS. Significant environmental improvements (in terms of pollutant load reduction to the receiving waters) can be achieved. The results illustrate the validity of the assumptions in the optimal control model formulation, and show that the slug flow approach provides a computationally efficient and sound formulation for the optimal control model and offers promise for practical implementation of optimal real time control.

Keywords: Interceptor sewer systems; pollution control; linear programming; dynamic programming

1. INTRODUCTION

Historically, urban drainage systems (UDS) were designed to provide hydraulically and economically effective transport of surface runoff and foul sewage from urban areas into receiving waters. The majority of UDS in the UK are combined sewer systems (CSS) where the raw sewage and surface runoff are transported through a common network. It is well recognised that this discharge of combined sewer flow into receiving waters has significant environmental implications [1] and this has resulted in the devastation of many urban waterways.

Interceptor sewer systems were conceived to reduce the environmental impacts on receiving waters by diverting the combined sewer flow from the original sewer outfalls to a Wastewater Treatment Works (WwTW). Interceptor sewer systems, generally, consist of large pipes, storage tanks and pumping stations. Such a system has been installed in the Liverpool area as part of the Mersey Estuary Pollution Alleviation Scheme (MEPAS). This scheme has been implemented in phases over the last two decades leading to significant improvement in the River Mersey's status, which has seen a return of fish species into the estuary. The Liverpool Interceptor Sewer runs parallel and close to the banks of the river where it intercepts the original combined sewer outfalls. The system is very complex being twenty-nine kilometres long with twenty-six intercept points where storm overflow structures form the junction between existing outfall sewers and the lower level interceptor sewer. The flow times within the sewer are therefore very large and the system potentially has millions of possible control states governing the entry of sewage into the interceptor during storm conditions.

Interceptor sewer systems have limited flow capacities that are periodically exceeded by sewer flows generated during storm events. Therefore, the systems are often designed to spill from overflow structures to mitigate surcharge of the interceptor. These combined sewer overflows (CSOs) cause intermittent impacts that can be significant. Currently in the UK, most interceptor sewers are operated locally with static operating rules to 'Formula A' consents. For example, the continuation flow rate through a throttle valve in an online overflow storage chamber may be governed by the chamber water level. Most usually a fixed throttle is adopted, i.e. a small diameter pipe or orifice plate is inserted and is designed to pass a specified flow when the chamber is almost full. However, some systems have moveable regulators (with monitoring facilities) such as the electro-mechanical penstocks in the Liverpool Interceptor, that enable flexible active and remote control. In this example, a sensor measures the chamber water level and a controller uses this information to regulate the throttle valve. This procedure is termed Local Control since only the local process measurements are used by the controller. This operating procedure has deficiencies since the loading of the entire interceptor sewer varies temporally and spatially. This is due to the heterogeneity of rainfall, the variations in response characteristics of the sub-catchments and the temporal and spatial variations in dry weather flow (DWF). Therefore, needless overspills are likely when storage is available elsewhere in the system. Global Control, in which the process measurements are taken across the system to operate the flow regulators throughout the system, enhances the efficiency of the control actions. Here, the control procedure reduces the frequency of spills by only overflowing when the entire interceptor sewer storage is fully utilised (if this is physically achievable).

A Global Control procedure can be developed by optimizing an operational model of the entire interceptor sewer. Examples of typical operational objective functions are minimization of flooding, minimization of operational cost and minimization of overspill volume. This last operational objective leads to an obvious result – fill the interceptor sewer as quickly as possible and keep it full. This maximizes the volumetric utilization in the sewer, thereby reducing total overflow volume. However, there is further scope for environmental improvements if the interceptor system is optimized for water quality consents. In this case, the objective function maximizes pollutant load retention within the interceptor sewer system.

There has been a significant amount of research undertaken in the control of urban drainage systems in general, notably by Schilling [10-12]. Nelen [8], Nielsen *et al.* [9], Khelil *et al.* [6] and Fuchs *et al.* [2] researched the **volumetric** control of urban drainage systems using various procedures but to the authors' knowledge there has been little research in the optimal **pollution** control of interceptor sewer systems. Gómez [3] undertook some research into the automatic control of interceptor sewers but this was not based upon optimization techniques. Weinreich *et al.* [13] researched pollution- based real time control of combined sewer systems and applied it to an interceptor sewer system. However, their formulation differs from the optimal pollution control model discussed here.

This paper describes the development of an optimal pollution control model for interceptor sewer systems from conceptualization to detailed validation. The formulation of two optimization alternatives (a Linear Programming (LP) model and a Dynamic Programming (DP) model) using a slug flow approach is presented. The models are tested on an idealized interceptor sewer system with a realistic but hypothetical storm event to confirm the viability of the optimization techniques. A post-processing hydraulic verification routine has been incorporated in which a quasi-steady approach is used to develop approximate interceptor sewer water profiles. Additionally, the WALLRUS software package [5] is used to validate the results from the hydraulic verification routine. The optimal pollution control model is applied to a simplified version of the northern leg of the Liverpool Interceptor Sewer System as a case study. In this application, a series of storm events were input into the model and the results from the control model, hydraulic verification routine and WALLRUS validation are discussed.

2. OPTIMAL POLLUTION CONTROL MODEL FORMULATION – LINEAR PROGRAMMING (LP)

The formulation of the optimal pollution control model allows the optimum solution to be obtained using LP or DP. The control models were developed using a slug flow approach and determine optimum inflow rates for an interceptor sewer system based on estimated pollutant concentrations. The slug flow approach and LP formulation are described below.

Various assumptions are made in the formulation of the control models including: a) all inflows and their respective pollution concentrations are known; b) hydrographs are piecewise constant; c) unintercepted flows are spilled to the river; d) all pipe flows are in the downstream direction; and e) there is complete control over the proportion of flow diverted into the interceptor from the CSOs.

The fundamentals of the optimal pollution control model are shown in Figure 1. Figure 1 represents a decision to be made at each intercept point where q_i is the control variable. $(Q_i - q_i)$ is the spill rate as a result of the control decision q_i. Q_i are hydrographs so Q_i and q_i are functions of time. Therefore, a second subscript is

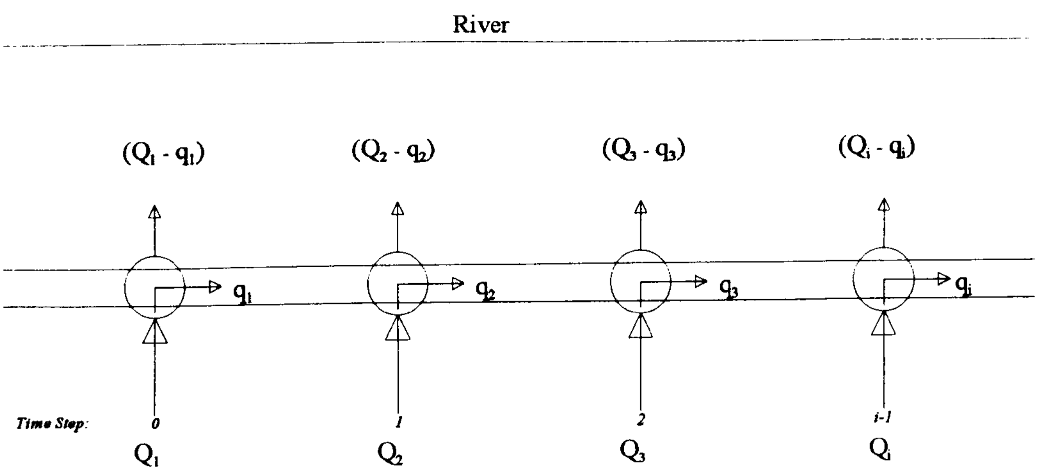


FIGURE 7 Theoretical basis of the control model.

included in the notation to allow for this, j, corresponding to a particular time step. For example, $Q_{1,2}$ corresponds to the inflow at intercept point 1 in time step 2.

For the purposes of model development, some further assumptions are initially imposed (and later removed). It is assumed that interceptor points are equally-spaced along the interceptor, flow velocity is constant, and therefore the time step can be chosen to be equal to the time of flow in the interceptor sewer between any two intercept points. As a consequence of this assumption, the model can be described as a chain of water travelling down the interceptor system as shown in Figure 2. A 'slug' of water, $q_{1,0}$, enters the interceptor at the extreme upstream end (intercept point 1) at time t = 0 and travels down the interceptor in the first time step. At t = 1 it arrives at intercept point 2 where it is incremented by slug $q_{2,1}$. The combined slug moves on down the sewer, accreting slugs $q_{3,2}$, $q_{4,3}$ etc., until the outfall is reached. The slugs of water are therefore treated as being separate in time and space. Therefore, an explicit constraint for hydraulic continuity between time step solutions within the optimization is not needed. This approach has hydraulic deficiencies since the slugs will of course interact to some degree but it enables a highly efficient computation of the control actions (or control strategy).

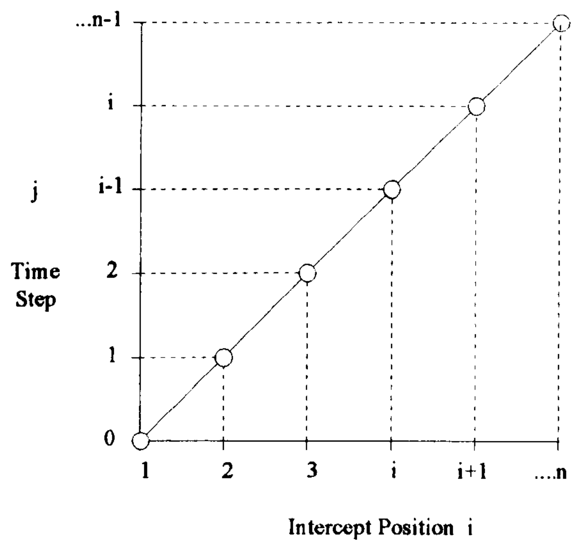


FIGURE 8 Chain of water commencing at time step 0.

A pollutant concentration factor, $\alpha_{i,j}$, is introduced into the model to facilitate the optimization of water quality s. For computational convenience here the pollutant concentration factor is defined as a coefficient assigned to each inflow at each time step. For general illustration this coefficient can be considered to range from 0 to 1, i.e. absolutely 'clean' through to absolutely 'dirty' inflows. A standard LP optimization model can now be set up.

Objective Function: Minimize pollutant load to the receiving water over variables $q_{i,j}$.

Thus,
$$Min\sum_{i=1}^{n}\alpha_{i,i-1}(Q_{i,i-1}-q_{i,i-1}) \tag{1}$$

where n is the number of intercept points. Or, since the total pollutant load $\sum_{i=1}^{n}\alpha_{i,i-1}Q_{i,i-1}$ is invariant for any storm event,

$$Max\sum_{i=1}^{n}\alpha_{i,i-1}q_{i,i-1} \tag{2}$$

subject to capacity constraints.

Constraints:
$$q_{i,i-1}\le Q_{i,i-1} \text{ for all i} \tag{3}$$

$$\sum_{j=1}^{i}q_{j,j-1}\le C_i \text{ for all i} \tag{4}$$

where C_i = Interceptor capacity at point i.

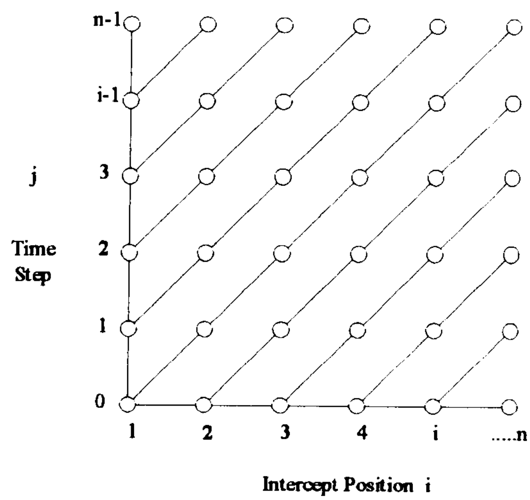


FIGURE 3 Complete model with chains from all time steps.

The objective function (2) has been developed in the LP model for the chain of water shown in Figure 2. This LP problem can be solved by any standard LP solver, in this case a computer program written in FORTRAN code using the Simplex Method. However this is just one chain of water running through the interceptor and there are other chains at different time steps. The complete chain model over a complete storm event is shown in Figure 3. This can be solved by the same method as described above but each chain is solved in sequence (i.e. j+1, j+2,…, n-1). The solution procedure commences at the lower right hand corner of Figure 3, i.e. at intercept point n in time step 0. In fact, there is no decision at this point since this represents the initial state of the sewer (cumulative DWFs). The chain of water to the left is the next solution to be determined. Here, the optimization routine determines how much inflow to add to the initial state of the interceptor at intercept point n (based on the pollutant concentration of that inflow). Further chains are processed in sequence. The methodology tracks the slugs through the interceptor and determines the amount of sewage that should be added from each catchment based on the pollutant concentrations at the appropriate time steps. The procedure continues until all the chains are solved throughout the time horizon and the full control strategy is obtained. Since, in this approach, the successive slugs of water are assumed not to interact, then the sequence of optimal controls derived each time step also represents the optimal control strategy for the entire event.

An extended version of the LP model relaxes the initial assumption of equally-spaced interception points along the interceptor sewer pipe. The interceptor pipe is now divided into time steps and there are intercept points on only some of these steps. This is shown in Figure 4 and is better able to represent a realistic system. The model can now control interceptor systems where the intercept points are at irregular intervals and times of flow between intercept points are not constant. This version of the model differs slightly from the original version in the definitions of the inflow rates and pollutant factors at the appropriate time steps.

The modified LP objective function in one time step is:

$$Max \ \sum_{i=1}^{n} \alpha_{i,t_i} q_{i,t_i}$$

where n is the number of intercept points, t_i is the time step position within the interceptor of intercept point i, $\alpha_{i,ti}$ is the pollutant concentration factor at intercept point i in time step t_i and $q_{i,ti}$ is the interceptor sewer flow rate after intercept point i

in time step t_i. The constraints for the solution are adjusted to coincide with the appropriate time step t_i. The solution procedure is shown in Figure 5 where the varying line gradients show the time step positions of the intercept points in Figure 4. This LP model is solved using the Simplex method. Again, the complete strategy is determined by adjusting the time step position (i.e. t_i+1, t_i+2,...etc.).

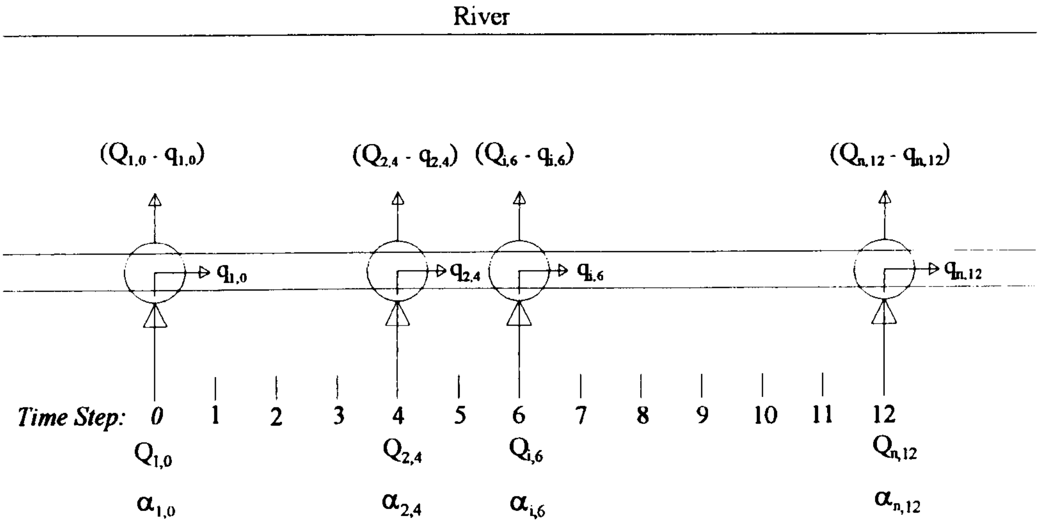


FIGURE 4 Extended version of control model.

3. OPTIMAL POLLUTION CONTROL MODEL FORMULATION – DYNAMIC PROGRAMMING (DP)

Dynamic Programming (DP) is an extremely fast and efficient method of solving multi-stage sequential optimization problems. Unlike LP, there is no standard solver such as the Simplex Method; each DP problem is formulated from first principles and is solved using a computer program unique to that problem.

The DP model is formulated in a similar fashion to the LP model, in that the model determines the optimum interceptor sewer flow rates for the chains in Figure 3. However, the difference between the models lies in the solution method. Figure 5 shows the fundamentals of the DP model. The slug of water travelling though the interceptor sewer is the DP state variable, S_i (limited by the interceptor capacity), and the algorithm determines the quantity of inflow from each catchment, d_i (the decision variable), to add to this slug based on the pollutant concentrations of all the inflows at the appropriate time steps. The cost of the decision, r_i, is the resultant pollutant load to the receiving water. The optimum solution is reached when the total pollutant load spilled from the entire interceptor sewer, $\sum_{i=1}^{n} r_i$, is a minimum.

Figure 6 shows an example of the DP solution method. In this example there are ten possible interceptor flow rates, each of which is a proportion of the sewer capacity. This capacity is the capacity at the extreme downstream section of the interceptor system. The selection of this proportion depends on the accuracy required and might be considered to represent the dynamic restrictions in setting of the control gates. However, the increase in accuracy will have a consequential increase in computational time. The DP solution method in Figure 6 appears to be complicated, however, the DP method only stores two stages in the memory at any one time. This is because the cumulative cost at any node is determined by adding the cumulative cost of the previous node to the link cost connecting those nodes. All preceding costs are then discarded.

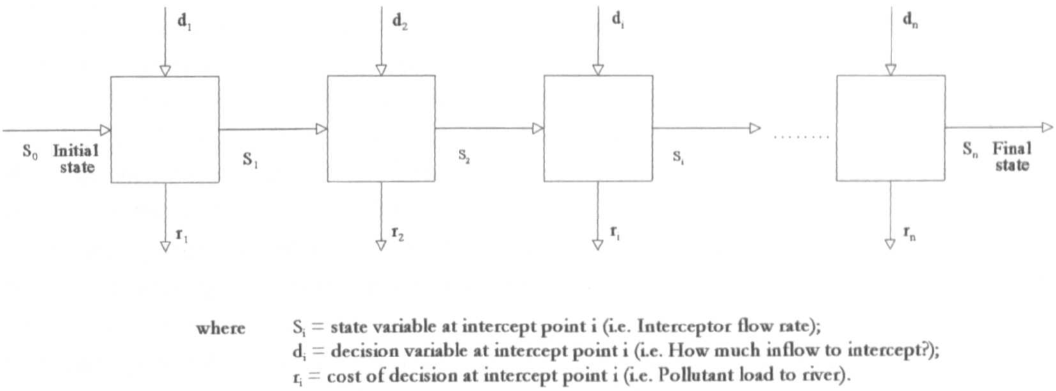


FIGURE 5 Fundamentals of DP solution method.

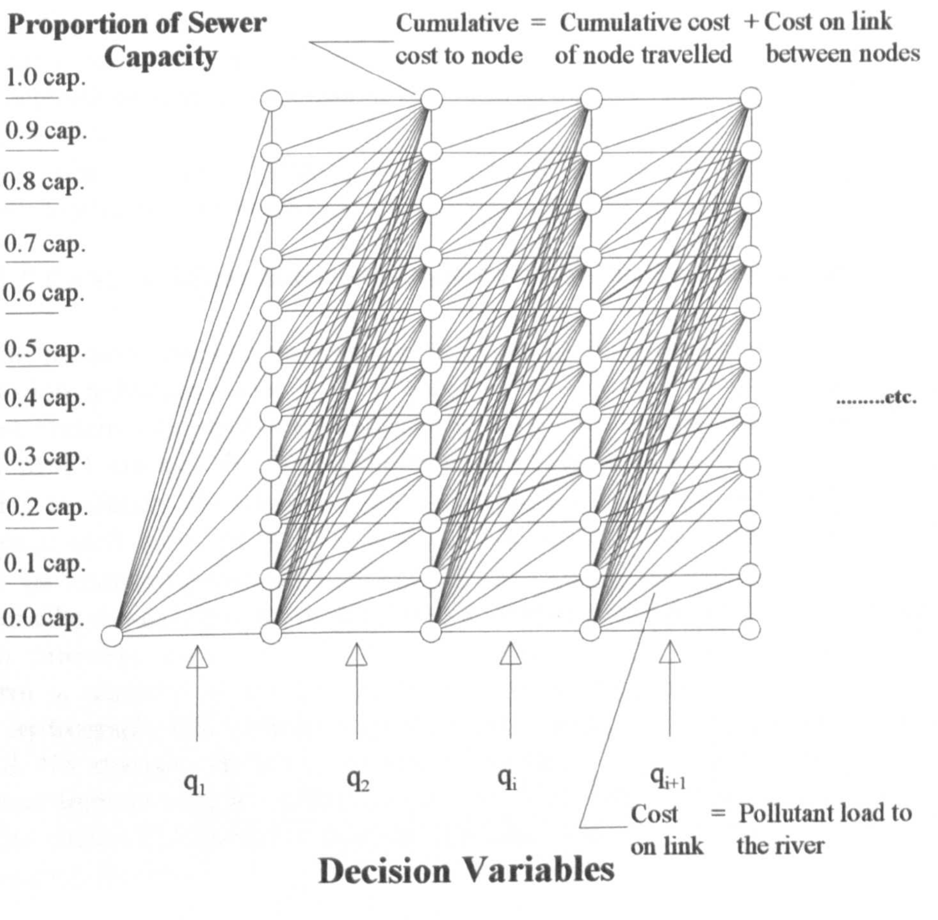


FIGURE 6 Example of DP solution method.

The DP solution method in Figure 6 is described as follows:

1. The DP method works through the system from left to right.
2. The starting cost (at the extreme left-hand node) is 0 since there are no spills.
3. The cost on the link for the decision q_1 to get to node proportion 0.0 times sewer capacity is the spill volume multiplied by the pollutant concentration factor (i.e.

pollutant load to river). Clearly, there is no increase in interceptor sewer flow; therefore the entire inflow volume is spilled.

4. The cumulative cost for the decision q_1 to get to node proportion 0.1 times sewer capacity is the cumulative cost at the node travelled from, plus the link cost between the nodes. As the cumulative cost equals 0 then the cumulative cost to 0.1 times sewer capacity equals the link cost. Note that for this node there is only one route possible but at other nodes further down the network there will be other routes possible.

5. Clearly, the cumulative costs to all the nodes in decisions q_1 equal the link costs that can easily obtained by the above method. These values are stored in the memory.

6. The method moves on to the next stage (q_2).

7. At any particular node within this stage there are various routes possible to arrive there. For example at interceptor flow 0.6 times sewer capacity for decision q_2 there are six possible routes possible. These six link costs can be determined by calculating the resultant pollutant load to the river. All the cumulative costs in the previous stage are known, therefore the cumulative costs of each of the six routes can be determined. The 'cheapest' (least cumulative pollutant load to the river) is stored in the memory and the route, which attained this cost, is indexed. All other costs are discarded.

8. The solution process is continued throughout the system until the 'cheapest' cumulative cost at the final node is attained.

9. A traceback procedure determines the strategy used in attaining this 'cheapest' final cumulative cost.

10. The optimum control strategy is determined by calculating the required inflows necessary to generate the interceptor flow rates in the DP solution.

4. TEST CASE – IDEALISED INTERCEPTOR SEWER SYSTEM

A simple fictitious interceptor sewer system has been used to test the viability of the optimization techniques used in the optimal control models and is shown in Figure 7. The test system has eight intercept points each with intercepted dry-weather flows (DWF) of 0.1 cumecs. In this application, the DWFs were deducted from the pipe capacities to obtain the effective sewer capacities. The interceptor pipe capacity C_i increases at each intercept point with a final capacity of eight cumecs. This is also the WwTW (at intercept point 8) treatment capacity. Each catchment is identical in layout and hydraulic design. One thousand possible proportions (settings) of the control gate at each intercept point have been considered for the DP method. A fictitious rainstorm is assumed to hit all the catchments at the same time and the resulting runoff hydrograph and pollutant concentration factors are postulated as shown in Figure 8. No attempt was made at synthesising runoff or pollutant concentrations but the concentrations roughly follow a first foul flush relationship. The model was run using the idealised interceptor system data and runoff hydrographs with pollutant concentration factors.

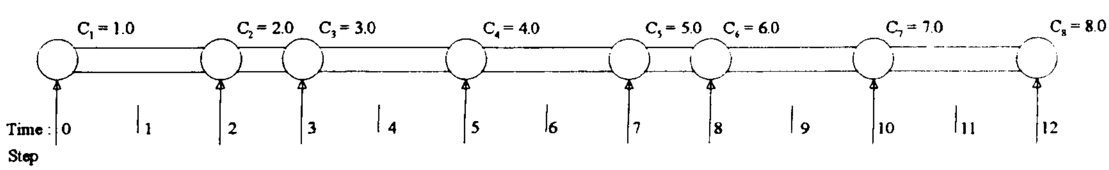


FIGURE 7 Idealised interceptor sewer system.

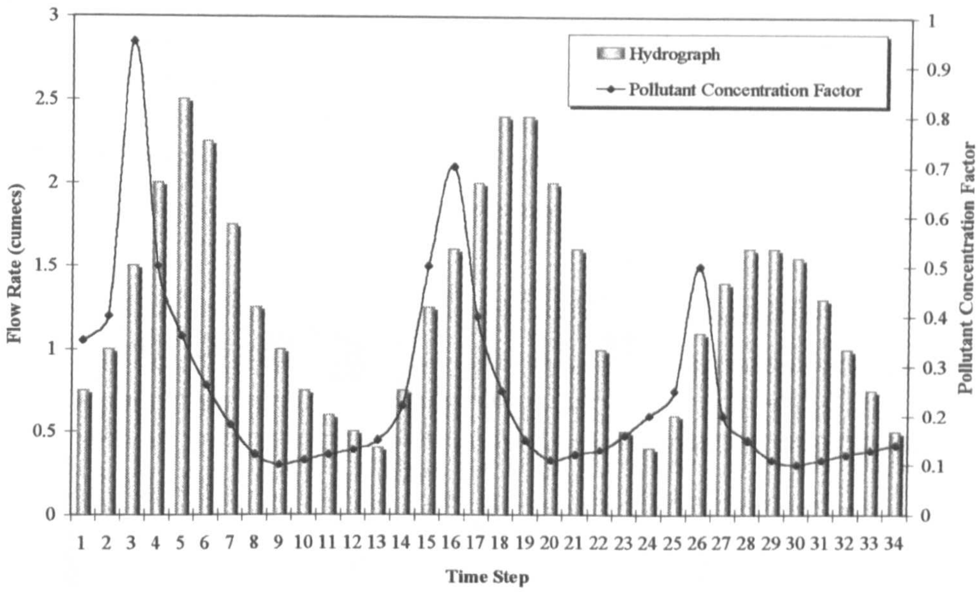


FIGURE 8 Runoff hydrograph and pollutant concentration factors.

Four different control procedures were considered:

Fixed Local Control: Intercepted flows are determined from local conditions at each independent interception point. Flows to the interceptor are governed by the use of flow restrictors (e.g. vortex devices) which restrict the inflow to a pre-set maximum. In this system the maximum flow rate was set at one cumec at every interception point. Clearly, the sum of these maximum flows must be less than or equal to the downstream interceptor and treatment works capacities. No account is made of the conditions in the interceptor system or conditions at other interception points. The method is volumetric based and no account is taken of the pollutant load of the flows.

Variable Local Control: This method determines intercepted flows using information about the conditions locally in the interceptor system. That is, if there is spare capacity locally in the interceptor, intercept flows up to this capacity can be permitted. The method is volumetric based and no account is made of the quality of the flows.

Global Control (LP): This method uses global information, including pollutant concentrations, to determine optimum strategies using the LP model.

Global Control (DP): This method uses global information, including pollutant concentrations, to determine optimum strategies using the DP model.

All the control procedures use the slug flow approach within the interceptor sewer; only the decision criteria differ. Volumetric optimization has not been included explicitly because the variable local control objective also fully utilises the available storage within the sewer (though spills will occur at different locations).

Figure 9 shows the control strategies from the various control procedures at all intercept points at time step 21. It shows the inflows, their respective pollutant concentration factors and the control strategies. The global control strategies in Figure 9 and 10 were obtained using the LP model. The local control strategies were obtained on volumetric criteria as defined above and no account was taken of the pollutant concentration factors.

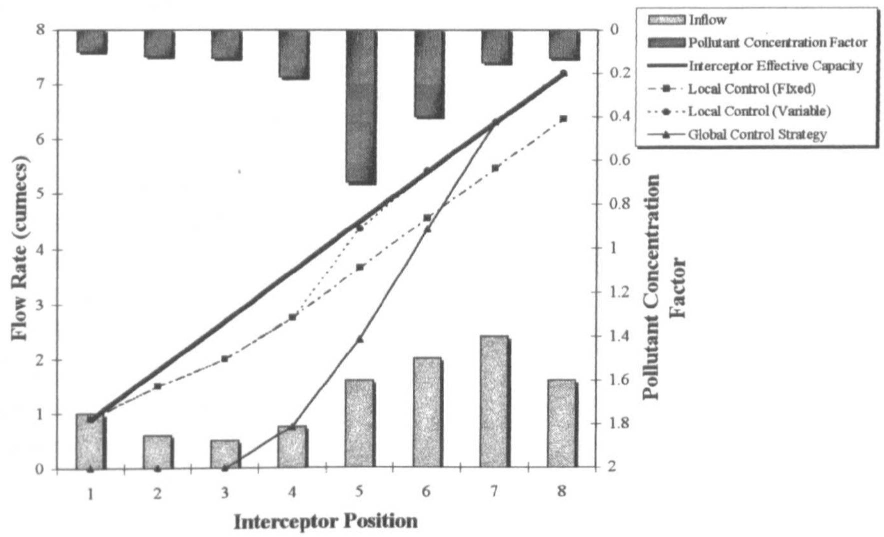


FIGURE 9 Strategies for chain of water in time step 21, (global control determined by LP model).

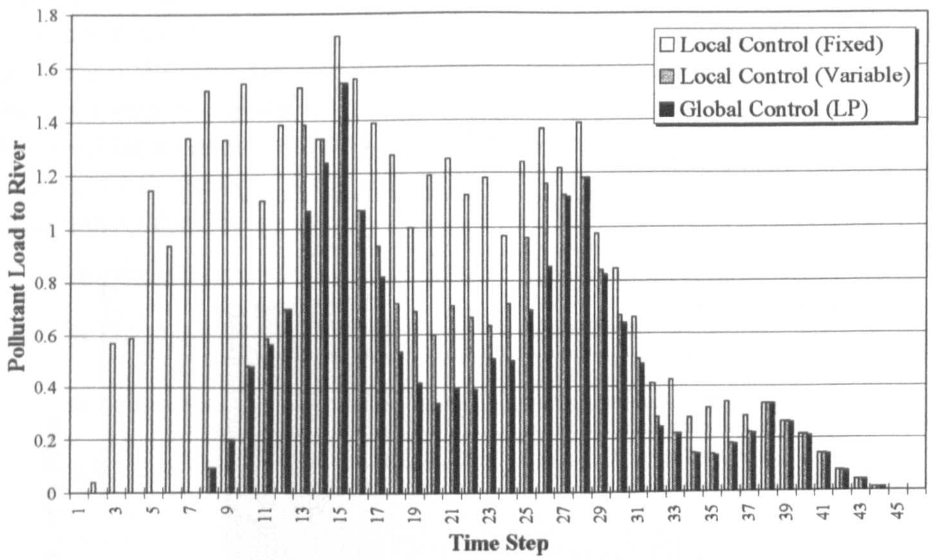


FIGURE 10 Comparison of local and global control strategies (determined by LP model) within the control horizon.

The DP results are almost identical to the LP results, as was expected because only the solution method differs between the two models. However, in some time steps the DP global control strategy spills slightly more pollutant load than the LP solution. This is explained by rounding errors inherent in the DP solution. There are one thousand proportions of sewer capacity in the solution and the extreme downstream sewer capacity is eight cumecs. Therefore, the resolution is to 0.008 cumecs since there are one thousand sewer proportions of 0.008 cumecs. This means that in certain circumstances the global control strategy will appear to spill approximately 0.01 cumecs more than the other control strategies. Obviously, the discrepancy would be less significant if a finer setting for the discretization was adopted.

Figure 9 clearly illustrates the deficiencies, in terms of pollutant load overspill reduction, of using fixed local control. In this case, the inflows at certain upstream interception points were less than the pre-set maximum flow rate and the interceptor storage was not fully utilised. The control procedure could not make allowances for this and, consequently, there were unnecessary overflows at the downstream interception points. This is shown in Figure 9 where the fixed local control flow rate line is below the pipe full capacity line at all times. The environmental efficiency of the control actions was improved using the variable local control strategy since it uses information about the interceptor system state. Therefore, this strategy would always fully utilise the capacity available locally within the interceptor system. This is shown in Figure 9 where the variable local control flow rate line is at the interceptor sewer capacity when possible. There is, however, an inherent danger of bias within the control decisions using this procedure. It may cause the downstream interceptor points to throttle back (i.e. closing of control gates) more readily than the upstream points. This is because the interceptor may already be at capacity because of decisions made upstream. Therefore, there may be operational or environmental difficulties with more frequent spills, or flooding, in the downstream sections of the sewer.

Figure 9 also illustrates the potential of the global optimal pollution control models. There are two particularly polluted inflows at intercept positions 5 and 6. The global control strategy did not intercept the flows at points 1 and 2 (relatively 'clean' flow) in order to have sufficient capacity to intercept these 'dirty' flows. The local control strategies did not make such allowances and as a consequence overflowed at intercept point 6. It was by pure chance that there was sufficient spare capacity within the system that the variable local control strategy did not spill at intercept point 5. In this time step the global control strategies had a sixty-nine percent improvement over the fixed local control strategy, and a forty-four percent improvement over the variable local control strategy. This improvement was measured in terms of pollutant load reduction to the river.

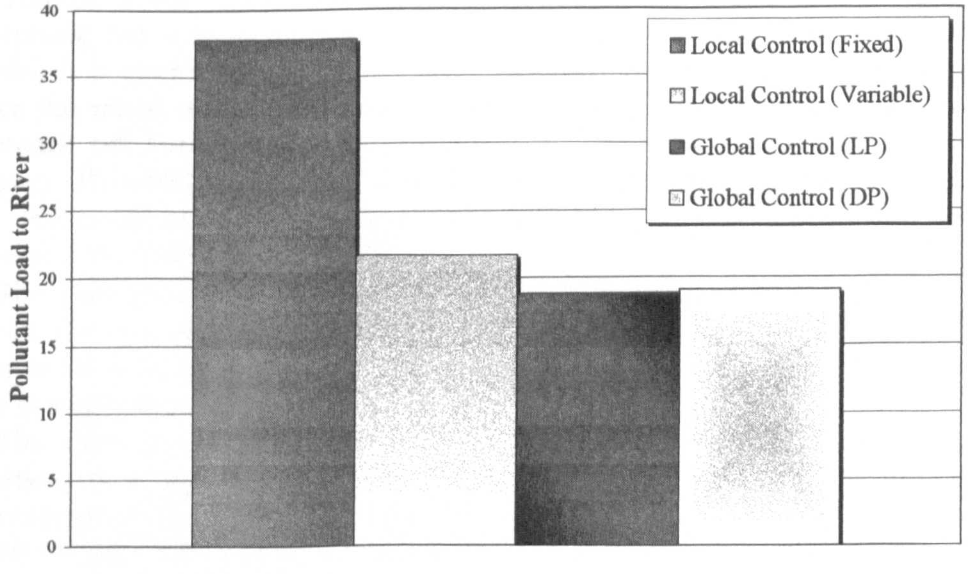


FIGURE 11 Overall comparison between control strategies.

Figure 10 shows comparisons of the solutions in each time step. They show that both the variable local control strategy and the global control strategies give significant improvements over the fixed local control strategy. The global control strategies have most improvement between time steps thirteen and twenty-six. At worst the global control strategy spills the same amount of pollutant load as the variable local control strategy.

Figure 11 shows an overall comparison of the four control strategies. The variable local control strategy provides a considerable improvement over the fixed local control strategy. Further improvements are achieved with the global control strategies. The LP model offers a marginal improvement over the DP model, however, this is explained by the discrepancies inherent in the DP model. The good performance of the variable local control strategy in this test case example is somewhat fortuitous since this strategy is volumetric and takes no account of pollutant load.

The results from this test case clearly illustrate the potential of optimal control models in interceptor sewer operation and the viability of both optimization techniques. The LP model runs considerably faster and yields exact solutions. However, the DP model may become more efficient as the complexity of the systems increase. In LP the computational time increases considerably as the number of variables increase. The computational time for DP solutions increases at a slower rate because more variables (intercept points) merely add stages to the method. At some point the relative computational demands for each model may reverse. Local control strategies that use information about the interceptor system state can significantly improve the performance of systems (in terms of pollutant load spills reduction) that use pre-set local control strategies. However, the results show that a further improvement can be achieved with the use of optimal control models using global information. Also, the global control models make more efficient use of the available storage within the interceptor system. There is an inherent danger of downstream interception points spilling, and possibly flooding, more frequently with variable local control strategies.

5. POST-PROCESSING HYDRAULIC VERIFICATION

The control model determines the intercept point time step positions within the interceptor sewer using the pipe full velocities. Generally, this is hydraulically acceptable but time is the critical parameter within the slug flow approach in the model. It is essential to be able to track the slugs accurately through the interceptor. Since the actual velocity of travel will deviate from the assumed pipe full velocity the approach will have a deficiency (and slugs of fluid will, of course, interact to some degree). Therefore, a post-processing hydraulic verification routine was introduced into the model to verify that the control strategies from the optimization algorithms provide a physically feasible solution.

The post-processing hydraulic verification routine determines the water profiles within the interceptor sewer in each time step solution. The procedure may be interpreted as 'snapshot' water profiles (such as that shown in Fig. 13(c)) in each time step throughout the control time horizon.

The water profiles are determined by a quasi-steady approach using the Manning equation. It is assumed that inflow q_{ij} will have reached one time step position downstream in the interceptor at the end of the time step. Therefore, the positions, based on pipe full velocity and size from the optimization module, of the slugs are known and the Manning equation is used to determine the hydraulic gradient required to transport the flow through the interceptor. Any hydraulic inconsistencies and positions of surcharging are then evident.

The procedure commences at the downstream boundary point and determines the water level at the previous time step position upstream so that there is sufficient hydraulic gradient for that flow rate. The routine then determines the water level for the upstream time step position before that, and so on. The critical depth is calculated at positions where pipe invert is discontinuous such as where the diameter of the sewer alters. If the water surface level from the Manning equation is lower than this depth, then the critical depth is used in the subsequent calculation of the water profile. Transitions in water profile can therefore be determined. The verification routine continues this procedure for all the control strategies throughout the control time horizon.

6. CASE STUDY – SIMPLIFIED NORTHERN LEG OF LIVERPOOL INTERCEPTOR SEWER

The northern leg of the Liverpool Interceptor Sewer has been simplified and used as a test case for the hydraulic verification routine. A longitudinal section of the sewer can be seen in Figure 12. Details ofthe interceptor sewer are given in Table I. The model was run with several runoff hydrographs and respective pollutant concentrations. The dry weather flows from each catchment were added to the rain hydrographs to obtain the total combined sewer flow. The runoff hydrographs consisted of three hypothetical storm events, of varying severity and loosely based on the catchment's response characteristics: a low storm event (~1.5-2 times fixed inflow setting), a medium storm event (~3-4 times fixed inflow setting), and a high storm event (~7-10 times fixed inflow setting).

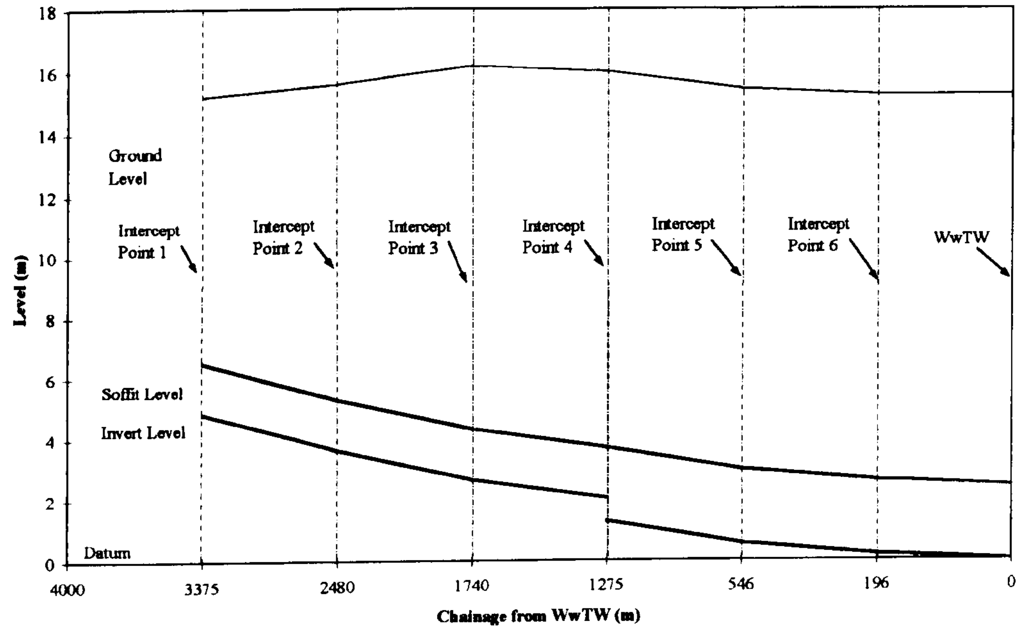


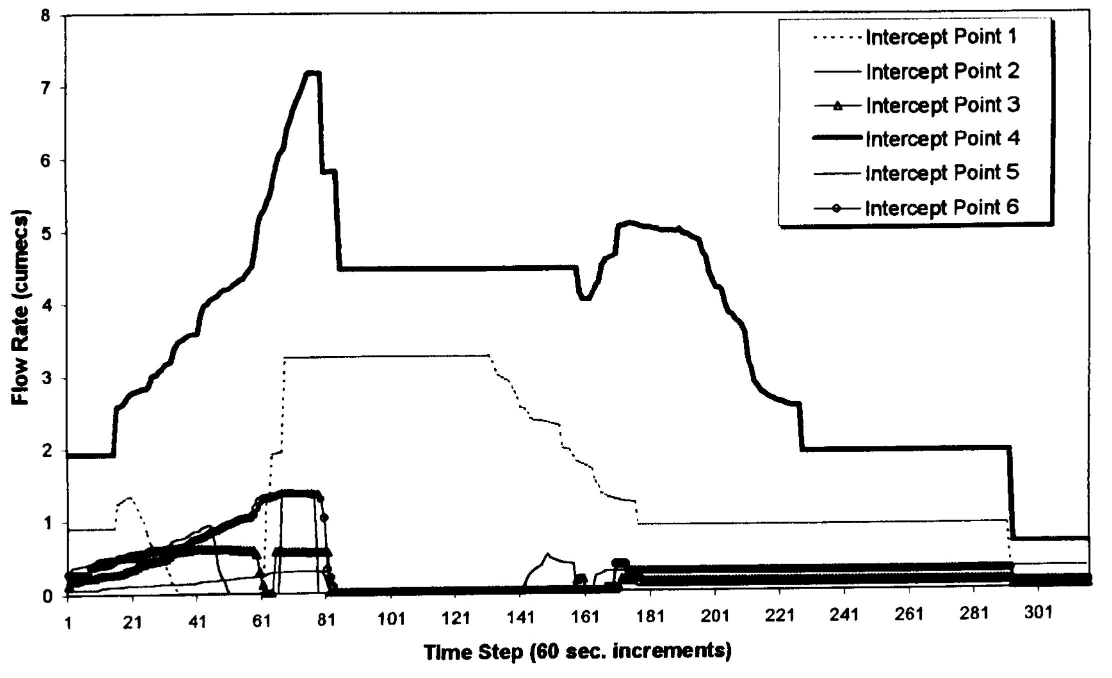
FIGURE 12 Longitudinal section of interceptor sewer.

TABLE I Input data for test interceptor sewer.

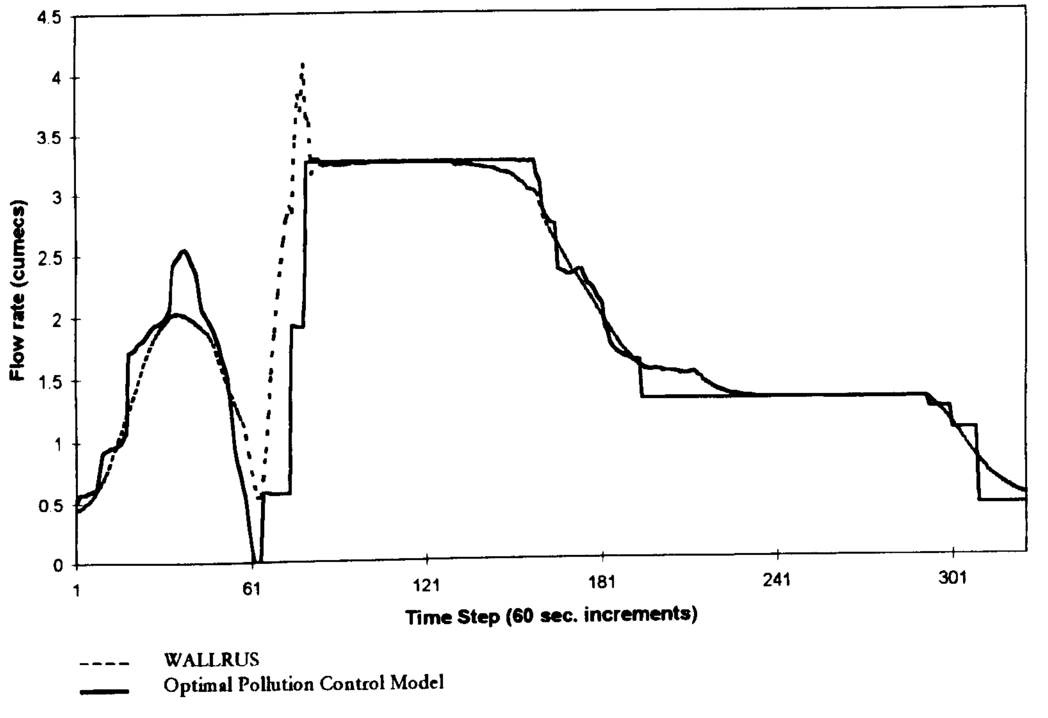
Intercept Point	Sewer Diameter (m)	Sewer Gradient	Sewer Capacity (cumecs)	D.W.F. (cumecs)	Fixed Inflow Setting (cumecs)
1	1.66	1/750	3.26	0.30	1.24
2	1.66	1/750	3.26	0.09	0.25
3	1.66	1/750	3.26	0.04	0.97
4	2.44	1/1000	7.72	0.64	2.82
5	2.44	1/1000	7.72	0.02	0.29
6	2.44	1/1000	7.72	0.09	0.31

Three control procedures were applied to the test case: fixed local control and variable local control (both of which take no account of the pollutant concentrations), and global pollution control (where the optimal control model uses global information including pollutant concentrations). The control strategies from the global pollution control model were determined using the Linear Programming routine. Figure 13 shows that the control strategy, the interceptor flow hydrograph, and the water profile in the interceptor for the medium intensity storm. Space precludes the display of any of the other cases.

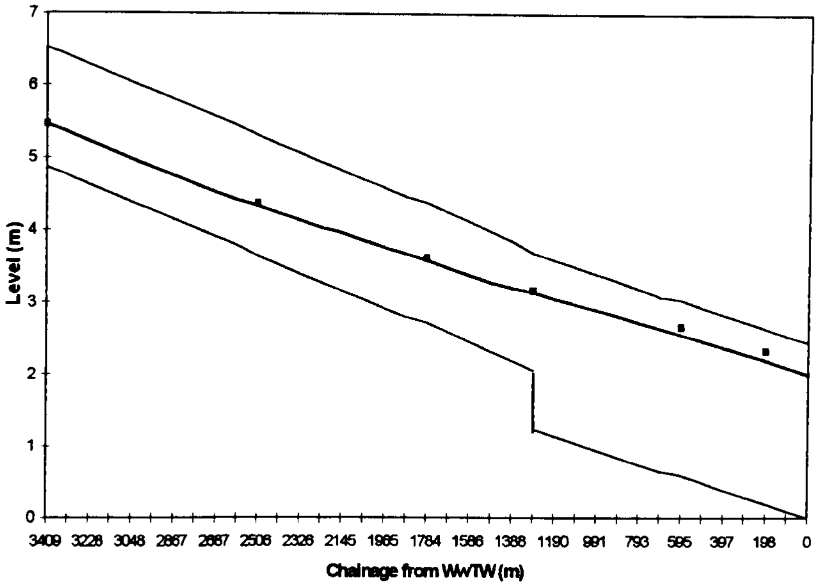
The control actions get increasingly severe as the storm severity increases, as would be expected since the inflow regulators (e.g. electro-mechanical penstock gates) have to restrict larger volumes of sewage to mitigate surcharge of the interceptor. A comparison between the control procedures can be seen in Table II and the results show that the variable local control procedure significantly reduce the pollutant load discharge to the receiving waters compared to the fixed local control, but the global pollution control model offer further enhancement. The results show that the environmental improvements decrease, compared to the fixed local control, as the severity of the storm increases. This is expected because spills are inevitable with larger inflows where the interceptor sewer storage is largely utilised. It must be stressed, therefore, that these results are likely to be a poor illustration of the potential of the optimal pollution control because the inflow hydrographs and pollutant concentrations were highly synchronised. It is likely that most improvements would be encountered when the hydrographs and corresponding pollutant concentrations varied spatially and temporally and that overall performance will be heavily weighted to the more moderate rainfalls that occur more frequently.



(a) Controls for the medium intensity storm.



(b) Interceptor flow hydrograph for the medium intensity storm.



(c) Interceptor water profile for the medium intensity storm (time step 190).

FIGURE 13 Case study – Liverpool interceptor sewer, northern leg. (a) Controls for the medium intensity storm. (b) Interceptor flow hydrograph for the medium intensity storm. (c) Interceptor water profile for the medium intensity storm (time step 190).

TABLE II Comparison of control procedures.

Condition	Pollutant Load to Receiving Waters (Spill Volume × Pollutant Concentration)			Improvement (GPC v FLC) %	Improvement (GPC v VLC) %
	Fixed Local Control (FLC)	Variable Local Control (VLC)	Global Pollution Control (GPC)		
Low Storm (~1.5-2 × Fixed Inflow Setting)	192.44	34.47	34.34	82.16	0.38
Medium Storm (~3-4 × Fixed Inflow Setting)	639.59	433.46	399.35	37.56	7.87
High Storm (~7-10 × Fixed Inflow Setting)	2146.86	1897.32	1823.37	15.07	3.40

The control strategies from the optimal pollution control model were verified in the post-processing hydraulic verification routine and these, in turn, were validated using WALLRUS [5]. Figure 13 shows sample results from both the hydraulic verification routine and WALLRUS (including the backwater effects).

The interceptor sewer hydrographs in Figure 13(b) from the hydraulic verification routine show the optimal control model's idealization of the interceptor sewer flow dynamics (i.e. all slugs of flow travelling at pipe full velocity). The results from WALLRUS have been included for comparison. This application of WALLRUS has shown some instability in the results particularly when rapid changes in inflow are imposed when the pipes are running close to full. This is clearly seen for the medium around time step 90. This instability probably occurs because of the numerical solution procedure in the WALLRUS code. However, the significance of the instability is considered negligible because the flow volumes within the instability are minimal, the solution recovers and the overall results compare well.

The hydrograph shown in Figure 13(b) shows remarkable correspondence between those generated by the relatively crude optimal pollution control model and those generated by WALLRUS. The discrepancies relate to differences in flow volumes, and those given by the optimal control model are generally conservative. The hydrographs provide strong evidence of the validity of the simple slug flow approach used in the control model.

The water profile in Figure 13(c), determined by the post-processing hydraulic routine, shows that the control strategies generated from the optimization were conservative and implies that there was additional volume available. Profiles for the other storms display similar characteristics and justify the idealization of the interceptor sewer system state used in the optimum control model, i.e. all slugs of flow travel through the interceptor sewer at the pipe full velocity irrespective of the water depth and surface slope. The inaccuracies in this idealization appear to be generally self-cancelling and conservative. The optimum control model permits all flows from zero to pipe full capacity at any point within the interceptor sewer. Therefore, the control strategies generated should not contain surcharging. The water profiles confirm these assumptions because the profiles do not even reach the interceptor sewer soffit.

7. CONCLUSIONS

A robust and computationally efficient method has been described for the pollutant load overspill minimization of interceptor sewer systems. The hydraulics of interceptor sewers have been idealized into a slug flow approach that enables the maximization of pollutant load retention within the sewer using Linear Programming or Dynamic Programming model alternatives. Several assumptions were imposed in the model to permit a computationally efficient solution for the control actions.

The results from the idealized test case interceptor sewer showed the viability of using Linear Programming and Dynamic Programming within the optimal pollution control models. Additionally, the results illustrated that significant environmental improvements (in terms of pollutant load reduction to the receiving waters) could be achieved with the use of the optimal control models, compared to fixed local control procedures.

The optimal control models have been verified using a post-processing hydraulic verification routine and have been validated against WALLRUS. On application to a simplified version of the northern leg of the Liverpool Interceptor Sewer with several storm events of varying severity, the optimal control model gives physically feasible solutions. These results also show that the optimal control model offers significant improvements over fixed local control procedures. The interceptor sewer hydrographs and water profiles illustrate the validity of the assumptions in the optimal control model formulation. Moreover, the results have illustrated that the slug flow approach is a computationally efficient and sound formulation for the optimal control model and offers promise for practical implementation of optimal real time control.

Further research will include the extension of the control models to include secondary storage structures such as overflow chambers. As a final step towards practical implementation, efficient methodologies will later be used for the simulation of inflows from rainfall [7] and for specification of time-varying pollutant concentrations [4].

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