A METHODOLOGY FOR THE EFFECTIVE EVALUATION OF THE PERFORMANCE OF WASTEWATER SYSTEMS

by

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This research programme was carried out in collaboration with the North of Scotland Water Authority

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I certify that this thesis is the true and accurate version of the thesis approved by the examiners

Signed (Director of Studies)

Date 22nd September 1997
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Many people played a part in the production of this research and I hope I have not neglected to offer my thanks to any of them in the following text.

Thanks must go to the collaborating institution, North of Scotland Water Authority, for part funding in the initial three years of the project and for employing me afterwards. On the note of funding, appreciation is gratefully offered to EPSRC (formerly SERC).

Many thanks are extended to my Director of Studies, Professor Richard Ashley who continually encouraged me and made me see my research in a different light. My supervisors Dave Blackwood, Chris Jefferies and Bruce Knowles also require appreciation for advice and guidance throughout the duration of the study. Thanks are extended to Norman Semple who encouraged me to think differently from the mould of common practice.

Vocational students came and went during the three year fieldwork collection, many are now firm friends and colleagues, and without their help and support of the divisional sewer squads, the practical aspects of the research would have failed. Thanks are offered to David Wotherspoon, Gordon Stenhouse and Andrew Jack for help on certain modelling issues, and to my dad for proof reading the thesis.

Last but not least, I wish to thank my wife Anne, who for the latter years of the study saw little of me (perhaps she wishes to thank me!) during writing of the thesis and who now has to put up with me full time.

I will leave you with a little anecdote regarding my feeling on field data collection, in particular that of storms, which some researchers have raised before. It's the fourth of July 1994, tension is high; Italy versus the Republic of Ireland in World Cup USA. Ray Houghton pounces on a loose ball at the edge of the eighteen yard box, he shoots, a perfectly weighted snapshot that captures the Italian keeper off his line. Jubilation ! A goal.
My extreme happiness at this event can only be matched by my dejection at the event occurring outside in the street, the heaviest rain during a dry summer explodes, the one I have been waiting for, streets are awash, sewers turn into foaming torrents, sediments flush (maybe) and overflows vomit forth the combined wastes of the citizens of Perth, but alas the samplers are quiet in Bridgend.

Need I say more.
ABSTRACT

An integrated methodology (WISPS) has been developed and presented which addresses areas of concern associated with the performance and rehabilitation of wastewater catchments. The methodology seeks to achieve effective solutions and to ease the decision making process when faced with multi-criteria problems. The methodology has been partly applied to the study catchment and others as means of testing the product of the research work. Through the application new information is presented in the form of value functions associated with the areas of concern under evaluation.

Holistic computer modelling of drainage catchments may seem theoretically viable but the work carried out has highlighted that there are severe limitations associated with sewer flow quality models and their ability to represent the behaviour of pollutants within sewerage systems. While UPM appears to be the pinnacle of 10 years of research, a vast understanding still needs to be sought by scientists and engineers in relation to the modelling of wastewater quality. A main conclusion is drawn that these models can only ever be calibrated, but never truly verified and therefore are of little practical use to engineers attempting to solve wastewater problems within time and budgetary constraints.

Information is presented on the way in which the public perceive water quality. This is shown to be based on the presence of land derived refuse within the watercourse corridor. Two of the urban watercourses studied, while being of low amenity value to those interviewed, were biochemically sound and were only judged unsatisfactory based on bankside refuse.

An approach is presented based on historical rainfall and regression analysis to determine the level of service afforded by sewerage systems in terms of flooding. It is recommended that this approach be utilised to identify the true flooding performance of catchments as opposed to the common practice of design storms.
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<td>Aesthetic Pollution</td>
<td>Visible and gross solids originating from wastewater systems.</td>
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<td>Area of Concern</td>
<td>Performance of a particular criteria associated with a wastewater system.</td>
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<tr>
<td>Combined Sewer Overflow</td>
<td>A hydraulic structure for relieving sewerage systems of excess flows.</td>
</tr>
<tr>
<td>Continuous Discharge</td>
<td>Term associated with wastewater treatment plant discharges to receiving watercourses.</td>
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<td>CSO Criticality Score</td>
<td>Scoring system used by the Author in assessing the aesthetic performance of CSOs within the Perth catchment.</td>
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<td>Design Storm</td>
<td>Statistical rainfall traditionally utilised by engineers for assessing the hydraulic performance of sewerage systems.</td>
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<tr>
<td>Direct Damage</td>
<td>Primary damage associated with a flooding or pollution event.</td>
</tr>
<tr>
<td>Dry Weather Flow</td>
<td>Flows in a sewerage system associated with domestic, industrial and infiltration flows.</td>
</tr>
<tr>
<td>Economic Evaluation of Environmental Goods</td>
<td>A term used for describing a number of techniques for assessing the benefits of environmental improvements.</td>
</tr>
<tr>
<td>Epic Sampler</td>
<td>A small bore sampler utilised in sampling wastewater flows.</td>
</tr>
<tr>
<td>Flow Survey Monitor</td>
<td>A device utilised to measure depths and velocities of sewer flows.</td>
</tr>
<tr>
<td>Formula “A”</td>
<td>Pass forward flowrate associated with the first spill of a CSO structure.</td>
</tr>
<tr>
<td>Historic Rainfall</td>
<td>Past rainfall used in the assessment of wastewater system performance.</td>
</tr>
<tr>
<td>Holistic Approach</td>
<td>An approach encompassing technical, economic, social and environmental aspects of wastewater system performance.</td>
</tr>
<tr>
<td>In Sewer Sediment</td>
<td>Inorganic/organic material deposited on the invert of sewers.</td>
</tr>
<tr>
<td>Indirect Damage</td>
<td>Damage associated with the secondary effects of a flooding or pollution event.</td>
</tr>
<tr>
<td>Intangible Damage</td>
<td>Damage which is difficult to evaluate in monetary terms.</td>
</tr>
<tr>
<td>Interested Parties</td>
<td>Those organisations or bodies having a vested interest in a wastewater system problem.</td>
</tr>
<tr>
<td>Intermittent Discharge</td>
<td>Discharge associated with SWO or CSO performance.</td>
</tr>
<tr>
<td>Level of Performance</td>
<td>Flooding performance of a sewerage system associated with design rainfall.</td>
</tr>
<tr>
<td>Level of Service</td>
<td>Actual flooding performance of sewerage system.</td>
</tr>
<tr>
<td>Major Area of Concern</td>
<td>A general criteria made up of areas of concern.</td>
</tr>
<tr>
<td>Non Use Values</td>
<td>Values associated with not using a particular environmental good.</td>
</tr>
<tr>
<td>Potency Factor</td>
<td>A factor used in wastewater quality modelling to determine the quantity of a particular pollution parameter that is associated with TSS.</td>
</tr>
<tr>
<td>Rain Gauge</td>
<td>A device used to record rainfall through a tipping bucket mechanism.</td>
</tr>
<tr>
<td>Surface Sediment</td>
<td>Sediment found on roof, permeable and road surface</td>
</tr>
<tr>
<td>Tangible Damage</td>
<td>Damage which can be evaluated in monetary terms.</td>
</tr>
<tr>
<td>Urban Pollution Management</td>
<td>A philosophy for controlling intermittent and continuous wet weather discharges to receiving watercourse.</td>
</tr>
<tr>
<td>Use Values</td>
<td>Values associated with using a particular good for benefit boating and fishing are examples.</td>
</tr>
<tr>
<td>Value Function</td>
<td>A graph showing the relationship between an arbitrary scoring system and physical data.</td>
</tr>
<tr>
<td>Weighting</td>
<td>A measure of importance associated with an area of concern.</td>
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<tr>
<td>Abbreviation</td>
<td>Definition</td>
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<td>--------------</td>
<td>---------------------------------------------------------------------------</td>
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<tr>
<td>ADWF</td>
<td>Average Dry Weather Flow</td>
</tr>
<tr>
<td>AMP</td>
<td>Asset Management Planning</td>
</tr>
<tr>
<td>AOC</td>
<td>Area Of Concern</td>
</tr>
<tr>
<td>BATNEEC</td>
<td>Best Available Technology Not Entailing Excessive Cost</td>
</tr>
<tr>
<td>BCA</td>
<td>Benefit Cost Analysis</td>
</tr>
<tr>
<td>BCR</td>
<td>Benefit Cost Ratio</td>
</tr>
<tr>
<td>BMP</td>
<td>Best Management Practices</td>
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<tr>
<td>BOD</td>
<td>Biochemical Oxygen Demand</td>
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<tr>
<td>BODFIL</td>
<td>Biochemical Oxygen Demand Filtered</td>
</tr>
<tr>
<td>CARP</td>
<td>Comparative Acceptable River Pollution</td>
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<tr>
<td>CEA</td>
<td>Cost Effective Analysis</td>
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<tr>
<td>COD</td>
<td>Chemical Oxygen Demand</td>
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<tr>
<td>CSO</td>
<td>Combined Sewer Overflow</td>
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<tr>
<td>CVM</td>
<td>Contingent Valuation Methodology</td>
</tr>
<tr>
<td>DO</td>
<td>Dissolved Oxygen</td>
</tr>
<tr>
<td>DWF</td>
<td>Dry Weather Flow</td>
</tr>
<tr>
<td>EES</td>
<td>Environmental Evaluation System</td>
</tr>
<tr>
<td>EMV</td>
<td>Expected Monetary Value</td>
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<tr>
<td>EOL</td>
<td>Expected Opportunity Loss</td>
</tr>
<tr>
<td>EQO</td>
<td>Environmental Quality Objective</td>
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<tr>
<td>EQS</td>
<td>Environmental Quality Standard</td>
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<tr>
<td>EU</td>
<td>European Union</td>
</tr>
<tr>
<td>FWR</td>
<td>Foundation for Water Research</td>
</tr>
<tr>
<td>MAC</td>
<td>Maximum Acceptable Concentration</td>
</tr>
<tr>
<td>MAOC</td>
<td>Major Area Of Concern</td>
</tr>
<tr>
<td>MLSS</td>
<td>Mixed Liquor Suspended Solids</td>
</tr>
<tr>
<td>MOSQITO</td>
<td>Modelling Of Sewer Quality Including Tanks And Overflows</td>
</tr>
<tr>
<td>NOSWA</td>
<td>North Of Scotland Water Authority</td>
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<td>NVSS</td>
<td>Non Volatile Suspended Solids</td>
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<tr>
<td>PCA</td>
<td>Performance Cost Analysis</td>
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<td>PFI</td>
<td>Private Finance Initiative</td>
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<tr>
<td>RAS</td>
<td>Return Activated Sludge</td>
</tr>
<tr>
<td>RBM</td>
<td>River Basin Management</td>
</tr>
<tr>
<td>SEPA</td>
<td>Scottish Environmental Protection Agency</td>
</tr>
<tr>
<td>SRM</td>
<td>Sewer Rehabilitation Manual</td>
</tr>
<tr>
<td>SRP</td>
<td>Soluble Reactive Phosphorus</td>
</tr>
<tr>
<td>STOAT</td>
<td>Sewage Treatment Operational Analysis Over Time</td>
</tr>
<tr>
<td>SWO</td>
<td>Surface Water Outfall</td>
</tr>
<tr>
<td>SWQO</td>
<td>Statutory Water Quality Objective</td>
</tr>
<tr>
<td>TON</td>
<td>Total Oxidisable Nitrogen</td>
</tr>
<tr>
<td>TSS</td>
<td>Total Suspended Solids</td>
</tr>
<tr>
<td>UC</td>
<td>Use Class</td>
</tr>
<tr>
<td>UES</td>
<td>Uniform Emission Standard</td>
</tr>
<tr>
<td>UPM</td>
<td>Urban Pollution Management</td>
</tr>
<tr>
<td>UPMAM</td>
<td>Urban Pollution Management Applications Methodology</td>
</tr>
<tr>
<td>UWWTX</td>
<td>Urban Wastewater Treatment Directive</td>
</tr>
<tr>
<td>VSS</td>
<td>Volatile Suspended Solids</td>
</tr>
<tr>
<td>WAS</td>
<td>Waste Activated Sludge</td>
</tr>
<tr>
<td>WSOPS</td>
<td>Wastewater Integrated System Performance Score</td>
</tr>
<tr>
<td>WQI</td>
<td>Water Quality Index</td>
</tr>
<tr>
<td>WWTP</td>
<td>Waste Water Treatment Plant</td>
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</table>
CHAPTER 1 INTRODUCTION

1.0 URBAN POLLUTION MANAGEMENT (UPM)

1994 saw the publication in the UK of the most important documentation to influence national wastewater system management in recent times. The UPM (FWR 1994a) manual was a culmination of a decade of research and development in the wastewater field. From the auspices of the Sewerage Rehabilitation Manual (SRM) and the River Basin Management (RBM) programme a new protocol has emerged which lays down the philosophy for controlling wet weather discharges to receiving watercourses from urban wastewater systems.

The UPM manual provides advice and instruction to engineers and water quality scientists alike. It is intended to provide a forum for those responsible for the management of wastewater systems and receiving watercourses. The key aim behind the manual is to protect watercourses from acute and chronic pollution from intermittent and continuous discharges. These discharges are complex because of vagaries in the weather, the transient quality of foul and storm discharges, and the physical, biological and chemical processes which control quality within treatment plants and wastewater systems.

UPM is firmly founded on computer modelling techniques and methodologies. Work carried out under UPM and previous endeavours have developed and made available packages which represent rainfall (STORMPAC), sewer hydraulics (WALLRUS/HYDROWORKST™), sewer flow quality (MOSQUITO/QM), receiving watercourses (MIKE 11) and wastewater treatment plant performance (STOAT). By modelling systems in this integrated manner, it is claimed that a better understanding of the problem to be solved is achieved.

The integrated use of these tools in an holistic approach provides the engineer with a medium through which regulatory standards with respect to aquatic life and amenity can be satisfied, while ensuring cost effectiveness of rehabilitation options.
UPM specifies intermittent water quality standards relating to un-ionised ammonia and dissolved oxygen to ensure rehabilitation works protect the aquatic environment. Aesthetic pollution of watercourses is addressed by specifying screening requirements for unsatisfactory Combined Sewer Overflows (CSOs) based on the amenity of the receiving water course. UPM has provided improved engineering designs relating to CSO structures and detention tanks.

Different methodologies and approaches within the UPM procedure are included, which allow the engineer to adopt the most effective line of investigation, to suit the complexity of any wastewater problem. Levels of complexity vary from simple design rules, such as Formula A, through to complex integrated modelling of all technical aspects of the catchment. The UPM procedure claims that applying complex modelling achieves more cost effective solutions.

Thus, capital funds can be utilised in producing cost effective solutions to wastewater system problems. This is important to ensure that funds are invested in the most secure manner, as Water Authorities and Companies are now, more than ever, questioned regarding investment by shareholders and the general public. UPM now potentially provides a tool to confidently deliver cost effectiveness and accurate compliance with regulatory standards.

While the areas of hydraulic modelling are recognised and applied throughout the nation, the realm of sewer flow quality and wastewater treatment plant modelling, have been little applied outwith the UPM programme. It is generally recognised that the use of these complex tools may end up in the hands of specialists, and consequently are difficult to apply by an engineer faced with time and budgetary constraints. The accuracy of these models under dry weather and storm conditions still requires to be determined fully.

UPM, while improving knowledge in many areas does not provide a means of determining priority upgrading of catchments in terms of holistic performance. Asset Management Planning (AMP) procedures (soon to be developed in Scotland), which are operable in England and Wales, define the level of service provided to the customer and identify deficient performance. However, no system
or methodology is apparent which compares all aspects of holistic wastewater system performance for one catchment against another.

Work has been carried out by the National Rivers Authority (NRA), Foundation for Water Research (FWR) and Office of Water Services (OFWAT) utilising Cost Benefit Analysis as a medium for prioritising upgrading work relating specifically to river water quality improvements. This work (FWR 1996) has recently been published and is intended to be adopted by English and Welsh Water Authorities to identify the most cost beneficial projects to be undertaken. It is recognised that the benefits associated with river quality improvements are difficult to evaluate in monetary equivalents and that non-use values do exist in relation to environmental parameters.

A methodology is clearly required which allows catchments to be effectively prioritised on holistic performance, in as many key areas of concern as possible. It is important that tangible and intangible aspects of performance are appraised, and judgements made relating to design criteria being exceeded based on a standard framework. The intangible aspects of sewer flooding and water pollution abatement are areas which must be evaluated and investigated if effective solutions are to be generated. Any methodology must make clear to the user what the key issues or problems are.

Many criteria such as economic, environmental and social issues can influence decision making processes, and consequently can control the selection of rehabilitation strategies. The major difficulty in dealing with these factors is that they incorporate value judgements and require a consideration of all the processes and activities which influence the dynamics of a wastewater system.

An approach incorporating such value judgements is essential if the most effective rehabilitation solution to a wastewater system problem is to be produced. The approach must be simple, transparent and easy to utilise. The approach must also compliment the existing UPM procedure with respect to modelling procedures and standards.
Following rehabilitation it is important that the performance of the catchment is continually monitored, to ensure that the rehabilitation option implemented is effectively solving the defined problem. This will ensure that capital expenditure has been invested wisely. Continual monitoring of the catchment will also define any ambiguities associated with rehabilitated performance.

It is very important for any developed methodology to complement the existing UPM procedure. Also, as few UPM studies have been carried out to date, it is fundamental that the UPM approach and the technical models embodied within it are subject to independent scrutiny and appraisal.

1.1 RESEARCH OBJECTIVE AND AIMS

It was important to carry out the research in relation to an operating wastewater system. The study catchment chosen was that of Perth in Tayside. The system of Perth was selected primarily due to the suitability of the system for modelling.

Perth has the largest treatment plant within Tayside and serves a population of around 42 000. CSOs are present, as are receiving watercourses of various sizes. The Water Services Department, of the then Tayside Regional Council required a hydraulic model of the sewerage system, and agreed to the development of a catchment plan based on the UPM procedure and to the initiation of the research project.

The main objective of the research was to develop a methodology usable by engineers which would aid in the prioritisation of catchments based on performance, selection of rehabilitation strategies, and monitoring of performance of existing/rehabilitated catchments.

This objective had the following aims:

1. Develop an integrated suite of computer analysis models of the drainage catchment of Perth in Scotland.
2. Critically examine the applicability of holistic modelling tools particularly those concerned with wastewater quality.
3. Examine methodologies for the evaluation of performance associated with wastewater systems.
4. Identify and appraise key criteria associated with drainage catchment performance.
5. Appraise public perception of water quality issues in Perth.
6. Develop and apply a methodology for the effective evaluation of wastewater system performance to the study catchment of Perth.
7. Utilise modelling tools to investigate and assess any inadequacies within the Perth drainage catchment.
8. Effectively provide solutions to inadequacies, if required, through the application of the methodology and UPM modelling tools.

1.2 OUTLINE OF THESIS

Chapter 2 discusses mankind’s approach to wastewater management and treatment throughout history, with particular reference to the City of London. Wastewater characteristics and wastewater systems are discussed, as is wastewater treatment. A brief summary of problems associated with wastewater systems is presented, which is further expanded on in later Chapters. Chapter 3 outlines the development of UPM from the early roots of the Sewerage Rehabilitation Manual through to the present day. A resume of the latest technology and methodologies is presented.

Chapter 4 argues the need for present technology to be complemented by techniques which encompass intangible, as well as tangible criteria when seeking effective rehabilitation strategies. A review is given of existing techniques and a method known as multi attribute utility analysis is chosen as the basis for the Perth methodology.

Chapter 5 presents the developed methodology entitled WISPS (Wastewater Integrated System Performance Score) and demonstrates how the methodology can be applied to derive a performance score for Perth and other catchments based on historical performance information.
Chapters 6 and 7 describe the development of computer models of the Perth drainage catchment. The fieldwork, calibration and verification of the models utilised is detailed. An appraisal of the performance and applicability of quality models is presented, as are recommendations for improvement. Chapter 8 shows how the WISPS methodology is integrated with the developed computer models in a detailed analysis of the performance of the Perth wastewater system.

Chapter 9 gives conclusions and recommendations as a result of the study. Appendices are presented which contain additional information relating to the study and details of papers written by the author during the study period.
CHAPTER 2 WASTEWATER AND WASTEWATER SYSTEMS

2.0 HISTORICAL ASPECTS

In the early 18th century all large cities in the world were united by one common bond: their stink. Human waste, offal and animal by-products lay in the streets untreated and degrading, resulting in a putrefied ooze that was the catalyst for heinous stenches. Man has always had a requirement for disposing of waste products away from his immediate living habitat. In many cities this simply involved tipping waste over the balcony or throwing excrement into the street where the problem became someone else's.

Although the ancient City of Rome employed a type of sewerage network to dispose of waste away from living quarters, the final “treatment plant” was the River Tiber which became filthier and filthier as waste degraded within it. This is an example of the “out of sight, out of mind” philosophy which has predominated for centuries and in some countries still controls wastewater policy.

Although man understood the need to transport wastewater away from his immediate living area, no thought was given to the final resting place of the wastewater. In medieval times, most waste was deposited in the streets and left to rot where it lay, until rain carried the waste away. Alternatively, watercourses were used for the transport of wastewater and consequently became no more than open sewers. Cesspools and middens were common disposal methods which were breeding grounds for life threatening disease.

Man, at this time, did not understand the relationship between improper disposal of waste and the risk of disease, due to the numerous pathogenic bacteria which caused epidemic diseases such as typhoid, cholera and dysentery. A classic example of the ignorance and attitude towards wastewater disposal can be displayed by examining the City of London.
2.1 THE PIECMEAL APPROACH TO POLLUTION MANAGEMENT

In 1594 Sir John Harrington invented a self emptying slop pan for Queen Elizabeth I. This invention was the forerunner of the modern WC, but did not meet with Her Majesty's approval and was not manufactured at a public level. Harrington like Da Vinci was a few hundred years ahead of his time.

Concurrently the streets of London and other large cities wallowed in their own waste products. In 1775 Alexander Cumming capitalised on Harrington's work and developed the WC further. With a pull of a handle and a flush, waste was effectively disposed of from the household. However, the consequences of the extra water flows required to operate this invention were not allowed for. The wastewater from thousands of WCs poured down into already overloaded cesspools, which ultimately burst into streets and brooks. Overnight streams which supplied drinking water were turned into no more than open sewers. In 1797 these streams in London were covered over as a solution to the disgusting mess transported in the watercourses. This approach treated the symptom rather than the disease. The "disease" reappeared with shocking consequences.

As no treatment was afforded to the waste discharged to the culverted watercourses, degradation was rapid, as was the build up of toxic and inflammable gases. In 1846 the River Fleet erupted from the ground in a massive explosion. In the process a steamboat was crushed against Blackfriars Bridge and decades of degraded by-products from London's dyeworks, tanneries, abattoirs and population spewed into the open air.

Massive outcry from the public, particularly the poor, who stated "they were living like pigs" followed this event; as did Cholera. Over 150 children out of every 1000 died from cholera due to a lack of proper sanitation. Due to early disposal methods waterborne disease was common, as a result of consumption of drinking water infected with pathogens. Germ theory and contagious theory were still not fully understood at this time.
A landmark in demonstrating contagious theory is demonstrated in the example of the Broad Street Pump. Dr John Snow (1813-1858) investigated the deaths of 500 people of Cholera in August 1854. When the handle of the local water pump was removed in Broad Street the outbreaks of Cholera in the area stopped. Further investigations showed that nappies of babies plagued by cholera were being washed, with the wash water draining to a nearby cesspool. The cesspool was found to be leaking three feet away from the Broad Street pump supply. Man now began to understand the need for proper sanitation.

2.1.1 Treating The Symptom Rather Than The Disease

1858 was a very hot year, with the result that the River Thames along its length stank due to the putrefied matter that had become deposited therein. The curtains of the House of Commons were anointed with Chloride of Lime to attempt to curb the stench arising from the nearby river. Tonnes of Carbolic Acid and Chalk Lime were tipped into the river to combat the smell; however these efforts were futile.

Consequently, plans to clean up London, driven by the Houses of Parliament were invited. One hundred and forty schemes were examined and the plan adopted was that of Bazalgette. Bazalgette proposed and constructed, a sewerage system of large egg-shaped tunnels, to transport foul and stormwater. Three vast tunnels were constructed on the north bank of the Thames and two on the south. These tunnels discharged raw sewage to the river.

The removal of wastewater and storm drainage through sewerage systems did not alleviate all health problems, as the source of disposal was often the nearest watercourse. This method of disposal created health problems for downstream users of watercourses receiving the untreated discharges. One example, is the outbreak of typhoid in the City of Aberdeen in 1964 which forced the Town Council to investigate methods of further improving the sewage disposal arrangements for the City (Donald and Wishart, 1987). The actions of the Town Council clearly show a wish to achieve cleaner living standards without the risk of life threatening disease.
Even though man clearly understood the link between improper sanitation and disease, he still did not give any thought to the consequences of where, or in what manner, his wastewater was disposed. The consequences of this outlook were demonstrated in London in the year 1878.

The paddle steamer *Princess Alice* collided with a coal barge near Berking and 623 perished in the Thames; not from drowning but from poisoning. The river was so filthy, putrid and toxic that the board of inquiry held after the incident declared that wastewater receive some form of treatment, before discharging to the receiving watercourse. This led to the first rudimentary wastewater treatment plants (WWTP) being constructed. These settled sewage and discharged straight to the watercourse. The settled sludge was transported by barges to be dumped at sea. By 1887 only liquid effluent was discharged to the Thames.

The treatment afforded to the wastewater of London remained the same, while the population increased, as it was no longer checked by disease. This led to larger and larger pollutant loads being discharged to the river. Bacteria present in the wastewater deprived the river of oxygen, until in 1947 the River Thames effectively died, with 80% of its flow attributable to untreated and treated sewage.

Again the solution to clean up the Thames was to treat the symptom and not the disease. More treatment was gradually given to the wastewater and by 1974 salmon again returned to the river after an absence of nearly 30 years.

### 2.1.2 A Lack Of Forethought? - The Aesthetic Nightmare

With the advent of new processes and technology, pollution from sewerage systems was not just a result of the biochemical degradation of the watercourse. Aesthetic pollution resulted from the development of sanitary products. Hard, shiny lavatory paper was developed in the early 1900s, soft paper in 1916, 1920 saw the sanitary towel manufactured, the tampon and condom followed in 1930, coloured toilet paper was manufactured in the 1950s and the panty liner and disposable nappy were developed in the 1960s.
These products, not being biodegradable, littered watercourses and beaches and were a common site on the Thames, where used condoms were classified as "Thames Goldfish". In fact, aesthetic pollution is still prevalent in many watercourses and beachfronts in the 1990s, almost 90 years on from the development of toilet paper.

Man has always recognised the need to dispose of his own waste products. The idea of handling your own excrement, is more abhorrent to today's society, than it was in Roman or Greek times. The disposal methods throughout the passage of time have varied between simply throwing waste from a window into a street, to cesspools, directly into rivers untreated and finally to disposing of treated effluent to watercourses.

Man has never approached the problems of wastewater from a source control or an educational viewpoint. Instead of controlling or regulating his use of water and the nature of the sanitary products used, he has treated the symptoms of pollution instead of addressing the source. The early engineers treated one problem at a time with no thought to the consequences of their actions and strategies. Indeed people had to die from diseases caused by filth, squalor and no sanitation before the powers at be acted.

To summarise, the problems of pollution, biochemical and aesthetic, are the result of mankind's inability and short sightedness in his approach to wastewater management. For too long the sewerage system has been seen as a separate entity from the wastewater treatment plant and the receiving watercourse. Today's engineers are faced with rehabilitating sewerage systems, and ensuring pollution and flooding are controlled, while safely transporting storm and foul sewage for disposal and treatment. This is carried out to ensure the quality of river and coastal waters meet with regulatory requirements. These targets must be achieved through an integrated, holistic and sustainable approach to wastewater management and rehabilitation if the folly of our forefathers is not to be repeated.
2.2 CHARACTERISTICS OF WASTEWATER

Wastewater is the by-product of man’s usage of water. Wastewater is generated from domestic activities and industrial processes. When storm runoff joins with human and industrial flows the wastewater is deemed to be combined. Wastewater within an urban drainage system contains many pollutants. The main characteristics associated with them are listed below (Andoh, 1994):

- Immediate oxygen demanding substances (e.g., soluble BOD)
- Delayed oxygen demanding substances (e.g., particulate BOD)
- Sediments and associated pollutants (e.g., heavy metals and priority pollutants)
- Refractory and persistent pollutants (e.g., pesticides)
- Obnoxious chemicals and substances resulting from industrial activity
- Pathogenic organisms (e.g., viruses and bacteria)
- Nutrients (e.g., nitrates and phosphates)
- Pollution from groundwater sources (e.g., sulphates and tidal infiltration)
- Aesthetically offensive inert materials (e.g., condoms and plastic strips)

2.2.1 Sources Of Pollutants

The sources of pollutants within a combined drainage system can be identified as arising from three main areas. These are listed below.

**Stormwater Inputs** as a result of runoff from roads, roofs and permeable areas

**Domestic and Industrial wastewater** linked closely to human activities

**Materials deposited within sewers** sewer sediments

Pollutants are attached to road and roof runoff. Roofwater is characterised by concentrations of substances which are only slightly higher than those observed in precipitation (Forster, 1990). Roads runoff is completely different, and can contain high amounts of suspended matter. In addition high levels of lead, cadmium, polycyclic aromatic hydrocarbons and mineral oil derivatives are associated with the runoff from roads (Xanthopoulos and Hahn, 1994). Heavy
metals and polycyclic aromatic hydrocarbons (PAHS) have been found attached to the sub 200 μm particles in sewer flows, with the degree of contamination related to landuse (Xanthopoulous and Augustin, 1992).

Domestic wastewater forms a temporally varying baseload of pollutants. The variation is a direct result of human activity. Domestic wastewater under peak dry weather flow (DWF) conditions is typically $2\frac{1}{2}$ times the average DWF. Peaks of domestic wastewater tend to be associated with washing and eating habits. Domestic wastewater contains large amounts of aesthetic pollution (Friedler et al, 1995). This type of pollution comprises material which gives offence to members of the public when discharged to watercourses from CSOs. Jefferies identified two categories of visual pollution (Jefferies, 1993). These are detailed below.

- Gross Solids can be defined as faecal matter, particles of paper and any other material greater than the arbitrary value of 6mm in any two dimensions with specific gravity close to unity.

- Visible solids are material which is identifiably sewage in origin and would be noticed by a casual observer walking on a river bank. The material is in effect plastic and paper strips which have virtually neutral buoyancy, and in many respects is the same as screenings material.

Material previously deposited within sewerage systems can be eroded into the flow during storm conditions. Five classes of in-sewer material have been identified by previous research (Crabtree, 1989) and various techniques exist for assessing the pollutant potential of in-sewer sediment (Crabtree and Forster, 1989). This material ranges from coarse, loose, predominately granular mineral material classed as a Type A through to Type E; fine grained material and organic deposits found in CSO storage tanks. Type C sediment is often found overlying Type A. Type C is a mobile, fine grained deposit, found in slack flow zones. Type C material forms the principal source for pollutants during storm flows except during extreme flows which can erode the underlying Type A sediment.
Wastewater, whether foul or combined, is transported within wastewater systems and these are discussed in the following section.

2.3 CHARACTERISTICS OF WASTEWATER SYSTEMS

A wastewater system is primarily designed to transport the waterborne waste products of human settlements and storm generated runoff to suitable treatment or disposal points. In the developed world, it is accepted that the use of the wastewater system is essential to avoid pollution of watercourses and protection against flooding.

Wastewater systems consist of conduits or pipes generally laid underground at constant gradients to allow the transportation of waste flows by gravity. These pipes are constructed of varying materials. Many materials have been used throughout the ages and date from tile drains in the Biblical era, brick in Victorian times, to concrete and glass reinforced plastic in more recent times. The type of material used for a sewer will depend upon the size, nature and volume of waste to be carried, depth of installation, construction technique and ground conditions. Sizes of sewer can vary between 150mm to 3000mm in diameter depending on the function and location of the sewer.

Wastewater systems are typically separate, partially separate or combined. A separate wastewater system employs two networks of pipes. These being a system which discharges all storm runoff to an available watercourse, and one which transports foul sewage to treatment. A partially separate system consists of two independent sewerage networks. One transports surface water runoff from roads to a watercourse, the other, carries the foul sewage and typically the runoff from roofs and backyards to treatment. A combined system transports foul and all stormwater within one network to a common point. A schematic of a wastewater catchment, showing inputs and outputs is shown in Figure 2.1.
Wastewater systems do not only consist of pipes beneath the ground. Many ancillary structures play a major part in the performance of a drainage network. Pumping stations are designed to lift wastewater flows when transportation by gravity is impossible. Pumping stations may contain pumps of different types, archimedian, centrifugal, mixed flow or axial flow. The specification of a particular type of pump depends upon the flow to be conveyed, the distance it is to be lifted and the nature of the flow to be pumped.

Other ancillaries include inverted syphons which are present on many wastewater systems, to allow flow to be transported beneath obstructions such as valleys and watercourses. Some systems contain grit or silt traps (Bertrand-Krajewski et al, 1995) which are designed to remove sediment from the flow. CSOs are provided on combined systems to discharge excess flows to the watercourse in times of rainfall, to protect urban areas downstream from flooding and to limit flow to treatment.
Alternatively, storage in the form of large sewers or tanks protect the conurbation from potential flooding arising from the wastewater system. Gates, weirs and flow controlling devices are utilised in times of heavy rainfall, to control the flow within the system, to ensure protection is afforded to the most vulnerable inhabited areas. The system as a whole should be operated efficiently, to ensure that flooding and pollution are controlled, whilst allowing the waterborne wastes of the human community to be transported to the source of treatment.

All wastewater is eventually returned to some part of the hydrological cycle whether it is treated or untreated. To ensure that the spread of communicable disease is controlled, to protect water supplies and to protect the environment, wastewater should be appropriately treated. Wastewater is treated by three main unit processes Physical, Biological and Chemical.

Receiving watercourses within drainage catchments are so called as they assimilate the discharges from wastewater systems. Storm and dry weather flows are passed from the treatment plant into river and streams. CSOs discharge pollutant loads to the watercourse during wet weather events and cause acute and chronic pollution. Many watercourses suffer from aesthetic pollution of the type described previously as the result of wet weather spills.

In essence, the watercourses are the final resting place for treated effluent and wastewater discharges from domestic and industrial sources. While being the final disposal point for effluent, many rivers are also utilised for potable water abstraction. It is of the utmost importance that pollution of receiving watercourses be controlled and minimised to avoid increased treatment costs, and the potential for the spread of waterborne disease.
2.4 PROBLEMS ASSOCIATED WITH WASTEWATER SYSTEMS

When wastewater systems fail to perform adequately the consequences are usually drastic. The consequences manifest themselves as pollution, flooding and structural collapse. These are briefly outlined below and detailed within Chapter 4.

2.4.1 Impact Of Pollutants On Receiving Water Courses

Pollutants are discharged to receiving watercourses from CSOs, SWOs (Storm Water Outfall), WWTPs and often directly from industrial processes. Pollutants containing oxygen demanding material, use up available oxygen within the watercourse during the degradation of organic pollutants (Balmforth, 1990). As a result, oxygen sags can cause fish kills and imbalances within the watercourse, subject to the pollutant load. Other toxic pollutants contained within discharges, such as ammonia, poison aquatic lifeforms.

Nutrients such as phosphates and nitrates cause eutrophication which ultimately leads to oxygen depletion. Pathogens and viruses discharged, can cause disease to users of the watercourse. Visual pollutants can cause aesthetic offence to those frequently using the watercourse, although they do not add significantly to biochemical degradation.

2.4.2 Flooding From Wastewater Systems

Flooding from wastewater systems occurs when the capacity of the drainage system is overloaded during storm events. Combined sewage is spilled from manhole covers, road gullies and sometimes WCs. The nature of floodwater from a drainage system contains large amounts of suspended material, faecal solids, rags and plastics.

Flooding of recreational areas causes loss of the particular amenity, while doing little financial damage. Flooding occurring on roads and streets causes dangerous driving conditions and increased journey times. Flooding in domestic catchments results in direct damage to properties and surrounding gardens. If commercial and
industrial areas are affected, then loss of business will occur along with stock damage. Associated with the physical occurrence of flooding, is the psychological worry which afflicted parties suffer every time the sky darkens and clouds gather.

2.4.3 Structural Aspects Of Wastewater Systems

To convey wastewater safely and efficiently the conduits transporting the waterborne material must be structurally sound. When collapse occurs, it does so, catastrophically. Sewer collapses are usually preceded by large degrees of subsidence in the road pavement above the pipeline.

The severity of sewer collapse influences the resulting level of traffic disruption. Vast quantities of finance are then devoted to emergency repairs of the collapse. Sewer collapses cause inconvenience, wasted resources, traffic disruption, flooding and pollution.

Wastewater systems should ideally provide acceptable levels of service in the three categories outlined above. To ensure this, the wastewater system must be managed and rehabilitated in an effective manner and all aspects considered integrally, not independently.

2.5 THE STUDY CATCHMENT OF PERTH

The City of Perth was chosen as the focus for the research presented in this thesis. The study originated from Tayside Regional Council Water Services Departments’ (now North of Scotland Water Authority, see Figure 2.2) requirement to investigate the effects of proposed development on the sewerage system, wastewater treatment plant and receiving watercourses within Perth City.

Perth is located within Tayside Region (see Figure 2.3) and has a population of approximately 42 000. The responsibility of delivering water services to the people of Tayside fell to the Regional Council prior to the reorganisation in 1996. The Water Services Department serves 363 000 people with public water amounting to 123 megalitres each day transported via 4 100 km of water mains (Tayside Regional Council, 1995). A population of 357 000 are served by 1 742
km of sewer and 50 megalitres are treated every day producing sewage sludge which is recycled to land.

The wish to examine the integrated performance of the City of Perth’s drainage system (see Figure 2.4) allowed the developing UPM strategy to be applied in conjunction with the research aspects of the study. The requirement to prevent pollution of watercourses and enhance the performance of the Perth sewerage system, while being utmost in the thoughts of present day engineers, was very important to their colleagues of the late 1800s working in Perth City.

Early references to the sewerage network of Perth can be found within the Archives department in the A.K. Bell Library in Perth. A Report (Young, 1862) On a System of Drainage for the City of Perth, contains the historical background to the development of the city’s drainage system and is discussed below.

2.5.1 Historical Perspective

Young’s Report lays out the methodology followed in the early 1860s to recondition and design the Perth sewerage system. The report states that steep gradients to central area sewers were not provided due to the flat topography of the area. However, to ensure that sewers remained “sweet and clean” ample flushing was provided through the connection of Lades (urban watercourses) to the sewerage system.

Lades were also added to the sewerage system to dilute sewage before discharge to the River Tay in an effort to lessen the impact on the river. Flushing points to the sewerage system were provided at many locations. Large quantities of water from the Lades were utilised by the city in the dyeing and manufacturing industries which were prevalent in the 1800s.

Reconditioning of the sewerage system was carried out in 1862 and involved the construction of a five feet catch sewer (interceptor) which collected all lateral drains. The carrying capacity of this sewer was estimated as 4 204 cubic feet per minute (1984 l/s) when full and 2 516 cubic feet per minute (1187 l/s) when 2/3 full. This sewer received combined wastewater from the surrounding areas.
2.5.2 Early Attempts At Urban Pollution Management In Perth

In 1925 a Report was commissioned to solve, and cost the water supply problem in Perth (Easton and Ker, 1925). This report was commissioned by Arthur Kinmond Bell and the work carried out by W.C. Easton and W. Arthur Ker. The report was designed to solve the problem associated with the close proximity of crude sewage discharges in relation to the water supply intake at the tip of Moncrieffe Island. Water was drawn through the gravel beds (to achieve filtration) before being supplied to the mains system. Investigations were requested due to the fears of contamination from nearby raw sewage discharges. The philosophy proposed by A.K. Bell was to:

"remove the Perth, including Scone sewage from the vicinity of the water intake, and dispose of it at such a point, and in such a manner, that pollution therefrom can never reach the intake."

The study produced a proposal to discharge the wastewater of the City on ebb tides at Easter Rhynd; the point where the River Earn joins the Tay. A simple catchpit, tanks and screens were proposed, capable of storing nine hours of average DWF. The policy adopted was to convey all wastewater to this point and allow no spills of storm sewage to the River Tay. These presumably were thought to pose a risk to the security of the water intake.

The estimated cost of the work proposed in the 1925 Report was £525 000. Easton’s scheme was never implemented by the city, and raw sewage continued to be discharged into the Tay until the late 1960s.

2.5.3 Recent Developments

Arthur Bell by a letter of Gift dated 1941, set aside property which was to be used by his Trustees to assist the Town Council in providing a system of drainage and sewage purification for the city. The gift in 1941 was in the order of £600 000. In order to make the best use of the money available, the Trustees commissioned the use of Babtie Shaw and Morton (Consulting Engineers) in the early 1960s, to develop and cost a suitable scheme for the city. A Report was commissioned in
1961 to examine the situation and to bring Easton’s original scheme up to date (Fraser, 1961).

This Report argued against Easton’s estimates of flow, and proposed that any scheme should deal with industrial wastes, domestic wastes and infiltration of 2¼ million gallons per day (118 l/s). The 1961 Report recommended the construction of a sewage purification works to the east of the city. This was argued to give more benefit than Easton’s scheme. Easton’s scheme was said to be only moving the problem of pollution further downstream. The purification works would encompass screening, grit removal, sedimentation, biological treatment and sludge disposal. The cost of this scheme which entailed pumping stations, pumping mains and purification works was estimated to cost £618 000 with running costs approximately £7 000 per annum. This scheme was further refined and detailed in a Report produced in 1964 (Fraser, 1964).

The 1964 Report makes allowance for discharging excess flows to the River Tay through CSOs. The flow at which excess would be spilled to the Tay is listed as 6 times the dry weather flow of sewage and three times the average daily flow of manufacturing wastes (or some other combination having the same volume). Excess flows would be treated to rudimentary standards. The design scenario was based on “a storm of such severity that it would not be expected to occur oftener than once a year on average” (presumably a one year return period).

The site selected for the purification works was an “island” called Sleepless Inch located 1¼ miles below the town. A recommendation was made in the Report that the works be designed to produce an effluent containing no more than 100mg/l of suspended solids and exerting no more a biochemical oxygen demand of 100mg/l (these consents are still in place, although full implementation of the Urban Waste Water Treatment Directive may change them). These standards were implemented due to the high volume of dilution available within the River Tay at the point of outfall.
The works was to designed to accommodate the dry weather flow from 50,000 people at 50 gallons per person per day (227 l/h/d) (131 l/s—design flow), plus industrial wastes 1 244 000 gallons per day (65 l/s). An allowance for future expansion for industrial flows was also added at 256 000 gallons per day (14 l/s). This gave an overall average design flow of 210 l/s. This figure equates to approximately ½ of Easton’s dry weather flow figure based on a design figure of 40 000. The estimated cost of this work was £795 000.

2.6 THE PRESENT DRAINAGE CATCHMENT OF PERTH

The present drainage system of Perth serves a population of around 42 000 and drains an area of approximately 15 km². The sewerage system is mainly combined with peripheral areas consisting of separate, partially separate and combined systems. The sewerage system associated with the city centre is flat in gradient whilst the networks in the surrounding subcatchments are steep. This configuration leads to the deposition of sediment along trunk and interceptor sewers in the central area.

There are three pumping stations associated with the drainage system; South Inch, Friarton and Willowgate. The latter station consisted of three dry well centrifugal pumps which conveyed the flow from the east side of the river, via a rising main slung from the railway bridge, to the sewer in Tay street. This pumping station has recently undergone total rehabilitation (to a wet well set-up) to alleviate damage, which occurred during the Tay flooding in January 1993 (Babtie Shaw and Morton, 1993).

The two remaining pumping stations which operate in series consist of archimedes screws which lift the wastewater to allow gravitational flow to the wastewater treatment plant (WWTP) at Sleepless Inch (McGeoch, 1974). Babties report proposed a rising main system, instead of the arrangement now in place. Presumably the present system was assessed as being more viable, during detailed design, than that proposed in the 1964 Report.
The WWTP consists of storm tanks, primary settlement, activated sludge aeration and final settlement tanks. The co-settled sludge produced is transported to agriculture and disposed of by injection. The consent standards are limited to biochemical oxygen demand (BOD) and total suspended solids (TSS), and these are 100mg/l respectively.

CSOs discharge to two watercourses within the Perth catchment, the River Tay and the Craigie Burn. There are five “hole in the wall” relief overflows situated along the Craigie Burn near Windsor Terrace, these have the potential to discharge unscreened combined wastewater to the burn. These relief pipes have flap valves to prevent backflow from the nearby Craigie Burn when in spate.

Six overflows of a similar type (only one has a flap valve) discharge unscreened sewage from the elderly system on the east side (Bridgend) of the River Tay (Young, 1866). The remaining CSO is immediately before the recently refurbished Willowgate pumping station. Rotating disc screens are utilised to allegedly give a level of screening equivalent to 6mm in two directions. An automatic penstock eradicates the effect of high river levels on the overflow system.

Two large overflows precede each of the archimedean pumping stations. Both are double sided weirs with automatic raking systems which do not function. The non-operation of the rakes has led to a massive build up of aesthetically polluting material on the screens at both overflow chambers. The outfall pipes from these overflows discharge via flap valves directly to the Tay. Consequently overflow discharge is often prevented or throttled, which results in surcharging of the main sewer.

The development of the Perth sewerage system and treatment plant from the 1800s to the present day is the result of increased awareness with respect to pollution and water quality issues. In Perth, Victorian engineers added the contents of urban watercourses to the sewers in an effort to keep them “sweet and clean” and to lessen the impact of discharges on the Tay.
The present day drainage system was promoted through the wish to eliminate the risk of water supplies being polluted. The trend of gradually affording more and more treatment to wastewater can be shown to be common to all large settlements. The development of a wastewater system for the City of London and awareness of water pollution has been traced earlier in the Chapter.

The need to manage wastewater systems to prevent failures is now recognised throughout the UK. In essence, engineers as with many other professions, have learned from their past mistakes. The appreciation of fully understanding the system and carrying out effective rehabilitation to the good of the entire system, not parts thereof, is the goal which must be strived for in the area of wastewater system management. This philosophy is the result of Urban Pollution Management (UPM) (*FWR 1994a*). The development of this approach is detailed in Chapter 3.
Figure 2.2 North of Scotland Water Authority
Figure 2.3 Tayside Region
Figure 2.4 The City of Perth
CHAPTER 3 THE DEVELOPMENT OF URBAN POLLUTION MANAGEMENT

3.0 INTRODUCTION

The following Chapter traces the development of Urban Pollution Management from the early days of the SRM through to the latest methodologies and tools to be deployed by engineers in dealing with the latest Directives from the European Union.

3.1 URBAN WASTE WATER TREATMENT DIRECTIVE

The driving legislation which influences wastewater policy in the UK is the EU UWWTD (*Urban Waste Water Treatment Directive*). This Directive is the most recent in terms of wastewater catchments. The Directive is a blend of the Uniform Emission Standard (UES) approach with the Environmental Quality Objective (EQO)/Environmental Quality Standard (EQS) philosophy (*Tyson et al, 1993*).

Under the EQO/EQS approach the principle is to establish the use requirements of the water body in question. These use requirements become the EQO for that particular receiving water (*Seager, 1993*). The EQS are then numerical standards which, if adhered to, will ensure the EQO is met for the watercourse. The use of computer modelling tools can ensure the EQS are met for the water body to achieve compliance with the EQO.

The UES approach is concerned with controlling effluent standards to the same particular figure regardless of local discharge circumstances. The use of this approach relates compliance to the use of a standard technology rather than a specified performance tailored to the needs of the receiving water.

The UWWTD has placed great emphasis upon the need to control wet weather discharges from CSOs, WWTPs and SWOs. Water Authorities, because of this Directive, are developing and costing their investment plans to meet legislative requirements. Private Finance Initiatives (PFI) are being considered by many
authorities to enable construction of treatment plants within the timescales stipulated.

Article 3 of the Directive requires member states to ensure that agglomerations of 2,000 or more are provided with collecting systems. New WWTP and existing plants are to conform with new sampling procedures and effluent standards laid down within the Directive. It further requires that collecting systems should satisfy the requirements of Annex I(a). This states that design, construction and maintenance will be undertaken with the best available technical knowledge not entailing excessive costs (BATNEEC), regarding the limitation of pollution from discharges from unsatisfactory CSOs.

CSOs are specified in the Directive as being unsatisfactory if they cause:

- significant aesthetic pollution;
- cause deterioration in river chemical or biological class;
- cause failure to comply with Bathing Water Quality Standards;
- operation in DWF conditions;
- operation in breach of existing consents and
- a breach of water quality standards.

The Directive required that laws, regulations and administrative procedures be set-up by June 1993. All populations of 2,000 or more should have a functioning collecting system by this date. Where there are special circumstances the required system must be in place by December 2000. All remaining implementations are required to be functioning by December 2005. Tighter timescales have been established for coastal areas but these are outwith the remit of this study.

Response to the UWWTD has been widely written about and discussed. An important point raised (Wright, 1992) is the impact of the Directive on the public and their perception of water quality. A recent survey of the Scottish populations’ attitude found that of 22 environmental topics covered, pollution of watercourses and the environment was the most important.
The response to this Directive by the wastewater industry within the UK has ultimately led to the development of Urban Pollution Management. While this procedure is the pinnacle of many years of research, it is important to investigate the historical development of UK wastewater philosophy which lead to UPM.

3.2 SEWERAGE REHABILITATION

In the early 1970s concern was expressed as to the condition of many of the sewerage systems in England and Wales, due to the frequent number of expensive structural collapses occurring during this period. A national programme of research was initiated, centred on the Water Research Centre (WRc), which resulted in the publication of the Sewerage Rehabilitation Manual (SRM 1) in 1984. This contained a detailed procedure for undertaking comprehensive investigations of complete sewerage catchment areas (Fiddes and Clifford, 1989).

The first edition of SRM allowed engineers to assess the renovation requirements for sewerage systems based on structural and hydraulic criteria. With the release of SRM(1) (WAA/WRC 1984) engineers began to realise that greatest asset was the “hole in the ground!” Hydraulic criteria for rehabilitation were generated from computer models of the drainage system, such as WASSP, developed from the Wallingford Procedure (Hydraulics Research, 1981).

The Wallingford procedure gave the engineer invaluable tools to aid the design and simulation of sewerage system hydraulics. Associated with hydraulic and structural problems were the problems of watercourses which received discharges from CSOs. The 1st edition of SRM was deficient in suggesting methods to deal with water quality problems, as the main thrust appraised solutions to structural and hydraulic inadequacies.
3.3 RIVER BASIN MANAGEMENT

SRM(1) explained how best to solve problems in relation to structural, hydraulic and, to a certain extent, economic criteria. SRM(1) made no attempt to suggest methodologies to address pollution occurring in watercourses as a result of CSO discharges.

An integrated holistic approach was required to address fully the problems of wet weather discharges from sewerage systems. A major research programme, entitled River Basin Management (RBM) commenced in 1985, and was funded by various institutions, which included the Department of the Environment (DoE) and the Science and Engineering Research Council (SERC).

Numerous polytechnics and universities undertook research activities over a period of years which helped develop the required computer models and techniques. The philosophy of River Basin Management was to formulate optimum methodologies which lead to the most cost effective control of urban runoff and CSO pollution (Ashley, 1988).

The RBM research programme set out to develop the ability to generate historic rainfall profiles, a sewer flow quality simulation model (MOSQITO) (Shamash, 1993), a receiving water impact model (SPRAT) (FWR 1990), relevant criteria and standards to be achieved and improved engineering solutions to drainage catchment problems. Work was also carried out to develop a wastewater treatment plant model (STOAT) (Dudley and Dickson, 1992). The philosophy was to use these models in an integrated fashion to establish the quality of discharges to receiving water courses during wet weather events.

This integrated approach was applied to pilot catchments in England and Wales, which had a high priority for rehabilitation. The idea being, to demonstrate if the models and methodologies developed in theory would operate in practice, and solve problems more accurately and economically than traditional analysis.
3.4 INTERIM APPROACHES FOR WATER QUALITY PLANNING

It was clear to the parties involved that the RBM programme would take time. This realisation led to interim approaches for water quality planning presented in a later edition of SRM II (WAA/WRC 1986) to address the problems of CSO discharge. These methods involved the use of standard pollutant concentrations applied to the hydraulics of CSO discharges to identify masses of pollutants spilled to receiving watercourses. The performance of CSOs were examined by using time series rainfall approaches (Henderson, 1986) and not statistical design storms.

A technique called CARP (Comparative Acceptable River Pollution) (WRC 1988) was developed to help engineers deal with the problems of CSO pollution in receiving water courses. This procedure is based upon comparisons between different river reaches, one of which must have known quality characteristics. This comparative approach did not require water quality standards to be achieved, but was used to tackle urgent problems and enabled valuable investigations to be undertaken (Barnwell and Fiddes, 1988). The preceding techniques and philosophies have been encompassed within the new UPM approach discussed below.

3.5 URBAN POLLUTION MANAGEMENT

The RBM programme transformed into the Urban Pollution Management (UPM) programme in the early 1990s. The thrust of the programme was to take into account the assimilative capacity of receiving watercourses from CSO spills, and to examine the performance of the whole drainage catchment in a holistic sense, to provide optimisation of pollution control and wastewater planning.

The programme strived to produce standards and methodologies which result in integrated wastewater system management and rational cost effective CSO pollution control. The methodologies are applicable to inland and coastal receiving waters based on a “fitness for use” philosophy. Major products from the study programme are listed over the page (FWR 1994a).
- Intermittent pollution standards based on un-ionised ammonia and dissolved oxygen
- Modelling tools:
  Rainfall STORMPAC;
  Sewer quality MOSQITO;
  Sewage treatment works STOAT and
  River quality represented by Danish model MIKE 11 (FWR 1992)
- Improved engineering designs
  CSOs
  Storage
- Implementation procedures:
  Interim; CARP, QUALSOC;
  Simple; SIMPOL;
  Complex; full modelling methods;
  planning techniques;
  compliance assessment procedures and
  applications guidance
- UPM Manual (published 1994)

The Urban Pollution Management Applications Methodology (UPMAM) was applied to the study catchments selected under the RBM programme, and resulted in a detailed knowledge of the building and understanding of the complex suite of models. UPMAM built up knowledge of the behaviour of the complex UPM models and stated that the use of these tools, results in an improved understanding of total system performance. This allowed more reliable and cost effective solutions, for major wet weather urban wastewater management schemes, to be identified through adopting the new fully integrated approach (FWR 1993a).

As a result of the evolution of UPM, it is now possible to simulate the behavioural characteristics of the wastewater system in its entirety and to the best of engineering ability. Potentially, any technical problem occurring in a wastewater system can be analysed and solved in terms of traditional engineering concepts by
the use of the integrated approach. UPM techniques and methodologies propose to offer cost effective solutions to catchment problems while achieving compliance with present and future legislation, through varying levels of modelling complexity.

The UPM procedure is presented in Figure 3.1, which shows the main components of the methodology and the relationship between each of them. For CSOs discharging to freshwaters three different approaches have been suggested by regulating water authorities in the UK \( \text{(SOED 1993)} \) in order to achieve the requirements of the Directive.

- Limited data methods such as Formula A \( \text{(SDD 1977)} \) are recommended for areas where the river dilution >8:1 (foul DWF : 5% low river flows).

- Interim procedures, QUALSOC \( \text{(WWA 1988)} \), CARP and sewer hydraulic models should be utilised where dilution <8:1.

- Complex modelling approach using the full UPM procedure where river dilution <2:1.

Aesthetic problems are solved by ensuring that no material greater than 6mm in two dimensions discharges to watercourses by installing screens on any new or unsatisfactory CSO. The approach suggested to deal with unsatisfactory CSOs, as specified in the Directive, is an EQO/EQS approach as problems will be solved based on site specific circumstances.

UPM studies have proposed that the application of the detailed full modelling procedures have produced more cost effective designs than the simpler techniques when applied to the same drainage catchment problem. The relationship between simple and complex with respect to solution costs is shown in Figure 3.2.
Preliminary assessment of problem

Is a planning study required?

Yes

Agree framework for environmental assessment

Decide on data and tools required

Assemble (or improve) data and tools

Establish Site specific standards

Develop Solution

Check solution compatible with other plans

Cost effective?

Yes

Obtain consent for solution

Proceed with detailed design

No

Identify improvements needed in data and models

Stop

Figure 3.1 The UPM Procedure
The use of the UPM approach will potentially allow water authorities to achieve holistic solutions, through integrated modelling, to simple and complex drainage problems and meet the required discharge standards concurrent with existing and proposed legislation in a cost effective manner. This holistic approach is shown in Figure 3.3.

![Figure 3.2 System Complexity And Solution Costs](image)

**Figure 3.2 System Complexity And Solution Costs**

### 3.6 POLLUTANT DISCHARGE STANDARDS

Discharge standards can be subdivided into three main sections; general standards, standards for protecting aquatic life and standards for protecting general amenity use. Under the general category are EQOs and EQSs, use classes and SWQOs (Statutory Water Quality Objectives).

At present non statutory water quality objectives are proposed to be replaced by Statutory Water Quality Objectives (SWQOs) which will provide a basis, in England and Wales, for the setting of water quality objectives to meet EU requirements for fisheries ecosystems, potable water supply abstraction, water sport activity, irrigation, livestock watering, harvesting of marine fish/shellfish and the protection of special ecosystems.
The setting of appropriate objectives and standards for river quality in Scotland is the responsibility of the former River Purification Boards (RPBs), now the Scottish Environmental Protection Agency (SEPA), and for the majority of drainage areas in Scotland it is expected that the simpler techniques will be utilised for analysing CSO discharges to watercourses due to the high levels of dilution contained within Scottish rivers.

SWQOs are to be split into two different types of water quality classification systems. Different use classes (UCs) for setting targets relating to the actual or proposed use of the water, on a statutory basis and a general quality assessment (GQA) scheme for assessing general overall progress on a periodic basis (Seager, 1993).

Fundamental standards for the protection of river aquatic life have been derived through research under the RBM and UPM programmes in the form of un-ionised ammonia (NH₃-N) and dissolved oxygen (DO). These pollutants have the most direct effect upon fish and invertebrates (Milne and Seager, 1990, 1991).

Working with DO and un-ionised ammonia is a complex process. To combat this, a set of intermittent standards based upon BOD and Total Ammonia, which are modelled by WWTP and sewer flow quality models, have been derived. These “derived standards” apply at the point of mixing and require no modelling of the watercourse being examined.

The presence of the derived standards does not negate the use of the fundamental intermittent standards. The advantage of the derived standards is that designs can be appraised in terms of their impact on the watercourse without the need for modelling of the receiving watercourse.
Figure 3.3 Components of Urban Pollution Management
Detailed modelling work by others (FWR 1994a) using the derived standards has shown that one year return period thresholds are most critical and if met then the shorter duration return periods will also be met. Although the 1 hour threshold can often be more critical than the 6 hour threshold, the difference is not great, and 6 hours is a more practicable duration to use (the difference is allowed for in the safety margins built into the derived standards).

The presence of the derived standards does not preclude the use of the fundamental standards where river conditions justify them; flat deep rivers, where DO levels are low for a large majority of the time or if a more cost-effective design is required.

Amenity standards have been developed based upon the frequency of CSO discharge and the amenity of the area in which the discharge takes place. The appropriate fundamental, derived and amenity standards (FWR 1994a) are shown in Appendix B.

3.7 UPM MODELLING TOOLS AND TECHNIQUES

The following section gives details of the pertinent computer modelling tools and techniques encompassed within the UPM philosophy. For additional information the reader is directed to the references.

3.7.1 WALLRUS/ HYDROWORKSTM

WALLRUS (Hydraulics Research, 1991) is the newest version of the Wallingford Storm Sewer Package WASSP. WALLRUS-SIM is the simulation mode and is used to analyse and represent the behaviour of a sewerage system. WALLRUS derives a run-off hydrograph from given rainfall parameters and then routes this through the specified sewer system. The package calculates flows and depths throughout the sewer system and graphical hard copies can be obtained showing flow and depth readings against time for specified pipes. WALLRUS also shows the position of above ground flooding and the volume of storm sewage involved.
WALLRUS-VIS allows the user to see flooding and surcharging which have occurred for a particular storm by producing a colour coded schematic of the sewerage network on screen.

The model deals with a large range of sewer ancillaries including overflows, on-line tanks, off-line tanks and pumping stations. This makes it ideal for the analysis of large complicated systems. However, its use is restricted to drainage layouts which have a dendritic structure and is therefore very difficult to apply to looped systems.

The model requires a large amount of information on the system, ground levels, invert levels, pipe lengths, pipe diameters, pipe roughness, impermeable areas, permeable areas, details on sewer ancillaries and information on the rainfall over the catchment being studied. A WALLRUS model is calibrated and verified by information gathered using short term sewer flow surveys. WALLRUS is the UK industry standard for hydraulic analysis of sewerage systems and provides the hydraulic base for the sewer flow quality model MOSQITO. WALLRUS was not developed explicitly under the RBM and UPM research programmes.

Recent commercial development has seen the release of HYDROWORKS™ (Hydraulics Research, 1995). This package is more mathematically stable (based on St Venant equations—full solution model) than WALLRUS and allows the engineer to examine more carefully the performance of the system, due to the ability to replay simulations interactively for storm events. This tool is now the mainstay of the industry with respect to computer modelling of drainage systems.

3.7.2 MOSQITO

MOSQITO (Hydraulics Research, 1991) models the quality of urban wastewater in sewers. The movement of sediments and pollutants are simulated within the sewerage system and pollutographs can be produced at any time period for any part of the drainage system. MOSQITO is the sewer flow quality package which accompanies WALLRUS. Common water quality parameters such as TSS, BOD,
COD and total ammonia are modelled as standard. The user can select other determinands if required.

Verification of a MOSQITO model involves installing flow monitoring units (Detectronic, 1991) and automatic sampling equipment at various points throughout a catchment to gather quality data on DWFs and storm flows. A verified WALLRUS model is essential before a MOSQITO model can be utilised.

Data for surface sediments and in-pipe sewer sediments should also be collected. A verified model will predict the pollutant loads discharging from the CSOs in a catchment area. This provides more accurate information on discharges than by using the Interim procedure (Crabtree et al, 1988). The model will also predict pollutant contributions entering a WWTP during dry weather and storm flows which can be modelled by the WWTP model STOAT.

At the time of writing Hydraulics Research have released HYDROWORKS-DM, a new quality module based on the hydraulic package of similar name. This package at present is based on the fundamentals of MOSQITO and a French sewer flow quality simulation FLUPOL. The DM package at present does not possess the ability to model the erosion or deposition of sediment from within the sewerage system.

3.7.3 STOAT

STOAT is a dynamic model which simulates the behaviour of WWTPs. The model can receive pollutographs from a sewer flow quality model and represent the performance of the unit processes. Storm tanks, primary settlement, activated sludge, biological filters, final settlement and other more complex processes are modelled within STOAT.

Determinands modelled include Volatile Suspended Solids (VSS), Non Volatile Suspended Solids (NVSS), soluble BOD and particulate BOD, Total Ammonia, NO\textsubscript{2} and DO (WRc 1991). STOAT requires flow and quality influent details at one to two hourly intervals over a four to five day period, including a storm event for calibration and verification. The model can be used to examine the
performance of the plant under differing storm conditions and to predict treated loads discharging to watercourses.

3.7.4 River Models

River models and impact assessment techniques are in some ways the most important developments from the UPM programme, as they model the direct effects of intermittent and continuous discharges upon water quality. The assimilative capacity of the receiving watercourse is just as important as the transport capacity of the sewerage system and the treatment capacity of the WWTP.

Many methods and models now exist to evaluate the quality of receiving watercourses, from simple desktop procedures to complicated computer packages. The choice of which approach to use depends on the answers required, the complexity of the problem and the cost involved. The UPM programme has identified the model MIKE 11 (Becker and Hutchings, 1992) to be suitable for application to catchment management plan studies. This model has been developed by the Danish Hydraulics Institute (DHI) and is a one-dimensional flow and water quality model.

It consists of a number of modules each performing an individual task. Data have to be collected on the catchment layout of the river system, channel characteristics and position of major continuous and intermittent inputs. Individual storm events are necessary to calibrate/verify the model. UPM procedures recommend the use of MIKE 11 in solving problems which require the full integrated modelling approach.

Other models in use are SPRAT and CARP. CARP compares the estimated pollution load to a river reach with pollution load discharged to a river reach of acceptable quality. SPRAT is a simple dynamic river flow and quality model developed by WRc. SPRAT is designed to predict river quality in an urban catchment during rainfall periods when CSOs are operating.
Impact assessments can be achieved by the use of methods such as QUALSOC (WWA, 1988). This uses a simple mass balance technique to evaluate the water quality downstream of a CSO. The water quality parameter used is the five day BOD value and the overflow impact is assessed against the Maximum Admissible Concentration value (MAC) in the downstream river appropriate to the river classification.

3.7.5 STORMPAC

STORMPAC is a package designed to utilise historic rainfall records and to generate long time series rainfall. The package runs through a windows interface and contains modules for generating artificial long time series of hourly rainfall for anywhere in the UK, accepting historical hourly time series from Met Office records, identifying storm events from either artificial or historical series and selecting events based on specifications input by the user.

The model can also disaggregate the hourly values for storm events into five minute intervals for use with sewer models. The main component of STORMPAC is the synthetic rainfall generator which can produce up to 25 years of rainfall records for anywhere in the UK (Cowpertwait et al, 1991) based on rainfall for each month of the year, grid reference, altitude and distance from the coast. The model has been tested against real rainfall records and has been found to be very accurate.

3.7.6 SIMPOL

The preceding deterministic hydraulic and quality models can be used to evaluate alternative designs to solve pollution problems in drainage catchments. Running a large time series of rainfall events with these models is time consuming. The use of a SIMPOL approach allows a large number of rainfall events to be run through a simplified catchment to assess design alternatives to a problem, in a fraction of the time taken with a complex model.

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SIMPOL has been created in a spreadsheet format (EXCEL) as part of the UPM procedure. In SIMPOL the sections of a sewerage system are represented by a system of tanks; surface tank, sewer tank, CSO tank and storm tank.

SIMPOL considers only one pollutant and this can be BOD or total ammonia. This is specified by the user. Pollutants are assumed to originate from DWF, surface sediments and in-sewer pipe sediments. The defined tanks in SIMPOL are calibrated against detailed WALLRUS, MOSQITO and STOAT models. SIMPOL also requires information regarding existing pollutant concentration and minimum flows in the watercourse being considered.

Once SIMPOL has been calibrated it can be run in one of three modes. The river intermittent mode estimates the in river BOD or ammonia concentrations which is exceeded for six hours with a return period of one year for a selection of rainfall events. This mode is likely to be most useful for inland watercourses. Other modes include spill frequency, marine impact and storm ranking.

3.8 THE NEED FOR A COMPLEMENTARY METHODOLOGY

Although the UPM procedure provides methodologies and techniques for rehabilitating the performance of wastewater systems and has undoubtedly advanced the knowledge base in many areas of integrated catchment management, there are further areas which require to be addressed.

It is unclear at this stage if the solutions produced through the complex quality models are sustainable with respect to actual perturbations, transients and fluctuations experienced by the sewerage system over extended periods of time. For a solution to be sustainable the cause of the problem must be addressed and not the consequence.
Sustainability issues are of key importance in society today (LGMB, 1993) and work is currently underway to resolve the conflict between economic, social and environmental issues associated with sustainable development and solutions (Payne and Gardiner, 1996). A number of studies have described comprehensive appraisals of sewerage system performance and comparisons between different options for future management (Nakamura 1981, Durschlag and Schilling 1990, Adamson et al, 1984).

UPM does not address in detail practical or economic aspects of source control (Rudolph and Balke, 1996) and best management practices (BMPs). Source control of runoff is widely implemented overseas (Fujita, 1996) and appears to offer advantages over existing approaches. Technical standards are now available for source control approaches, notably in Germany (ATV 1990). Source control is beginning to emerge in the UK and is gaining in popularity.

UPM provides a method of proving that regulatory standards are met during design. This does not necessarily mean that these standards will be sustainable over the design life of a solution. Only if all possible fluctuations and scenarios are examined during design, can the engineer be sure that the solution will deal with all combinations of input. This is an onerous task. Therefore, the performance of a rehabilitated catchment must be monitored with respect to all areas of performance to identify if the problem has truly been solved.

Prioritising the need for upgrading of wastewater systems is an area in which many Water Authorities are interested. May 1994 saw the initiation of a project controlled by FWR into assessing the benefits of river water quality improvements (FWR 1996).

The purpose of this project is to provide operational and planning staff within the Water Authorities with a means of assessing the benefits of improvements in water quality of rivers within their catchments. Three decision contexts are taken into account; whether or not to undertake a single project; which of a number of
mutually exclusive projects to undertake and which of a number of mutually compatible revenue competing projects to undertake.

The analysis is wholly concerned with upgrading the quality of a watercourse to a higher level and the costs/benefits associated with doing so. The idea of the analysis is to expend capital in the area which has the biggest benefit to society. The project utilises Benefit Cost Analysis (BCA) as the decision making tool which has been widely used in the analysis of river inundation and for quantifying the benefits associated with ecological improvement (Boddington, 1993).

Clearly prioritisation of capital expenditure is an important issue. What is slightly concerning is the fact that the decision analysis procedure is associated with only the benefits accruing from improvement in water quality issues. UPM provides a holistic approach (regarding all aspects of system performance) to wastewater management and therefore should a prioritisation procedure be based solely on water quality improvements?

A methodology/technique is required which allows engineers to prioritise upgrading works based on the performance of catchments under their control. The methodology should holistically consider and evaluate all areas of concern associated with a wastewater systems performance. These areas of concern are typically associated with pollution, flooding, structural aspects and water quality issues.

Such a methodology would allow engineers to collectively examine the relative performance of catchments. The methodology should ideally complement UPM and work integrally to aid in decision making between effective rehabilitation options. Also the methodology must be capable of monitoring a rehabilitated catchments performance, to ensure any solution implemented is solving the defined problems and controlling the risk of failure.
Any methodology must satisfy rigorous standards and these are summarised below (Green et al, 1989).

- **Elucidation** - as a result of the appraisal the decision makers should be more knowledgeable about the issues and trade-offs involved between the different objectives, and the performance of the different objectives.
- **Simplification** - the method must enable the clarification and simplification of supporting data so that the data does not overwhelm the decision maker.
- **Feasibility** - the resources required to carry out the analysis must be appropriate to the timing and importance of the decision.
- **Completeness** - the method must be able to encompass all the significant differences between impacts of different rehabilitation options.
- **Rigour** - the method must be internally consistent and where value judgments are required these cannot be left to the analyst.
- **Reliability** - the results of the analysis should not depend upon the individual who undertakes the analysis.

Traditionally the assessment of risk has been abrogated to a simple selection of design storm return period (WRc/WAA 1995) for flooding analysis. The process is to select an appropriate design storm and size the drainage conduits on the flows generated from the storm. The validity of this approach is unclear particularly when applied separately to each of the components in a large and integrated urban drainage system.

Attempts have been made to improve the UK approach (Hydraulics Research, 1981) and to devise a methodology for flood risk assessment (Penning-Rowsell and Chatterton, 1977), but these have not been applied extensively to urban wastewater systems. They are generally applied to river catchments. Stochastic based approaches to risk assessment for sewerage systems have been proposed elsewhere, notably in the USA (Yen, 1990) and Denmark (Nielsen and Harremoes, 1996).
Environmental assessment and the quantification of intangibles relating to, for example, the visual impact of gross sewage solids on riverbanks, has not been considered to any great extent, although some attempts (Dee et al., 1973) (House and Sangster, 1991) have been made to develop techniques for the evaluation of environmental criteria.

The major difficulty in dealing with these factors is that they incorporate moral value judgements (Semple, 1991) and when considered in the broadest holistic sense require a consideration of all the processes and activities which could possibly influence the dynamics of a wastewater system within a particular urban drainage catchment. These activities are not restricted to technical criteria but include environmental, social and economic criteria (Ashley and Goodison, 1991).

Similarly, (Delannoy, 1990) and (Del Treste, 1990) have developed methods which allow the user to see the benefit of such projects and choose the most beneficial solution. However, no appreciation is given to intangible damages and no costs are attributed. Also, the inputs to these methodologies have been developed from empirical and theoretical models and not from dynamic simulation models.

BCA analysis in itself is not a new approach. However, the evaluation of intangible environmental benefits makes the technique difficult to apply in the area of wastewater systems. The technique of BCA, the evaluation of benefits and other approaches are examined in Chapter 4 with the purpose of selecting a suitable medium on which to base the complementary methodology to be applied to the Perth wastewater system.
CHAPTER 4 A TECHNIQUE FOR THE EFFECTIVE EVALUATION OF THE PERFORMANCE OF WASTEWATER SYSTEMS

4.0 INTRODUCTION

In the following sections a review of existing techniques appropriate to project appraisal and decision making is presented. The purpose of the analysis is to determine a suitable method on which to base the development of the holistic approach to be applied to the wastewater system of Perth.

4.1 METHODOLOGIES AND TECHNIQUES USED FOR PROJECT EVALUATION

One common methodology, used predominately by the business sector, is that of financial analysis. This is related around profit or loss for a particular firm or society. A “good” project does not cater for the individual or a group, it is only concerned with maximising the benefit to itself (Local Government Operational Research Unit, 1978). Financial analysis cannot be applied if the whole society is to be considered. Therefore, this approach should be discarded as a decision making tool for choosing between options with respect to projects in the Public domain.

Cost Effective Analysis (CEA) has been generally adopted in the wastewater industry. For example, a sewerage system is designed for a particular return period or flow, for the minimum cost. Solutions achieve a given aim at the lowest cost from a series of alternatives. The aim may well be a specified number of occurrences of flooding in ten years or a number of overflow spills in a particular period. The cost applies to the money incurred in building the project to meet the criteria. No appreciation is given to damage costs associated with a lack of performance from the designed system under conditions other than the design input.

Performance Cost Analysis (PCA) allows a solution to achieve a certain performance standard or a specified cost. Different levels of performance are identified and their costs compared. The solution chosen is the one deemed to
provide adequate protection at an acceptable cost. A choice is offered between levels of protection at varying costs in comparison to the cost effective approach which gives choice between different costs of building the same project to achieve a specified level of protection.

Benefit Cost Analysis (BCA) has been applied predominately to river inundation. Rehabilitation solutions are examined in detail and all benefits (tangible and intangible) and associated costs are evaluated. Costs refer to the construction of a particular project, whereas benefits are associated with the aversion of damages related to not building the project. In theory, this method of analysis can allow the decision maker to see the best solution to a particular problem.

Like any methodology there are flaws relating to the specific applicability to wastewater projects. Evaluation of benefits relating to prevention of pollution are particularly difficult to quantify in monetary terms. Alternative solutions are judged based on the Benefit Cost Ratio; the ratio of the present value of benefits to the present value of the costs. Costs and benefits are discounted through the application of a factor which relates the value of a sum of money in the future to the value at present.

The net present value of an alternative is the sum of all the present values of all benefits less the sum of the present values of all costs. The favoured alternative is the project which has the maximum net benefit or if costs always exceed benefits, that which shows the minimum net cost. This may be particularly true where the averted damage is small compared to the cost of implementing the project (Harris and McCaffer, 1989).

Minimising deviations from targets is a technique which involves the principles of goal programming. It is a progressive methodology which starts with the most important priority according to the decision maker, and its deviation from a preset goal or target. The option with the minimum deviation from the target is consequently selected. For example, an overflow discharge to a bathing beach is
required to meet the criterion of three spills per bathing season, but also to be
designed and constructed within budgetary requirements. Designs are put forward
which represent 2, 3, 4, 5, 6 and 7 spills per bathing season. The design chosen
represents the spill criterion which achieves the standard or deviates from the
standard least.

Linear programming is one approach available to the decision maker when
analysing complicated scenarios consisting of many variables and constraints on
these variables. Linear programming has many applications in the industrial world,
and works well when every variable in the model behaves in a linear fashion and
remains linear over the range of the model, and the number of feasible solutions to
the problem are limited by constraints on the solution. Optimal solutions can be
found by using linear programming in situations where least cost mixes of
concrete are required and transportation problems regarding optimal movement of
earthworks.

Applications for linear programming techniques to particular civil engineering
problems are numerous and wide ranging. The existence of well defined
characteristics in a problem should become recognisable and should intuitively
suggest linear programming as a possible solution (Templeman, 1982).

Simplex methodologies utilise the principles of linear programming. More
variables are added to each constraint equation under consideration to ensure that
the constraints achieve equality. These variables, which are termed slack, can be
negative or positive to achieve equality in the constraint equations. If a constraint
already achieves equality it is said to be tight. In graphical format the amount of
slack in a variable gives a rough indication of how far a point is from a constraint
boundary line. The major advantage of the simplex method is that only feasible
points are considered in the solution set and that the solution is highly
computable.
These techniques are adequate when dealing with simple problems and give the decision maker an idea of which approach to take to the problem being addressed, but they do not consider the likelihood of different outcomes occurring (Coyle, 1972).

These techniques are also of little use when states of nature are involved, for example when considering the probability of the severity of a rainfall event and the consequences resulting from the event. Probability must be utilised when states of nature are involved in the decision making process, if a clear understanding of the payoff to be achieved under varying circumstances is to be understood.

Decision analysis is critical if an effective solution is to be identified for a particular problem. This not only applies to drainage rehabilitation, but to any problem which has many options, with different probabilities relating to states of nature and outcomes relating to these states of nature. This type of analysis has been applied historically to the aversion of damage from hurricanes by seeding with silver iodide crystals (Howard et al., 1972). The medical industry have utilised decision making theory models in expert systems for the identification of diseases and the required treatments (Betaque and Gorry, 1971). Techniques utilising neural networks (Loke, 1996) for decision analysis in drainage engineering have emerged and been applied successfully.

Decision making under conditions of uncertainty and risk can be eased by the use of techniques which involve probability approaches (Delleur, 1981). The two main approaches are Expected Monetary Value (EMV) and Expected Opportunity Loss (EOL). Recent work (Geldof, 1996) has developed strategies to considered uncertainty through statistical approaches based on Monte Carlo Techniques.

The EMV technique considers the probabilities associated with states of nature particular to the problem, for example, those probabilities associated with wave height exceeding 3m on a given date or the volume of snowmelt runoff between given times. The probabilities can be assessed from past data regarding the
particular phenomenon being studied or can be chosen subjectively by the decision maker. The EMV technique finds the course of action which when repeatedly used gives the largest reward to the decision maker (Ossenbruggen, 1984).

The EOL technique is directly related to that of EMV and Opportunity Loss, but concentrates on the least payoff which is to be achieved. The main problem with probability approaches is the selection of probability to use in the calculations. Even if past probabilities are known, there is no guarantee they will repeat themselves given a certain set of circumstances for a complex problem.

Although these techniques involve states of nature in their criteria and the payoff is adjusted by the probability of the occurrence being studied, they both do not take into account the magnitude of the reward and its circumstances. They do not reflect the decision makers attitude or willingness to accept reward or loss.

People clearly value money or choices in different ways depending on their outlook. Environmental and social factors are greatly subjective depending on the perception of those involved. One decision maker can view the same decision totally differently from another depending on their judgement of the risk involved. This phenomenon can be dealt with using the principles of utility. Utility theory expresses a decision maker's preference for a particular outcome or set of circumstances.

Multi-attribute utility theory involves measuring the changes in utility when faced with a series of alternatives. The principle of utility is closely related to the risk nature of an individual facing the dilemma. The problem with this approach is that even if enough individual utilities are measured, they do not necessarily reflect the preferences of the whole society possibly affected by the proposals on offer.
Environmental Impact Assessment is a technique which involves the thorough investigation of environmental and social factors affected by proposed options. The technique can generate masses of information which often are of a conflicting nature. Techniques such as the Leopold Matrix can be employed for social and environmental impacts.

Once a solution has been reached for a particular problem through the application of any appropriate methodology, the sensitivity of the solution to changes in the constants must be investigated. Re-working of the problem would prove costly if it contains large quantities of constraints. Sensitivity analysis provides a methodology for investigating the changes to the optimum solution as a result of changes in the objective function coefficients, constraint coefficients and constraint boundaries.

An all encompassing methodology must provide a means whereby a solution which gives the greatest protection to all aspects of the environment and society is attainable at an acceptable cost. From the review of techniques it is clear that any methodology must deal with technical, environmental, social and economic criteria in a structured fashion.

Benefit Cost Analysis allows benefits (averted damages) associated with rehabilitation improvements to be converted into tangible cash sums. Therefore, the technique allows the user to see benefits accruing from a capital investment. The techniques available, especially to evaluate environmental benefits, are complex and if all benefits are not evaluated accurately flaws in the methodology can be produced.

Multi-attribute utility analysis allows preferences for certain outcomes or scenarios to be evaluated and can be applied to the more intangible aspects of water pollution and sewerage system performance. This methodology allows solutions to be appraised with respect to the values and opinions of those parties ultimately affected.
The techniques of BCA and multi-attribute utility theory are worthy of further investigation as they both offer the possibility of assessing tangible and intangible criteria, associated with the performance of wastewater systems in a structured fashion. The other techniques are rejected due to their lack of suitability with respect to the evaluation of intangible criteria.

4.2 BENEFIT-COST ANALYSIS

BCA is used by decision makers to decide if a project is in the best interests of society, and therefore is appropriate for regulatory bodies such as Water and River Authorities. The structure of BCA is essentially associated with defining the improvement in social welfare for society when expenditure is invested and resources allocated. BCA is a complementary technique, to be used in conjunction with technical approaches.

The willingness-to-pay is an important concept used in BCA. The objective of BCA is to further the wishes of society in accordance with the opinion of individuals. Thus the benefits offered by an alternative are based on the aggregation of the parties involved. Willingness-to-pay is evaluated through the examination of the provision of a good or service, such as that provided by a sewerage rehabilitation proposal with respect to flood damage. This service is defined as a public good and cannot be traded in the market place in the normal economic manner. For river flooding projects, the willingness-to-pay is estimated through the aggregation of expected losses.

BCA is only concerned with benefits/damages to society as a whole and not financial losses to individuals. If benefits are associated to one party and similar damages are experienced by another then no net gain is presented to society. Benefits and costs which accrue in the future are discounted to present day values through the application of a discount factor set by the Treasury Department *(HM Treasury, 1984)*. There is considerable debate about the application of discount factors associated with Public projects.
Uncertainty in BCA, as with many techniques, is investigated through the application of sensitivity analysis. This involves the adjustment of key criteria to examine the effect on the outcome of the appraised solution.

Effectively, the application of BCA involves the complete evaluation of all benefits (averted damages) associated with a scheme and the relationship associated with the cost of the scheme all discounted to a base date. Benefit cost ratios are compared for alternative solutions to a problem. Commonly, incremental ratios are examined to investigate if providing an additional benefit to society can be justified.

From this the decision maker can see the most viable option for any catchment. Alternatively, the decision maker can see which option gives the largest benefit to society as a whole. The main area of concern is the evaluation of the damages associated with states of nature. This and the applicability of BCA to wastewater system analysis is presented in the following section.

4.3 DAMAGE ASSOCIATED WITH WASTEWATER SYSTEM DISCHARGES

Engineers are concerned primarily with the benefits to be gleaned from reduction in damage caused by sewage flooding, structural collapses and pollution of watercourses. Flooding from a sewerage system arises due to the inability of the conduits to convey the flows generated by storm events. The increase in urban development in many large towns has led to an increase of storm runoff and frequency of flooding.

Structural collapses are the result of inappropriate maintenance of sewerage systems. Many core sewers in towns are elderly and up until the development of SRM received scant attention. Indeed, only when collapses started to occur did engineers develop a successful methodology for dealing with the problem (WAA/WRC, 1984). Pollution has been discussed in Chapter 2, and generally is a result of the untreated discharges of combined sewage, storm water runoff and industrial effluent into watercourses. These discharges cause acute and chronic
pollution which can kill aquatic lifeforms and degrade water quality for other uses. It is apparent that there are different types of damage associated with the failure of drainage systems, and these will now be considered individually.

4.3.1 Damages Associated With Flooding From Wastewater Systems

Flooding from wastewater systems occurs when the capacity of the sewerage system is exceeded due to the volume of runoff from precipitation. Precipitation is commonly in the form of rain, although rapidly thawing snow can cause flooding from sewerage systems, notably in Scandinavia (Thorolfsson and Brandt, 1996). The occurrence of snowmelt is not generally considered when designing or rehabilitating the capacity of urban storm drainage systems.

Flooding will emerge from the drainage system at the lowest available point depending on the topography of the catchment. This can be at road gullies, manhole covers or in extreme cases, WCs or showers in basements. The depth of flooding is dependent both on the volume of flooding and the floodable area region.

Most available research work concerning damage from inundation to catchment areas, applies to that caused by river flooding and not to that arising from sewage flooding. When damage is caused by a river flood, it tends to be vast and may last for a few days at a time, depending on the characteristics of the flood. Sewage flooding is short lived, intermittent and may be confined to one street or a small area of typical landuse.

Often flooding from the sewerage system is not reported unless severe. Table 4.1 shows the consequences of damages associated with flooding from sewerage systems (Hydraulics Research, 1981).
4.3.2 Techniques Used To Evaluate Damage Resulting From Sewage Flooding

Accurate modelling of the distribution of flood water over a catchment is required to give accurate flooding depths. Depths can be used to evaluate damage costs resulting from flooding associated with hydraulic inadequacy. The first step is to assess the depth of flooding on the catchment. A suggested technique for doing so is detailed below (Davies, 1992).

- Determine where the flooding will emerge from the system. This will normally be at a gully or manhole (in the street or adjacent connected properties) at the lowest point in the area.

- Examine the slope of the ground, the presence of barriers such as walls and kerbs that may contain the flow causing surface ponding, or divert the flow.

- Examine the slope of the ground in order to establish which way flow will travel on exiting the system.

- Examine the data for the existence of neighbouring sewerage systems, watercourses, adjacent pervious areas or adjacent properties, in order to determine where the flooding will runoff to. For example, flooding from a partially separate system may discharge into a storm sewer system without causing any discernible flooding.

- From the model prediction of surface flooding, examine the data to determine the volume of flooding predicted.

- Examine the data to evaluate what area will be affected by surface flooding. From the predictions of volume and area, the depth of flooding can be evaluated.

This procedure can be utilised in conjunction with techniques which relate depth of inundation to damage costs. Unfortunately, this type of approach for sewerage systems would be very difficult to apply in reality. Verification of above ground flow paths would be problematic to confirm, as flooding may not occur during the period of investigation.
This type of approach would require depth/damage curves to be generated for various types of catchment (e.g. residential, recreational and commercial). While this has been developed for river flooding, shallow sewage flooding rarely causes severe financial damage. The verification of a topography model for above ground flows would be difficult, especially if few flooding events are captured during the study period.

<table>
<thead>
<tr>
<th>Property</th>
<th>Risk</th>
<th>Results Of Flooding</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highways</td>
<td>Surface flooding</td>
<td>Pedestrian splashing, loss of amenity, traffic delays, accidents, structural damage to road pavement</td>
</tr>
<tr>
<td>Domestic Properties</td>
<td>Flooding from surface water or storm sewage</td>
<td>Loss of amenity, access difficulties damage to properties, structural damage, health risk</td>
</tr>
<tr>
<td>Commercial and Industrial</td>
<td>Flooding from surface water or storm sewage</td>
<td>Inconvenience to customers, difficulty with vehicle and staff movement, direct damage to stored goods and materials, damage to plant and machinery, consequential losses to workers/consumers through loss of production or stock, serious structural damage, risks to health, risks to life</td>
</tr>
</tbody>
</table>

Table 4.1 Consequences Of Flooding From Sewerage Systems

Damage evaluated by a depth/damage methodology tends to be of the direct type. Direct and indirect damage are discussed in the following section.
4.3.3 Direct And Indirect Damage

Damage can be divided into two categories: direct and indirect. Direct damages consider damage to property and the magnitude depends, broadly, upon the type and size of property, as well as, and crucially, upon the depth and nature of flooding. The method of assessment of the magnitude of these losses is well established (Penning-Rowsell and Chatterton, 1977).

Indirect losses from flood damage tend to be losses which are regarded as resulting from the secondary affects of the flooding. Traffic disruption and retail losses are in this category. Traffic disruption can be the first activity affected by flooding from sewerage systems. Ponding from road gullies generally precedes flooding from manholes. Disrupting traffic results in losses of four kinds (Green, 1983).

- Consumption of petrol and oil is higher if average speeds are drastically reduced.
- Journey times are increased.
- If delays are very long, then some goods may lose their value in transit (e.g., fruit and newspapers).
- The occupants of the vehicles lose the opportunity cost of the extra time they spend in transit.

The above are critical if the road network is experiencing a large degree of frequent flooding (from river or sewerage system). Small localised flooding incidents, during wet weather, will not cause large indirect damages to develop. It is generally not worth assessing traffic disruption benefits unless the road affected is a heavily utilised road and is flooded at regular intervals (Parker et al, 1987). The indirect damages are those caused through interruption and disruption of economic and social activities as a consequence of flooding and thus might also be termed “consequential losses” (FHRC, 1983).
The definition of indirect loss must be examined and the differentiation between financial loss and economic loss explained. An example defines this difference appropriately. A firm manufacturing bottled water in a developing country may suffer flood damage to manufacturing ability to produce the product. This will result in a financial loss to the firm which may be severe depending on the depth of inundation.

If the demand on this firm to produce bottled water can be quickly and efficiently met at the same costs by a rival firm then the economic indirect losses to the receiving community will be low even though the financial losses to the flooded firm may be substantial. In general, the more specialised a service, and how much that service is valued and difficult to replace, the greater the indirect losses will be. The ratio of indirect damages to direct damages tends to be lower when considering highly specialised industries.

Overall, indirect losses will generally be low in relation to direct damages when retail, office and housing forms the majority of the properties affected. Indirect losses can contribute a relatively large proportion of the losses in industrial areas, or where a significant fraction of the capacity of the road network is affected (Penning-Rowsell and Green, 1990). The indirect losses associated with shallow sewage flooding will be mostly concerned with road flooding.

As the occurrence and nature of flooding within many catchments from the sewerage system appears to be localised and shallow, direct damages will be small and indirect damage even less. Hydraulic models of sewerage systems can pinpoint the floodable areas at risk, and the frequency of flooding can be examined through the application of rainfall events (design or historical).

The important criterion in many cases is the frequency of occurrence, rather than the damage costs associated with sewage flooding. This may not be the case in some catchments where frequent flooding causes large degrees of damage. Specific cases must be dealt with on their own merits.
4.3.4 **Tangible And Intangible Damages**

Damage can be further classified into “tangible” and “intangible”. Tangible damages can be evaluated in monetary terms and those which cannot are classified as intangible. The intangible damages have proven in the past very difficult to evaluate with confidence.

Research by others (Green *et al*, 1987) in this area suggests that after allowances for the interdependencies between the severity of the different impacts, the non-monetary impacts of a flooding incident are about twice as important as the value of physical damage caused to the household and its contents.

Intangible damage includes factors such as stress, anxiety and ill health resulting from the intrusion of flood water. These parameters must be evaluated, because if they are ignored as a result of not being easily valued, a full account of the damage will not be accounted for within a benefit cost analysis. A method which can be adopted for deriving monetary equivalents of non-monetary impacts is that of “bootstrapping”.

Bootstrapping requires a detailed interview questionnaire with flood victims. This involves deriving a relationship in the form of an equation between the subjective severity of direct damage cost and the actual financial loss suffered. If this can be achieved the equation can be inverted and the severity entered for the indirect damage costs incurred during a flood event, and thus the financial equivalent be derived (Green and Penning-Rowsell, 1988).

The floods analysed in Table 4.2 range from very severe sea flooding to shallow to frequent sewage flooding (Parker *et al*, 1987). The approach of bootstrapping would be difficult to apply to incidences of sewage flooding unless the flooding occurred frequently.

Also, appraising severity relies on subjective analysis which can cause ambiguities dependent on the judgement exercised by respondents. It is interesting to note that the above survey agrees more readily on the severity of direct damage (to house structure) than that associated with intangible aspects (stress of flood).
<table>
<thead>
<tr>
<th>Criteria</th>
<th>Swalecliffe</th>
<th>Uphill</th>
<th>Southgate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damage to house structure</td>
<td>5.0</td>
<td>5.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Damage to replaceable contents</td>
<td>9.0</td>
<td>7.0</td>
<td>0</td>
</tr>
<tr>
<td>Loss of memorabilia</td>
<td>10.0</td>
<td>7.0</td>
<td>-</td>
</tr>
<tr>
<td>Health effects</td>
<td>7.5</td>
<td>5.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Stress of flood itself</td>
<td>10.0</td>
<td>N/A</td>
<td>6.5</td>
</tr>
<tr>
<td>Evacuation</td>
<td>10.0</td>
<td>6.0</td>
<td>-</td>
</tr>
<tr>
<td>Disruption</td>
<td>10.0</td>
<td>10.0</td>
<td>6.0</td>
</tr>
</tbody>
</table>

N/A = not asked
- = no household suffered impact
Score based on scale 1 to 10 (0 no impact, 10 extreme impact)

Table 4.2 Relative Severity Of Different Impacts Of Flooding

Non monetary impacts have been shown to depend on five groups of variables (Green, 1988):

- The characteristics of the flood events:
  depth, rate of rise, time of the year at which the flood occurs, whether a warning is received, degree of contamination of the water, and duration of flooding.

- The characteristics of the individual household affected:
  income, prior health status, personality type, social competency, insurance coverage, household age structure, degree of social support, proportion of able bodied adults in the household.

- Prior experience of flooding.

- The characteristics of the dwelling unit affected.

- Degree of flood susceptibility:
  i.e., basements, caravans and bungalows

Many complex variables are involved when non-monetary impacts are to be assessed relating to flooding. Even if all the relevant criteria were generated, the site specific nature and public perception of the problem would lead to varying results. If the bootstrapping approach was to be carried out it needs to be directed at each problem area. Results or equations developed in one catchment may not
be directly applicable to another, i.e., any technique should take into account the people who are to be affected in the way that utility can. In the case of flooding from a sewerage system, many of the drastic events attributable to river inundation detailed above do not apply.

Intangible benefits, although not normally evaluated, form a large part of the benefits of such work. Indeed MAFF guidelines for river flood defence schemes, on the whole only take tangible benefits into account in any analysis (MAFF, 1993). This approach is suitable for technical appraisal of options for defence, but any defence system must also protect people and give “peace of mind” (a direct benefit but intangible).

Intangible benefits which are difficult to assess are particularly relevant to water pollution abatement. Any methodology seeking to produce an effective solution to a wastewater system problem must consider pollution and the damages associated with it. This is discussed in the following section.

4.4 DAMAGE ASSOCIATED WITH POLLUTION GENERATED FROM WASTEWATER SYSTEMS

The cost associated with the pollution of watercourses is generally intangible in nature and can frequently be classified as subjective, as the perception of pollution can vary entirely from one bankside user to another. Tangible elements can be evaluated when the costs of water treatment are considered, fishery revenue can be examined and the costs of clean up operations after CSO discharges are determined.

It is necessary to examine the nature of such damage before progressing to attempts at evaluation. It is proposed that pollution from wastewater systems may result in:

- decreased recreational and amenity use of a watercourse;
- a decrease in the value of fisheries;
- increased costs of potable water treatment;
- increased possibility of transmission of waterborne disease;
• smell associated with degrading sewage material;
• aesthetic impairment and
• the degradation of aquatic ecosystems.

4.4.1 Decreased Recreational And Amenity Use

If a watercourse suffers from acute and chronic pollution there will be drastic
effects upon the quality of the water and any life systems which depend upon this
quality will be affected dependent upon the degree of pollution. If the general
public are attracted to the watercourse for reasons related to the flora and fauna,
and this attraction degrades or ceases to exist, then enjoyment of the attraction
will decrease. If revenue is based upon the visiting public (e.g., brown trout
fishery) then a loss could occur which would be tangible. There are techniques
which exist (and are discussed later) to evaluate this damage but in general terms
this proves very difficult as evaluation of loss in these circumstances is variable in
value and personal.

Generally watercourses in Scotland, as elsewhere are seen as a public “good” (in
the economic sense), something to be enjoyed, and not at an excessive cost to the
person enjoying the recreation. Should there be a public contribution towards the
cost of controlling river pollution, to keep an appropriate stretch of watercourse
to the level of service required for optimum recreational use?

4.4.2 Decreased Value In Fisheries

This can be split into inland watercourse and coastal areas. Fisheries in Scotland
which raise a financial income inland tend to be based around salmonids (salmon,
rainbow trout, brown trout and sea trout). The River Tay is a typical example of
this. People travel from all over the world to fish here and will pay extravagant
costs. If pollution begins to materialise for any reason, the migratory fish will
decline steadily over a period of time. Any revenue based on the presence of these
fish, and the quantity of that presence, will be affected in the long term by the
level of pollution that occurs.
Coastal fisheries also suffer from the effects of pollution. Shellfish businesses are directly affected by the quality of water in which the crop lives and breeds. The presence of heavy metals, generated from sewage pollution, within the flesh of mussels and scallops can render them unsuitable for sale and consumption. A positive tangible benefit can be achieved by removing the pollution problem and allowing harvesting to recommence.

4.4.3 Increased Costs Of Potable Water Treatment

Where there are CSO and WWTP discharges above a water intake in a river system, which has generally low volumes of flow, then extra costs above that required to treat good quality supply water will be incurred. If action is taken to remove or improve the quality of upstream discharges, then a benefit is accrued in terms of the amount of chemicals and time taken to produce potable water to the level required by regulatory standards.

In Perth no upstream discharges affect the quality of the Tay due to the massive assimilative capacity of the river, and thus the costs of treating supply water will vary, but not according to the variation in upstream pollution, but because of the nature of the river water and the amount of sediment transported. In certain site specific cases however, there may be tangible benefits from the removal of upstream discharges in relation to the costs of potable water treatment.

4.4.4 Increased Possibility Of Waterborne Disease

In general the spread of waterborne disease within inland watercourses has been controlled, through the application of disinfection and chlorination techniques, applied to water treatment works to ensure the spread of disease is impossible. If large pollutant loads are discharged to watercourses that are not used for water supply and these areas are used for water contact sports then the risk of disease may be significant.
Social benefits are accrued if the pollutant discharges are reduced/treated and the
risk of waterborne disease is minimised. The problem of waterborne disease is rife
in developing countries, where infected water supply systems from untreated
sewage discharges, have a far greater effect upon the spread of disease than they
do in developed countries.

Even in developed countries waterborne disease is evident, due to coastal
discharges. Many surfers, swimmers and yachtsmen experience ear, eye and more
serious infections as a result of contamination from viruses in coastal waters.
Present European standards relate to total coliforms and faecal coliforms and not
viruses.

The problems of coastal discharges are being addressed as part of water
authorities programmes to meet the UWWTD. The implementation of coastal
treatment will deal with the pollutant potential of discharged loads to coastal
waters. Water Authorities argue that only a few people have been affected by
disease from contact with the receiving coastal water. This introduces the
important concept of the social value of health. A question could be asked as to
the value placed on health, and the requisite number of people to be infected
before a water authority would implement remedial measures, if this were not
enforced by the UWWTD.

Waterborne disease and the effects on population raises social questions and
values related to peoples health. This is intangible and hence requires subjective
criteria to be appraised. With respect to Perth, and the majority of watercourses in
the UK the problems of waterborne disease (and the benefits associated with
eradication) do not exist due to sufficiently high water quality of the watercourses
used for contact sports and recreation.
### 4.4.5 Aesthetic Pollution Of Watercourses

Aesthetic pollution includes typically slicks of sewage, fish kills, coloration, and visible solids derived from CSO spills. Aesthetic pollution appraisal of watercourses can also include the presence of land derived waste within the watercourse. Visible solids are material which is identifiably sewage in origin and would be noticed by a casual observer on a river bank (Jefferies, 1992). Examples of sewage derived litter are shown in the table below (FWR 1993b).

<table>
<thead>
<tr>
<th>Examples of items of sewage derived litter which may be found in rivers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tampon Residues</td>
</tr>
<tr>
<td>Other Sanitary products including backing strips</td>
</tr>
<tr>
<td>Nappy Liner remains</td>
</tr>
<tr>
<td>Grease Balls</td>
</tr>
<tr>
<td>Other plastic items</td>
</tr>
<tr>
<td>Rags</td>
</tr>
<tr>
<td>Faeces</td>
</tr>
<tr>
<td>Cotton Buds</td>
</tr>
</tbody>
</table>

Table 4.3 Examples Of Items Of Sewage Derived Litter

With the likelihood of standards being set for aesthetic pollution (Seager, 1993), it is necessary to consider the perception of this problem and the associated damage. The perception of aesthetic pollution is a variable criterion and consequently is perceived differently by bankside users. Recent work carried out by the Foundation for Water Research, regarding aesthetic pollution, has drawn the conclusions below, regarding the problem and its public perception (FWR 1994b).

- The main sources of riverine litter are fly-tipping and from sewage inputs. The majority of such litter is plastics.
- In England and Wales the NRA receive numerous reports of pollution incidents from the general public which are unlikely to cause environmental problems.
- The visual state of the water, is the most important factor in influencing the public’s perception of water quality. The presence of gross solids on river banks and beaches have far less impact than their presence in the water.
• Sewage derived contaminants have a much greater negative impact on the public’s enjoyment of a visit to a river or beach than any other aesthetic pollution indicator.

• Sewage derived products are not regularly seen, or at least not recognised by the public. There is also an unwillingness to talk about products such as sanitary towels and condoms. However, when identified, condoms appear to have less of a negative impact than sanitary products when seen on a beach or river bank.

• A large proportion of the public do not associate the presence of sewage derived material in the water or on the beach or river bank as coming from the water.

• The public considered that more, or improved sewage treatment would be the most effective solution to the problem of sewage derived contamination. Legislation and improved consumer awareness would also be effective.

• Aesthetic pollution needs to be controlled at source.

It is hard to believe that sewage derived sanitary products are not recognised by the public, as the majority of men and women all use, or are aware of their partners using condoms and tampons. The last statement in the above list is very hopeful, although correct in principle. Sewage derived material can be controlled to an extent by “bag and bin it” campaigns but the practicality of these still has to be proven.

Obvious benefits can immediately be seen if this type of disposal route was chosen. It is likely that less aesthetic pollution would appear in watercourses and on beaches. However, convincing the public to follow this disposal route involves changing their attitude towards the status quo. This will only be achieved over generations and through education of the young and old alike.

In reality, even if large percentages of the population participated in bag and binning their aesthetic material, Water Authorities would still provide screens on CSOs to prevent the discharge of gross solids and to meet EU requirements. Rehabilitation of sewerage systems clearly results in benefits with respect to water
quality. A summary of the benefits associated with improvements in water quality is shown in Table 4.4 (Green, 1992). From the table it can be seen that the major benefit is associated with out of stream recreation and non-use values.

<table>
<thead>
<tr>
<th>Category of Benefits</th>
<th>Magnitude of benefits</th>
<th>Variance between watercourses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abstraction for irrigation</td>
<td>low</td>
<td>high: average value low because few watercourse can support this usage</td>
</tr>
<tr>
<td>Abstraction for potable supply</td>
<td>low</td>
<td>high: high value where technically feasible</td>
</tr>
<tr>
<td>Abstraction for cooling</td>
<td>low</td>
<td>low: not technically feasible in many areas</td>
</tr>
<tr>
<td>Hydropower uses</td>
<td>low</td>
<td>low relatively low: instream uses inhibited by physical characteristics of rivers.</td>
</tr>
<tr>
<td>Instream recreation</td>
<td>low</td>
<td>Relatively low number of such visitors as anglers and canoeists variance determined by number of visitors-very large numbers of visits made to average river corridor for picnics and walks</td>
</tr>
<tr>
<td>Out of stream recreation</td>
<td>high</td>
<td>high: development benefits in urban centre may be very large</td>
</tr>
<tr>
<td>Amenity value to</td>
<td>relatively low</td>
<td>high: development benefits in urban centre may be very large</td>
</tr>
<tr>
<td>neighbouring land uses</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Commercial fisheries</td>
<td>low</td>
<td>low</td>
</tr>
<tr>
<td>Navigation</td>
<td>low</td>
<td>low</td>
</tr>
<tr>
<td>Non-use value</td>
<td>high</td>
<td>not known</td>
</tr>
</tbody>
</table>

Table 4.4 Relative Magnitude Of Different Categories Of Benefit

Perception is based on sight and smell and the general public have a good idea of what is poor water quality and in many cases a river of good quality can be seen as polluted when the public tend to base their assumptions in terms of watercourse attractiveness.
The general public are becoming increasingly more aware of pollution aspects relating to the water industry, and of the major impact aesthetic pollution has upon the use of a watercourse for recreation.

Significant studies on the subject of perception have been carried out by the Flood Hazard Research Centre at (the former) Middlesex Polytechnic. A person's perception of water quality is largely influenced by their relationship with the watercourse, education, social status and what their individual interest is in environmental issues. The factors affecting the public's perception of water quality are listed in Tables 4.5 and 4.6 (House and Sangster, 1991).

<table>
<thead>
<tr>
<th>Indicators of good water quality</th>
<th>Indicators of poor water quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>Many fish</td>
<td>Muddy water</td>
</tr>
<tr>
<td>Can see the river bottom</td>
<td>Rubbish on banks</td>
</tr>
<tr>
<td>Adults fishing</td>
<td>Water an unusual smell</td>
</tr>
<tr>
<td>Kingfishers</td>
<td>Dead fish</td>
</tr>
<tr>
<td>Waterfowl</td>
<td>Water an unusual colour</td>
</tr>
<tr>
<td>Plants in river</td>
<td>Oil in water</td>
</tr>
<tr>
<td></td>
<td>Protruding rubbish in river</td>
</tr>
</tbody>
</table>

Table 4.5 Public Perception Of Water Quality

The effect on the general public, of the presence of aesthetic pollution within a watercourse is variable and to some extent subjective. Bankside derived waste affects perception drastically and may result in a lowering of the recreational and amenity value of a watercourse.

Much of the damage associated with pollution of watercourses is variable in value and intangible in nature, and as a consequence extremely difficult to evaluate in monetary terms. Studies from European countries have indicated that the costs associated with water pollution are not trivial and demand attention. Table 4.7 shows estimated costs associated with water pollution.
<table>
<thead>
<tr>
<th>Primary indicators</th>
<th>Secondary indicators</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Good Water Quality</strong></td>
<td><strong>Good Water Quality</strong></td>
</tr>
<tr>
<td>Can see the river bottom</td>
<td>Adults fishing</td>
</tr>
<tr>
<td>Many fish</td>
<td></td>
</tr>
<tr>
<td>Water fowl</td>
<td></td>
</tr>
<tr>
<td><strong>Poor Water Quality</strong></td>
<td><strong>Poor Water Quality</strong></td>
</tr>
<tr>
<td>Protruding rubbish in the river</td>
<td>Green Scum on surface</td>
</tr>
<tr>
<td>Foam on water</td>
<td>Rubbish on banks</td>
</tr>
<tr>
<td>Water an unusual smell/colour</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.6 Primary And Secondary Water Quality Indicators

<table>
<thead>
<tr>
<th>Country</th>
<th>Damage per year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Germany potable water (1991) groundwater (1986) measurable damage to rivers and lakes</td>
<td>780 DM million 4.1 to 6.9 DM billion in excess of 17.6 DM billion</td>
</tr>
<tr>
<td>Italy (1974) coastal waters inland waters</td>
<td>6 billion lire 19 billion lire</td>
</tr>
<tr>
<td>France (1978)</td>
<td>10150 FF million</td>
</tr>
<tr>
<td>The Netherlands (1990)</td>
<td>300-930 Dfl million</td>
</tr>
</tbody>
</table>

Table 4.7 Estimates Of Water Pollution Costs


Benefits accrue if the damage is prevented. Quantification of these environmental benefits within a methodology is difficult. A number of techniques have been applied to environmental economics and these are considered in the following section.
4.5 TECHNIQUES AVAILABLE TO EVALUATE DAMAGES FROM WATER POLLUTION

The economic benefits of water pollution improvement arise from two main sources in the UK, the increased recreational and amenity use value of the particular river corridor and through benefits associated with “non-use” values (Green and Tunstall, 1990a). Use values are clearer and easier to define. These are values which are associated with a change in enjoyment experienced by visitors due to an improvement in water quality of the watercourse they are visiting. Non-use values are motivations which exist to preserve a “good”, even when the public do not visit the watercourse for amenity or recreational use.

Before any evaluation of environmental improvement can take place it is first necessary to determine those characteristics of river water quality which can affect the visitor’s enjoyment and how the public assess water quality (Burrows and House, 1989). User benefits of sewerage rehabilitation and water quality improvements can only be assessed provided the public can directly see that an improvement has occurred or their enjoyment increased because of a reduction in flooding or pollution. Standards such as the intermittent discharge standards proposed by WRc (FWR 1994a), can be used to determine scientifically the acceptable level of discharges during wet weather on watercourses. Recreational users of the watercourse may not see the effect of a reduction in dissolved pollutants such as, un-ionised ammonia in the river for periods of time, but they can appreciate an improvement in terms of a reduction in the aesthetic pollution associated with a reduced number and volume of spills.

Generally, the change in individual enjoyment is small when measured in monetary terms, however, in many cases due to the large number of visitors to a popular river corridor the overall benefit after, say CSO rehabilitation, is large. If a river corridor suffers environmental damage then visitors will either continue to visit and experience a reduction in enjoyment, or visit an alternative location which gives some enjoyment at some cost. In the first case, the economic loss is simply the value of the reduction in enjoyment. Where visits are transferred to another
site then the economic loss is the difference for the two sites in enjoyment value net of the costs incurred making the visits (Green and Tunstall, 1990b). The gains resulting from an increase in enjoyment from water quality improvement can be evaluated similarly, however, it is more difficult to assess the increase in enjoyment associated with visitors being attracted to the improved site and away from another site.

In the case of environmental goods economists have theorised that non-use motivations exist. Thus an individual may be willing to pay to preserve a site in the hope of visiting it at a later point in time (option value); to preserve the good for later generations of their family (bequest value) or because they derive a good for simply knowing that the good exists (existence value) (Krutilla, 1967). Figure 4.1 shows non-use and use values, and gives examples of each.

The existence of these non-use values makes the application of neo-classical economics, which deals mainly with the supply and demand of private goods, not suited for the assessment of environmental criteria. Thus methodologies have evolved for the evaluation of environmental goods. Table 4.8 summarises these techniques (Green, 1992).

![Figure 4.1 The Elements Of Total Economic Value (Hodge, 1995)](image-url)
The recreational benefits of water quality improvements are substantial to the users of the watercourse. These benefits have been defined as increased pleasure to visitors of the river, the benefits associated with an increase of visitors to the watercourse and amenity increases to developments neighbouring the watercourse.

Economic evaluation of improvements in water quality are hard to quantify but may be assessed. In defining the economic benefits of river quality improvement the engineer is faced with four main criteria. What is the good of which they desire more or less, who benefits, why do they benefit and how may this benefit be evaluated?

Three methods from Table 4.8 may be considered applicable. The suitability of these methods in relation to wastewater catchments is discussed. These methods are Travel Cost, Hedonic Pricing and Contingent Valuation.

<table>
<thead>
<tr>
<th>Technique</th>
<th>Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shadow Prices</td>
<td>Market prices adjusted to reflect opportunity costs: where market prices exist, easiest approach</td>
</tr>
<tr>
<td>Least Cost Alternative</td>
<td>A bundle of techniques, including &quot;aversive expenditure&quot; and &quot;Shadow Project&quot;, often useful when no other technique is suitable. Based upon the principle that the value of providing a good by one means cannot exceed the cost of the cheapest alternative method of providing that good</td>
</tr>
<tr>
<td>Travel Cost Method</td>
<td>Only applicable to the evaluation of recreational benefits. Of questionable use in the UK as a significant proportion of recreational travel is on foot</td>
</tr>
<tr>
<td>Hedonic Price Method</td>
<td>If an environmental good might be expected to be reflected in house prices, then potentially its value could be extracted by this means. Theoretically sound, its applicability in the UK housing market has still to be established. Theoretically, could be used to separate out other components of value where a good offers a bundle of characteristics (e.g., to value fuel conservation in cars)</td>
</tr>
<tr>
<td>Contingent Valuation Method (CVM)</td>
<td>The application of social science methodology to asking individuals what value they place on a good. Appears to work extremely well for use values</td>
</tr>
</tbody>
</table>

Table 4.8 Techniques For The Economic Evaluation Of Environmental Goods
4.5.1 The Travel Cost Method

This approach is limited to the assessment of the recreational value offered by an environmental good through the relationship with another set of attributes. The other set being time and money expended by the party to receive the good. The basic principles of this method are simple in that the number of visitors is multiplied by the costs incurred to reach the good. The method is often used for the provision and management of outdoor recreation. A recent study (Willis, 1991) used the method to assess the recreational value of the Forestry Commission in Great Britain.

The area around the recreational site is divided into concentric zones of equal travel costs. Assumptions are made that all travel costs from the same zone are similar, and that all other factors which may influence travellers to visit the site, or not, are similar. Data are then collected on the costs incurred by visitors travelling from each zone and from where they have travelled. The second stage is to estimate the relationship between the cost of travel and the proportion of the population from each concentric zone that visited the site. This produces what is known as the trip generating function. This is shown in Figure 4.2.

![Figure 4.2 Trip Generating Function](image)

This function is utilised to produce a "demand curve", shown in Figure 4.3, for the recreational site and a consumer surplus can be derived. This surplus is the total willingness to pay for the use of the site.
This demand curve is produced by calculating the number of visits for a range of possible entry fees. Various entry fees are levied and the consequential reduction in visits estimated from the trip generating function for each zone. From this it is then possible to draw the demand curve. The complete area under the curve represents the total value for the site.

![Demand curve](image)

**Figure 4.3 Demand Curve For Recreational Site**

This method can be applied in the case of wishing to determine the existing recreational value of a river or stream. Indeed, it is plausible to use the approach after environmental decline or improvement, to determine the change in consumer surplus. This will be directly related to the level of environmental change.

The approach must be used with caution if used to predict new consumer surplus due to increases in visitor numbers and costs associated with a particular level of improvement provided, before the improvement is implemented. This would require the analyst to estimate the increase in visitors and associated costs.

Many visitors travelling to urban watercourses receiving CSO discharge do so by foot, and thus the application of travel cost would be difficult to justify in many cases. The basis of the method assumes that the costs of travelling reflect the strength of the good to be gained. Can this mean that if no-one visits a watercourse or incurs zero costs in reaching it, then no value is held for the watercourse?
Thus the Travel Cost methodology while being valid in other areas of environmental evaluation is not wholly applicable to engineers considering effective rehabilitation strategies associated with urban watercourses. There may be cases where the methodology can be applied to assess the increase in benefits associated with water pollution improvement post rehabilitation. This would provide information regarding the effectiveness of the rehabilitation with respect to the numbers of visitors to the site.

4.5.2 Hedonic Pricing Method

This method relies on using a proxy attribute to help evaluate the criteria being investigated. Recent work (Perman, 1995) suggests using house prices to evaluate the importance relating to clean air. The method is to establish a relationship between house prices and the quality of ambient air standards through regression techniques. Similar work (Pennington et al, 1990) has been carried out regarding house prices and noise pollution in Manchester. Earlier work carried out in Australia (Abelson, 1979) related property values to traffic noise and property outlook.

The methodology involves developing a statistical relationship between attributes associated with the properties (independent) and the price of the property (dependent). The independent variables can consist of size, number of rooms, neighbourhood, housing density and the environmental variable (air quality, noise or water quality). A large sample of properties must be assessed and a statistical analysis is carried out to derive a relationship between the dependent and independent values. Thus the relationship can be used to determine the result of small changes in the environment variable while holding all other variables constant. If the intention is to evaluate large changes in the environmental attribute then a demand curve requires to be evaluated.

With reference to water quality, it is probable that those living in properties bordering rivers do so, as they are prepared to pay for the level of environmental quality offered by the watercourse. If a watercourse is to be upgraded, through
CSO elimination, from Class 3 to Class 2 then applying Hedonic Pricing would require the definition of a relationship between a river quality parameter, say BOD and the housing characteristics. This may be possible, but is unlikely, as the methodology can result in no statistical relationship being derived, as was found in the Manchester study. No significant statistical relationship was found between levels of noise and house prices.

Thus any benefit calculated from environmental evaluation by this manner should be treated with caution. Thus the application of Hedonic Pricing to assess the benefits of water pollution abatement cannot be recommended as the best approach.

4.5.3 The Contingent Valuation Method

The CVM approach is based on the application of social survey techniques which ask the public how much they value a visit to the particular good under examination, and consequently assess the increase in the value of the good following remedial measures.

This technique is the most popular of all available methodologies for the evaluation of environmental benefits, and is the only one capable of determining non-use values. Table 4.9 summarises results of CVM surveys carried out in the UK on the recreational value of environmental resources (Green et al, 1992).

<table>
<thead>
<tr>
<th>Environmental Criteria</th>
<th>Mean value per visit by an adult</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rivers</td>
<td>£80p</td>
</tr>
<tr>
<td>Canals</td>
<td>£40p</td>
</tr>
<tr>
<td>Forests</td>
<td>£60p</td>
</tr>
<tr>
<td>Beaches</td>
<td>£7 to £8</td>
</tr>
</tbody>
</table>

Table 4.9 Recreational Value Of Environmental Resources In The UK

Contingent Valuation has so far proved to be the most appropriate methodology for the evaluation of environmental goods and has been applied to determine such criteria as the damage inflicted upon the environment from the Exxon Valdez oil
spill (Carson et al, 1992). As a result, it has had much criticism. The method concentrates on asking carefully structured questions and analysing the response by the participants regarding the value they place on a good, and how much they are willing to pay for some degree of change or improvement. The method is open to question as many people when asked to value some good (that they have probably not thought about before) find it difficult, and clearly people of different backgrounds may answer differently to the same question.

For public goods the total benefit is simply the average value of good per person multiplied by the number of people affected. Problems occur when trying to assess the economic benefits of use values and non-use values, or intrinsic values. The evidence from studies carried out in the past is that non-use benefits may on occasion exceed use benefits. Studies have also shown, that there are major problems when trying to place confidence in the reliability of methods which elicit non-use values.

Generally, non-use values are evaluated separately from use values due to the difficulty in trying to define non-use criteria. Surveys carried out have shown that the general public put a high value upon non-use motivations, but dividing lines between non-use and use motivations are hard to derive, due to the problem of specifying the split between the non-use and use value a particular good has. Generally non-use values are derived by interviewing members of the public who do not have a direct relationship with the environmental area under evaluation.

Moral beliefs tend to rank highly in the minds of the public “We should not cut down rain forests in the Amazon Basin, or kill whales for oil”. These moral beliefs affect the public’s opinion on water quality issues. A classic example was when the oil tanker Braer ran aground in the Orkney Islands and spilled thousands of gallons of crude oil and seriously endangered aquatic eco-systems. Many people when voicing their opinion on the matter stated that this was a disaster and put a value of good on getting the slick cleaned up.
Even though many people were not affected by this disaster a good was placed by a large percentage of the population on cleaning up the environment around the area of the spilled oil. Clearly, people place moral value on matters which do not directly affect them. The author while writing this Chapter carried out preliminary communications to ascertain the general public’s attitude regarding pollution of watercourses. Many people answered that pollution should not exist, whether they used the particular watercourse in question or not. A clear example of non-use value and very difficult to evaluate.

It is clear that characteristics of an environmentally important site which give it particular ecological value may not be perceived to be economically important. Thus, before goods can be assessed, it is necessary to have established the characteristics of the good which are perceived to be important. This applies to any question of perception to any criteria. A benefit must be perceived to have occurred or no benefit is appreciated (Green et al, 1989).

CVM appears to work well when applied to many environmental cases, as willingness to pay/willingness to accept compensation, can be determined for any environmental issue. However, the technique utilised in preparing and carrying out structured interviews is very important to eliminate possible errors, bias and to ensure correct dissemination of information to the participants.

Studies have often found differences between willingness to pay and willingness to accept compensation, and this remains a bone of contention with many analysts. One such study is outlined below. Also, the amount people will offer in terms of payment is directly affected by their income, and perception of the question at large. In many cases when comparison has been carried out between hypothetical studies and real choices, differences are apparent.

One study (Bishop and Heberlein, 1979) carried out a Contingent Valuation study of duck hunting permits in Wisconsin and found a willingness to pay for the permits ranging between $11 to $21, and a willingness to accept compensation for surrendering permits of $68 to $121. Subsequently real cash offers were
presented to hunters and it was found that the mean actual willingness to accept compensation was $63. This suggests bias in the hypothetical willingness to accept compensation for this loss of permits. In conclusion the CVM approach has been applied in many studies, and has been accepted by Courts of Law in the USA for the evaluation of environmental damage.

CVM is the only methodology available to evaluate non-use values associated with environmental economics, although this is not wholly recommended due to the complex issues associated with these values. The methodology should be used with care and great skill is required in structuring the questions and deriving responses. As a consequence CVM studies tend to be expensive. This may preclude the methodologies application in many urban drainage studies.

4.5.4 Discounting Of Environmental Benefits

The application of economics requires that benefits be calculated based on the population which experiences the gain or loss of enjoyment. Evidently, there must be a relationship between what the benefits are biochemically, as determined by an "expert", and how these biochemical improvements interlink with identifiable benefits in terms of the public’s perception of river corridor quality.

Application of BCA raises issues of discounting environmental benefits. Discounting of costs or benefits involves reducing costs or damages occurring in the future to the present day. This is because an individual will generally put more value on a benefit which is available to him/her, than if the same benefit is available in twenty years time.

The extent of this preference is reflected in the calculation of the so-called discount rate, and can include changes in inflation (WAA/WRC 1986). As a result benefits or costs occurring far into the future may be substantial, but when discounted, prove to have a very minor impact on the benefit-cost ratio of the project.
Considerations of inter-generation equity, the rights of the present generation to sacrifice the opportunities of future generations, have led to proposals that a different discount rate should be used for environmental goods.

Growth factors can be applied to environmental benefits to allow for changes in time. It is evident that a particular environmental good may not become apparent well into the future of the project, such as the abolishing long established chemical dumping to a watercourse or the effect of abolishing sludge dumping to the North Sea after 1998.

The value of benefit derived from projects may increase or decrease depending on the time period that good is examined over. As the availability of a particular good decreases its value will increase. This growth and recession is difficult to assess, and is even more difficult to evaluate accurately, and allow decisions to be made reflecting the most effective course of action to be taken.

The previous discussion highlights the problems in identifying benefits associated with the alleviation of flooding and pollution events. Many of the intangible benefits are the most important, but cannot be fully evaluated in monetary units. Many of the applications are detailed methodologies, such as CVM and could not simply be applied by an engineer when trying to solve a wastewater system problem faced with tight time and budgetary constraints.

Many of the techniques have had much criticism in terms of their applicability. Regardless of the methodology used, the analysis should clearly define the issues at stake. The UPM procedure is to be complemented by an assessment procedure, as previously detailed in Chapter 3, funded by FWR, OFWAT and the NRA. This assessment is based on BCA and suggests the use of any of the evaluation methods previously discussed to derive benefits. This is specifically aimed at water quality improvements and does not deal with other benefits relating to other performance parameters associated with the wastewater system.
Indeed the use of social sciences (CVM) is more suited in application to the consideration of benefits associated with beaches and highly used recreational areas, than to most small watercourses suffering CSO discharge.

The derivation of benefits associated with rehabilitation schemes is clearly a difficult task when intangibles and non-use values are considered. The application of BCA may only be applicable to the tangible benefits that can be evaluated. Even these, as shown, can be extremely difficult to evaluate in the case of flooding from a sewerage system. Generally, drainage rehabilitation works are not examined using BCA. The point shows the effect of not considering all the benefits within an analysis. Unless the tangible damages associated with the performance of the existing system far outweigh the costs, BCA should not generally be applied to urban drainage system analysis.

The issue of public peace of mind, whether as non-use values relating to water pollution or anxiety protection from flooding is an area which requires satisfaction. It cannot be done successfully through the application of standard techniques, as has been discussed, due to the difficulties in deriving values relating to intangible criteria. People perceive a problem in a different way depending on their background or knowledge of the problem in question. This is why conflict often arises between the informed engineer and the general public. Many of the techniques previously discussed to evaluate intangibles are complex, and must be carefully executed for meaningful results to be produced.

The issues raised in the preceding text can be directly related to moral values and multiple objectives. Non-use values are clearly important when water pollution is investigated and the intangible anxiety caused by sewage related flooding cannot easily be measured in monetary terms. Therefore a methodology which takes into account these factors is critical in the effective solution of drainage problems.

Economic efficiency is not now the only criterion for evaluation of rehabilitation projects, and as a consequence, the area of multi-attribute utility analysis is now examined for applicability in the approach to be developed and applied to Perth.
4.6 MULTI ATTRIBUTE UTILITY ANALYSIS

It is very difficult, as has been shown, to measure changes in water quality in terms of financial gain or loss, or indeed the disruption associated with shallow sewage flooding. The units (mg/l and volume or frequency) used to measure these criteria do not relate directly to monetary damage or improvement costs. They are difficult to encompass in any holistic methodology which requires monetary values as a measuring vehicle.

An alternative approach involves relating preferences of outcomes or scenarios on a suitable scoring regime. Almost any subjective or objective criteria can be investigated through the application of value functions (Edwards, 1971 and 1977) and multi-attribute utility analysis (Snell, 1994).

4.6.1 Value Functions And Water Quality Indices

The public perception of river or stream quality has been shown to depend upon the relationship of an individual with the watercourse in question and of their awareness to water quality problems (House and Sangster, 1991).

The need for a simple objective and reproducible numeric scale to represent aggregate water quality has been fundamentally recognised (House and Newsome, 1989), (SDD 1976), (Brown et al, 1972), (Harkins, 1974) and (Horton, 1965). These quality indices allow beneficial and detrimental effects upon watercourses to be evaluated by means of a simple aggregate function and value functions which take into account a specified range of chemical and biological determinands. To produce value functions relating to water quality requires numeric scales to be developed (O'Connor, 1972).

A recent study (House, 1988) developed a family of water quality indices based on value functions. These were termed Water Quality Index (WQI), Aquatic Toxicity Index (ATI), Potable Sapidity Index (PSI) and Potable Water Supply Index (PWSI). The WQI determinands investigated in the study were Dissolved Oxygen (DO), Ammonia (NH₄), Biochemical Oxygen Demand (BOD),
Suspended Solids (SS), Nitrates, pH, Temperature, Chlorides and Total Coliforms. Four stages were followed in the development of the index system:

(i) Determinand selection.
(ii) Transfer of determinands to a common scale.
(iii) The development of determinands to a common scale.
(iv) The selection of an appropriate aggregation function.

The aggregate function used (House, 1988) is shown in Equation 4.1.

\[
WQI = \frac{1}{100} \sum_{i=1}^{n} (q_i w_i)^2 \quad \text{Equation 4.1}
\]

\( q_i \) = represents the rating for the ith determinand
\( w_i \) = represents the weighting for the ith determinand
\( n \) = represents the number of determinands

Of the four stages of development, determinand selection and rating curve growth are the most important. Rating curves (or value functions, an example is shown below) are used to relate determinand values (e.g., concentration of BOD sampled) to values of environmental quality (based on an arbitrary scale). Environmental quality scores are multiplied by weightings and a WQI score is calculated based on the aggregate function.

Figure 4.4 Value Function For Dissolved Oxygen
Similar work has been carried out on the quality of Acid Mine Drainage (Gray, 1996) where a water quality index was used to classify mine drainage water. Most studies agree on the use of the aggregation equation proposed by the (SDD 1976) and used by House for developing water quality scores as it allows more discernment between high quality waters than by utilising a simple weighted average.

This type of approach effectively allows reproducible scores to be produced which can represent the quality of watercourses by taking into account all sampled parameters. This approach can be used in a monitoring role to determine the environmental quality of river reaches at present (from routine sampling of determinands) and any changes in that quality due to modifications in the discharges to the river from WWTPs and CSOs. From House's work WQI scores are related to river classifications. Thus the WQI can be used to show the particular quality of a study reach in terms of the National Water Council (NWC) classification scheme as shown in Table 4.10.

The NWC expresses non-statutory river quality objectives in terms of the rivers use. This classification system is being replaced by statutory water quality objectives for watercourses in England and Wales. The new classification system is split into two: (i) different use classes (UCs) for setting targets relating to the actual or proposed use of the water, (ii) A general quality assessment (GQA) scheme for assessing overall progress on a periodic basis.

The WQI system is extremely useful for monitoring watercourses over time. While the intermittent standards proposed by WRc (see Appendix B) are useful as a basis for design, the application of the WQI can show how a rehabilitation option affects a watercourse in reality over a period of time. Thus, value functions of this form can relate biochemical improvements integrally and reflect the success of sewerage rehabilitation on a watercourse.
The use of an index system and value functions has been used to investigate other environmental criteria. One such system, used to evaluate environmental impacts, is that of the Environmental Evaluation System (EES) (Dee et al, 1973). The EES provides ways of measuring environmental impacts of varying solutions to environmental problems in units known as Environmental Impact Units (EIU). In the EES, environmental quality is defined as lying between 0 and 1, 0 being very bad quality while 1 being very good quality. The transfer of a real value for a determinand to a value of environmental quality is done through a value function, as with the WQI.

<table>
<thead>
<tr>
<th>NWC classification</th>
<th>WQI score</th>
<th>Quality Narrative</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>71-100</td>
<td>Indicates river quality suitable for all high value uses including Potable Water Supply, game fisheries, contact recreation and high quality industrial abstractions at low treatment cost</td>
</tr>
<tr>
<td>2</td>
<td>51-70</td>
<td>Indicates water of reasonable quality suitable for high value uses including Potable Water Supply after conventional treatment, good coarse fisheries, indirect contact sports and most industrial abstractions at moderate treatment cost</td>
</tr>
<tr>
<td>3</td>
<td>31-50</td>
<td>Indicates polluted water with generally moderate value uses including Potable Water Supply after advanced treatment, indirect contact sports and reasonable to sporadic coarse fish populations</td>
</tr>
<tr>
<td>4</td>
<td>10-30</td>
<td>Indicates badly polluted waters of low economic value requiring large investment in treatment facilities if they are to be upgraded. Usually restricted to non-contact recreational uses, sewage transport and navigation.</td>
</tr>
</tbody>
</table>

Table 4.10 Relationship Between WQI Score And NWC Classification
As with the WQI, the shape of the value function is very important. Value function development must be done from consultations with chemists, biologists and engineers to achieve the most realistic and accurate rating curve. In the EES, weightings are given to each of the determinands to show their relative importance to one another and their effects on the environment.

This approach is open to criticism by those who state that the rating curve development can be flawed. Careful procedures must be followed to ensure that the curve developed is correct before being utilised in the analysis.

\[ \sum_{i=1}^{m} (V_i)_1 W_i - \sum_{i=1}^{m} (V_i)_2 W_i = EI \]  

\[ EI = \text{Environmental Impact} \]
\[ (V_i)_1 = \text{Value of environmental quality of parameter } i \text{ with project} \]
\[ (V_i)_2 = \text{Value of environmental quality of parameter } i \text{ without project} \]
\[ W_i = \text{relative weight (importance) of parameter } i \]
\[ m = \text{total number of parameters} \]

(-) A loss in EI constitutes an adverse environmental impact
(+ ) A gain in EI constitutes a beneficial impact.

The EES also considers other criteria to that of water quality based on similar rating curves. Some of the value functions address human factors, such as cultures, social interactions and the perception of atmosphere.

Recent work \((Day\ and\ Fenner,\ 1996)\) used a similar type of approach to the EES in the evaluation of the land drainage consent system. Each feature occurring along a river bank is given an impact rating based on a positive impact, negative impact or null. A score of +1, -1 or 0 is attributed respectively. The impact ratings (IR) for three areas are considered; visual evidence of pollution, ability to pass peak flow and environmental impact are calculated by taking away the sum of the negative impacts from the sum of the positive impacts. This figure is then divided
by the total number of features within the particular impact area. The equation is shown below.

\[ IR = \sum_{i=1}^{m} (\sum_{j=1}^{N} PI - \sum_{j=1}^{N} NI) \times \frac{1}{N} \] ..........................Equation 4.3

IR= Impact Rating for the river type  
PI and NI= Positive and Negative impacts  
N= Total number of bankside features  
m= Different impact criteria

The solution of a drainage catchment problem must be achieved in relation to the persons or groups of people directly affected while achieving regulatory standards and solving technical criteria. This highlights the need for communication between participants. Also, it should be noted that one party can view an occurrence or plan of action completely differently depending on their preferences. Many techniques suggested to evaluate such problems fail in terms of their applicability when faced with multiple criteria, multiple audiences, pressure groups, budget constraints and tight deadlines (Schultz, 1989).

Previous work has shown the use of value functions to assess the holistic quality of water, degree of environmental impact and river impact ratings. The principles are all the same, in that the criteria are weighted and assessed and reproduced as a score. This type of analysis deals with tangible and intangible criteria as well as assessing objective and subjective aspects of a problem. Above all a technique must be *simple* to apply, all encompassing, and *transparent* i.e., easy for others not involved in the detail to understand the results and how they were produced.

Decisions between alternative drainage rehabilitation scenarios must be taken based on sound analysis involving social, economic, hydraulic and environmental criteria. These criteria are objective and subjective.
Utility theory expresses the decision makers (group or individual) preference for a particular outcome for any given criteria \((Raiffa, 1968)\) and areas of concern. Utility theory probes the value structures of involved parties with reference to given criterion. In essence, utility theory allows the preferences of the parties towards a problem to be developed in a structured fashion.

In doing so, all important attributes are evaluated, and not simply glossed over, as is frequently done to avoid over complication of the situation. Drainage engineers are required to provide a service to the public. The public are customers, the group who must be satisfied and appropriate definition of the groups' value functions for criteria should be taken into account. These quantities should be evaluated from public consultation or from some kind of aggregation of individual views, rather than on any single individual's viewpoint.

The application of utility theory can allow the engineer to metaphorically drag the decision maker to the waters edge by the ear, and show them what they can achieve technically, environmentally, economically and socially for a given amount of cash. Utility theory provides a base for the evaluation of group decisions. Uncertainty plays a controlling role in engineering decision making, simply due to the fact that the weather which produces rainfall and runoff to the wastewater system is transient in nature. Utility theory accommodates uncertainty into the analysis through direct questioning of the problem being analysed and value functions developed through analysis.

Simplified techniques based on utility \((Edwards, 1977)\) rely, as do the water quality indices, on the use of value functions for criteria which influence a particular strategy or approach. Mathematical approaches proposed by \((Raiffa, 1969)\) and \((Keeney, 1972)\) are available to develop value functions for areas of concern. However, these are complex and not suited for general engineering applications. The simplified techniques rely on the participants drawing the value functions directly. An outline of the approach is presented.
1. This requires the person or organisation whose utilities are to be maximised to be identified. There may be a number of bodies whose input to the problem is required. The identities of the organisations or authorities whose aims must be satisfied must be clearly chosen.

2. This requires the issues and decisions under consideration to be identified. For example, for a drainage catchment the objective may be to identify the most effective rehabilitation strategy.

3. This requires the identification of the areas of concern associated with system performance.

4. This requires the definition of how the areas of concern are to be measured. (e.g. frequency, load or complaints)

5. This requires that the areas of concern be ranked in order of importance. Any criteria should be modified by cutting down or adding to the list after consultation with those parties identified in step 1. The parties should then have the chance to rank the dimensions in order of preference. A list is drawn up with the most important criteria at the top and the least at the bottom. This can be carried out individually or through a group approach. The group approach can be applied, as it allows arguments to be raised and discussed, and gives all participants a common information base.

The areas of concern are rated in importance preserving ratios. This process involves taking each of the ranked dimensions and assigning a number to the least important dimension (10 representing the least important dimension). The next task is to score the remaining dimensions relative to the scoring system. The next least important dimension is examined and the question is asked “how much more important (if at all) is this compared to the least important dimension (10)”. This is repeated until all dimensions have been covered. If the
second least important criterion was given a score of 20 it would be said to be twice as important as the least important criterion.

6. The importance weights are summed and each is divided by the sum and multiplied by 100. This gives each area of concern a weighting factor relative to its importance within the context of the problem.

7. A series of value functions (graphs) are drawn up which represent the attribute under consideration and give a reasonable scale for the attribute in context with the problem being examined. Flooding for example may range between a frequency of 0 events in a particular period and 10 events. These ordinates, and the values along the X axis are related to utility scores between 0 and 100 on the Y axis. The choice of a 0 to 100 scale is arbitrary.

8. Various techniques can be employed to derive the graphs. Indifference questioning can be employed, but the simplest approach is to allow the interviewee to draw the graph which explains the preferences associated with the particular problem. Each strategy is examined and the attributes affected scored from the developed value functions. Value functions can be purely objective and subjective depending on what is being evaluated. Utility scores for the areas of concern are calculated from the application of a simple weighted aggregation equation (see page 124).

9. Assessment of the defined objective utilising the developed utility scores.

This type of approach was utilised to evaluate 15 planning permit applications for development within the District of Venice, Los Angeles (Edwards, 1977). The applications were of varying nature consisting of single family, duplex, triplex and multiple dwellings. The decision-making body in this case was the South Coast Regional Commission, who were faced with the task of choosing between the permit applications and selecting the most appropriate one.
The areas of concern to be evaluated and how they were measured are listed.

- Size of development: The number of square feet of the coastal zone taken up by the development.

- Distance from the mean high tide level: The distance between the nearest edge of the development and the mean high tide line, measured in feet.

- Density of the proposed development: The number of dwelling units per acre for the development.

- On site parking facilities: The percentage of cars brought in by the development for which car parking space is provided as part of the development on site.

- Building height: The height of the development in feet (17½ feet per storey)

- Unit Rental: The dollar rental per month (on average) for the development.

- Conformity with land use in the vicinity: The density, measured on a five point scale, from much less dense, to much more dense of the development, relative to the average density of adjacent residential lots.

- Aesthetics of the development: A rating on a scale from poor to excellent.

Steps 5 and 6 were followed to rank the above criteria in importance preserving ratios. Step 7 was then followed to calculate weightings for each of the entities. Step 8 was then completed which resulted in value functions being produced. An example is shown in Figure 4.5. The weighting is shown in brackets.

![Figure 4.5 Value Function For Permit Evaluation](image_url)
The 15 permit applications were assessed on the above value functions and steps 9 and 10 followed to allow the most favourable permit application to be chosen from the set. The most favourable option being the one with the maximum aggregate score.

The application of multi-attribute utility theory to this particular problem has demonstrated a methodology which can be used to aid decision makers in choices between scenarios which encompass subjective and objective criteria. The approach is both simple, transparent and relatively quick to use. In conclusion, multi-attribute utility theory potentially provides the decision maker with a methodology which is structured, subjective and objective, all encompassing and provides solutions to problems on the principles of those it will ultimately effect.

4.7 SELECTION OF TECHNIQUE

The two favoured methods, BCA and multi-attribute utility analysis, were compared against a list of points necessary for their successful application to the proposed holistic methodology. These points are amplified on page 96.

<table>
<thead>
<tr>
<th>Comparative Assessment Criteria</th>
<th>Benefit Cost Analysis</th>
<th>Multi Attribute Utility Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Ability to encompass tangible criteria</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2. Ability to encompass intangible criteria</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>3. Ability to aid holistic assessment</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>4. Data requirements</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>5. Degree of specialism required</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>6. Simplicity of use</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>7. Ability to deal with multi-criteria problems</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>8. Ability to aid in prioritisation of catchments</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>9. Ability to aid in monitoring performance</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>10. Ability to integrate with UPM approach</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>11. Ability to integrate economic information</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>12. Ability to aid in definition of existing problems</td>
<td>0</td>
<td>1</td>
</tr>
</tbody>
</table>

**Relative Score For Technique**

| Relative Score For Technique | 7 | 11 |

Table 4.11 Comparative Assessment Criteria- Selection Of Adopted Technique

Greyed squares indicate the superiority of one approach over the other.
1. Does the method have the ability to utilise tangible criteria in the analysis. (Yes=1, No=0)

2. Does the method have the ability to utilise intangible criteria in the analysis. (Yes=1, No=0)

3. Can the methodology be used to holistically assess the performance of drainage catchments. (Yes=1, No=0)

4. What is the level of data required for the method. (High=0, Low=1)

5. What degree of specialism is required to carry out the method. (High=0, Low=1)

6. What level of simplicity is associated with the method. (High=1, Low=0)

7. What ability does the method have to encompass multi-criteria associated with drainage system performance. (High=1, Low=0)

8. What ability has the method to aid in the prioritisation of catchment rehabilitation. (High=1, Low=0)

9. What ability does the method have to monitor the existing/rehabilitated catchments. (High=1, Low=0)

10. What ability does the method have in terms of integrating with the existing UPM procedure. (High=1, Low=0)

11. What ability does the method have in terms of defining problems within an existing catchment. (High=1, Low=0)

12. What ability does the method have to integrate economic information into the analysis. (High=1, Low=0)

Following the comparative analysis, shown in Table 4.11, it is clear that both methods are similar in terms of suitability for the use in the holistic assessment of the performance of wastewater systems. However, as shown in Table 4.11, multi-attribute utility analysis has the advantage in terms of applicability when simplicity, data requirement and level of specialism are considered.

As has been discussed previously, many of the damages associated with wastewater system performance are intangible in nature and extremely difficult to evaluate. Benefits (averted damages) must be calculated as accurately as possible if a BCA analysis is to hold true. Techniques available to evaluate intangibles are available. However, they require large degrees of data, are expensive and involve specialist activities, such as social surveys.

Multi-attribute analysis has the additional advantage, as shown in Table 4.11, over BCA of allowing system performance to be assessed with regard to specified areas of concern. The technique can also be used to select between rehabilitation options and to monitor the holistic performance of an existing or rehabilitated...
system. Data requirements are low and the level of simplicity is high. However, it is important that weighting factors and value functions associated with parameters under evaluation be determined accurately. It must be recognised that multi-attribute utility analysis does not directly take into account economics. Also, the two methods are not necessarily alternatives but are complementary and present different sorts of information to the decision-maker.

From current work (FWR, 1996) discussed in Chapter 3, it is apparent that the UK water industry are pursuing the avenue of BCA as a method for prioritising upgrading works required to improve water quality. The problems concerning the evaluation of benefits have been documented in this Chapter as have the techniques used to evaluate the scale of these benefits.

These BCA approaches will be carried out initially based on desktop studies, determining the size of population to benefit, the scale of average benefits to each visitor and the spread of the benefits throughout the future. The scale of this task should not be underestimated and may prove to be very difficult to arrive at accurate figures. The application of sensitivity analysis to any work carried out using this approach will show how the range of figures relating population, benefits and timescale will affect the decision making process. It is proposed that most studies will need to carry out CVM studies of the watercourse in question.

There is also the question of non-use values and how these will be evaluated. It appears that this criterion will be estimated from previous work and not specifically calculated. Either way, it is likely that this type of work, especially the fieldwork which requires detailed social survey techniques, will be carried out by specialists. It is important to consider all aspects of system performance in a decision making context. BCA has been applied to flooding from river catchments but the approach would be difficult, as has been discussed, to apply to flooding from wastewater systems.
It may be that in future BCA will be utilised to prioritise all aspects of system upgrading; flooding, structural and pollution performance. This will be a difficult task, as has been shown through previous studies, and specifically because of the intangible nature of many of the benefits resulting from wastewater system rehabilitation.

Therefore, for the purposes of this study it is proposed not to use BCA as the evaluation technique in the methodology to be developed. This has been determined with reference to the criteria in Table 4.11. The technique of multi-attribute analysis, using weightings and simple value functions, has been demonstrated, in the past, to provide an adequate approach for analysing multi-criteria problems. This makes it suitable for dealing with the integrated performance of wastewater systems.

Another advantage of the approach is that it can be used to aid in the selection of rehabilitation options, through comparison with existing system performance or standards (similar to the EES system). The technique can also be used in a monitoring role, as with Houses' WQI, where index scores are developed based on routine sampling to investigate improvements or degradation in water quality in specific river and stream reaches. A similar approach can be developed for the performance of the whole sewerage system utilising the technique.

A methodology based on multi-attribute utility analysis will be developed and applied to the drainage system of Perth, in conjunction with integrated modelling tools, as a means of holistically analysing the performance of the catchment. The methodology has been entitled WISPS; Wastewater Integrated System Performance Score and is detailed in Chapter 5.
CHAPTER 5 DEVELOPMENT OF THE WISPS METHODOLOGY

5.0 INTRODUCTION

The methodology for the effective evaluation of wastewater systems (WISPS) is intended to allow engineers to (i) evaluate and prioritise catchments with respect to performance criteria, (ii) aid in the appraisal of rehabilitation options and (iii) monitor catchment performance.

The methodology is based on multi-attribute utility theory and uses simple value functions and weightings to derive scores for the integrated performance of wastewater systems. This approach has been discussed in Chapter 4. The WISPS methodology is presented in Figures 5.1, 5.2 and 5.3.

5.1 INTERESTED PARTIES

Before developing the WISPS methodology it was important to assess the main interested parties to ensure that as much information was provided from outside bodies as possible.

The two main interested parties were defined as the North of Scotland Water Authority and the General Public. During the derivation of weightings for the Areas of Concern a page was advertised on the internet to gauge independent engineering views.

5.2 MAJOR AREAS OF CONCERN AND AREAS OF CONCERN

A list of criteria, shown in Figure 5.4 was developed, which describes Major Areas Of Concern (MAOC) associated with the performance of wastewater systems. These are sub-divided into Areas Of Concern (AOC) as detailed. It is hypothesised that the criteria are generally applicable to all drainage catchments and not restricted to Perth. In certain catchments, other criteria may be relevant and can be added to the list.
Definition of objectives and interested parties

Carried out through interviews with operational/divisional staff and existing recorded performance information.

Major areas of concern

Areas of concern

Selection and definition of attributes for areas of concern

Ranking and weighting of areas of concern

Develop value functions for areas of concern

Examine available historical performance data

Develop historical wastewater integrated system performance score (WISPS)

Ranking of catchments to establish criticality based on WISPS score

Selection of catchment for rehabilitation or investigation

Aggregation equation (eq 5.1) applied to calculate WISPS for catchment

Progress to stage II

Figure 5.1 Prioritisation–Stage I of Methodology
Appropriate modelling/monitoring/investigation of wastewater system

Available techniques and tools from UPM Manual

Are you assessing a rehabilitation option?

Yes

No (1st Pass)

Detailed assessment of areas of concern from modelling/investigations/surveys

Assessment of rehabilitated areas of concern

Data input to value functions for areas of concern

Are you assessing a rehabilitation option?

No

Apply constraints: time, budget, land, political etc.

Through application of aggregation equation (eq 5.1)

YES

Detailed WISPS of rehabilitated drainage catchment

Generate rehabilitation options (Stage IIA)

Evaluation of selected rehabilitation option. (Check compliance with UPM standards)

Compare detailed and rehabilitated WISPS, offer options to “decision makers”

Progress to stage III

All rehabilitation options modelled and assessed?

YES

Figure 5.2 Evaluation of Rehabilitation Options~ Stage II of Methodology
Implement and construct selected solution for wastewater system

Set-up and maintain systems and procedures to monitor the performance of the rehabilitated system with respect to the areas of concern

Assess WISPS score for rehabilitated catchment on a monthly basis or alternative timescale depending on resources required/available

Assess and compare yearly (average) WISPS for rehabilitated system with rehabilitated designed system

Satisfactory system Performance?

YES

Continue monitoring

NO

Return to stage II

Figure 5.3 Monitoring of Rehabilitated Performance–Stage III of Methodology
5.3 SELECTION OF ATTRIBUTES FOR AREAS OF CONCERN

Attributes define how the AOC are measured. Each MAOC is discussed in the following sections along with subdivided AOC and their attributes. An explanation is presented regarding the choice of attribute. Each AOC is discussed with respect to the methodology and some areas are rejected as they are defined as being duplicated within other AOC.

5.3.1 Hydraulic Performance Of System

Flooding (attribute; frequency of flooding events per year).
Infiltration (attribute; relative percentage of the average DWF entering WWTP).
Surcharging (attribute; frequency of surcharging).
Sedimentation (attribute; percentage of system affected by substantial deposition).

Many attributes could be selected to represent flooding performance; flooding volume, floodable area, flooded depth or total damage inflicted. Damage requires depth to be defined and as stated in Chapter 4 is very difficult to achieve accurately for sewage flooding. Holistic damage costs would have to include
intangible anxiety and stress which are even more difficult to assess. Flooding volumes are feasible due to the ability of hydraulic models to predict volumes of flooding discharged.

However, volumes themselves while giving a feel for the performance of the system, do not give any real indication of the repeatability of the problem. The attribute chosen therefore is associated with the occurrence of flooding, i.e., the frequency, based on the premise that any act of flooding is unacceptable and the reoccurrence of flooding is highly influential in terms of assessing the performance of a wastewater system. Flooding analysis is to be carried out utilising historical data and the attribute is defined as flooding events per year on average with respect to available historical rainfall data and not design storms. This analysis is carried out in Chapter 8 as part of the detailed assessment in Stage II of the methodology.

Infiltration can be expressed as an average flowrate, but this gives no indication as to the relative quantity present. A better approach is to express the parameter as a percentage of the overall average DWF entering the treatment plant. This allows the engineer to see the quantity of infiltration present in relation to domestic and industrial flow.

Surcharging is clearly related to flooding, as sewers have to surcharge before flooding can occur. The definition of an historical level of service with respect to surcharge is difficult to achieve, without complicated analysis of each storm within an historical series, and the consequential effect upon the surcharge regime of the system. It is proposed that the frequency of flooding attribute be used to infer the frequency of surcharge.

Alternatively, design storms can be applied to assess the level of performance with respect to surcharging. Generally, Water Authorities express performance indicators in relation to flooding frequency and not surcharge. Therefore this AOC is omitted from the methodology. To support the rejection it is argued that
the Structural Performance Grade (see section 5.3.2) is finalised with respect to
the frequency of surcharging that a sewer experiences.

The attribute used to describe sedimentation is based on the percentage of sewers
substantially affected by sedimentation within a catchment area. Substantially is
defined as being a sediment depth more than 10% of nominal sewer diameter.

5.3.2 Structural Performance Of System

Grading of Critical Sewers (attribute; Length of critical sewer with grading
≥A3/B3; expressed as a percentage of total length of category A and B sewers).

This attribute relates to critical sewers which form the main core of any sewerage
system. Of main interest to engineers are the sewers within Category A and B. A
sewer will fall into these categories if the engineering costs of repairing the sewer,
in the event of failure, are likely to be high, if traffic delay costs are high as a
result of sewer failure and if the sewer is considered to be strategically important.

Most sewer rehabilitation teams operate a system of plans which show critical
sewers with respect to nomenclature laid down in the Sewer Rehabilitation
Manual. Internal condition grades are obtained by CCTV work and walk through
surveys in larger diameter sewers. The grading is shown below.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Implication</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Collapsed or collapse imminent</td>
</tr>
<tr>
<td>4</td>
<td>Collapse likely in foreseeable future</td>
</tr>
<tr>
<td>3</td>
<td>Collapse unlikely in near future but further deterioration likely</td>
</tr>
<tr>
<td>2</td>
<td>Minimal collapse risk in short term but potential for further deterioration</td>
</tr>
<tr>
<td>1</td>
<td>Acceptable structural condition</td>
</tr>
</tbody>
</table>

Table 5.1 Internal Condition Grades

The worst defect along the individual sewer length is utilised to give a singular
internal condition grade for that length of sewer. Supplementary data is used to
transfer the internal condition grade to a structural performance grade. This
supplementary data is typically related to surrounding soil type, surcharge
frequency, groundwater, maintenance problems, construction standard, heavy
traffic loading on sewers with minimum cover and specific evidence of continuing deterioration. If the sewer in question does not suffer from any of the above criteria, the original internal grading score stands. Information is provided within SRM which specifies the awarding of performance grades.

Critical sewers can therefore be classified as A1 through to A5, and B1 through to B5. Within any sewerage system it is important to ensure all sewers are maintained to a suitable level of structural performance. For the purposes of this analysis the attribute to describe structural performance is expressed as the percentage of Category A and B sewers which have a grading score of 3 or above (i.e., the percentage of sewers scoring A5, A4, A3, B5, B4, B3 in relation to the total number of category A and B sewers).

5.3.3 Pollution Performance Of System

CSO Discharge (attribute; average frequency of spill/month).
Aesthetic Pollution (attribute; impact of sewage derived waste present).

Formal assessments of CSOs and their performance are required to be carried out by authorities in England and Wales for Asset Management Planning (AMP) purposes. The new Water Authorities in Scotland will require the similar assessments in the near future.

CSOs in Scotland will require upgrading to protect amenity standards more so than river quality. Scotland is fortunate in that many of the larger conurbations, or towns, are situated near areas of natural dispersion, such as the North Sea and large rivers. Acceptable pollution loads can be spilled to these watercourses and thus alleviate the need for large volumes of storage and treatment.

Water Authorities in England and Wales have moved towards an agreed assessment procedure for Combined Sewer Overflows. This is necessary to evaluate the performance of the overflows and to rank them in terms of their priority with respect to upgrading. NRA regions have developed assessment techniques for overflows within their boundaries. Most assessment techniques revolve around the following criteria.
1. **Aesthetic Impact**: This impact tends to be evaluated in terms of the number of visible and gross solids around the CSO, on the banks and in the watercourse. Some regions count the number of solids as an impact assessment, whereas others simply judge the impact on the presence of such material around the CSO discharge zone. This parameter is highly noticeable from a bankside users' viewpoint, as well as being identifiable as originating from a sewer to a less well informed recreational user. Aesthetic pollution of this type has more impact in areas of high amenity.

2. **Dry Weather Flow Operation**: This criterion is judged by simply viewing the structure under normal DWF conditions. This gives an indication as to the over loading of the system. It may be important to visit the overflow at different times of the day to observe DWF operation, as this phenomenon will likely take place under peak DWF.

3. **Complaints**: A history of complaint does not necessarily detail every time a CSO operates or the effect it is having upon the watercourse it discharges into. A record of complaints may indicate how aware the complainer is to matters of pollution. This may or may not be as a result of the complainer having an interest in water quality matters. The most complained about CSOs may not be the ones which operate more than others. They may be in fact those CSOs which are discharging to high amenity areas of the watercourse or the structures which are highly visible to the public.

4. **Fish Mortality**: Fish mortality is an important criterion which must be addressed when examining CSO discharge. However, the presence of dead fish does not indicate that a discharge of combined sewage has poisoned them by reducing DO levels or increasing levels of un-ionised ammonia. If a clear link between CSO discharge and dead fish can be obtained then this is very important. However, this appears to require the presence of an observer to witness the discharge and then to observe if fish are killed. Dead fish can be as result of many forms of pollution, farm waste, chemical dumping and natural
causes. Fish kills should be logged if they have occurred but should not form part of an assessment technique.

5. **Sewage Fungus**: Sewage fungus is utilised by some NRA regions for the assessment of CSO impact. However, this criterion can be perceived in different ways by different observers and can result in impact varying dependent on the observers perception of the spread and concentration of sewage fungus. The presence of fungus does indicate frequent overflow discharge. Fungus may grow more quickly dependent on the water conditions i.e., temperature and chemical content and this should be borne in mind when considering impact on rivers of different characteristics.

6. **Public Access**: CSO structures which are easily accessible to the public, such as those near playparks or footpaths, are more important than those hidden in shrubbery or away from areas frequented by bankside users. The degree of accessibility is important in cases where two overflows are causing the same aesthetic and biochemical problems but one is easily accessible to the public and one is not. The one which is easily accessible must be dealt with first.

7. **Biological Impact**: The effect of CSO discharges upon macroinvertebrates is an excellent way of assessing the polluting performance of any overflow structure. Most techniques which are used by regulators involve carrying out kick tests and invertebrate counts upstream and downstream of CSOs. Once the collected invertebrates have been split into families, Biological Monitoring Working Party (BMWP) scores can be calculated and differences between upstream and downstream scores can be examined to see the effects of pollution upon the insect community in the river.

This technique works well if the physical conditions within the watercourse are suitable for kick samples to be taken. Also the river upstream of the structure being examined needs to be relatively unpolluted. This gives high levels of
macroinvertebrates compared to the concentration of these animals below the CSO under examination, for the results of storm discharge to be assessed. If a river is polluted from the headwaters down it will be impossible to assess the performance of the overflow based in biological terms by sampling families of macroinvertebrates.

8. **River Sediments:** The build up of pollutants within river sediments around CSO discharges can be good indicators of the polluting effects of storm discharges. The majority of NRA regions refer to river sediments in their assessment techniques but do not necessarily sample and analyse this material. It is clear that the presence of black odoriferous sediment within a watercourse near a CSO, is an indicator that the CSO is operating frequently, especially if the sediment is well established.

Within the UPM manual, a methodology is recommended for assessing the impact of CSOs (Milne and Clark, 1994a, 1994b). This revolves around examining the quantities of sewage derived litter, quantities of sewage fungus, public complaints, pollution incidents, history of dry weather operation and impact on water quality class or objectives. The impact of each criterion is given a classification between A and E; A being the most unsatisfactory. Thresholds are adopted to specify which CSOs are satisfactory, unsatisfactory or very unsatisfactory, based on the classification system.

It was decided that the impact of CSOs within the Perth system would be appraised based on the quantity of aesthetic pollution present, degree of public accessibility and the occurrence of operation, as these three criteria are considered to be the most important in terms of prioritising the performance of a CSO structure.

The attribute chosen for assessing the pollution performance of CSOs is related to the frequency of spill over a defined historical period. This is based on the premise that less spills are better than more. Volume of spill per month or pollutant load
spilled over a defined period of time can be used. Information relating to spilled pollutants can be gained from site specific sampling. However, by considering frequency, each event is taken as being potentially acutely polluting to the receiving watercourse, in the same way that an occurrence of sewage flooding is abhorrent to the general public, notwithstanding the volume present.

Aesthetic pollution is assessed by visual inspections. The quantity of sewage derived waste present near the CSO is defined as the attribute for the aesthetic criteria. The scale of judgement is based on simplicity and specified in Table 5.2, i.e. quantity of sewage derived waste local to CSO or group of CSOs; large amount =10, medium amount =25, small amount =50, trace amount =75, none=100. Public accessibility is important in terms of the impact of aesthetic pollution and this is based on the following scale; high access =10, medium access =25, low access =50, very low access =75, no access =100. The two factors are weighted as 0.75 and 0.25 respectively and are simply multiplied by the scaling figures shown to achieve a CSO criticality score which can range from 10 (large quantity present and high public access: very unsatisfactory) to 100 (no aesthetic pollution present and no public access: very satisfactory).

<table>
<thead>
<tr>
<th>Sewage Derived Material Present</th>
<th>Definition</th>
<th>Access</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>none</td>
<td>0</td>
<td>none</td>
<td>no public access</td>
</tr>
<tr>
<td>trace</td>
<td>1-2 items</td>
<td>very low</td>
<td>seldom accessed by public</td>
</tr>
<tr>
<td>small</td>
<td>2-6</td>
<td>low</td>
<td>casual access by public</td>
</tr>
<tr>
<td>medium</td>
<td>6-10</td>
<td>medium</td>
<td>frequent access by public</td>
</tr>
<tr>
<td>large</td>
<td>&gt;10</td>
<td>high</td>
<td>unavoidable access by public</td>
</tr>
</tbody>
</table>

Table 5.2 CSO Criticality System
5.3.4 Performance Of Wastewater Treatment Plants

Compliance with standards (attribute; percentage compliance with standards).
Odour (attribute; frequency of complaints).

The attribute used to judge the performance of WWTPs is based on percentage compliance with consent standards. This attribute can be easily evaluated from information concerning routine sampling and monitoring carried out by regulatory authorities (SEPA) and from Water Authorities themselves.

The attribute for odour could be expressed in terms of concentrations of Hydrogen Sulphide and other commonly occurring nuisance gases (e.g., Mercaptans). However, smell is a subjective criteria and it was felt that the occurrence of complaints from the general public regarding an odour source was a better assessment vehicle. The drawback of utilising a system of evaluation based on complaint is that all public complaints regarding WWTP odour may not be attributable to the treatment plant.

5.3.5 Performance Costs

Tangible costs associated with system lack of performance (attribute; cash).

These are costs associated with clean-up after CSO discharge, or flooding events, or the increased costs for potable water treatment associated with poor quality raw water affected by pollutant discharge. These costs may include fines from the appropriate river authority in relation to breached consent standards from WWTPs.

In many catchments these are not easily defined and to develop a value function for this criterion would be very difficult. In any case, a lack of performance in defined areas may already be measured in other areas of concern, albeit not in cash terms. Therefore, if this information is easily available it is suggested that the data be used as additional detail on the performance of a wastewater system, but does not explicitly form part of the WISPS methodology.
5.3.6 Receiving Water Courses

Quality of Receiving Watercourses (attribute; River Board classification).
Amenity of Receiving Watercourses (attribute; high, moderate, low, non)

The quality of receiving watercourses is judged from historical information or from a database of records and/or from selected sampling of watercourse quality. This AOC is associated with the long term water quality of the receiving water course based on regulatory sampling. It is recognised that acute pollution of watercourses can occur during CSO/SWO events. Long term water quality is related to presence of CSO, SWO and WWTP discharges, and it was thought prudent to include an area of concern which showed the decision maker the quality of receiving water course and a value function which showed the preference for each of the classifications (1A, 1B, 2, 3 or 4).

The amenity of watercourse can be related to the recent UPM classification, which specifies screening requirements in relation to CSOs dependent on the amenity of the surrounding area. The amenity is classified as high, medium, low and non. At present the amenity value of watercourse is not the responsibility of the Water Authority, however, it may well be in the future.

It is likely for the foreseeable future in Scotland that District Councils will be responsible for the amenity of watercourses. If discharges to receiving water courses are reduced, or improved biochemically or aesthetically, then the amenity of the watercourse is potentially increased. For more people to visit a river site a benefit must be perceived to have occurred. Also, the amenity value of a watercourse is dependent upon those who have a relationship with the stream or river.

Therefore, this AOC is not included explicitly within the WISPS methodology, but is appraised, similarly to performance costs, for the purposes of additional information.
5.4 RANKING AND WEIGHTING OF AREAS OF CONCERN

Ranking and weighting of the AOC were carried out in importance preserving ratios as specified in Chapter 4. A sample of ten staff currently working within Capital Procurement, Environment and Quality and Asset Management Planning were interviewed and their responses are shown in graphical form.

![Frequency of flooding](image1)

Figure 5.5 Weightings For Frequency Of Flooding

![Aesthetic Pollution on Riverbanks](image2)

Figure 5.6 Weightings For CSO Aesthetic Pollution

![Structural Integrity of Sewerage system](image3)

Figure 5.7 Weightings For Structural Integrity
Figure 5.8 Weightings For WWTP Compliance

Figure 5.9 Weightings For Infiltration

Figure 5.10 Weightings For CSO Discharge

Figure 5.11 Weightings For Receiving Watercourse Quality
A full discussion of the responses is contained within Appendix M. Average weightings were calculated and the areas of concern ranked according to the most important criteria. These are shown below in tabular and graphical form.

<table>
<thead>
<tr>
<th>Area of Concern</th>
<th>Weighting</th>
<th>Ranking</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Integrity of Sewerage System</td>
<td>0.140</td>
<td>1</td>
</tr>
<tr>
<td>Flooding</td>
<td>0.138</td>
<td>2</td>
</tr>
<tr>
<td>WWTP Compliance with Consents</td>
<td>0.132</td>
<td>3</td>
</tr>
<tr>
<td>CSO discharge to Receiving Waters</td>
<td>0.110</td>
<td>4</td>
</tr>
<tr>
<td>Aesthetic Pollution</td>
<td>0.109</td>
<td>5</td>
</tr>
<tr>
<td>Receiving Water Course Quality</td>
<td>0.101</td>
<td>6</td>
</tr>
<tr>
<td>Sedimentation</td>
<td>0.095</td>
<td>7</td>
</tr>
<tr>
<td>Infiltration</td>
<td>0.094</td>
<td>8</td>
</tr>
<tr>
<td>Odour from WWTP</td>
<td>0.080</td>
<td>9</td>
</tr>
</tbody>
</table>

Table 5.3 Ranking And Weighting Of Areas Of Concern
5.5 DEVELOPMENT OF VALUE FUNCTIONS

Value functions were next developed for each AOC. These were derived by allowing respondents to sketch their preferences on presented blank graphs. The graphs shown have been produced from averaging the respondents curves. Development of the value functions are discussed in Appendix M.
Figure 5.16 Value Function For Infiltration

Figure 5.17 Value Function For Sedimentation

Figure 5.18 Value Function For WWTP Odour
Figure 5.19 Value Function For CSO Discharge

Figure 5.20 Value function For WWTP Compliance

Figure 5.21 Value Function For Structural Integrity
5.6 REVIEW OF HISTORICAL PERFORMANCE DATA

Following the methodology shown in the flowcharts all available historical performance data were collated for the Perth catchment. This information allowed performance scores to be calculated from the value functions using Equation 5.1 (shown in section 5.7, page 124). Drainage catchments scoring highly are more efficient than those achieving low scores. Scores range between 0 (worst) and 100 (best).

5.6.1 Historical Flooding Performance

Scant information could be found regarding the historical flooding performance of the Perth drainage system. Interviews were held with divisional engineers, inspectors and superintendents in an attempt to derive data. While specific
information could not be found regarding the average frequency of flooding attributable to overloading of the sewerage system, areas subject to flooding were provided. It was thought that flooding did occur on an infrequent basis. Consequently a value of two flooding events per year was estimated.

Although NoSWA operate a customer services “hotline” regarding any complaints, this system while logging a flooding event does not specifically attribute the occurrence to overloading of the sewer. Many of these occurrences on inspection appear to be the result of chokes in the system or blocked road gullies. There is clearly a requirement to record significant flooding events occurring within a drainage catchment, as this would provide critical data regarding the actual hydraulic performance of sewerage systems.

5.6.2 Sedimentation Performance

From examination of sewer record plans, available from a survey of the central area system in Perth, it is apparent that sedimentation is present within the network. From interviews with divisional staff it became clear, that while sediment was widely distributed throughout the central area, the sediment did not appear to be generally of great depth with respect to pipe diameter. It was estimated that approximately 25% of the system suffered from substantial sedimentation.

5.6.3 Infiltration Performance

Figures regarding the ADWF entering the WWTP at Sleepless Inch were available from the Quality and Treatment section within Water Services. The ADWF quoted was 220l/s. Analysis was carried out utilising this figure, population, known industry and water usage rates to identify the potential quantity of infiltration present.

The population of 42 000 was assumed to have a water usage of 180 l/h/d. Industry was estimated as contributing 25l/s. Domestic and industrial inputs were calculated to contribute 112 l/s as an ADWF. In essence, there appears to be 107l/s of extra flow. Analysis was carried out on the quantities of treated water
supplied to the Perth catchment. Water supplied from Gowans Terrace is in the order of 14 000 m³/d. This equates to a water usage, minus of industrial flows, of 282 l/h/d. This is a very extreme figure and is indicative of leakage within the water distribution network.

General figures for leakage from water networks are recognised to be in the field of 20% to 30%, with the latter figure more realistic. It was assumed that 30% of the extra flow (50l/s) arriving at the WWTP was due to leakage and the remainder 58l/s, due to true groundwater infiltration. The relative proportion of infiltration, is serious and appears to contribute to 50% of the ADWF.

5.6.4 Structural Performance

Very little information was found to exist regarding the structural performance grades of the critical sewers within the Perth catchment. It generally recognised that many of the sewers within the city centre are critical due to the depth, structure and traffic loading. Therefore 15% of the critical sewers were estimated as being of Grade A3/B3 or worse.

5.6.5 CSO Performance

It is known that the two major overflows prior to the Friarton and South Inch pumping stations operate frequently, and discharge screened sewage to the Tay. CSOs within the Craigie catchment are also known to operate but less frequently. CSOs are present within the Bridgend catchment but little was known about their performance. Consequently, CSOs were estimated to spill to receiving watercourses twice a month on average.

5.6.6 CSO Aesthetic Performance

No information was available relating to the degree of aesthetic pollution affecting the watercourses within the catchment. This is not routinely monitored by NoSWA. It was estimated that as no complaints had been raised regarding this material that the quantity present would be small (score of 50), and as the CSOs
are reasonably accessible (score of 25) that an average score of 44 (0.75*50+0.25*25) should be attributed on the developed criticality scale.

5.6.7 WWTP Performance

Recent figures for the performance of the treatment plant are shown in Table 5.4 for BOD, TSS and COD for final effluent. It can be seen that over the period 8/9/94 to 5/1/95 that the plant has failed consent standards only once for TSS. A review of samples taken at Sleepless Inch from 05/11/75 to 19/03/92 shows approximately three recorded failures in 20 years of operation, an effective compliance of 100%.

<table>
<thead>
<tr>
<th>Date</th>
<th>BOD (mg/l)</th>
<th>TSS (mg/l)</th>
<th>COD (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/9/94</td>
<td>47</td>
<td>54</td>
<td>170</td>
</tr>
<tr>
<td>19/8/94</td>
<td>70</td>
<td>72</td>
<td>192</td>
</tr>
<tr>
<td>21/7/94</td>
<td>52</td>
<td>31</td>
<td>186</td>
</tr>
<tr>
<td>16/6/94</td>
<td>58</td>
<td>148</td>
<td>306</td>
</tr>
<tr>
<td>26/5/94</td>
<td>33</td>
<td>35</td>
<td>125</td>
</tr>
<tr>
<td>27/4/94</td>
<td>15</td>
<td>18</td>
<td>78</td>
</tr>
<tr>
<td>28/4/95</td>
<td>44</td>
<td>45</td>
<td>142</td>
</tr>
<tr>
<td>23/3/95</td>
<td>8</td>
<td>22</td>
<td>56</td>
</tr>
<tr>
<td>1/2/95</td>
<td>5</td>
<td>9</td>
<td>48</td>
</tr>
<tr>
<td>5/1/95</td>
<td>16</td>
<td>27</td>
<td>67</td>
</tr>
</tbody>
</table>

Table 5.4 Perth Wastewater Treatment Plant Final Effluent Data

5.6.8 WWTP Odour Performance

Odour complaints have been received from the public regarding the WWTP at Sleepless Inch on numerous occasions in recent years. Table 5.5 summarises the complaints. The frequency equates to approximately 12 complaints a year on average.

<table>
<thead>
<tr>
<th>Year</th>
<th>Complaints</th>
</tr>
</thead>
<tbody>
<tr>
<td>1992</td>
<td>1</td>
</tr>
<tr>
<td>1993</td>
<td>10</td>
</tr>
<tr>
<td>1994</td>
<td>34</td>
</tr>
<tr>
<td>1995</td>
<td>10</td>
</tr>
<tr>
<td>1996</td>
<td>2 (to date May 1996)</td>
</tr>
</tbody>
</table>

Table 5.5 Odour Complaints Frequency
5.6.9 Receiving Watercourse Performance

Large percentages of Scotland’s watercourses are clean and free of water pollution (SOE 1990). The water quality of Scotland’s rivers, estuaries and lochs have had a net increase of 968km in the length of Class 1 quality rivers between 1980 and 1990.

The length of class 4 rivers has been reduced by 56% during the same period. Also more than 96% of Scottish rivers classified on the basis of biology are either Class A or B. 89% of biological sites have shown an improvement between 1980 and 1990.

Review of SEPA analysis has shown that the biochemical quality of the Tay in the Perth area is excellent. During the summer period of 1995 the quality of the Tay at Perth can be seen in the Table 5.6.

<table>
<thead>
<tr>
<th>Date</th>
<th>Temp</th>
<th>DO mg/l</th>
<th>% Satn</th>
<th>BOD₅ mg/l</th>
<th>NH₃N mg/l</th>
<th>TON mg/l</th>
<th>Total Pug/l</th>
<th>CONDY 25 us/cm</th>
<th>CHLA ug/l</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/8/95</td>
<td>20.5</td>
<td>10.70</td>
<td>119</td>
<td>0.93</td>
<td>0.09</td>
<td>0.15</td>
<td>18.38</td>
<td>83.4</td>
<td>2.38</td>
</tr>
<tr>
<td>15/8/95</td>
<td>20</td>
<td>10.7</td>
<td>118</td>
<td>1.22</td>
<td>0.08</td>
<td>0.13</td>
<td>23.65</td>
<td>82</td>
<td>5.19</td>
</tr>
<tr>
<td>22/8/95</td>
<td>21.5</td>
<td>9.59</td>
<td>109</td>
<td>0.72</td>
<td>0.23</td>
<td>1.16</td>
<td>24.5</td>
<td>86.20</td>
<td>3.99</td>
</tr>
</tbody>
</table>

Table 5.6 Water Quality Of River Tay

Even in the very dry spell of the summer of 1995 the Tay contained volumes of flow shown below. Flows shown in Table 5.7 while low for the Tay at Perth clearly have enough dilution to absorb CSO spills ($Q_{95}$ for the Tay is 44.3 cumecs).

<table>
<thead>
<tr>
<th>Date</th>
<th>8 August 1995</th>
<th>15 August 1995</th>
<th>22 August 1995</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow (m³/s)</td>
<td>38</td>
<td>33</td>
<td>29</td>
</tr>
</tbody>
</table>

Table 5.7 Selected Flows From River Tay

In general terms, the CSOs located within Perth do not appear to present a long-term biochemical threat to the watercourses in the areas. The River Tay has been shown to be a class 1A river based on regulatory sampling which defines the long term water quality. Little is known about the receiving water quality during CSO spills.
5.7 HISTORICAL WISPS FOR PERTH

Through carrying out this approach, areas of information have been estimated based on local knowledge of system performance, to provide input to the methodology. This will always be necessary with respect to wastewater systems. More efficient records of performance need to be developed by Water Authorities. This would allow a better understanding of the wastewater systems under their control, and allow identification of inadequacies more efficiently.

The historical information gathered for Perth was used to establish a historical WISPS score for the system. This is shown in Table 5.8. The arithmetic mean equation shown below in 5.1 was applied. It must be stressed at this stage that the score is an outline figure, and has been produced by utilising historical and estimated information by through Stage I of the methodology. A sensitivity analysis was carried out on the WISPS technique and is presented in Appendix M.

\[
\text{WISPS} = \frac{n = 9}{\sum_{i=1}^{n} w_i q_i} \quad \text{Equation 5.1}
\]

\(n\) = number of areas of concern
\(w_i\) = weighting attributed to the \(i\)th area of concern
\(q_i\) = performance score of \(i\)th parameter

Similar historical WISPS scores were derived for the wastewater catchments of, Coupar Angus, Almondbank and Forfar. These were developed for comparative purposes, and to demonstrate the general applicability of the method.

The same weightings and value functions developed previously were applied to evaluate the other test catchments. This is valid, as the original value functions and weightings were not developed for a specific wastewater catchment. Historical WISPS for the test catchments are shown in Table 5.10 and in Figure 5.24.
The sewerage system of Coupar Angus is recognised to be in poor structural condition in certain parts of the central area. Very little flooding has been recorded over the years in Coupar Angus, and receiving water course quality is good. There are no CSOs present on the sewerage system, resulting in no spills and no aesthetic pollution. Sedimentation is present within a length of sewer leading to the WWTP, but is not substantial in terms of the attribute definition in section 5.3.1. Infiltration is estimated from flow loggers to be high and the WWTP performs well. No odour complaints have been received in recent years.

Almondbank is small village to the North of Perth. The system has few structural defects and no frequent flooding has been reported. Two CSOs are present and discharge frequently to the River Almond; a good quality river. Aesthetic pollution is present and can be seen downstream of CSO locations. The WWTP is hydraulically and biologically overloaded and performs poorly. Infiltration is small, according to flow logger information, and a short length of trunk sewer suffers from substantial sedimentation. No odour complaints have been received regarding the WWTP.

The town of Forfar is a moderately sized settlement in the Angus area of NoSWA. The sewerage system is recognised as having some structural problems in various locations. Flooding is reported regularly at certain locations within the system. Sedimentation is known to be substantial in the trunk sewer leading to the WWTP. The WWTP performs poorly and is presently under reconstruction. CSOs discharge frequently to Forfar Loch (a sensitive watercourse) resulting in poor water quality and large amounts of aesthetic pollution. Infiltration, from flow records is thought to be average, and odour complaints from the existing WWTP are a regular occurrence.

On inspection of the scores from the four catchments shown in Table 5.10, it was evident from discussions with engineers, and past system behaviour, that the holistic performance of the wastewater systems under consideration was being described adequately by the WISPS methodology.
<table>
<thead>
<tr>
<th>Area of Concern</th>
<th>Average Weighting</th>
<th>Attribute Value</th>
<th>Performance Score</th>
<th>WISPS Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Integrity of Sewerage System</td>
<td>0.140</td>
<td>15% (estimate)</td>
<td>50</td>
<td>7</td>
</tr>
<tr>
<td>Flooding</td>
<td>0.138</td>
<td>2/year</td>
<td>50</td>
<td>6.9</td>
</tr>
<tr>
<td>Receiving Water Course Quality</td>
<td>0.101</td>
<td>1A</td>
<td>100</td>
<td>10.1</td>
</tr>
<tr>
<td>WWTP Compliance with Consents</td>
<td>0.132</td>
<td>100%</td>
<td>100</td>
<td>13.2</td>
</tr>
<tr>
<td>Aesthetic Pollution</td>
<td>0.109</td>
<td>44</td>
<td>50</td>
<td>5.45</td>
</tr>
<tr>
<td>CSO discharge</td>
<td>0.110</td>
<td>2</td>
<td>30</td>
<td>3.3</td>
</tr>
<tr>
<td>Infiltration</td>
<td>0.094</td>
<td>50%</td>
<td>20</td>
<td>1.88</td>
</tr>
<tr>
<td>Sedimentation</td>
<td>0.095</td>
<td>25%</td>
<td>40</td>
<td>3.8</td>
</tr>
<tr>
<td>Odour from WWTP</td>
<td>0.080</td>
<td>12</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Historical Perth WISPS</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>51.6</strong></td>
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</table>

Table 5.8 Historical WISPS For The Perth Wastewater System

![Prioritisation of Wastewater Catchments](image)

Figure 5.24 Prioritisation Of Wastewater Catchments Utilising WISPS
<table>
<thead>
<tr>
<th>Wastewater System</th>
<th>Structural (0.140)</th>
<th>Flooding (0.138)</th>
<th>Water Quality Compliance (0.101)</th>
<th>WWTP (0.132)</th>
<th>Aesthetic Pollution (0.109)</th>
<th>CSO Spill (0.110)</th>
<th>Infiltration (0.094)</th>
<th>Sedimentation (0.095)</th>
<th>Odour Complaints (0.080)</th>
<th>Historical WISPS</th>
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<td>30</td>
<td>20</td>
<td>40</td>
<td>0</td>
<td>51.6</td>
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Table 5.9 Historical WISPS For Various Catchments Within Tayside
If deciding between these wastewater systems in terms of rehabilitation, efforts should be concentrated and prioritised on Forfar. Following the application of the WISPS methodology to Perth, Coupar Angus, Almondbank and Forfar a decision table was drawn up relating the need to rehabilitate a catchment based on historical WISPS. The levels tabulated below are superimposed on Figure 5.24.

<table>
<thead>
<tr>
<th>Historical WISPS</th>
<th>Rehabilitation Requirement</th>
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<tbody>
<tr>
<td>0 to 30</td>
<td>Requires rehabilitation</td>
</tr>
<tr>
<td>31-69</td>
<td>Possible rehabilitation—more data required</td>
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<tr>
<td>70-100</td>
<td>No immediate rehabilitation—monitor performance</td>
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Table 5.10 Historical WISPS—Rehabilitation Requirement

Table 5.10 is based on a limited application of the methodology and numerous wastewater catchments need to be appraised to develop the WISPS methodology further, and to draw more conclusions on the requirement for rehabilitation based on historical scores.

The WISPS methodology has been applied to four wastewater systems within Tayside. The historical scores for each catchment describe adequately the performance of each of the wastewater systems. As the focus of this research project is the wastewater system of Perth this catchment requires detailed appraisal.

Stage II of the methodology is applied to the Perth catchment in Chapter 8 where detailed WISPS are generated for comparison against any proposed rehabilitation options. For detailed WISPS to be generated, computer modelling and investigations require to be carried out. The development of hydraulic and quality models for the Perth system is detailed in Chapters 6 and 7.
CHAPTER 6 HYDRAULIC MODEL DEVELOPMENT FOR THE PERTH SEWERAGE SYSTEM

6.0 REQUIREMENT FOR A HYDRAULIC MODEL

A model of the Perth sewerage system was developed primarily to examine the hydraulic capacity of the drainage network. Until the present study took place very little was understood regarding the behaviour of the Perth system under wet weather events. A model was required by NoSWA to evaluate the performance of the system, and to examine the effects of proposed new developments on the capacity of the network. The model utilised initially was the industry standard WALLRUS. This model was converted to HYDROWORKSTM during the course of the study.

As one of the aims of the project was to investigate sewer flow quality, through the development of a MOSQITO model, the hydraulic model was required to be sufficiently detailed to function accurately with the sewer flow quality model.

6.1 DEVELOPMENT OF WALLRUS MODEL

This Chapter documents the construction, calibration and verification of the Perth WALLRUS model. The model was developed over a 2½ year period from 1991 to 1993. The author was aided by summer vocational students from the University of Abertay Dundee. The model was converted to the more mathematically stable platform HYDROWORKSTM in 1994. This tool was utilised for the analysis carried out in Chapter 8. The model has been utilised by NoSWA for examining the performance of the Perth sewerage system and for aiding in sewer rehabilitation strategies.

6.2 MODEL DEVELOPMENT STRATEGY

The building of a hydraulic model for any sewerage system can be a complex task, depending on the size of model to be constructed, and the level of detail necessary. As stated, the Perth model required to be accurate to work effectively with the sewer flow quality model. If flows and depths are wrong within a hydraulic model then there is no point in attempting to model flow quality until the hydraulic ambiguities are resolved.
High accuracy consequently requires large amounts of information relating to the sewerage network. A CCTV (close circuit television) inspection and manhole survey had been conducted in 1988 throughout the central area in Perth (core sewers). Although this information was highly detailed, similar information was not available for the subcatchments and peripheral areas.

It was decided that the best course of action was to break the catchment down into subcatchments. This was carried out by analysing existing record plans available at the Perth area office. This served a multitude of purposes; the subdivision of subcatchments lead to an understanding of how the system was structured, available manhole information on each subcatchment was collated, ancillary structures were noted and preliminary flow logger positions were selected. Each subcatchment is briefly described below. The subcatchments are outlined in Figure 6.1 and the modelled network shown in Figure 6.2.

6.3 SUB CATCHMENT DEVELOPMENT

6.3.1 Moncrieffe

The subcatchment of Moncrieffe is located to the south of Perth. Moncrieffe is predominately residential with dwellings running from Cromlix Road to Glendevon Road. The sewerage is combined and due to the steep nature of the catchment, pipes were found to be generally below 375mm in diameter. Moncrieffe outfalls into a 1200mm diameter interceptor sewer draining to the Friarton CSO and pumping station.

Two culverted watercourses are present within Moncrieffe. These originate from Magdelene Hill and Friarton Hill. The culverted streams meet on the west side of Edinburgh Road where they combine into one conduit which varies in diameter. The resulting pipe discharges to the interceptor sewer at Friarton Buildings.

6.3.2 Bridgend

The subcatchment of Bridgend is located on the east bank of the River Tay. The subcatchment comprises two areas; Bridgend and Gannochy. The population is approximately 2 600 and the area is predominately residential with no industry.
The sewerage is fully combined and gravitates towards the Willowgate Pumping Station (WPS), where wastewater is pumped through a rising main (300mm in diameter) across the railway bridge, to discharge into the main interceptor in Tay Street on the west bank of the river.

In the outlying areas of the subcatchment the sewers are steeply graded, and fall to meet a 600mm diameter interceptor sewer laid parallel to the river bank (Young, 1866). The sewer is very flatly graded and consequently sediment deposits are numerous along this length. CSOs are located along the interceptor and discharge directly to the Tay.

The first CSO is located within the grounds of a private garden in Mansfield Place. This structure consists of a high level relief pipe terminating at the river wall. On inspection the continuation pipe from the CSO was found to be running ¾ full under DWF conditions. This is due to sedimentation in downstream pipes. The CSO operates frequently and results in severe surcharging which uplifts the cover from the manhole and floods the neighbouring gardens.

Further south a high level relief pipe discharges to the Tay at the rear of a bakery at No.2 Main Street. Operation of this overflow was only witnessed once during the study, but evidence was present of aesthetically polluting material at the base of the river wall, below the CSO discharge. Immediately adjacent a CSO originates from the sewer running through the cross-roads at west Bridge Street/Gowrie Street/Main Street/East Bridge Street. This CSO is “controlled” by a low sided weir and again discharges to the Tay at the rear of the Bakery. Fifty metres downstream another high level relief pipe is present, but is at such a high level relative to the sewer invert, that operation is extremely rare and was never witnessed.

A similar CSO is located opposite the Stanners Island in the area adjacent to a small slipway. This CSO is a high sided weir. The discharge pipe protrudes from the river wall a considerable distance above the neighbouring ground level. The overflow pipe has a flap valve to prevent back flow from the Tay during flood conditions.
Figure 6.2 Perth Sewerage System
Modelled Network
The remaining overflow precedes the WPS. This CSO was originally a single sided weir with raked screens (up to 1994). The rakes, on inspection did not operate and the screens were totally blinded with solids. The point of CSO discharge is some 500m downstream of the pumping station into a channel of the River Tay. This structure has recently undergone rehabilitation.

The main sewer leading to the WPS experiences large degrees of surcharging and flooding has been noted from manholes between Queens Bridge and the northern tip of Moncrieffe Island.

6.3.3 Tullton

The subcatchment of Tullton is located to the north of Perth's city centre. It comprises of Tulloch and Muirton. The subcatchment is predominately residential with railway marshalling yards located centrally. Tulloch's sewerage is generally combined and foul, as is Muirton's. Surface water systems in the Tulloch area drain to the Newton Burn and the Town Lade. Similar systems in the Muirton area drain to the Lade and to a small burn which runs through a neighbouring golf course. Tullton has a population of around 7,000 and no industry is present.

The foul and combined sewers gravitate from Tulloch to the Crieff Road Nursery School, where they meet and continue as a 600mm diameter pipe which conveys flow through Stanley Crescent and Muirton Place, where the sub-area of Muirton connects. The 600mm diameter sewer finally outfalls to an aged interceptor sewer just upstream of the Bells Sport Centre.

6.3.4 Rannoch

The Rannoch subcatchment is located to the west of Perth, between the subcatchments of Hillyland and Craigie. The catchment is predominately residential and no industry exists. The population is approximately 6,500. This subcatchment consists of two hilly areas which are located to the north and south of Rannoch Road. The sewerage system gravitates down the slopes of these hills to be collected by sewers running along Rannoch Road.
The sewerage system is partially separate. The surface water system carries roads runoff. This system gravitates to a large diameter culverted watercourse known as the Goodlyburn. The watercourse originates at the top of Rannoch Road and gravitates along this road and discharges into the Town Lade at the rear of the Fairfield Estate. The partially separate system follows the same route, and finally outfalls as a 750mm diameter sewer to the Crieff Road, where it flows down into the city centre via Dunkeld Road and Atholl Street.

6.3.5 North Muirton

The North Muirton subcatchment is located to the north-west of the city centre and has a population of around 4600. The sewerage system is separate, and the foul sewers outfall to a 750mm diameter sewer, which runs the length of the North Inch to connect into the main interceptor below Perth Bridge. The surface water from North Muirton discharges to the River Tay via twin 1200mm diameter SWO pipes, which are flap-valved to prevent reverse flow from the Tay under flood conditions.

North Muirton contains a small industrial area. A larger industrial area is located to the west; the Inveralmond Industrial Estate contains various units, from FMC (slaughter house) to Pullars (dry cleaners), with foul flows connecting to the North Muirton system. The surface water from Inveralmond outfalls to the River Almond at two discharge points. The sewerage system in this area is very slackly graded and is oversized for the population it serves.

6.3.6 Craigie

The Craigie subcatchment is located to the south-west of the city centre and spans from the A9 in the west to the South Inch in the east. The subcatchment has a population of around 7600 and the sewerage is a mixture of partially separate, separate and combined. Sewerage in the west is predominately separate. As the system progresses eastward it gradually reverts to combined drainage. Any surface water not drained by the foul and combined system discharges to the Scouring/Craigie Burn via a separate system. This burn runs from the Broxden Roundabout in the west to discharge to the Tay in the area of Perth Prison.
CSOs are present in this catchment and are located along the length of Windsor Terrace. These overflows are of the “hole in the wall” type and provide surcharge relief for the system in this area. Discharge is to the Craigie Burn. Four of the overflows have badly fitting flap valves which prevent back flow from the burn during spate conditions.

6.3.7 Central

The central area is defined by the borders of the neighbouring subcatchments. The area is residential on its outskirts and becomes progressively more commercial towards its locus. The sewerage system is combined and many pipes suffer from sedimentation due to slack gradients and frequent surcharging. The population is around 6 000.

Two pumping stations are located along the length of the main interceptor sewer. These are the South Inch Pumping Station (SIPS) and the Friarton Pumping Station (FPS). Both stations contain two archimedes screws which lift the flow up to enable gravity flow to the WWTP at Sleepless Inch. Both installations are preceded by CSOs.

The overflows at both stations are high sided weirs with raked bar screens of 25mm spacing. The rakes do not operate, and as a result total blinding of the screens has occurred (see Appendix C Plate 1). This causes obstruction to any overflow discharge and effectively decreases the operational weir length and the coefficient of discharge for the structure. The outlets to the River Tay at both SIPS and FPS are controlled by flap valves (see Appendix C Plates 3 and 5).

These flaps prevent the Tay from flooding the sewerage system during tidal cycles, but they also effectively prevent any successful discharge from the weirs from reaching the Tay, as the flaps can be held shut by high levels in the river. This obviously exacerbates the surcharging problems experienced along the length of interceptor sewer.
A double inverted syphon is present on the system adjacent to the Prison Area, where the sewerage system runs underneath the Craigie Burn. From record drawings the invert level upstream of the syphon is lower than the invert level at the downstream point. This configuration controls the level in the sewer upstream of the syphon. It is expected that sediment is present within the dual syphons. Both syphons appear to operate under DWF conditions and this is an indication of the overloading of the system.

6.3.8 Hillyland

The subcatchment of Hillyland is located between the subcatchments of Rannoch and Tullton. Hillyland has a population of around 800. The catchment is steep and mainly residential with small industrial units to the west of the catchment. Old housing exists along the Crieff Road with newer developments on the steep hillside to the south of the catchment. The newer housing is serviced by a separate system with surface water draining to the Town Lade near Perth Crematorium.

Foul flows from the new developments drain to a 375mm sewer in Crieff Road. It was expected that the roofed areas from the multi-storey flats located on the hillside drain to this 375 diameter pipe. Another two pipes gravitate along the Crieff Road. The first of these is a “road ditch” pipe which runs beneath the pavement on the south side of the Crieff Road. This sewer drains the surface water from the Crieff Road. The road ditch connects into the Goodlyburn Culvert and discharges to the Town Lade. On inspection wastewater solids were found in this pipe suggesting there are foul connections to the road ditch pipe somewhere along the Crieff Road.

A smaller pipe varying between 225mm to 300mm also runs down the Crieff Road. This sewer has been assumed to drain the old housing located along the Crieff Road, taking foul flows and possibly roof water. This was termed the “old pipe”.
6.4 FIELD DATA COLLECTION

Following work carried out previously, it was clear that a large quantity of data was required relating to the sewerage system and ancillary details in all the outer subcatchments. A team of student engineers were employed to carry out manhole surveys and to record the information on STC25 cards. Each subcatchment was surveyed in turn and the data input to the model sequentially. Area measurement was carried out concurrently, following walkovers of each individual subcatchment.

A sediment survey was carried out by the author and colleagues to collect data on positions and depths of sediment present in the sewerage system. This survey was required to correctly model the reduced hydraulic capacity of affected sewers, and to produce information for use with the MOSQITO model in the next stage of the study.

Various sewers were entered over a period of time ranging from 10/3/93 to 9/6/93. The city centre area was surveyed during the night of 9/6/93. Surveyed depths were input to the model, along with depths taken from the manhole survey carried out in 1988. All depths stated are only accurate at the time of survey and are indicative values rather than precise values. The depths were measured at the mouth of pipes and therefore may not be wholly representative of sediment depths along the length of the pipes.

The site specific approach was utilised to ensure accurate information was gained on the presence of in-sewer sediment, and to ensure adequate representation within the hydraulic model. Prior to the above survey investigations were made into the build-up of sediment in other areas of the Perth sewerage network. The main areas are discussed below.

Sediment was found to be present along the length of the Bridgend interceptor sewer (600mm diameter) from Mansfield Place to WPS. This is due to the slack gradient of the sewer, frequent surcharging and insufficient maintenance. Depths varied between 30mm to 150mm.
The Craigie sewer also suffers from sedimentation across the South Inch. Sediment depths varied between 30mm to 145mm at the time of survey. This sediment was of a coarse nature, typically class A with a small percentage of fines. Due to the nature of the system velocities in this sewer can be low during dry weather and this results in sedimentation.

The interceptor sewer which runs adjacent to Bells Sport Centre near the North Inch had sediment depths varying between 20mm to 150mm. The sediment nature varied between coarse grit to organic fines. The sediment build up is associated with the low gradient and the flow being held back by a 900mm diameter sewer connecting into the interceptor and causing a backwater effect. The sediment build-up in this area was pronounced.

Sediment is also located in the 1200mm diameter sewer running between the SIPS and the FPS. This is due to the slack gradient of the pipe and the effect the inverted syphon has upon the flows (see central subcatchment). A survey was carried out in June 1993 on the sewer below the FPS. Sediment build-up was noted around the area where the 1100mm diameter sewer splits into three smaller diameter sewers, and negotiates two 90° bends over a short length of pipe.

6.5 SEWERAGE SYSTEM ANCILLARIES

Following subcatchment development it was clear that various ancillaries are located throughout the sewerage system of Perth. The majority were CSOs and bifurcations. These are summarised in Appendix D and their significance in relation to the performance of the system is highlighted. The main ancillaries which control the system are the South Inch, Friarton and Willowgate pumping stations. Each of these stations is preceded by a CSO as detailed in section 6.3. The pumping stations are described briefly below.

6.5.1 South Inch Pumping Station

The South Inch Pumping Station is located on the main interceptor sewer, downstream of the railway bridge on the west bank of the Tay. The station contains two archimedian screw pumps, a 10hp pump and a 25hp pump. The
screws are controlled by an ultrasonic device and the sequence of operation is as follows: (1) small pump only, (2) large pump only and (3) both pumps together. The latter condition once selected by the control system is maintained until the level is reduced to a specified limit.

No records were available for head/discharge relationships regarding the archimedian screws. Drop tests were carried out upon the two screws. From these results, and a documented maximum output for each pump, head/discharge relationships were derived for the large screw and the small screw at SIPS to allow their representation within the WALLRUS model.

It was initially thought that modelling of the screws could be done by utilising the facility of inserting a user defined head/discharge relationship. This was attempted but WALLRUS refused to use the selection of flows inserted in the record type. When the pumps switched on in the model, WALLRUS used the maximum flow regardless of the head difference.

The screws were eventually modelled using the design head and design flow parameters associated with the pump record within WALLRUS. This involved specifying a design flow and design head from the graphs developed for both screws.

The actual operation of the pumps could not be modelled by WALLRUS, as the pumps have more than one switch off level. It was decided to model the two screw system as three screws to combat the problem (one screw being a dummy).

6.5.2 Friarton Pumping Station

The Friarton Pumping Station contains two archimedian screw pumps, a 20hp pump and a 50hp pump. The screws are controlled by an ultrasonic device and the sequence of operation is identical to the SIPS.

No records were available for head/discharge relationships regarding the archimedian screws as with the SIPS. Relationships for head/discharge were derived for the large screw and the small screw at FPS. These were derived from
known maximum discharges for the screws and from logger data at Site 1015. The method of modelling for the FPS was similar to that utilised for the SIPS.

6.5.3 Willowgate Pumping Station

The Willowgate Pumping Station was historically a wetwell/drywell configuration. The station consisted of 3 vertical spindle centrifugal pumps. Only two of these pumps were operational while the third remained on standby for emergency purposes during the study period.

The pumping station was subject to heavy inundation from the River Tay during mid January 1993. Such was the damage to the pumping station that a contract was let for a new replacement station and overflow (commissioned early 1996).

The modelled duty point of each pump was specified from records as 50l/s against a total head of approximately 15m. The construction contract effectively replaced like with like, and the new pumping station outputs the same maximum carry-on flow of 100l/s. With no drop test information the pumping station was modelled as fixed discharge pumps which output 45l/s each.

6.6 INSTRUMENTATION

Concurrently with area measurement and model development, instrumentation was installed to collect calibration and verification data. This is described below as are the various locations where instrumentation was installed throughout the catchment.

6.6.1 Rain Gauges

Detectronic tipping bucket 0.2mm rain gauges were installed in four positions throughout the study period. These are listed below:

- On the roof of TRC WSD water treatment works, Gowans Terrace.
- Railway Station yard, located on ground.
- Broomhill Avenue, private garage roof.
- Burghmuir covered reservoir (See Appendix C Plate 2).
Throughout the duration of the study period rain gauges performed adequately. Gauges did malfunction occasionally; due to freezing and heavy winds blowing instruments from roof tops. One gauge installed in a yard near Perth Railway Station was flattened by an excavator during construction work.

Input tests were carried out upon gauges during interrogation to ensure accurate data were being collected. When malfunctions were noted gauges were replaced with new units. Only three gauges were available for the study, which was less than the number recommended in WRc's *guide to short term flow surveys*. Spatial variation was noted especially on short duration "peaky" rainfall events.

Many rainfall events were recorded for verification and every attempt was made to use storms which had in excess of 5mm in volume and were above 12mm/hr intensity. Events used for verification can be seen in Table 6.1.

### 6.6.2 Flow Survey Loggers

IS 32 Detectronic flow loggers (*Detectronic, 1991*) were used to measure depths and velocities throughout the Perth sewerage system. These provided actual flows and depths in the system during rainfall events for comparison with model predictions. Logger calibrations were done regularly at each monitoring site. This involved measuring depths with a metre stick and measuring velocities with a portable ultrasonic velocity probe or valeport propeller meter; preferably with the propeller meter. Where large errors were apparent loggers were removed and replaced by newly calibrated devices.

Due to the limited number of instruments available for the study loggers had to be moved around from site to site instead of carrying out the more appropriate block flow survey. Hydraulic conditions at logger sites highly influenced accuracy and on a number of occasions units had to be removed and repositioned to gather more suitable data. Flow loggers were positioned at 32 locations throughout the Perth sewerage system. These are shown in Appendix D.
6.6.3 Scan Arx Units

Scan Arx units, set-up to measure level were positioned on the CSO outfall pipes at the South Inch and Friarton pumping stations. These units recorded river levels on the flap valves during tidal and storm conditions in the river. The accuracy of these units were limited with the sensor head being masked by river debris on many occasions. The data provided a useful comparison between levels in the river and corresponding levels in the sewer during storm conditions. Calibration checks on these units were not carried out as often as they should have been due to the limitations imposed by the tide preventing access to the outfall chambers.

6.6.4 Data Retrieval

Data were retrieved on a weekly basis from rain gauges and flow loggers during the study period. Data were retrieved using a Sanyo Laptop (4mb, 20mghz) and Husky Hunter PCs. Problems were encountered using both systems. Problems with the Hunters were usually associated with battery failures and overloading of the memory. A common problem encountered was the malfunction of the communication lead used to transfer information from the logging device to the Laptop or Hunter.

6.7 MODEL STABILITY

Following full construction of the model, stability runs were carried out using specified storms to test the model under severe conditions. The model behaved reasonably well and produced the volume balances indicated. Runs were carried out with M5-60min and M50-60min storms. The overall volume balance for the former storm was +2.99% and for the latter +0.94%. This indicated that the model was extremely stable under severe hydraulic conditions.

Heavy surcharging occurred upstream of the SIPS in the model and volumes of flooding were produced. The areas in which flooding occurred with the design storms were areas in Perth where above ground flooding had been observed during the study period. This was classed as historical verification.
Areas suffering from localised flooding included; Tulloch Works, outside Bells Sports Centre, South Inch and various positions along the Bridgend interceptor sewer.

However, from studying volume balances at the affected tanks in the model no major volume imbalance occurred. Some pipes were noted to display negative flows. This was deemed possible due to the heavy surcharging and flat gradients of certain parts of the sewerage system. A listing of the Perth model is presented within Appendix D.

Simplifications made during the construction of the model, do not appear to affect predicted areas of flooding. Modelling simplifications are discussed in Appendix E.

6.8 MODEL VERIFICATION

Verification was a laborious process due to the nature of the flow survey employed. A block survey would have been better and would have led to the model being verified quicker. The installation of more rain gauges would have given more information on spatially varied events.

Verification events were of a blanket nature, i.e., the volume, intensity and duration were similar across the catchment at each of the three rain gauges for the chosen event.

All verification fits are reasonable and are within the realms of accuracy quoted in WaPUG's *Code of Practice for Hydraulic Modelling*. Verification graphs are presented in Appendix F along with a discussion on the accuracy of verification. Storms used for verification are shown in Table 6.1.
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<td>29/3/93</td>
<td>6</td>
<td>27.8</td>
<td>1146</td>
</tr>
<tr>
<td>10/6/93</td>
<td>12</td>
<td>2.2</td>
<td>38</td>
</tr>
<tr>
<td>22/8/92</td>
<td>12</td>
<td>7.6</td>
<td>196</td>
</tr>
<tr>
<td>26/4/93</td>
<td>6</td>
<td>2.6</td>
<td>240</td>
</tr>
<tr>
<td>26/6/93</td>
<td>6</td>
<td>9.6</td>
<td>608</td>
</tr>
<tr>
<td>23/7/93</td>
<td>6</td>
<td>4</td>
<td>332</td>
</tr>
</tbody>
</table>

Table 6.1 Storm Events Used For Verification

6.9 CONVERSION OF WALLRUS TO HYDROWORKS™

Due to the problems of using WALLRUS the Perth hydraulic model was subsequently converted to HYDROWORKS™ in late 1994. This allowed accurate modelling of the archimedean screw pumping stations and CSOs, inverted syphon and triple pipes along the length of the interceptor, due to the greater flexibility of the modelling tool now employed to represent the sewerage system.

Modifications were made to the model to represent the overflows at the pumping stations and the inverted syphons more efficiently. The advantage of this tool is that long sections and plan views can be used in replay mode which allows the engineer to view the sewerage system’s behaviour during chosen rainfall events. This tool was used for the hydraulic analysis of the sewerage system described in Chapter 8.
Subsequent work by a complementary research project utilising the Perth HYDROWORKS™ model has carried out flow logging in other locations within the catchment, and fits have been excellent with respect to flow, depth and velocity. The hydraulic model WALLRUS formed the basis for the sewer flow quality model MOSQITO. The construction of this model and the WWTP model STOAT are presented in Chapter 7.
CHAPTER 7 DEVELOPMENT OF WASTEWATER QUALITY MODELS

7.0 INTRODUCTION

This Chapter discusses the development of deterministic quality models for the wastewater system of Perth. The models under consideration are MOSQITO (Wallingford Software, 1993) and STOAT (Dudley and Dickson, 1992). The Chapter looks at data collection, calibration and verification of the models.

The model MOSQITO and its limitations for an holistic approach are examined in detail and conclusions are drawn which imply that sewer flow quality modelling is not sufficiently developed to be utilised in an integrated modelling approach without a great deal of caution.

7.1 THE DEVELOPMENT OF A MODEL FOR PERTH WWTP~STOAT

Sleepless Inch Wastewater Treatment Plant was built in 1971 to deal with the City of Perth’s wastewater. Previous to the commissioning of this plant all wastewater was discharged to the River Tay from various outfalls.

The plant is located to the south east of the City and contains screening, grit removal, storm settlement, primary settlement, activated sludge aeration and final settlement. Details of the plant are presented in Appendix K.

Sludge from satellite treatment works around the Perthshire area is injected into the sewerage system immediately downstream of the Friarton pumping station. This results in very variable and acute loadings of the plant. Table 7.1 shows results of influent TSS sampled (compositely) prior to the STOAT study taking place.
Concern was expressed early in the study as to the magnitude of the variance in the influent with respect to these sludge loadings. Also questioned, was the effect the discharges would have on the ability of the model to represent the plant as it really operated under temporal loadings and not just under the conditions experienced during the data collection exercise.

7.2 QUALITY STANDARDS

Consent standards for the WWTP are relatively high, which reflects the dilution available in the River Tay. The standards are 100mg/l for Total Suspended Solids and Biochemical Oxygen Demand.

Throughout the history of the plant, performance has been extremely good with respects to consent standards. A synopsis of results is shown in the table below for the period 5/11/75 to 22/2/92.

<table>
<thead>
<tr>
<th>Times</th>
<th>Date</th>
<th>TSS (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1140-1240</td>
<td>14/2/93</td>
<td>809</td>
</tr>
<tr>
<td>1540-1640</td>
<td>14/2/93</td>
<td>1777</td>
</tr>
<tr>
<td>0730-0830</td>
<td>15/2/93</td>
<td>821</td>
</tr>
<tr>
<td>1200-1300</td>
<td>24/2/93</td>
<td>4186</td>
</tr>
<tr>
<td>1800-1900</td>
<td>25/2/93</td>
<td>2702</td>
</tr>
<tr>
<td>0200-0300</td>
<td>25/2/93</td>
<td>994</td>
</tr>
<tr>
<td>1200-1300</td>
<td>4/3/93</td>
<td>4423</td>
</tr>
</tbody>
</table>

Table 7.1 Total Suspended Solids Contained Within Influent At Sleepless Inch

Table 7.2 Synopsis Of Wastewater Quality At Sleepless Inch 1975 To 1992
7.3 STOAT MODEL CONSTRUCTION

7.3.1 Introduction

Dynamic wastewater treatment plant models such as STOAT are intended to model and predict performance over dry weather periods and storm events. Modelled determinands are generally flow, TSS, BOD, ammonia, DO and oxidised nitrogen. The STOAT model incorporates a number of constituent process models. The processes applicable to the modelling of Sleepless Inch are listed below:

- Storm and primary tanks are represented in STOAT by models previously developed (Lessard and Beck, 1988). These models have proved robust and satisfactory.
- The Activated sludge process is represented in STOAT by the WRc activated sludge model (Jones, 1978). Some alternative packages use the IAWQ Activated Sludge Model No.1. The latter is the most prevalently used model to represent the activated sludge process.
- The final settling process is modelled by the currently accepted model of (Takacs et al, 1991).

The Perth STOAT model was built with advice and assistance from staff at WRc, Swindon. Perth was the first model to be constructed using the enhanced STOAT software which ran through a DOS WINDOWS environment. Working with WRc staff, the model took a week to calibrate and verify, to an adequate level of accuracy.

7.3.2 Field Data Collection

Data collection was carried out in conjunction with the Water Services Department Quality and Treatment section based at Clatto Laboratories. Sample analysis was carried out by the same department in accordance with quality control procedures.
The data collected are specified below:

- Flows were monitored at positions shown on Fig 7.1 (marked F); screened sewage before and after overflow to storm tanks, flows of settled sewage, return storm sewage and sludge, flows of final effluent, flows of treated storm sewage overflowed and flows of any sewage overflowed prior to aeration.

- Flow monitoring was carried out using Detectronic IS32 flow monitors. Levels were monitored at the RAS pumphouse and each of the sludge holding tanks. These were measured using Warren Jones WJ 460 units.

- Wastewater quality was monitored at positions shown on Fig 7.1 (marked Q); screened sewage before overflow to storm tanks (see Appendix C Plate No.10), storm tank overflow, storm tank return, settled sewage, MLSS in each of the aeration legs, DO in each pocket of the centre aeration lane (see Appendix C Plate No.11), final effluent from three clarifiers at one point (see Appendix C Plate No.12) and MLSS at the RAS pumphouse.

- All samples were two hourly composites analysed for BOD, NH₃-N and TSS, TON and SRP. Every fourth sample was analysed for soluble BOD and non-settleable TSS.

- Daily SSVI measurements were taken from each lane of the aeration unit.

- Daily temperature was taken from crude sewage after screening.

Samples of primary sludge were taken whenever the primary tanks were desludged and formed a composite sludge sample for each day of monitoring. Data is presented showing metal contents within the composite sludges sampled in Table 7.3.

- The frequency of activated sludge surplusing was recorded by level monitors.
Figure 7.1 Sleepless Inch Flow And Quality Monitoring
In general the SSVI results for the samples taken during the study period indicate performance of a poorly settling sludge, as they are valued above 150. This is due to the variable nature of the influent and the uncontrolled nature of the activated sludge process. The drop in SSVI results shown on the graphs corresponds to the influx of storm water. Analysis of primary sludge shows the solids to be around 5%, which may be indicative of sludge thickening in the primary tanks.

<table>
<thead>
<tr>
<th>Date</th>
<th>19th</th>
<th>20th</th>
<th>21th</th>
<th>22th</th>
<th>23th</th>
<th>24th</th>
<th>25th</th>
<th>26th</th>
<th>27th</th>
<th>28th</th>
</tr>
</thead>
<tbody>
<tr>
<td>%ge dry matter</td>
<td>5.56</td>
<td>5.34</td>
<td>5.51</td>
<td>4.10</td>
<td>4.66</td>
<td>6.55</td>
<td>5.61</td>
<td>7.68</td>
<td>6.74</td>
<td>6.03</td>
</tr>
<tr>
<td>Copper(mg/kg)</td>
<td>230</td>
<td>238</td>
<td>235</td>
<td>231</td>
<td>213</td>
<td>248</td>
<td>235</td>
<td>243</td>
<td>237</td>
<td>231</td>
</tr>
<tr>
<td>Zinc(mg/kg)</td>
<td>572</td>
<td>581</td>
<td>561</td>
<td>584</td>
<td>520</td>
<td>564</td>
<td>538</td>
<td>589</td>
<td>593</td>
<td>543</td>
</tr>
<tr>
<td>Nickel(mg/kg)</td>
<td>17.8</td>
<td>23.60</td>
<td>26.8</td>
<td>21</td>
<td>20.9</td>
<td>20.50</td>
<td>19.40</td>
<td>35.20</td>
<td>23</td>
<td>19.3</td>
</tr>
<tr>
<td>Cadmium(mg/kg)</td>
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<td>2.30</td>
<td>2</td>
<td>2.30</td>
<td>2.20</td>
<td>2</td>
<td>2.3</td>
<td>2.60</td>
<td>2.70</td>
<td>1.9</td>
</tr>
<tr>
<td>Lead(mg/kg)</td>
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<td>403</td>
<td>403</td>
<td>416</td>
<td>382</td>
<td>426</td>
<td>405</td>
<td>444</td>
<td>415</td>
<td>372</td>
</tr>
<tr>
<td>Chromium(mg/kg)</td>
<td>33.6</td>
<td>38.90</td>
<td>38</td>
<td>2.3</td>
<td>34.50</td>
<td>36.50</td>
<td>34.30</td>
<td>36.40</td>
<td>36.2</td>
<td>36.2</td>
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<tr>
<td>Molbdenum(mg/kg)</td>
<td>4.73</td>
<td>4.93</td>
<td>4.68</td>
<td>4.15</td>
<td>4.10</td>
<td>5.02</td>
<td>4.6</td>
<td>4.5</td>
<td>4.6</td>
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</tr>
<tr>
<td>Arsenic(mg/kg)</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
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<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Selenium(mg/kg)</td>
<td>1.33</td>
<td>1.64</td>
<td>1.47</td>
<td>1.40</td>
<td>1.24</td>
<td>1.24</td>
<td>1.27</td>
<td>1.35</td>
<td>1.49</td>
<td>1.19</td>
</tr>
<tr>
<td>Mercury(mg/kg)</td>
<td>2.26</td>
<td>1.78</td>
<td>2.39</td>
<td>1.78</td>
<td>1.96</td>
<td>1.68</td>
<td>1.93</td>
<td>2.07</td>
<td>1.75</td>
<td>1.74</td>
</tr>
</tbody>
</table>

Table 7.3 Metal Content Within Primary Sludge April 1993

Data used to calibrate and verify the STOAT model were collected between 18/4/93 and 30/4/93. This period proved to be wet with intermittent rain occurring, on all but three days (typically Scottish DWF conditions).
Precipitation occurring during the STOAT study is shown in Figure 7.3. Figure 7.4 shows average (two hourly) inflows over the study period measured before overflow to storm settlement.

The storm event used for verification was collected on the 19/4/93 and dry days were experienced on 24/4, 27/4 and 29/4 (although rain fell on the catchment on the 28th-29th no apparent increase in flow to the plant was observed).

![Figure 7.3 Rainfall During STOAT Monitoring Period](image)

Analysis carried out on the data showed that the sludge import from smaller works around Perthshire had a significant effect on the influent reaching the plant. Typical mean SS:BOD values of 350:250mg/l for a dry day frequently included peaks of around 1000:600mg/l between 1300 hours and 1700 hours. In contrast, mean values of 145:81mg/l were recorded on a Sunday when no imports took place. Most solids settled out within the primary tanks.
A comparison is shown relating the data from 27-28 April, 28-29 April and 29-30 April in the following graphs. This data shows marked variation over the period 10am to 6pm GMT. The data are similar at all other times. This clearly indicates that DWF quality is grossly influenced by the input of satellite sludge to the treatment plant.
TSS (kg/day) loadings are compared below. The data indicates variation through the period 10am to 6pm which is concurrent with the import of sludge to the plant. The results show remarkable agreement for the time periods 2 to 8 and 20 to 24 (outwith the time of sludge import).

From Figure 7.8 it can be seen that the DWF profile for the works is very similar for each of the days graphed and high levels of infiltration are present (approximately 150l/s) in the early hours of the morning. A fuller discussion of
infiltration and its effects on the performance of the drainage catchment are given in the section 7.12.

![Comparison of DWF-Perth STOAT model](image)

**Figure 7.8 Comparison of Influent DWF**

Figures 7.9 and 7.10 show typical dry weekday and weekend quality data for the influent sewage. The graphs presented show a marked difference in the influent relative to weekday and weekend behaviour, again highlighting the impact of imported sludge on the plant.

![Comparison of influent Total Suspended Solids Concentrations-Perth STOAT model](image)

**Figure 7.9 Comparison of Weekday/Weekend TSS**
Rainfall affected the quality data for the Sunday shown. The data presented for the Sunday should only be taken in comparison with the other dry days up to and including 1400 hours. After this point data are affected by storm flows.

Analysis of the storm data collected for the Sunday (25/4-26/4 after 1400 hours) shows an interesting occurrence with respect to the performance of the primary settlement tanks. In general terms, the same hierarchical behaviour (see Figures 7.11 and 7.12) with respect to influent quality can be seen. Weekday settled BOD is more concentrated than Saturday, which in turn is more concentrated than Sunday.

The TSS performance is not so conclusive, it can be noted that the TSS increases dramatically from 1400 hours to 1800 hours for the Sunday data. This may be the effect of settled sludge being disturbed by the increase in flows entering the primary tanks and consequently causing settled solids to be resuspended into the flows exiting the primary tanks. This is supported by the solids content of the primary sludge indicating thickening and a large reservoir of sludge in the primary tanks.
Consent standards have been indicated on Figures 7.11 and 7.12. This clearly shows that the tanks are extremely efficient in removing a significant proportion of the incoming load.

From discussions with operatives and local staff it is apparent that the primary tanks are not desludged as frequently as they should be, due to the limited capacity of the sludge thickening tanks at the plant (compared to the loading imposed by the imported sludge). This leads to a build up of sludge within the primary tanks and consequently anaerobic conditions.
When desludging of the tanks takes place odours are very much in evidence. At the time of writing the author is presently supervising construction for specific odour abatement measures for the sludge holding tanks at Perth.

Figure 7.13 shows MLSS from lane one of the activated sludge plant and indicates the effect of typical weekday and weekend variation in influent quality. The MLSS drops in value over the period of the storm event previously discussed.

Data collected from another storm event occurring on the 19/4/93 were examined to see if similar washout behaviour was exhibited by the primary tanks. Influent data are shown in Figures 7.15 and 7.16.
The influent quality for the storm event shows an increase in ammonia, TSS and BOD with the arrival of the storm flows. However, it cannot be concluded that this is a flush from the sewerage system arriving at the plant. It may be caused by the normal dry weather flow quality receiving dilution. It also may be influenced by discharges from sludge tankers. The ammonia after reaching a peak receives dilution. Primary settlement tank performance related to this event is shown in Figures 7.17 and 7.18.
From examination of the figures it can not be concluded that the primary tanks suffer from resuspension of sludge during storm flows. However, it is likely this is the case given the operational nature of the primary tanks, with respect to the frequency of desludging.

Final effluent varied between 10mg/l and 20 mg/l for TSS and between 5mg/l and 10mg/l for BOD in dry weather during the study period. Ammonia levels did not change throughout the treatment process and averaged around 15 mg/l. Unfortunately, a build up in the concentration of MLSS within the mixed liquor of the activated sludge units occurred. This phenomenon caused difficulties in the
calibration and verification stages of model development. It is believed that this is due to the uncontrolled behaviour of the activated sludge process.

![Comparison of final effluent BOD-Perth STOAT model](image)

**Figure 7.19 Comparison Of Final Effluent BOD**

![Comparison of Final effluent TSS-Perth STOAT model](image)

**Figure 7.20 Comparison Of Final Effluent TSS**

The final effluent data shown in Figures 7.19 and 7.20 compares the performance of the plant with respect to weekday and weekend operation and shows the performance to be comfortably within the consent standards. The data for Sunday show a peak of suspended solids leaving the plant at approximately 2200 hours. This is suspected to be associated with the peak of solids washed out from the primary tanks by the impact of the storm occurring on the Sunday.
7.4 CALIBRATION OF THE PERTH STOAT MODEL

7.4.1 Primary Settlement Tanks

Calibration of the model was carried out following standard approaches (Dudley and Dickson, 1992). The primary tanks and activated sludge process were calibrated together as surplus sludge is co-settled in the primary tanks. Initially, there was insufficient settlement in the primary tanks. This was improved by increasing the settleability of the solids to 0.9, the proportion of particulate BOD to 0.8, and settleable fraction of that BOD to 0.9. These were the maximum values which could reasonably be assumed with respect to the crude sewage measured data. Improvements in the activated sludge calibration led to a good match for TSS, BOD and ammonia for DWF. These are shown in Figures 7.21, 7.22 and 7.23.

Figure 7.21 Calibration of Settled TSS

Figure 7.22 Calibration of Settled BOD
7.4.2 Activated Sludge

Dissolved oxygen was initially chosen for the control mechanism for the activated sludge plant. The DO set points for each pocket were set at 1mg/l, 1.5mg/l and 4mg/l which was deemed appropriate from the monitored data. KdA was estimated as no data were available for this parameter; a value of 12 was chosen. The Waste Activated Sludge (WAS) rate was set at 3.2 l/s from data collected on site.

Initially simulations with this data did not prove satisfactory and the control was altered to a MLSS set point of 1650 mg/l. This value was typical of measurements taken within the MLSS from 27/4 to 28/4. More problems were apparent when simulations were carried out as the liquor never reached setpoint. The cause of this was identified as a wrong flowrate of RAS.

A value of 120l/s (calculated empirically) was entered into the model and this proved to solve the problem and an improvement in calibration results occurred. This value was derived from a mathematical mass balance across the aeration system. This figure was confirmed to be correct following on site checks of the RAS flowrate carried out using a valeport propeller meter.

The maximum possible settling velocity (Vo) and the exponential constant for hindered settling (k) for the final settling tanks were estimated using SSVI data collected. The value of Vo was 1.52 compared with a default value of 5.625. This proved to be low and Vo was increased to 2.5. The value of k was estimated at 0.00051. The exponential constant for settling at low solids concentrations (p)
was increased to 0.03. These changes resulted in a very good match for ammonia. Suspended solids predictions were good but the shape and the timing of the profile were poor. BOD predictions were generally good. Figures 7.24, 7.25 and 7.26 show DWF calibration results.

Figure 7.24 Calibration of Final Effluent TSS

Figure 7.25 Calibration of Final Effluent BOD
7.4.3 Storm Tanks

Calibration of the storm tanks was carried out using data from 19th-20th April. This calibration was limited as overflow only occurred over a short period of time. However, the following points were noted. The level of overflow from the tanks was under predicted. The actual overflow took place over $1\frac{\text{ }}{2}$ hours, (less than 10l/s for the last $\frac{1}{2}$ hour). Average values for measured composite SS:BOD:NH$_3$-N in mg/l were 160:63:7.6 for the first hour. Predicted values for SS:BOD:NH$_3$-N in mg/l were 350:97:7.7. It can be seen that SS and BOD are overpredicted by 119% and 54% respectively. Ammonia predictions however are very good.

7.5 VERIFICATION OF THE PERTH STOAT MODEL

Verification of the model was achieved by running storm data through the plant. Only one storm, sufficient to cause a storm overflow was recorded at the beginning of the data collection exercise. Although calibration of the model was not fully successful, runs were carried out in order to achieve a feel for the ability of the model to predict under extreme circumstances. This was performed by running a five day influent series, 18th to 23rd April, through the model which included the storm event on the 19th of April (see Appendix G for verification graphs).
7.5.1 Storm Tanks

The data collected for the storm tanks during the event 19th-20th were used to calibrate the storm tank process, and therefore it was not possible to verify storm tank behaviour.

7.5.2 Primary Tanks

Suspended solids results for the storm itself were poor. Peak TSS were overpredicted. The last three days results were much improved, although peak solids levels appear to increase steadily. By contrast, the BOD results for the storm are reasonable, although the peak is underpredicted by about 20mg/l. The following three days show over predictions of peaks for two of the days, one being over 100%. Ammonia predictions however are good. Timing of the predicted data and measured data are also in agreement. These predictions are shown in Appendix G.

7.5.3 Activated Sludge Process

The activated sludge process was not satisfactorily calibrated before verification was attempted. Timing of predicted and measured results for the storm data match well and while ammonia predictions are good, TSS and BOD are both overpredicted. These predictions are shown in Appendix G. WAS flows were reduced to zero for several days in the model. This was due to wasting not taking place because the MLSS setpoint was not reached within the activated sludge process. The setpoint as previously described was 1650mg/l, a typical value for the day used for calibration, but not for the day of the storm event. A typical value for MLSS on the day of the storm event would have been around 1000mg/l. The varying MLSS values within the activated sludge process with respect to the set value within the model are suspected of being the source of the errors.

7.6 CONCLUSIONS ON STOAT MODEL

7.6.1 Settled Sewage

A good fit between predicted and observed data has been achieved for dry weather data. Timing of peaks are within two hours and peak concentrations are
within approximately 5% of measured peaks. To achieve these matches high settleable fractions were used, but this is appropriate because of the import of sludges from other treatment plants to Sleepless Inch. A reasonable match has been achieved for the storm weather data. The timing of peaks for ammonia is very good indicating the correct number of CSTRs (continuous stirred tank reactors) have been used to model the primary tanks. Settled TSS and BOD are overpredicted for the storm event.

7.6.2 Final Effluent

BOD is reasonably predicted during dry weather, while predicted TSS concentrations are good the profile is poor. Both DO and MLSS are set at measured levels in the model. As DO measurement is probably less accurate than MLSS, oxygen limitation is the most likely cause of the mismatch. Storm predictions are generally poor. TSS over prediction coincides with peaks in settled sewage and may be due to the MLSS set point being too high, leading to low, and sometimes zero wastage rates. Some of the BOD over prediction will be due to the TSS over prediction.

However, there is still a significant over prediction of BOD at times of high flow. This is a recognised weakness in the STOAT model and is due to the poor applicability of the activated sludge model for BOD at low retention times. Ammonia prediction is good. The increase in MLSS over the 12 days of data collection has made it difficult to verify the model due to the variability of the plants response to the variable influent.

7.6.3 Storm Tank Effluent

Storm tank calibration is very crude due to the small amount of data collected as a result of the small duration of the storm sampled. Another storm needs to be collected to enable verification of the storm tank process. This storm requires to be of sufficient volume of flow to the storm tanks to allow more quality data to be collected. Data were not collected to further verify the storm tanks due to lack of time and resources.
7.6.4 Discussion On STOAT Model Of Perth

It is apparent from examination of the data that the treatment plant produced good quality effluent in accordance with the specified regulatory standards for BOD and TSS. The activated sludge plant did not nitrify and levels of ammonia leaving the plant were comparable with those entering the plant.

There are major concerns over the validity of the models calibration with respect to the sludge import. As stated previously, this import of sludge from satellite treatment works is injected at an upstream pumping station. It is highly variable in nature and calibration parameters for each of the processes were adjusted to achieve accurate calibration based on one day of DWF quality analysis.

Each time a slug of imported sludge is injected, the quality of the influent may be drastically different, and the models performance under these differing conditions from that on which it was calibrated must be brought into question. This is clearly an example of the real variability of pollutants within the sewerage system and the consequential difficulty in attempting to produce deterministic detailed representations of this variability. Accuracy of the model was acceptable under storm verification runs, which involved running a number of consecutive days of data including the storm event through the model.

Levels of MLSS fluctuated from 1000mg/l to 2000mg/l during the data collection period and a median value was settled on for calibration and verification. The plant is not controlled with respect to dissolved oxygen and MLSS and consequently the model may perform differently from the real system, when real MLSS figures change from the figure used to calibrate the model. The inaccuracies associated with the models prediction of final effluent during the storm verification are not all due to the integrity of the model to represent the biological processes, but are due to the variances in the biological processes within the works responding to a highly variable influent.

The STOAT exercise demonstrated that the model could predict the performance of an activated sludge works with a reasonable degree of accuracy. However,
doubts exist as to the models ability to predict long term pollutant performance under varying conditions of MLSS in the reactor and variable content within the influent.

7.7 DEVELOPMENT OF A SEWER FLOW QUALITY MODEL FOR THE SEWERAGE SYSTEM OF PERTH-MOSQITO

7.7.1 Introduction

MOSQITO is a sewer flow quality simulation model which utilises the WALLRUS computer package to provide hydraulics to transport the pollutants being modelled. MOSQITO simulates the movement of sediments and pollutants within sewerage systems. MOSQITO was developed over a period of some ten years, an indication of the complexity of the phenomenon attempting to be modelled. In the Spring of 1995 the replacement for MOSQITO was released.

The new model, HYDROWORKS-DM, runs in parallel with HYDROWORKSTM. Unfortunately, producing a more glossy package, might look better, but still does not address the fundamental problems discussed towards the conclusion of this Chapter, and in the opinion of the author may be a step in the wrong direction. The work presented here relates to MOSQITO 1.5, and its application to the drainage network of Perth, but the conclusions equally apply to any sewer flow quality model based on the same deterministic principles.

7.8 THE THEORY BEHIND MOSQITO

MOSQITO models the movement of pollutants and sediments through the application of advection and dispersion equations. The underlying assumption is that pollutants and sediments travel at the same speed as the pipefull velocity. The effect of dispersion is assumed to be negligible as a result of the high velocities of flow compared to the dispersion speed of the pollutants against the flow. Clearly, fine sediments may travel at the speed of the velocity but coarser material will not.

Sediment is transported through the sewerage network in the same way that dissolved pollutants are. Very fine sediment is carried as washload (this is not effectively modelled by MOSQITO). Coarser sediment (at present fine and coarse
fractions in MOSQITO) is transported by the Ackers and White equation as a mixture of suspended load and bed load. The transport rate is regulated by velocity of flow, hydraulic radius, width of sediment bed, sediment size and sediment density.

The Ackers and White formula is subject to certain limitations:

- Sediments in sewers are widely graded whereas the formula is relative to narrowly graded sediments.
- The formula was developed for steady state conditions whereas the conditions during rainfall events are unsteady.
- The formula was based on alluvial rivers with an unlimited supply of material available for erosion. In sewers there is only a limited supply of erodible material from the sewer bed.
- The formula ignores fine suspended load (i.e., sediments in the silt/clay range).

Sediment deposits are modelled in MOSQITO in two layers. The active layer of unconsolidated sediment, consisting of mainly organic material is generated automatically by the programme depending on the dry weather flow conditions. The storage layer must be defined by the user from knowledge of the system. The storage layer is consolidated and lies beneath the active layer. This layer is a mixture of organic and inorganic material. This material has a shear strength which must be overcome before erosion can take place. The storage layer contains interstitial fluid which may contain high concentrations of pollutants.

Pipe sediment is eroded into the flow when the shear strength of the sediment is exceeded. Pollutants are released into the flow and transported through the sewerage system. The release of pollutants from eroded sediments is recognised as being a substantial component of the pollutant load during storm events (Ashley et al, 1993 and 1994 and Verbanck, 1994).
7.9 THE STRUCTURE OF MOSQITO

There are four sub models in MOSQITO which represent:

- Washoff from catchment surfaces.
- Foul water inflow.
- Pollutant behaviour in pipes and channels.
- Pollutant behaviour in ancillary structures.

7.9.1 Washoff From Catchment Surfaces

This submodel represents the removal of sediments and pollutants from catchment surfaces. This is represented by a modified form of a model first developed by Price and Mance. Washoff from roads and the consequent discharge from gully pots are not modelled explicitly, but are taken into account during the calibration process. The most important process contributing washoff is erosion of surface sediment by rainfall impact. The surface washoff model is severely limited in several ways due to the small set of field data used to calibrate the submodel within MOSQITO.

7.9.2 Foul Water Inflow

MOSQITO has the ability to generate dry weather flows and concentrations for foul water inflow. The user can enter these flows as distributed over area, as individual flows or a mass flow. The distributed area method is recommended. Default values for average flows and pollutants are provided with the programme. Diurnal factors are also provided for pollutants and flows.

7.9.3 Pollutant Behaviour In Pipes, Channels And Ancillary Structures

Manholes and junctions within the sewerage system are treated as simple mixing chambers. The pollutants in all inflows are mixed to give a uniform outflow concentration. Dissolved pollutants within CSOs with storage or tanks are assumed to have the same concentration in the inflow as the outflow and overflow. Sediments are allowed to settle, as are the pollutants attached to them. The settlement model is based on classical sedimentation theory using the surface loading rate of the tank. The efficiency of the structure is defined by a factor which varies between 0 and 1 (0 being a fully mixed tank and 1 being a fully
efficient tank). Little information is available on the efficiency factors applicable to modelling sewerage system structures. The engineer must estimate a figure for the efficiency factor for a CSO structure, which could drastically influence the results. Some suggest building a physical model of the structure (Gent et al, 1994) to gain information on efficiency, a ploy which seems to belittle deterministic computer modelling.

7.9.4 Determinands Modelled By MOSQITO

MOSQITO has the ability to model BOD, COD, TSS, ammonia and other user defined determinands. Pollutants can be dissolved or attached to the sediments in transport. Dissolved pollutants are considered to travel at the same velocity as the flow they are within. Pollutants attached to sediments are assumed to be eroded and deposited at the same rate as the sediment fractions they are attached to. The determinands chosen for the Perth MOSQITO model for calibration and verification purposes were COD, TSS and ammonia.

Sediments originating from the catchment surface and from the erosion of pipe deposits are assumed to have fixed amounts of pollutants attached to them. MOSQITO allows masses of these pollutants to be attached to masses of sediment (kg pollutant/kg sediment) through the application of potency factors.

7.10 CALIBRATION AND VERIFICATION OF THE PERTH MOSQITO MODEL

7.10.1 Data Collection—Dry Weather Flow

For MOSQITO calibration and verification to be successful DWF and Dry Weather Pollutants (DWP) must be modelled accurately. For the Perth model this necessitated the selection and monitoring of quality and flows during dry weather at a number of key sites within the catchment. DWF and DWP must be monitored at positions far enough up in the subcatchments, so that no upstream sedimentation can influence true DWP quality. However, the monitoring points must be sufficiently far down the system to enable commonly used flow loggers to accurately monitor depths and velocities.
<table>
<thead>
<tr>
<th>Site No.</th>
<th>Location</th>
<th>Pipe details</th>
<th>Subcatchment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1001</td>
<td>In pathway adjacent to Tescoes Superstore</td>
<td>375mm diam</td>
<td>Moncrieffe</td>
</tr>
<tr>
<td>1002</td>
<td>Above Tulloch Works</td>
<td>375mm diam</td>
<td>Tullton</td>
</tr>
<tr>
<td>1003</td>
<td>Inveralmond Industrial Estate</td>
<td>600mm diam</td>
<td>North Muirton</td>
</tr>
<tr>
<td>1004</td>
<td>Tayflatts Cottages</td>
<td>375mm diam</td>
<td>Bridgend</td>
</tr>
<tr>
<td>1009</td>
<td>Culverted Watercourse</td>
<td>600mm diam</td>
<td>Moncrieffe</td>
</tr>
<tr>
<td>1010</td>
<td>Gowans Terrace Backwash</td>
<td>300mm diam</td>
<td>North Muirton</td>
</tr>
<tr>
<td>1011</td>
<td>Rannoch Road</td>
<td>750mm diam</td>
<td>Rannoch</td>
</tr>
<tr>
<td>1012</td>
<td>Grassed area, adjacent to Bute Drive</td>
<td>750mm diam</td>
<td>North Muirton</td>
</tr>
<tr>
<td>1015</td>
<td>Inlet to Friarton PS(outfall)</td>
<td>1200mm diam</td>
<td>Central</td>
</tr>
</tbody>
</table>

Table 7.4 Sampling Sites—Perth MOSQUITO Model Summer 1993

The sites selected for quality monitoring are summarised in Table 7.4 and the relevant data sets from each site are shown in Table 7.5. The exercise involved the installation of Epic flow samplers and IS32 loggers to collect quality and flow data respectively (See Appendix C Plate No.s 7 and 9).

Samplers were controlled to collect data every hour from the DWF sites. This involved programming the samplers to take a composite sample over the hour, made up of sample shots taken every 15 minutes. Samplers frequently malfunctioned and required constant maintenance in the field.
<table>
<thead>
<tr>
<th>Date</th>
<th>Site 1001</th>
<th>Site 1002</th>
<th>Site 1003</th>
<th>Site 1004</th>
<th>Site 1011</th>
<th>Site 1012</th>
<th>Site 1015</th>
</tr>
</thead>
<tbody>
<tr>
<td>25 Mar</td>
<td>19</td>
<td>24</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30 Mar</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 April</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 June</td>
<td>18&lt;sup&gt;w&lt;/sup&gt;</td>
<td>24&lt;sup&gt;w&lt;/sup&gt;</td>
<td>24&lt;sup&gt;w&lt;/sup&gt;</td>
<td>19&lt;sup&gt;w&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9 June</td>
<td>18</td>
<td>24</td>
<td>23</td>
<td></td>
<td>21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23 June</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28 June</td>
<td></td>
<td></td>
<td>24&lt;sup&gt;w&lt;/sup&gt;</td>
<td></td>
<td></td>
<td>24&lt;sup&gt;w&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>6 July</td>
<td>24</td>
<td></td>
<td></td>
<td></td>
<td>24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19 July</td>
<td>24&lt;sup&gt;w&lt;/sup&gt;</td>
<td></td>
<td></td>
<td></td>
<td>24&lt;sup&gt;w&lt;/sup&gt;</td>
<td></td>
<td>24&lt;sup&gt;w&lt;/sup&gt;</td>
</tr>
<tr>
<td>24 Aug</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>25 Aug</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>18</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>27 Aug</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>23</td>
<td></td>
</tr>
<tr>
<td>1 Sept</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>2 Sept</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>6 Sept</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>7 Sept</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>Total</td>
<td>4</td>
<td>3</td>
<td>5</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>7</td>
</tr>
</tbody>
</table>

Superscript w indicates weekend sample sets. Snap samples were also collected from the culverted watercourse and Gowans Terrace (sampling of rapid gravity filter backwash discharging to modelled sewerage system)

Table 7.5 Summary of DWF Quality Collected Summer 1993

Site 1015 was taken as the modelled system outfall. This site was immediately prior to the FPS CSO. As stated previously in this Chapter sludge from satellite treatment works is discharged into the sewerage system at this point. Limited information was available regarding the quantity of sludge discharged daily and no information could be traced relating to qualitative aspects. Therefore, it was decided to verify the quality within the system up to this point, thereby excluding the effect of sludge discharges.

A criterion was specified during sampling that if precipitation had fallen in the 24 hours prior to the sampling start time the collected data were abandoned. This practice ensured that the data collected were as truly representative of DWF quality as possible. Unfortunately, the summer of 1993 proved to be intermittently wet and data collection for DWF was a prolonged tedious exercise which caused frustration. Many sets of samples had to be abandoned due to small amounts of
rain affecting the data (not sufficient to be classed as storm events, but enough to disrupt dry weather flow quality).

Equipment failures were persistent and one began to develop an understanding of how to carry out onsite repair work, to ensure as many samplers as possible were operating. However, the collection of site specific data was fundamental if the model was not to be based on the “average” default values for pollutant determinands supplied with the programme. This intense approach was adopted to ensure the data used in the model were representative of sewer flow quality in Perth.

Due to the limited amount of equipment (four samplers) and workforce (three, including the author) sampling was undertaken as a rolling front exercise. Data collection began in April 1993 and was complete by September 1993. Any rainfall events logged were used to recheck WALLRUS verification. Composite samples were taken of DWF sewage and were analysed for BOD, COD, BODfil, COD, CODfil, TSS and ammonia at the WWTCs' laboratories.

7.10.2 Surface Sediment Data Collection

Surface sediment samples were collected from the catchment in various locations to identify the pollutant characteristics associated with surface sediment. No sediment was collected from roofs, although a number of samples were collected from road networks within the Perth wastewater catchment. Roads were assumed to be the most prevalent source of surface sediment.

Sediments were collected by utilising a large industrial vacuum cleaner. The procedure involved marking off an area of 10m x 2½m and brushing and hoovering any sediment within the area into sample bags. This data was not used specifically for DWF calibration and verification, but gave an indication of the pollutant potential of surface sediment from different locations (useful for storm calibration and verification). Surface sediments were collected from the subcatchments of Moncrieffe, Craigie, City Centre and Bridgend. Data were collected on the 5 of August 1993 and are summarised in Table 7.6.
### Table 7.6 Surface Sediment Data Collected 05/08/93

The previous table shows figures highlighted in bold to indicate the variability of pollutants attached to the surface sediment analysed. Surprisingly, the city centre sampling site had a lower polluting potential than the domestic catchments.

#### 7.10.3 Pipe Sediment Data Collection

Variability of an even greater nature was shown during the collection and analysis of in-sewer pipe sediment data (See Appendix C plate No.8). Although the user can utilise the supplied default values to represent the characteristics and quality associated with in-sewer pipe sediment, it was decided to sample and analyse the sediment found within the Perth sewerage network at various locations. As with the site specific sampling of the DWF quality, this strategy was employed to try to limit the possible sources of modelling error. Analysis followed standard procedures of blending and mixing the samples (Crabtree and Forster, 1989) to release the pollutants attached to the sediment before analysis was carried out.
<table>
<thead>
<tr>
<th>Subcatchment sampling site</th>
<th>COD (mg/l)</th>
<th>BOD (mg/l)</th>
<th>NH₃ (mg/l)</th>
<th>TSS (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28/7/93</td>
<td>28/7/93</td>
<td>28/7/93</td>
<td>28/7/93</td>
<td>28/7/93</td>
</tr>
<tr>
<td>Bridgend- Sundial Blended</td>
<td>10100</td>
<td>1062</td>
<td>42</td>
<td>29120</td>
</tr>
<tr>
<td>Mixed</td>
<td>5500</td>
<td>787</td>
<td>32</td>
<td>10560</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>2600</td>
<td>375</td>
<td>36</td>
<td>2850</td>
</tr>
<tr>
<td>Craigie South Inch Blended</td>
<td>8100</td>
<td>1227</td>
<td>41</td>
<td>31160</td>
</tr>
<tr>
<td>Mixed</td>
<td>4000</td>
<td>952</td>
<td>27</td>
<td>6130</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>1200</td>
<td>457</td>
<td>27</td>
<td>1700</td>
</tr>
<tr>
<td>Central- Bells Site Blended</td>
<td>8800</td>
<td>1490</td>
<td>69</td>
<td>27150</td>
</tr>
<tr>
<td>Mixed</td>
<td>4000</td>
<td>1984</td>
<td>58</td>
<td>29730</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>400</td>
<td>1407</td>
<td>75</td>
<td>5330</td>
</tr>
<tr>
<td>Bridgend- Willowgate Blended</td>
<td>16000</td>
<td>3197</td>
<td>50</td>
<td>27910</td>
</tr>
<tr>
<td>Mixed</td>
<td>12400</td>
<td>3306</td>
<td>55</td>
<td>13870</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>5800</td>
<td>1931</td>
<td>52</td>
<td>4450</td>
</tr>
<tr>
<td>Tullton- Tulloch Works Blended</td>
<td>16400</td>
<td>3362</td>
<td>62</td>
<td>40700</td>
</tr>
<tr>
<td>Mixed</td>
<td>7600</td>
<td>1602</td>
<td>52</td>
<td>14690</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>3400</td>
<td>1272</td>
<td>46</td>
<td>2820</td>
</tr>
<tr>
<td>Central- North Inch Blended</td>
<td>47200</td>
<td>7798</td>
<td>176</td>
<td>64670</td>
</tr>
<tr>
<td>Mixed</td>
<td>23200</td>
<td>5781</td>
<td>134</td>
<td>29400</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>10200</td>
<td>3581</td>
<td>153</td>
<td>7680</td>
</tr>
<tr>
<td>Bridgend- Graveyard Blended</td>
<td>13600</td>
<td>1334</td>
<td>84</td>
<td>63580</td>
</tr>
<tr>
<td>Mixed</td>
<td>3000</td>
<td>417</td>
<td>65</td>
<td>8330</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>1200</td>
<td>417</td>
<td>52</td>
<td>3260</td>
</tr>
<tr>
<td>Arran Rd</td>
<td>8800</td>
<td>2067</td>
<td>160</td>
<td>45040</td>
</tr>
<tr>
<td>Mixed</td>
<td>2900</td>
<td>1059</td>
<td>126</td>
<td>8480</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>2000</td>
<td>600</td>
<td>126</td>
<td>3570</td>
</tr>
<tr>
<td>North Muirton</td>
<td>50000</td>
<td>6100</td>
<td>345</td>
<td>94360</td>
</tr>
<tr>
<td>Mixed</td>
<td>12200</td>
<td>2067</td>
<td>241</td>
<td>19970</td>
</tr>
<tr>
<td>Diss/fine</td>
<td>6500</td>
<td>1242</td>
<td>231</td>
<td>8580</td>
</tr>
</tbody>
</table>

Table 7.7 Pipe Sediment Data Collected from Perth Sewerage System
The quality associated with the in-sewer sediments shows great variability both temporally and spatially. The levels of pollutants contained within these pipe sediments are typical of sediment deposits. All sediment analysed was Class A in nature.

### 7.11 MOSQITO MODEL CALIBRATION

Following the success of the data collection exercise for DWF (finished September 1993) the model required calibration. This process involved various stages, primarily to allow the model to represent accurately the observed DWF and DWP at the sampling sites. The approach followed to achieve calibration is described below.

#### 7.11.1 Dry Weather Flow Data Averaging

MOSQITO, as with most sewer flow quality models, requires the definition of average DWF for each subcatchment being modelled. The model also requires diurnal factors to allow the daily variations of flow, due to population and industrial habits, to be modelled. Average flows for each of the subcatchments defined in Table 7.8 were easily calculated, by utilising the catchment areas defined in the WALLRUS model above the sampling points, and the flow data collected during the quality sampling exercise. Average flows are expressed in m³/s/km².

<table>
<thead>
<tr>
<th>Subcatchment</th>
<th>Area Contributing</th>
<th>Average Flow</th>
<th>Average Flow per Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>North Muirton</td>
<td>0.693 km²</td>
<td>0.0152 m³/s</td>
<td>0.022 m³/s/km²</td>
</tr>
<tr>
<td>Tullton</td>
<td>0.626 km²</td>
<td>0.0019 m³/s</td>
<td>0.031 m³/s/km²</td>
</tr>
<tr>
<td>Craigie</td>
<td>1.270 km²</td>
<td>0.0153 m³/s</td>
<td>0.012 m³/s/km²</td>
</tr>
<tr>
<td>Bridgend</td>
<td>1.704 km²</td>
<td>0.0048 m³/s</td>
<td>0.028 m³/s/km²</td>
</tr>
<tr>
<td>Rannoch</td>
<td>0.1696 km²</td>
<td>0.0124 m³/s</td>
<td>0.0792 m³/s/km²</td>
</tr>
<tr>
<td>Moncrieffe</td>
<td>0.1389 km²</td>
<td>0.0024 m³/s</td>
<td>0.0170 m³/s/km²</td>
</tr>
<tr>
<td>City Centre</td>
<td>0.7910 km²</td>
<td>0.0166 m³/s</td>
<td>0.0210 m³/s/km²</td>
</tr>
<tr>
<td>Carrier</td>
<td>0.0000 km²</td>
<td>0.0000 m³/s</td>
<td>0.0000 m³/s/km²</td>
</tr>
<tr>
<td>Hillyland</td>
<td>0.4830 km²</td>
<td>0.0542 m³/s</td>
<td>0.0110 m³/s/km²</td>
</tr>
</tbody>
</table>

Table 7.8 Average Flow From Subcatchments
Carrier pipes were defined as pipes having no contributing dry weather flow. It can be observed from the data presented in Table 7.8 that although Perth is predominately domestic, there is great variation in the average flow per unit area, for all subcatchments being monitored and modelled. Diurnal factors were calculated by dividing the average hourly flows monitored at each of the sites by the daily average flow for that site. This averaging exercise was carried out over three separate DWF days at each site, to achieve a realistic average flow and set of diurnal profiles for each subcatchment.

The diurnal profiles calculated for the Perth model are shown in Table 7.9. It can be observed that the diurnal profiles produced are similar at certain parts of the day. However, this is a direct result of the removal of infiltration from the analysis. If the infiltration flows had been left in the analysis the modelled production of diurnal patterns would have been incorrect and showed great variation between the DWF patterns for the subcatchments. Infiltration is discussed in 7.12.

Variation between the catchments, in terms of volume, of the same land-use is clear when the diurnal factors are applied to the average flow/unit area (to produce diurnal flows). Rannoch has been omitted from this part of the analysis. The flow/unit area for the Rannoch subcatchment was based upon roof area (partially separate subcatchment) whereas the remainder of average DWF were calculated based upon total contributing area to each pipe, and thus to compare Rannoch against the others would be invalid.

Analysis of Figures 7.27 and 7.28 shows that the population of the Perth catchment behave similarly with respect to the variation and the time of variation of DWF. However, it can be clearly seen that the quantities of flow produced from catchments, which appear to be of similar land-use are substantially different. Thus, if these phenomenon are to modelled accurately then site specific approaches, like the one carried out in Perth are mandatory to the success of sewer flow quality modelling.
Figure 7.27 Comparison Of Diurnal Flow Factors

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>North Muirton</th>
<th>Tullton</th>
<th>Rannoch</th>
<th>Bridgend</th>
<th>Craigie</th>
<th>Moncrieffe</th>
<th>City Centre</th>
<th>Hillyland</th>
</tr>
</thead>
<tbody>
<tr>
<td>0900</td>
<td>1.296</td>
<td>1.981</td>
<td>1.712</td>
<td>1.782</td>
<td>1.855</td>
<td>1.424</td>
<td>1.380</td>
<td>2.470</td>
</tr>
<tr>
<td>1000</td>
<td>1.270</td>
<td>1.478</td>
<td>1.853</td>
<td>1.683</td>
<td>1.907</td>
<td>1.210</td>
<td>1.380</td>
<td>1.137</td>
</tr>
<tr>
<td>1100</td>
<td>1.242</td>
<td>1.169</td>
<td>1.575</td>
<td>1.527</td>
<td>1.578</td>
<td>1.090</td>
<td>1.340</td>
<td>1.125</td>
</tr>
<tr>
<td>1200</td>
<td>1.303</td>
<td>1.473</td>
<td>1.395</td>
<td>1.333</td>
<td>1.543</td>
<td>1.375</td>
<td>1.310</td>
<td>0.842</td>
</tr>
<tr>
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<td>0.060</td>
<td>0.001</td>
<td>0.332</td>
<td>0.069</td>
<td>0.051</td>
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<td>0.001</td>
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<td>0.000</td>
<td>0.341</td>
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<td>1.543</td>
<td>1.601</td>
<td>1.200</td>
<td>1.060</td>
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</table>

Table 7.9 Diurnal Flow Factors
7.12 THE EFFECT OF INFILTRATION UPON SEWER FLOW QUALITY MODELLING

Infiltration within a sewerage network is the consequence of the structural deterioration of the sewerage system. As pipes age and are subjected to external loads and surcharging, the condition of the pipe allows groundwater from the surrounding area to enter the sewer. In some cases of surcharge, storm sewage can enter the ground water table through exfiltration.

From examination of flow data collected during the building of the WALLRUS, STOAT and the MOSQITO models, it was estimated that large quantities of infiltration were present within the Perth network. As MOSQITO requires information on DWF and DWP, infiltration had to be explicitly addressed during model calibration. This was primarily due to its potential to dilute sewage quality and because it is generally not a diurnally varying phenomenon.

Infiltration is clearly vital to the accuracy of DWF and DWP predictions within a sewer flow quality model. At the time of modelling no references could be found to indicate this was believed to be a significant problem. If data is collected from a sampling site and no appreciation is given to the quantity of infiltration present, then the quality data at this site may not be representative of the real foul flow and quality characteristics of the subcatchment being sampled.
Consequently, if data are assumed to represent the characteristics of this type of land-use, and are applied to other similar subcatchments, then errors in prediction will occur. This will lead to the user adjusting calibration data erroneously, when in fact the mistakes are associated with the sampled flow and quality data due to infiltration being present.

For the Perth model, infiltration was estimated at each of the sampling sites from flow logging data, and removed as a constant baseflow for the DWF figures used in the averaging process. This allowed real diurnal variation and average flowrates of foul flow to be assessed. The infiltration figures were added to the model by the use of dummy pipes to give the correct flows at the sampling sites. In between sampling sites, infiltration was added based on the logged flows. From a knowledge of the catchment the location of entry of infiltration into the sewerage system was estimated.

Work by the author has indicated vast quantities of infiltration are present within the sewerage network and consequently feed into the WWTP. Calculations show that to produce the volumes of flow entering the plant during the early hours of the morning (150l/s), the population of Perth (approx 42 000) would have a *per capita* water usage of around 330l/h/d.

Dry weather flows measured in the inlet channel of the WWTP, after screening, in the early hours of the morning averaged around 150l/s. These flows are extraordinarily large for a catchment the size of Perth. Quality data sampled from the inlet are representative of a highly dilute domestic sewage. Retention time is being used up within the WWTP and expenditure wasted pumping and treating the infiltration flows. The TSS and BOD values sampled below are all below the consent standards of Sleepless Inch and theoretically could discharge during this time period direct to the River Tay after screening and grit removal.
Table 7.10 Dilute Influent Quality At Sleepless Inch

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>TSS (mg/l)</th>
<th>COD (mg/l)</th>
<th>Ammn (mg/l)</th>
<th>BOD (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BST</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0700</td>
<td>52</td>
<td>124</td>
<td>9.23</td>
<td>53</td>
</tr>
<tr>
<td>0730</td>
<td>38</td>
<td>73</td>
<td>6.44</td>
<td>34</td>
</tr>
<tr>
<td>0800</td>
<td>46</td>
<td>74</td>
<td>4.99</td>
<td>31</td>
</tr>
<tr>
<td>0830</td>
<td>73</td>
<td>98</td>
<td>4.84</td>
<td>30</td>
</tr>
<tr>
<td>0900</td>
<td>87</td>
<td>79</td>
<td>6.21</td>
<td>27</td>
</tr>
</tbody>
</table>

The dilutant effect of infiltration is demonstrated, when ammonia data sampled in the subcatchments are compared with those listed above entering the treatment plant.

A full understanding of the actual processes occurring within the sewerage system such as the quantity and location of infiltration is mandatory, rather than the approach which relies on default values and tweaking calibration parameters to produce a model until a representation of reality is achieved.

7.13 DRY WEATHER POLLUTANT AVERAGING

MOSQITO requires average pollutant characteristics to be entered representing the quality determinands to be modelled. For the Perth model, this involved the averaging of pollutant data to produce diurnal pollutant factors and average concentrations for COD, TSS and ammonia. Also, potency factors require to be calculated which represent the amount of a pollutant attached to suspended solids (e.g., kg pollutant/kg TSS).

The sampled data was adjusted to allow for the presence of infiltration and its dilutant effect upon the data at the sampled points. This lead to averaging being carried out which produced concentrations (for input to the model) higher than were sampled in the laboratory.
This adjusted data represents foul flow from similar subcatchments to the one sampled when no infiltration is available to dilute the foul flow entering the system at the sampling point.

As stated previously, clean infiltration was added to the model to give the same characteristics in the model at the sampling site as were measured in the laboratory. Data averaging was carried out over the two to three days of DWF quality sampled from each site.

The input data for the Perth MOSQITO model is presented in Table 7.11. Actual field concentrations are less than those shown, as they are affected in reality by infiltration.

<table>
<thead>
<tr>
<th>MOSQITO Subcatchment</th>
<th>TSS (mg/l)</th>
<th>BOD filtered (mg/l)</th>
<th>COD filtered (mg/l)</th>
<th>Ammn (mg/l)</th>
<th>BOD potency factor</th>
<th>COD potency factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>N Muirton/Inver</td>
<td>456</td>
<td>205</td>
<td>511</td>
<td>59</td>
<td>0.102</td>
<td>1.944</td>
</tr>
<tr>
<td>Tullton</td>
<td>229</td>
<td>56</td>
<td>256</td>
<td>33</td>
<td>0.390</td>
<td>3.150</td>
</tr>
<tr>
<td>Rannoch</td>
<td>299</td>
<td>124</td>
<td>410</td>
<td>59</td>
<td>0.420</td>
<td>1.040</td>
</tr>
<tr>
<td>Bridgend</td>
<td>315</td>
<td>201</td>
<td>527</td>
<td>37</td>
<td>0.930</td>
<td>1.500</td>
</tr>
<tr>
<td>Craigie</td>
<td>315</td>
<td>90</td>
<td>432</td>
<td>37</td>
<td>0.500</td>
<td>1.370</td>
</tr>
<tr>
<td>Moncrieffe</td>
<td>200</td>
<td>68</td>
<td>262</td>
<td>40</td>
<td>0.680</td>
<td>1.250</td>
</tr>
<tr>
<td>City Centre</td>
<td>456</td>
<td>205</td>
<td>511</td>
<td>59</td>
<td>0.1</td>
<td>1.944</td>
</tr>
<tr>
<td>Carrier Pipes</td>
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<td>0</td>
<td>0</td>
<td>0</td>
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</tr>
<tr>
<td>Hillyland</td>
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<td>56</td>
<td>256</td>
<td>33</td>
<td>0.390</td>
<td>3.150</td>
</tr>
</tbody>
</table>

Table 7.11 Average Input Data For Modelled Perth Subcatchments

Figure 7.29 illustrates the differences exhibited by variation in pollutants throughout a typical diurnal period. Although in places the factors are closely related, there are points which exhibit substantial variation from one another, indicating that site specific variation factors must be evaluated when attempting work of this type, if errors are to be minimised. Once average pollutant concentrations are applied, it can be clearly seen that variation is significant even between catchments of similar land use such as those illustrated.
Diurnal Pollutant Factors from selected domestic Subcatchments -
Perth MOSQITO model

Figure 7.29 Diurnal Pollutant Factors

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>North Muirton</th>
<th>Tullton</th>
<th>Rannoch</th>
<th>Bridgend</th>
<th>Craigie</th>
<th>Moncrieffe</th>
<th>City Centre</th>
<th>Hillyland</th>
</tr>
</thead>
<tbody>
<tr>
<td>0900</td>
<td>1.090</td>
<td>1.582</td>
<td>1.182</td>
<td>1.528</td>
<td>1.312</td>
<td>1.242</td>
<td>2.040</td>
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<tr>
<td>1000</td>
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<td>0.924</td>
<td>0.482</td>
<td>1.230</td>
<td>1.211</td>
<td>1.540</td>
<td>0.874</td>
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<tr>
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<td>0.952</td>
<td>0.885</td>
<td>1.023</td>
<td>1.210</td>
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<td>1.070</td>
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<td>0.940</td>
<td>0.808</td>
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<td>0.745</td>
<td>0.940</td>
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</tr>
<tr>
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<td>0.702</td>
<td>0.860</td>
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<td>0.962</td>
<td>0.900</td>
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<tr>
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<tr>
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<td>1.164</td>
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</tr>
<tr>
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<td>0.936</td>
<td>1.245</td>
<td>1.810</td>
<td>1.709</td>
</tr>
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</table>

Table 7.12 Diurnal Quality Factors
Although averaging has to be adopted to produce simplified data to allow realistic simulations to be carried out, it is apparent from the work carried out, that the variation of pollutants within the Perth sewerage system is extreme. Figures 7.31 and 7.32 illustrate the point perfectly, and raises the question can sewer flow quality models accurately predict the variability of a real system? Calibration for a particular set of circumstances is possible, but verification may not be truly achievable and this should be borne in mind by those utilising sewer flow quality models. Recent work (Friedler and Butler, 1995) associated with the quality and quantity of domestic wastewater demonstrates the inherent variability of this phenomenon.
7.14 SEWER AND SURFACE SEDIMENT CHARACTERISTICS

Sediment characteristics require to be defined to allow MOSQITO to model the movement of sediment throughout the sewerage system. Although MOSQITO can model up to nine fractions of sediment, it was only possible to define the sediment as fine or coarse (two fractions). All sediment associated with DWF and surface sources is assumed to be fine. The in-sewer pipe sediment is assumed to be a mixture of fine and coarse sediment (predominately coarse).

Regarding in-sewer sediment, MOSQITO allows eight pollutant mixtures to be modelled. Six of these are for gully pot mixtures and the remaining two are for surface sediment and in-sewer pipe sediment respectively. This effectively leads to the “averaging” of all collected pipe sediment data to produce a “typical” pipe sediment for the system being modelled. With regard to Perth, it was apparent that the pipe sediment deposits exhibit a wide degree of spatial and temporal variability, and thus averaging was fundamentally inaccurate, but had to be accepted.

Wet bulk density, shear strength, moisture content, (%) fine, (%) coarse, BOD concentration and COD concentration parameters all required to be determined, averaged and then input into the SED.PRS file.
MOSQITO also required the initial depths of in-sewer deposits. The suggested methodology which consisted of running the model for a few days with typical dry weather flows to indicate accumulations of sediment was not followed. The data collected during the WALLRUS study relating to sediment depths and locations were used to give an accurate representation of the distribution of in-sewer sediment within the model.

The next stage was effectively to run the model and check the predictions in relation to the sampling points in the subcatchments. DWFs were predicted accurately for the monitoring locations without any adjustment to the site specific data being necessary. Suspended sediment required to be verified next. In general, the model deposited sediment in too great a quantity, and anomalies were apparent between predicted and observed TSS concentrations at the sampling sites.

This was remedied by reducing the default density of the fine sediment to 1010 kg/m³ (this parameter cannot be measured effectively in the field, and the adjustment was therefore deemed plausible). Following this adjustment, observed TSS and predicted matched adequately for all subcatchments. Figure 7.33 shows the accuracy of TSS verification for the monitoring Site 1004 (Bridgend).

![Figure 7.33 DWP Verification TSS Site 1004](image-url)
The verification of pollutants proved to be a problem initially as total COD was being utilised as input data to the model. This led to gross overpredictions when compared to the observed data. The reason lay in using total COD values instead of filtered COD values. Amendment to the data input rectified this problem.

Figure 7.34 shows the accuracy of COD verification at the Site 1015 (Inlet to Friarton pumping station: modelled outfall of the system). The modelling of ammonia did not constitute a problem and observed/predicted values matched closely at all sampling sites.

All DWF verification graphs are shown in Appendix G, and following the completion of the verification procedure, it was concluded that the model had been calibrated to a reasonable degree of accuracy for DWF. The accuracy is a reflection of the site specific data collection approach, the non-reliance on default values and the unadjustment of critical parameters to achieve verification.
The work carried out on the DWF verification of the Perth MOSQITO model, highlighted critical areas of importance, which have a fundamental bearing on the use of deterministic quality models in integrated studies. These are discussed in detail in the following section.

7.15 THE APPLICABILITY OF DETERMINISTIC SEWER FLOW QUALITY MODELLING AND THE CONSEQUENCES FOR HOLISTIC CATCHMENT STUDIES

The work carried out on dry weather flow has shown significant variations in pollutant concentration and flow patterns from subcatchments with the same characteristics (mainly domestic). This highlights the need for site specific data to be collected. Also, the influence and dilutant potential of infiltration has been discussed. If sewer flow quality models are to be built accurately, more time needs to be spent in really understanding the phenomenon which is trying to be modelled.

It is apparent that DWP can be modelled to an acceptable degree of accuracy, following detailed calibration and collection of site specific data. However, the work has shown that DWF quality is varied and as a result any models constructed will only represent the system accurately under calibrated conditions.

Concern is expressed as to the accuracy of modelled predictions under storm conditions. Deposited sediment and pollutants can be eroded during wet weather events and thus the modelling of this occurrence is of high importance if modelled predictions are to be valid (Gent et al, 1995).

This is where the application of deterministic sewer flow quality models fail in terms of their attempt to model reality. The failure of the models are not necessarily due to the limitations of any particular package, but are primarily the result of trying to represent complex and highly variant processes which are not fully understood (Verbanck et al, 1994, Gent et al, 1995).
Sewer flow quality models were intended to model the movement of sediment through a sewerage system and the consequential release of pollutants during storm conditions. The underlying theory in the models for sediment erosion and transport has proved difficult to transfer from fluvial hydraulics to the conditions found within drainage systems.

Recent work (Arthur, Ashley and Nalluri, 1995) has shown that the modelling of site specific sediment transport for inorganic near bed transport has proven difficult to achieve with any degree of accuracy. Predicted transport rates from empirically developed equations from laboratory experiments, compared to observed, are quoted as lying between -50% and 100% of the measured value range.

If site specific sediment transport models cannot produce reasonable accuracy, can it be seriously expected that sediment transport can be modelled accurately throughout an entire sewerage system. Other recent studies have confirmed the difficulty in attempting to model sediment transport (Coghlan, Ashley and Smith, 1995). This is due to the difficulty in measuring the controlling parameters such as average particle size, grading, specific gravity and settling velocities under dry weather and storm conditions.

Consequently, if the movement of sediment cannot be predicted accurately by empirical methods throughout the sewerage system, can it be expected that the release of pollutants can be predicted by deterministic quality modelling. The levels of accuracy necessary for use with a river model to conclude if acute pollution meets intermittent discharge standards are not supplied by current sewer flow quality models.

If the data collected for in-sewer deposits for Perth are considered, vast differences can be seen between the associated pollutants. Although it must be stated that not all of the pollutants attached to the sediment will be liberated during storm events, significant temporal and seasonal variations exist to conclude that there are no typical sediment characteristics. Deterministic models attempt to take these processes into account but fail, as they usually rely on data collected
from short term studies, and do not allow for the significant changes in a phenomenon which is as variable as the living system supplying the raw material.

This variability of pollutants attached to sediment is not confined to large spatial variation but can be seen in a recent study of a 48m length of interceptor sewer in the catchment of Dundee (Hutchison, 1995).

<table>
<thead>
<tr>
<th>Date</th>
<th>Sampling Point</th>
<th>1/12/94 BOD(mg/l)</th>
<th>17/12/94 BOD(mg/l)</th>
<th>13/1/95 BOD(mg/l)</th>
<th>27/1/95 BOD(mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>17/12/94</td>
<td>1</td>
<td>715</td>
<td>1430</td>
<td>513</td>
<td>688</td>
</tr>
<tr>
<td>17/12/94</td>
<td>2</td>
<td>165</td>
<td>150</td>
<td>550</td>
<td>532</td>
</tr>
<tr>
<td>13/1/95</td>
<td>3</td>
<td>385</td>
<td>450</td>
<td>430</td>
<td>450</td>
</tr>
<tr>
<td>27/1/95</td>
<td>4</td>
<td>743</td>
<td>751</td>
<td>422</td>
<td>275</td>
</tr>
</tbody>
</table>

Table 7.13 Temporal Variability Of Pollutants Attached To Sediments Within 48m Length Of Interceptor Sewer

Sewer flow quality models are calibrated and verified on data collection exercises carried out over small sections of the year. These are effectively snapshot representations of the system performance. The model may only represent performance for conditions based on the limited data collected. Generally, the model is “calibrated” to fit the field data.

Many models are verified on storm flows and quality resulting from rain events that do not cause CSOs to operate. Consequently, the model will only be representative of system performance in similar conditions to which it was calibrated. Spatial variability of sewage quality is extensive (Brown and Jones, 1993) and seasonal variation (Ashley et al, 1994) is important. Any quality model will not be realistic in terms of system performance throughout a typical year unless extended data collection exercises are carried out on the system (these are never carried out for the purposes of sewer flow quality models, because of expense and time constraints).
Also once a model has been developed, analysis carried out and a solution proposed, the engineer must be aware that the solution developed is based upon a model with all the above limitations. Therefore, if major changes take place to the catchment in the near future, these may drastically alter the performance of the designed solution, as the system will perform differently from the verified model used in the analysis. This raises the question of designed performance as opposed to performance in practice once implemented.

Typical catchment changes may be; population (baseflows and sediments), industry (a brewery coming on line), weather profiles (runoff and CSO performance), operation and maintenance of the system (sediment depths and hydraulic profiles) and discharge standards. Therefore, can it really be expected that an engineer use these sewer flow quality models to effectively design a solution which is sustainable, unless the solution is conservative. Engineers do not need complex sewer flow quality models to produce conservative design solutions.

Engineers require tools which help them reach decisions economically and accurately. However, after data collection calibration and verification these models may only be at best +/- 50% accurate when compared to the observed data (Jack and Ashley, 1996). They are indicative models, and should not be assumed to be precision tools because of their attempt to model complex pollutants.

Generally, sewer flow quality models require the specification of a tank efficiency factor to model the performance of an existing CSO with storage, or indeed to predict the performance of a proposed storage solution. In certain circumstances an efficiency factor for an existing tank may be estimated through data collection. A proposed tanks efficiency can only be estimated. One method that has been suggested is to construct a physical model of the tank to achieve the figure required. This appears to be a contrived way of attempting to model a proposed solution by the use of a physical model (and their limitations with respect to representing particle deposition and erosion) and a deterministic model.
Sewer flow quality models are very sensitive to many factors, most of them are unmeasurable to the degree of accuracy required, due to the fact that they are impossible to measure within in-sewer flow conditions. The two least easily measurable physical parameters, settling velocity and shear strength, are the most significant in determining deposition and erosion and consequently pollutants released (Gent et al, 1995).

It is generally recognised that there are problems with techniques associated with sewer sediments and wastewater sampling. It is unclear particularly in the area of wastewater sampling during DWF and storm flows, if the characteristics of the discharge being sampled are wholly representative of the flow passing the sampling point. During storm conditions more turbulence and mixing occurs and the problem may not be as significant.

However, under DWF conditions, on which quality models are calibrated, gradients of pollutants exist particularly in large sewers. It is important to sample accurately and representatively, to ensure calibration pollutants and factors are as close to actual characteristics as possible.

All of the above makes the application of these tools highly questionable until fundamental research (if demanded) solves the key issues. Many site specific sediment transport studies are striving to quantify the controlling factors associated with the movement and deposition of sediment transport (Arthur, Ashley and Nalluri, 1995). However, while this site specific work is important it has been concluded recently that a universal sediment transport model is very unlikely (Verbanck et al, 1994).

Consequently, the availability of these tools do not alleviate the pressure on the engineer when trying to make decisions relating to the implementation of effective designs. When faced with conditions of uncertainty regarding the performance of the system, the engineer will design conservatively. This is precisely what sewer flow quality models were intended to stop, through their proposed ability to model complexity.
At the time of writing the prospect of a sediment transport equation that works globally is unrealistic, the hope of a sewer flow quality model that can deal effectively with all the limitations previously mentioned is perhaps a decade away, and the further commercial development of these tools is definitely a case of "running before walking".

The inaccuracies and inappropriateness of sewer flow quality models and their lack of accurate predictions begins to call into question the integrity of the entire complex UPM approach, when utilising these tools in conjunction with river and WWTP models. If the sewer flow quality model is inaccurate or has large error bands associated with the predictions, then this output data must not be utilised as input data to the other models in the process without extreme caution.

If this data is used the prediction from wastewater treatment plant models may be inaccurate or not representative of the plant's performance in reality. Similarly, predicted pollutant loads discharging to water courses may not be representative and consequently errors will occur within the river model being implemented. The effect is to introduce more uncertainty to the problem.

Faith cannot be put in the long term predictions of a sewer flow quality model to achieve accurate input data to a WWTP. It is unwise to assume that predictions from a set of models should be used to decide capital investment if the models are not constructed to take into account the wide degree of variance in pollutant quality which is known to occur.

Consequently, for the Perth study it was decided that storm quality data would not be collected under the realms of this research project. Further work through a closely related research project based on the Perth quality models has confirmed the inaccuracies associated with sewer flow quality modelling (Jack and Petrie, 1995). This work showed that the MOSQITO package was limited with respect to quality performance during storm conditions and did not effectively model the release of pollutants from eroded sediment. Also, the model appeared to be insensitive to large changes in potency factors associated with the in-sewer pipe sediment.
Through the above research project the MOSQITO model for Perth has been converted to HYDROWORKS-DM. A large storm data collection exercise was carried out to provide storm verification of the quality model. DM readily produced the dry weather quality performance shown in the MOSQITO model. DM at present does not model the deposition and erosion of in-sewer pipe sediment. This is a planned release some time in the “near future”.

Storm verification was attempted but was deemed inaccurate for moderate rainfall events. Low intensity rainfall events, when erosion was deemed not to take place were of reasonable accuracy. This is believed to be wholly associated with no modelling of sediment erosion and deposition taking place.

The manufacturers of DM are currently working on a sediment transport model to incorporate sediment erosion and deposition. This is proposed to take full advantage of all sewer sediment research. Until sewer flow quality models can effectively model the erosion and deposition of sediment, and the consequential release of pollutants, research should be the primary target while commercial organisations should not “write cheques that their models can’t cash”.

It is still necessary for the engineer to appraise the level of pollutants being discharged to watercourse in the catchment over long periods of time, and by each rainfall event falling on the catchment. An alternative current approach, albeit an improved interim procedure, is based upon the hydraulics of the system and quality monitoring. SIMPOL has been discussed and offers the engineer the ability to assess discharges from CSOs and storm tanks using hydraulics and quality data.

SIMPOL is utilised in conjunction with historic rainfall profiles giving the ability to examine performance over 20 years or more. SIMPOL is spreadsheet based and thus can analyse thousands of events in very small amounts of time. SIMPOL requires calibration against detailed models. For hydraulics WALLRUS or HYDROWORKSTM can be used. The purpose of calibration is to allow SIMPOL to accurately predict hourly spill volumes and total volumes from the CSOs.
When being utilised for quality predictions it is recommended (*FWR, 1994a*) that SIMPOL is calibrated against MOSQITO models when examining CSO performance. This is not advisable in the author’s opinion due to the inaccuracies associated with the present sewer flow quality models. SIMPOL can be used in River Impact mode to assess compliance with the derived intermittent standards, and spill frequency mode to assess the number spills from CSOs under investigation. Quality parameters that are modelled by SIMPOL are total ammonia and total BOD, both are compliant with the derived intermittent standards.

An alternative approach for calibrating SIMPOL for quality analysis is not to use detailed sewer flow quality models, but to actually use data sampled from CSO spills. Many detailed quality models are verified from in-sewer storm data and not from spill data. The definition of an efficiency factor for the modelled storage is difficult to quantify. Therefore calibrating against actual CSO spill quality appears attractive and more realistic.

As the detailed quality model of Perth is not to be applied, due to the limitations discussed, a SIMPOL model was produced to examine the performance of the CSOs within the catchment. This is detailed in Chapter 8.

### 7.16 SUMMARY OF QUALITY MODELS DEVELOPED FOR THE PERTH WASTEWATER SYSTEM

The use of sewer flow quality models to predict pollution has been brought into question and consequently they may be of limited use in integrated approaches with other deterministic models. The WWTP model STOAT has the ability to represent the performance of treatment plants adequately. The underlying theory behind UPM is integration of these tools. The sewer flow quality model forms a weak link in the integrated chain.

For Perth, the WWTP performs very efficiently during dry weather flows and appears not to suffer during wet weather events. The intention behind this section of the research programme was to develop tools and apply the integrated
approach to Perth. The research has shown that there are difficulties in the application of these tools which must be addressed. On reflection, it is disappointing that the decision not to apply the quality models had to be taken due to the tremendous amount of work that had been carried out.

Positively, a WWTP model has been produced, the quality performance of the system has been investigated and a database of wastewater quality is now available for the drainage system of Perth.
CHAPTER 8 DETAILED APPLICATION OF WISPS METHODOLOGY

8.0 INTRODUCTION

Uncertainty regarding the accuracy of information exists, relating to performance in the AOC associated with flooding, CSO spill and structural integrity for the system of Perth. Further investigation of all areas of concern is important to develop a detailed WISPS score for comparison with rehabilitation option scores, if rehabilitation is found to be required. Stage II of the methodology is now being followed.

Due to the inadequacies relating to the sewer flow quality model the use of this tool was abandoned. However, the WISPS methodology requires to assess the spill frequency and impact of CSOs within the catchment. This was carried out utilising the SIMPOL tool described in Chapter 3. The SIMPOL tool was also used to identify rainfall events liable to give flooding through examination of those events predominating large CSO discharge. The flooding performance of the Perth system was analysed utilising historical rainfall in an attempt to derive the actual level of service provided by the system, as opposed to assessing the level of performance with design storms.

This stage of the methodology is concerned with developing detailed performance scores for consideration against rehabilitation options and the selection of the most effective rehabilitation strategy. The computer models developed and additional information gathered are now utilised to develop a detailed WISPS score relating to the wastewater system.

8.1 STORMPAC

Twenty years of rainfall data local to Perth were purchased from the Meteorological Office for the Perth area. This data ranged from 1970 to 1989. The model STORMPAC was utilised to generate a synthetic rainfall series for the Perth catchment and this was compared against the local historical data as a means of testing and checking the STORMPAC software. Synthetic and historical data are compared in Table 8.1.
### Table 8.1 Comparisons Of Monthly Average Rainfall

<table>
<thead>
<tr>
<th>Month</th>
<th>Perth Data (mm)</th>
<th>STORMPAC (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>86.1</td>
<td>96.0</td>
</tr>
<tr>
<td>February</td>
<td>52.4</td>
<td>49.1</td>
</tr>
<tr>
<td>March</td>
<td>65.6</td>
<td>60.3</td>
</tr>
<tr>
<td>April</td>
<td>41.6</td>
<td>44.2</td>
</tr>
<tr>
<td>May</td>
<td>47.4</td>
<td>45.7</td>
</tr>
<tr>
<td>June</td>
<td>57.5</td>
<td>63.1</td>
</tr>
<tr>
<td>July</td>
<td>58.1</td>
<td>50.3</td>
</tr>
<tr>
<td>August</td>
<td>63.6</td>
<td>69.6</td>
</tr>
<tr>
<td>September</td>
<td>67.4</td>
<td>69.5</td>
</tr>
<tr>
<td>October</td>
<td>74.2</td>
<td>69.2</td>
</tr>
<tr>
<td>November</td>
<td>66.5</td>
<td>64.5</td>
</tr>
<tr>
<td>December</td>
<td>73.9</td>
<td>77.6</td>
</tr>
<tr>
<td>Annual Total</td>
<td>754.3</td>
<td>759.1</td>
</tr>
</tbody>
</table>

The generated synthetic data compares well with the published data. STORMPAC can utilise daily rainfall information or daily mean values (from monthly averages). The STORMPAC manual recommends that daily information is used if it is available. Although daily data were available for the Perth area from local rain gauges, the STORMPAC simulation was found to represent the monthly averages equally well with monthly published totals amended as daily data. A further check was made on the distribution of events within the synthetic series, by comparing it with the daily rainfall totals for Perth across a range of total depths.

### Table 8.2 Distribution Of Rainfall Daily Total Depths (% Of Total Number Of Events)

<table>
<thead>
<tr>
<th>Daily total depth</th>
<th>&lt;3mm</th>
<th>3 to 10 mm</th>
<th>10 to 15 mm</th>
<th>&gt;15mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perth Data</td>
<td>79</td>
<td>16</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>STORMPAC</td>
<td>79</td>
<td>17</td>
<td>3</td>
<td>2</td>
</tr>
</tbody>
</table>

The synthetic series appeared to underpredict the number of larger storm events when compared to the recorded daily totals. This test was carried out with version 1.2 of STORMPAC which can reproduce the statistical characteristics of up to a two year return period storm.
The next release of the software is intended to extend the range up to a ten year return period. A minimum data requirement of monthly totals from local rain gauges is recommended. It was not possible to test the disaggregation routine against recorded local data.

8.2 CREATION OF SIMPOL MODEL OF PERTH

The creation of a SIMPOL model for any sewerage system requires a simplification of the real system being studied. This is carried out by calibrating the SIMPOL model against a detailed hydraulic model. The key elements required for the SIMPOL spreadsheet are adequately described in the UPM Manual (FWR 1994a).

The representation of the catchment is the most crucial part of the simplification process. The location of CSOs within Perth were already known and this aided conceptualisation of the catchment into a SIMPOL format. To represent the system the SIMPOL model was split into nine catchments which included one dummy catchment to enable SIMPOL to model the network configuration.

The SIMPOL representation is shown in Figure 8.1. The Craigie CSOs were considered together due to their close proximity of discharge. The CSOs within Bridgend were defined as two separate areas (upper and lower) in the simplification process. Each major CSO at SIPS and FPS was modelled separately to investigate the relative impact of each structure on the River Tay.

A design storm was run through the detailed model of the catchment to assist in identifying the key throttle points within subcatchments. Ten rainfall events, with varying UCWI, intensities and durations were utilised to calibrate SIMPOL against the detailed model.

Calibration was based on deriving an adequate match for total spill volumes and hourly spills between the CSOs in the detailed model and those in SIMPOL. Once calibration was adequate the performance of the CSOs with respect to the 20 years of historical data were examined.
Figure 8.1 SIMPOL Representation of Perth Sewerage System
8.3 CSO PERFORMANCE

Following satisfactory calibration of the Perth SIMPOL model the 20 years of rainfall data were run through the SIMPOL model. Over five thousand rainfall events were processed and a spill frequency analysis showed the following results presented in Figure 8.2.

![Spill Frequency Analysis Perth SIMPOL model](image)

**Figure 8.2 Total Historical CSO Spills From Perth Sewerage System**

The above analysis shows that, on average, CSO discharge takes place 68 times a year to receiving watercourses, which equates to six spills per month. During the "bathing season" when receiving watercourse assimilative capacity is low, CSOs operate 24 times a year on average, again equating to six times a month.

Figure 8.3 shows annual spill volumes associated with the CSO discharge. On average 293 785 m\(^3\)/year of combined wastewater is spilled to the receiving watercourse. During the period June to September 118 272m\(^3\) is spilled during four months. This equates to 40% of the total CSO spill.
CSOs or groups of CSOs which contribute the most potential acute pollution during wet weather to the receiving watercourse are detailed in Table 8.3. Average volumes/year were derived from the SIMPOL analysis.

<table>
<thead>
<tr>
<th>CSO Group</th>
<th>Average yearly spill volume</th>
<th>Contribution to total average yearly spill volume(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Craigie</td>
<td>1087 (90 m³/month)</td>
<td>0.4</td>
</tr>
<tr>
<td>Bridgend</td>
<td>29 434 (2 453 m³/month)</td>
<td>10</td>
</tr>
<tr>
<td>Friarton</td>
<td>10 705 (892 m³/month)</td>
<td>3.6</td>
</tr>
<tr>
<td>South Inch</td>
<td>252 559 (21 046 m³/month)</td>
<td>86</td>
</tr>
<tr>
<td>Total</td>
<td>293 785 (24 482 m³/month)</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 8.3 CSO Spill Volumes

It can be seen that the SIPS CSO is the greatest contributor of potential pollution to the receiving watercourse. All other CSOs, with the exception of Bridgend, contribute very small percentages of the total volume spilled. Through utilising the SIMPOL tool an appreciation of the spill frequency and spill volume from the CSOs in Perth has been established, based on historical rainfall analysis and the package STORMPAC. This detailed information is now available for use with the WISPS methodology.
8.4 HYDRAULIC PERFORMANCE

Traditional practices suggest the use of design storms for the analysis of sewerage system performance. However, some attempt should be made to evaluate the historical level of service associated with the catchment. This can be used, as opposed to relying on incomplete sewer flooding archives, for indications of actual system performance.

The historical level of service provides critical information for sewerage system operators and managers to set a level of service indicator. This is an important point relating to the new Water Authorities recently established in Scotland, as they will likely follow the progress of their counterparts in England with respect to DG5 indicators for flooding. The approach adopted in the study is detailed below.

8.4.1 Flooding Performance-Historical Level Of Service

The areas at flood risk were identified by running standard design storms through the hydraulic model of the Perth system. These areas were recorded as recreational, commercial, domestic or industrial. Domestic representing housing, commercial associated with city centre areas, recreational describes typically parkland, and industrial areas are associated with manufacturing or process operations, such as those carried out in the Inveralmond and North Muirton Industrial Estates. Areas at risk are shown in Table 8.4.

One year and two year return period design storms of durations varying between 15 minutes and 120 minutes were used to identify areas likely to flood. Five and ten year return period events were also analysed. These latter events did not cause flooding in any different areas from the lower return periods. The volume of flooding was greater due to the increased severity of the storms.
The only area identified in the model as flooding which gave concern was that downstream of the South Inch pumping station. Flooding has never been reported here, although severe surcharging was noted during verification of the WALLRUS model.

Thirty rainfall events were selected from the historical series which caused substantial CSO discharge as defined by the SIMPOL analysis. These events were run through the detailed hydraulic model and floodable volumes produced within the subcatchments were noted. The hydraulic model was run without any tidal influences on the flap valved CSOs at SIPS and FPS. This gave the best possible protection to the modelled system against flooding.

A multiple linear regression equation was derived for the whole system between floodable volume (m$^3$) and the characteristics of the rainfall event causing the flooding; volume (mm), peak intensity (mm/hr) and duration (hours). The regression equation is shown in Table 8.5. The development of the regression model is detailed in Appendix L.
<table>
<thead>
<tr>
<th>Catchment</th>
<th>Constant (A)</th>
<th>Duration (mins) (B)</th>
<th>Volume (mm) (C)</th>
<th>Intensity (mm/hr) (D)</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whole</td>
<td>-312.599</td>
<td>-0.478</td>
<td>32.298</td>
<td>34.661</td>
<td>0.92</td>
</tr>
</tbody>
</table>

Flooding = A + B(duration) + C(volume) + D(intensity)

Table 8.5 Parameters For Regression Equation For Flooding Analysis

On the whole the regression equation developed appears to be a reasonable representation of the frequency of flooding, with the R² for the entire system being particularly encouraging.

A spreadsheet was utilised to allow floodable volumes to be derived from the full historical series (20 years) defined by STORMPAC. Each storm producing a total flooding volume greater than 50m³ was defined as a flooding event. The attribute for the area of concern relates to the frequency of flooding.

<table>
<thead>
<tr>
<th>Catchment</th>
<th>Average Frequency of Flooding (calculated over 20 years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whole</td>
<td>146 occurrences; 7.3/year</td>
</tr>
</tbody>
</table>

Table 8.6 Average Frequency For Flooding-limit 50m³

From the analysis carried out it is apparent that Perth has a historical level of service equal to approximately 7 flooding events/year based on a flooding event being greater than equal to 50m³. Using a figure of 50m³ can be open to question with respect to the model’s accuracy, unmodelled storage and varying ground levels.

A further analysis was carried out utilising various values of unmodelled storage. The results are shown in Table 8.7. This was carried out to assess the magnitude of historical flooding and to give more confidence in predictions of the regression model for use with the methodology.
Catchment Unmodelled storage(m3) Average Frequency of Flooding Length of unmodelled pipe (m) (0.150mm diameter)
Whole 100 5.6/year 5 659
Whole 150 4.2/year 8 488
Whole 200 3.45/year 11 317
Whole 250 2.85/year 14 147
Whole 300 2.1/year 16 976

Table 8.7 Average Frequency Of Flooding Relative To Values Of Unmodelled Storage

All results produced from the regression model, with the twenty years of rainfall data, appear reasonable. From Table 8.7, it is apparent that assuming different levels of unmodelled storage ultimately effects the predicted number of average flooding occurrences. It can be concluded that the regression model is sensitive, as is expected, to volumes of unmodelled storage. All the areas at risk in Table 8.4 are in close proximity to the central area of Perth. During model construction an attempt was made to evaluate and include various unmodelled main pipes. However, no account was taken of unmodelled storage in lateral domestic connections. This value in large drainage catchments can be high. Values are included in Table 8.7 which show the length of drain required, assuming 150mm diameter, to accommodate the various values of unmodelled storage. These values are not unfeasible.

Ideally, the exact value of unmodelled storage should be included into the hydraulic model. Then, the regression model developed from analysis of predicted flood volumes from the hydraulic model, would be more suitable for predictive purposes. Using data from the hydraulic model, it is apparent that approximately 9 000m of sewer are modelled in the central area of Perth. Assuming that connections are distributed along 75% of this length, connections are therefore present on 6 750m of central area sewer. If there are two lateral connections every 10m of sewer then this results in 1350 lateral connections. Further assuming
that each lateral is 12m long and is 150mm in diameter 286m$^3$ of unmodelled storage requires to be accounted for. This figure is an estimate of the unmodelled storage present in the Perth system affected by flooding. Based on this estimate, it is recommended that a figure of 2 flooding events/year be adopted as the historical level of service for the sewerage system of Perth. This figure compares well with the estimated level of service stated in Table 5.8. From the flow survey used for verification of the hydraulic model, it is apparent that Perth does suffer from regular flooding in certain locations and this historical data supports the data from the regression analysis.

The use of this approach has shown that historical levels of service can be derived relatively easily for wastewater systems. For Perth, the regression model predicted reasonably accurately the level of service associated with system performance. However, it should be noted that the inclusion of unmodelled storage clearly affects the prediction of flooding events. Future studies using this technique should make all attempts to include relevant data on unmodelled storage in the sewerage system model. Consequently, it is recommended that a figure of two flooding events per year be utilised in the detailed WISPS methodology.

8.5 SURCHARGING PERFORMANCE

The frequency of surcharging can be inferred from the flooding analysis to be in the order of less than one in six months. Analysis using the hydraulic model was carried out using standard design storms (1, 2, 5 and 10 year with 15, 30, 60 and 120min durations) to examine the behaviour of the system under surcharge.

Analysis shows that the core sewerage system of Perth has a design level of performance with respect to surcharge in the order of less than one year. This analysis was undertaken assuming low river levels. This attribute is not to used in the methodology explicitly.
8.6 SEDIMENTATION PERFORMANCE

The estimate of 25% of the system suffering from substantial sedimentation was investigated by carrying out sediment surveys. Sediment was found to exist within the Perth sewerage network in many locations. The sediment locations and depths are accounted for in the hydraulic model. Of the 1100 pipes modelled 16% suffered from sedimentation. The greatest quantity of sediment was found in the city centre area and was attributed to the slack gradients of some of the older sewers in this vicinity and frequent surcharging of the system.

Depths of sediment found in the pipes were approximately 12% of the nominal diameter of the affected pipe. Examples are shown in Table 8.8. From results of sediment tested during the MOSQITO data collection exercise it is evident that sediment is overwhelmingly Class A in origin and highly inorganic. From the analysis it is apparent that in the order of 16% of the sewers within the Perth catchment suffer from deposition ≥ 10% of the nominal pipe diameter.

<table>
<thead>
<tr>
<th>Pipe Diameter</th>
<th>Sediment Depth</th>
<th>Percentage</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1143mm</td>
<td>120mm</td>
<td>10</td>
<td>Main leg to WWTP</td>
</tr>
<tr>
<td>1500mm</td>
<td>150mm</td>
<td>10</td>
<td>Main leg above SIPS</td>
</tr>
<tr>
<td>580mm</td>
<td>75mm</td>
<td>13</td>
<td>Main leg to WPS</td>
</tr>
<tr>
<td>1350mm</td>
<td>200mm</td>
<td>15</td>
<td>Main leg at Perth Bridge</td>
</tr>
<tr>
<td>1085mm</td>
<td>100mm</td>
<td>10</td>
<td>Craigie Sewer at South Inch</td>
</tr>
<tr>
<td>1150mm</td>
<td>150mm</td>
<td>13</td>
<td>Main leg Bells Sport Centre</td>
</tr>
</tbody>
</table>

Table 8.8 Selected Relative Sediment Depths

8.7 INFILTRATION PERFORMANCE

The presence of substantial infiltration within any sewerage system is a matter which demands urgent attention. Depending on the quantity present, infiltration controls sewerage capacity which could be utilised by DWF (new developments) or storm flows. Infiltration has been identified in the Perth catchment, from flow
logging work as being in the order of 150l/s. This can be seen on Figure 7.8 showing influent to the WWTP. This is more than the original calculations shown presented in 5.6.3.

This value may largely result from old Lades (see Chapter 2) which were never cut-off as intended. From analysis of the flows entering Sleepless Inch (from commissioning to present day) it is apparent that a large base flow has always been present. If the source of this infiltration can be identified and eradicated there are possibly major benefits to be gained with respect to the performance of the WWTP and capacity within the sewerage system.

This infiltration has a dilutant effect upon the wastewater entering the treatment plant (see Chapter 7). Although the treatment plant operates admirably, the introduction of percentage removal standards based on BOD, TSS and COD may cause a problem. Simply because trying to remove 70% of a dilute influent parameter is more difficult than doing so with a normal strength wastewater. This problem will only be exacerbated at Perth, if, as is proposed at the time of writing, the import of sludge to the treatment plant is abandoned. As the average DWF entering the treatment works is 240l/s infiltration is approximately 60% of this flow.

8.8 STRUCTURAL PERFORMANCE

Large quantities of CCTV work have been carried out on the Perth sewerage network over the last five to ten years and it was disappointing to find, on detailed investigation, that no definitive record had been produced showing the structural performance grades of the critical sewers within the system. Consequently, the estimated figure stated in Chapter 5 had to be utilised. This is an important performance parameter and detailed information regarding the structural condition of the Category A and B sewers must be obtained.
8.9 AESTHETIC POLLUTION PERFORMANCE

Each of the CSOs in the Perth drainage catchment were examined for the purposes of assessing the impact of the aesthetically polluting material (see Appendix C Plates 4 and 6) discharged, based on the scoring system previously developed in section 5.3.3. The assessment procedure is intended to identify the CSOs which have the most impact on the surrounding waterside environment and give engineers a strategy to prioritise those in most need of rehabilitation. Surveyed criticality scores are shown in Table 8.9. From the scoring system implemented, it can clearly be seen that the CSOs which require attention primarily, are those located at the South Inch and Friarton Pumping stations. The Bridgend river bank manholes also require attention due to their operation.

<table>
<thead>
<tr>
<th>CSOs</th>
<th>Location</th>
<th>Quantity</th>
<th>Accessibility</th>
<th>Criticality Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Craigie Burn</td>
<td>Windsor Terrace</td>
<td>none (75)</td>
<td>high (10)</td>
<td>78</td>
</tr>
<tr>
<td>South Inch</td>
<td>Shore Road</td>
<td>small (50)</td>
<td>very low (75)</td>
<td>56</td>
</tr>
<tr>
<td>Friarton</td>
<td>Shore Road</td>
<td>small (50)</td>
<td>medium (25)</td>
<td>44</td>
</tr>
<tr>
<td>Bridgend</td>
<td>Mansfield place</td>
<td>trace (75)</td>
<td>none (100)</td>
<td>75</td>
</tr>
<tr>
<td>Bridgend</td>
<td>Rear of Bakery 2 No. CSOs</td>
<td>trace (75)</td>
<td>low (50)</td>
<td>69</td>
</tr>
<tr>
<td>Bridgend</td>
<td>Stanners</td>
<td>none (100)</td>
<td>medium (25)</td>
<td>82</td>
</tr>
<tr>
<td>Bridgend</td>
<td>Riverbank manholes</td>
<td>small (50)</td>
<td>medium (25)</td>
<td>44</td>
</tr>
<tr>
<td>Bridgend</td>
<td>Willowgate</td>
<td>small (50)</td>
<td>very low (75)</td>
<td>57</td>
</tr>
<tr>
<td>Average</td>
<td>65 (small to trace) quantity</td>
<td>48 (low)</td>
<td>63</td>
<td></td>
</tr>
</tbody>
</table>

Table 8.9 Aesthetic Assessment Of CSOs Within Perth Catchment
On average the CSOs within the Perth catchment were found to discharge trace to small amounts of aesthetic pollution in areas which are generally of low accessibility to the general public. The surveyed average score of 63, compared to the estimated figure of 44 in section 5.6.6, is used as input data to the methodology for aesthetic pollution performance from the Perth CSOs.

However, the nature of the River Tay at Perth is such that any aesthetic pollution discharged during wet weather events is likely to be “swallowed up” and transported a considerable distance during tidal cycles. Therefore, the analysis of aesthetic pollution is based on the material left behind, which can have an impact on the public, and is not an indicator of effective CSO screening performance.

**8.10 WASTEWATER TREATMENT PLANT PERFORMANCE**

From the STOAT model produced the performance of Sleepless Inch at present is excellent, particularly with respect to the loading from imported sludges. There appears to be, from the monitoring carried out, no drop in effluent quality with respect to the consent standards during storm conditions. Consequently, the figure utilised in Chapter 5 (100% compliance) has been carried forward into the detailed analysis.

The only problem at the present associated with the WWTP is the apparent odour problem which occurs during desludging of the primary tanks to the sludge thickeners and during the filling of contract tankers when sludge is uplifted for disposal to agriculture.
8.10.1Odour From Wastewater Treatment Plant

Complaints regarding odour nuisance have been received since the opening of the works and have occurred intermittently until the present day. Several housing developments are located on the opposite bank of the River Tay notably near Kinaunans Castle and the Walnut Grove area. A recent survey by WRc was utilised to identify if complaints were justified. A table is presented showing on site measurements of Hydrogen Sulphide (measured by WRc).

<table>
<thead>
<tr>
<th>Location</th>
<th>Mean Hydrogen sulphide Concentration (ppb by volume)</th>
<th>Mean Hydrogen sulphide Concentration (ppb by volume)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Survey 1 (28 Sep)</td>
<td>Survey 2 (30 Sep)</td>
</tr>
<tr>
<td>Inlet</td>
<td>1.40</td>
<td>2</td>
</tr>
<tr>
<td>Inlet (top of screws)</td>
<td>3.66</td>
<td>36</td>
</tr>
<tr>
<td>Inlet (under covers)</td>
<td>18.67 PPM</td>
<td></td>
</tr>
<tr>
<td>Inlet (top)</td>
<td>360</td>
<td>2.25 PPM</td>
</tr>
<tr>
<td>Large AC Filter Exhaust</td>
<td>1.08 PPM</td>
<td></td>
</tr>
<tr>
<td>Small AC Filter Exhaust</td>
<td>3.98</td>
<td></td>
</tr>
<tr>
<td>Inlet Channel to screens</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Screens</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Grit Removal</td>
<td>7</td>
<td>2</td>
</tr>
<tr>
<td>Primary Tanks</td>
<td>34.6</td>
<td>53</td>
</tr>
<tr>
<td>Downwind</td>
<td>12.10</td>
<td>16</td>
</tr>
<tr>
<td>Upstream</td>
<td>1.5</td>
<td>1</td>
</tr>
<tr>
<td>Sludge Sewpass</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Before takeoff</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>During takeoff</td>
<td>14.67 PPM</td>
<td>29 PPM</td>
</tr>
<tr>
<td>Sludge Splitting Well</td>
<td>4.65 PPM</td>
<td></td>
</tr>
<tr>
<td>Sludge Thickening tank</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tank weir</td>
<td>13 PPM</td>
<td></td>
</tr>
<tr>
<td>1m downwind</td>
<td>27</td>
<td>1.67</td>
</tr>
<tr>
<td>Filling Tank</td>
<td></td>
<td>135</td>
</tr>
<tr>
<td>Tanker (top)</td>
<td>&gt;50 PPM</td>
<td>5 PPM</td>
</tr>
<tr>
<td>5m Downwind</td>
<td>22.38</td>
<td></td>
</tr>
<tr>
<td>15m Downwind</td>
<td>13.37</td>
<td></td>
</tr>
<tr>
<td>20m Downwind</td>
<td>167</td>
<td></td>
</tr>
<tr>
<td>25m Downwind</td>
<td>6.53</td>
<td></td>
</tr>
<tr>
<td>30m Downwind</td>
<td></td>
<td>40</td>
</tr>
<tr>
<td>Works approach road</td>
<td>2.45</td>
<td></td>
</tr>
<tr>
<td>Echo Castle</td>
<td>1.80</td>
<td></td>
</tr>
<tr>
<td>Kinaunans Castle Road</td>
<td>3.25</td>
<td>2.13</td>
</tr>
<tr>
<td>Lay by A90 (A85)</td>
<td>3.10</td>
<td>2.8</td>
</tr>
<tr>
<td>Kinaunans Junction (35m of A90(A85))</td>
<td>1.93</td>
<td>2.6</td>
</tr>
<tr>
<td>Kinaunans Church</td>
<td>2.40</td>
<td></td>
</tr>
<tr>
<td>ESSO Garage A90 (A85)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A90 (A85) lay-by</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Walnut Grove</td>
<td>1.45</td>
<td>2.6</td>
</tr>
<tr>
<td>(Opposite works)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 8.10 Selected Hydrogen Sulphide Samples From Sleepless Inch

215
The threshold odour concentration for hydrogen sulphide is in the order of 2ppb and from the survey work carried out it can be seen that this level is exceeded in many cases and is therefore the probable cause for complaint.

Areas identified as causing an odour problem include, sludge distribution wells, primary settling tanks, inlet screws, activated sludge tanks and tanker loading facility. The main cause of the odour problems however, is the addition of sludge from satellite WWTPs in Perthshire at the FPS. The average number of complaints has been utilised in the methodology.

8.11 PERFORMANCE COSTS

Analysis of the Perth drainage catchment has shown that at present, no costs arise due to poor performance by the wastewater system. These costs may be important in other catchments and have been discussed in Chapter 4.

8.12 RECEIVING WATER COURSE QUALITY

Numerous CSOs are present within the Perth drainage catchment and discharge to receiving watercourses which are of different characteristics. The CSOs located along the length of Windsor Terrace discharge to the Craigie Burn. However, the operation of these ancillaries was never witnessed during the study and it is believed that they operate very infrequently, and only in extreme conditions to alleviate flooding of the neighbouring area. The infrequency has been demonstrated through analysis of the SIMPOL model of the Perth sewerage system. Consequently they do not present a recurring pollution problem within the Craigie Burn.

The details of the CSOs within the Bridgend subcatchment have been discussed previously, as have the manholes which flood directly into a small backwater of the Tay. These structures do operate frequently to relieve the system from possible flooding.
Bacteriological analysis of the Tay was carried out by SEPA in the area of Queens Bridge and shows the following data for levels of Total Coliforms (TC), Faecal Coliforms (FC) and Faecal Streptococci (FS).

<table>
<thead>
<tr>
<th>Site</th>
<th>Date</th>
<th>Time</th>
<th>TC/100ml</th>
<th>FC/100ml</th>
<th>FS/100ml</th>
<th>FC/FS</th>
</tr>
</thead>
<tbody>
<tr>
<td>River Tay</td>
<td>4/4/95</td>
<td>1420</td>
<td>2900</td>
<td>450</td>
<td>320</td>
<td>1.4</td>
</tr>
<tr>
<td>River Tay</td>
<td>11/4/95</td>
<td>1450</td>
<td>7400</td>
<td>590</td>
<td>54</td>
<td>10.9</td>
</tr>
<tr>
<td>River Tay</td>
<td>18/4/95</td>
<td>1345</td>
<td>4500</td>
<td>610</td>
<td>150</td>
<td>4</td>
</tr>
<tr>
<td>River Tay</td>
<td>8/8/95</td>
<td>1450</td>
<td>1900</td>
<td>600</td>
<td>310</td>
<td>1.9</td>
</tr>
<tr>
<td>River Tay</td>
<td>15/8/95</td>
<td>1430</td>
<td>&gt;100 000</td>
<td>21400</td>
<td>150</td>
<td>142</td>
</tr>
<tr>
<td>River Tay</td>
<td>22/8/95</td>
<td>1410</td>
<td>22000</td>
<td>1010</td>
<td>110</td>
<td>9</td>
</tr>
</tbody>
</table>

Table 8.11 Bacteriological Quality Of River Tay At Perth

Ratios are presented relating Faecal Coliforms to Faecal Streptococci in Table 8.11. These are useful in determining the source of the pollution. The ratio for domestic animals is usually less than 1 and that for humans is in excess of four (Metcalf and Eddy). The samples taken 11/4, 18/4, 15/8 and 22/8/95 clearly indicate the source of pollution as human. This source is likely to be that of the large CSO at South Inch Pumping Station.

The levels of bacteria and the ratios presented are indicators of pollution originating from a human source. Upstream discharges above Perth may be responsible for these levels of bacteria in the Tay. The spills from the SIPS CSO, while not causing a biochemical problem because of the large dilution, may be subjected to movement by tidal cycles within the Tay. As a result, pockets of bacteria may become effectively static or move downstream slowly while experiencing die off.

Discharges from the SIPS CSO have been observed to hug the riverbank from the point of discharge. The Friarton Pumping Station also contains an overflow of similar type to the one found at South Inch. However, this CSO while operating does not discharge the quantities of volume associated with the South Inch CSO. The River Tay can be defined as a Class 1A river with respect to long term water quality. Detailed analysis has shown the historical information to be accurate.
8.13 AMENITY USE AND PUBLIC PERCEPTION OF WATERCOURSES

The technical investigation has shown that the watercourses in Perth appear to cope adequately with the discharge of CSO and WWTP effluent, mainly due to the large assimilative capacity within the River Tay. A public perception survey was carried out to investigate how the public viewed the quality of the water bodies. Data derived were not used in the WISPS methodology, but were useful in comparing public opinion to technical facts relating to the quality of the watercourses. Results of the survey can be found in Appendix H. The River Tay, Town Lade and Craigie Burn were each subject to investigation.

8.13.1 The River Tay

The east and west banks of the Tay, the Craigie Burn, and the Town Lade were each affected by wastewater solids and domestic refuse to differing degrees. The east bank of the Tay suffered from small amounts of wastewater solids throughout the area surveyed. However, it is unlikely that these, because of their locations, would be easily spotted by bankside users (and consequently would not greatly influence perception).

Deadwood and rubbish were also common and were concentrated in certain areas, which detracted from the aesthetic value of the area. However, the presence of the Norrie Miller Walk, and the pathway along a large proportion of the length of bank studied, suggests this bank is popular for recreation.

The west bank of the Tay was found to be in two cleanliness categories, north of Perth Bridge was found to be of excellent quality with no wastewater solids and only small amounts of land derived litter present. This stretch was widely used for recreation, especially by walkers and anglers. South of Perth Bridge, evidence of wastewater discharges were prevalent as was the deposition of litter.
8.13.2 The Craigie Burn

The quality of the Craigie Burn was generally good, though a trace number of wastewater solids were found downstream of the overflows at Windsor Terrace. Again, these wastewater solids were not obvious unless one, as in the survey, was particularly seeking them out.

No evidence of wastewater solids were found upstream of the overflows, and large amounts of land derived waste were confined to areas of high public access. The area between Edinburgh Road and the top of Windsor Terrace was well used, despite this being the stretch most affected by wastewater discharges. The quality of the Burn from Balmoral Place to the source was very good with only occasional occurrences of litter and other debris.

8.13.3 The Town Lade

The Town Lade throughout Perth was so badly affected by land derived rubbish that few points along its length could be described as attractive. The Environmental Improvement financed by the Perthshire Enterprise Company, Perth and Kinross District Council, Scottish Enterprise Tayside and Tayside Regional Council has improved the quality of the Lade, but has not necessarily made it more attractive (construction of man-made banks and channels). Very little of the length of the Lade seemed to be used for recreation, possibly for these reasons.

8.13.4 General Discussion On Public Perception Of Perth's Waterways

Throughout the study it was found that the poor visual (perceived) quality of the water did not necessarily mean poor biological quality, as aquatic plants and fish were found in areas which were highly unattractive, due to the presence of rubbish and debris. The areas most accessible to the public tended to be the most heavily littered and were the most widely used.

To summarise, the discharge of wastewater to the waterways of Perth has had minimal impact. The environmental quality of the smaller waterways from a visual perspective, is on the whole poor, due to large quantities of general land derived
The effect of domestic refuse was by far the most influential in terms of controlling the waterways attractiveness. Due to the small amounts of wastewater solids that were present it is proposed that the public would tend to base their opinions on the amount of refuse present rather than the effect of any wastewater discharges.

Despite the presence of rubbish along the Tay, the public perception is that of a clean river and is therefore valued by the public as such. The Craigie Burn was seen by the public to be dirty, due to rubbish, and so was found to be of a low value. Although combined wastewater generally does not discharge into the Town Lade (with the exception of the foul connection into the Goodlyburn) the presence of refuse, oil and the discoloration of the water has led to the public perception of a dirty waterway which therefore was found to be of low value.

It can be said that the environmental quality of the waterways was found to be at its poorest where evidence of wastewater solids and land derived rubbish was at its greatest. Hence, the public perceived these areas to be dirty and consequently of little value. Areas found to be clean by the public were given a far higher value and were used far more frequently for recreation.

The over-riding feeling amongst the members of the public interviewed in Perth was that a better sewerage system should be employed to prevent discharges, and that the clearance of litter should become a frequent event. With the implementation of these views the environmental quality of the waterways in Perth could be improved, thus leading to an area more highly valued by the public.

It is clear that the general public in Perth do have an awareness of water quality and are concerned about maintaining the benefits from preventing pollution occurring. As other researchers have found, when carrying out similar more detailed surveys, it is what the public sees and smells that influences their opinion in environmental matters.

The public believe the Lade and Craigie Burn to be polluted and of poor quality based solely on land derived waste and not on sewage derived solids. The
The identification of sewage derived solids may be clearer in areas where overflow events occur frequently and the users are aware of these products, but in Perth overflow events are rare in the smaller water courses and when they do occur in the Tay the river is sufficiently large to only leave traces of these products. It is true that these items will only be spotted by users of the watercourse and in Perth most accusations of aesthetic pollution were levelled at waste originating from land and not the sewerage system.

The public were aware that in some cases wastewater is discharged to the watercourses in the area but did not demand that this was an outrage and must be stopped. The author believes this to be a case of “what the eye does not see the heart does not grieve about”. From the work carried out it is evident that Perth does not have a major wet weather pollution problem as most of the major discharges from overflows take place to the River Tay which has enough dilution to deal with the polluting load.

In general the public were most concerned for the removal of land derived refuse from the watercourse in question. This highly influenced their opinion as to the cleanliness of the watercourses. While the aspects of the watercourses are part of this study, the responsibility of cleaning land derived waste from them, and the consideration of the most effective way to do this is not.

8.14 SUMMARY ON PERFORMANCE OF PERTH CATCHMENT

From the analysis of the Perth catchment it can be seen that in general terms the present performance of the catchment falls into two distinct areas. From a water quality viewpoint no long-term problems are apparent within the watercourses associated with CSO and WWTP discharges. Biochemical analysis carried out by SEPA has shown the Tay to be very clean although the levels of bacteria sampled during the dry spell of 1995 are concerning.

Aesthetic pollution is present in trace to small amounts from CSO discharges and generally is in areas where the material would only be observed by keen bankside users. Overwhelming evidence shows that the general public in Perth were agreed on the best way of improving watercourse quality; by removing land derived waste.
refuse. This aesthetic pollution was very influential in determining the value placed upon the smaller watercourses in Perth.

The sewerage system was affected by sedimentation to a small degree and the depth of sediment encountered was not excessive. Also, most sediment was found to be highly inorganic in nature and therefore will not contribute excessively towards pollutant load during storm events.

Infiltration was found to be some 60% of the average dry weather flow entering the WWTP and consequently is absorbing capacity which would improve the flooding and surcharging performance and which could be taken up by flows from any proposed new developments.

The WWTP was assessed as performing well within the consent standards. It has been shown that in some cases the treatment afforded by the primary sedimentation tanks is enough to meet the discharge constraints. Although the WWTP is subject to very high concentrations of TSS and BOD during the weekdays from sludge import, the plants efficiency does not decrease. Concern is expressed regarding the plants ability to perform adequately if consent standards in the future are to be based on percentage removal of pollutants. This is particularly important if the import of sludge ceases as the influent will effectively become a very dilute domestic wastewater.

Odour has become a problem at the plant due to insufficient desludging capacity within the sludge holding tanks. Also, the policy of pumping sludge to tankers through open hatchways exacerbates the situation and nearby residents have voiced opinion regarding odour.

The historical level of service provided by the sewerage system with respect to flooding has been shown to be in the order of 2/year in the affected locations. Many of the areas affected are mainly recreational and are not domestic or commercial. Surcharging performance has a level of service less than that associated with flooding. Analysis of the CSO discharge regime has shown that the CSOs in the Perth catchment operate frequently and on occasion, particularly
from SIPS and FPS, discharge large volumes to the River Tay. However, water quality analysis has shown that there are no long term affects upon the Tay as a result of assimilating the CSO discharge from these two overflows. Following the analysis of the catchment the performance information was used to derive a detailed WISPS score for the wastewater system of Perth. This is shown in Table 8.12.

The detailed score compares very favourably with the historical score for Perth. However, this is mainly due to the historical evidence being an accurate estimate of modelled and surveyed performance. Most Water Authorities will hold adequate historical information on receiving water course quality, WWTP performance and odour complaints. Most English Authorities will hold information on aesthetic pollution from CSOs, through surveys carried out for AMP purposes. Areas of concern associated with historical flooding, CSO spill, infiltration and sedimentation generally will require detailed assessments, via models or surveys, to generate performance information.

It may be the case that for larger systems like Perth, a large degree of historical information exists. It is proposed that for many smaller systems historical information may be limited and detailed studies will be necessary if a holistic appreciation of system performance is to be gained. It may be possible, in the presence of accurate historical information, to use outline WISPS scores for comparison against rehabilitation option scores.

However, this requires work to be carried out on number of systems, whereby historical scores can be compared to detailed scores. If doubt surrounds any one area of concern then data should be collected to ascertain a correct estimate for the parameter under investigation. As with any technique the approach is only as reliable as the data fed into it.
<table>
<thead>
<tr>
<th>Area of Concern</th>
<th>Average Weighting</th>
<th>Attribute Value</th>
<th>Performance Score</th>
<th>Detailed WISPS Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Integrity of Sewerage System</td>
<td>0.140</td>
<td>15% estimate</td>
<td>50</td>
<td>7</td>
</tr>
<tr>
<td>Flooding</td>
<td>0.138</td>
<td>2/year</td>
<td>50</td>
<td>6.9</td>
</tr>
<tr>
<td>Receiving Water Course Quality</td>
<td>0.101</td>
<td>1A</td>
<td>100</td>
<td>10.1</td>
</tr>
<tr>
<td>WWTP Compliance with Consents</td>
<td>0.132</td>
<td>100%</td>
<td>100</td>
<td>13.2</td>
</tr>
<tr>
<td>Aesthetic Pollution</td>
<td>0.109</td>
<td>63</td>
<td>60</td>
<td>6.54</td>
</tr>
<tr>
<td>CSO discharge to Receiving Waters</td>
<td>0.110</td>
<td>6</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Infiltration</td>
<td>0.094</td>
<td>60%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Sedimentation</td>
<td>0.095</td>
<td>16%</td>
<td>60</td>
<td>5.7</td>
</tr>
<tr>
<td>Odour from WWTP</td>
<td>0.080</td>
<td>12</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Detailed WISPS</td>
<td></td>
<td></td>
<td></td>
<td>49.4</td>
</tr>
</tbody>
</table>

Table 8.12 Detailed WISPS Score For Perth

8.15 GENERATION OF REHABILITATION OPTIONS

The generation of rehabilitation options for any wastewater system is an important procedure and must be carried out in a structured manner. Stage IIA of the WISPS methodology deals with the generation of rehabilitation options. What is important is that all options are considered holistically and not as piecemeal solutions to specific problems. Stage IIA, shown in Figure 8.5 contains three main areas. These are discussed below.
Define objectives and key constraints: All objectives must be clearly stated and understood. The objectives relate to the level of service provided by the system against flooding, level of infiltration to be accommodated, frequency of odour complaint, level of aesthetic pollution to be tolerated, percentage of sewers in critical condition, proposed river water quality, WWTP compliance targets and other important criteria. Constraints to option generation must be stated. These are effectively budgetary, political, legislative and environmental. Objectives will be concerned with improving the performance of the existing system to a predefined level (if no legislative standards apply) or a specified acceptable standard. Legislative standards in Scotland are applicable to CSO discharges, WWTP compliance and water quality. At present no specific legislation exists relating to the level of service with respect to flooding frequency, quantity of infiltration, structural stability, allowable sedimentation and odour emission. To a certain extent the last two criteria have no legislative target for upgrading. The targets are defined by engineering judgement and advice from manuals of good practice.
**Option Generation:** This phase is associated with “brainstorming” to develop a range of options. Any options deemed infeasible due to constraints or technical criteria should be rejected and the reasons for rejection documented.

**Option Evaluation:** This involves the examination of shortlisted options to investigate the effect on improving the holistic performance of the wastewater system. Capital and revenue costs should be developed for each option under detailed consideration.

Following modelling of rehabilitation options, rehabilitated WISPS are calculated for each (see stage II of methodology, Figure 5.2). These are compared with existing detailed scores of the wastewater systems performance. The rehabilitation options are then offered to the “decision makers”. The most effective solution is then selected for implementation. The most holistically effective rehabilitation option provides the greatest increase in WISPS score while meeting all constraints.

**8.16 MONITORING OF REHABILITATED PERFORMANCE**

It is of critical importance that following the implementation and construction of a rehabilitation option within a wastewater system, the performance of the system is monitored. This must be carried out in each of the rehabilitated areas of concern and compared with existing performance. This ensures that the rehabilitation option is actually effective in solving the problems which it was designed for.

This is simply carried out by installing monitoring equipment, and developing procedures to record performance data on each of the areas of concern under consideration. This may involve the installation of sampling and logging equipment to monitor emissions and hydraulics. CSOs must be regularly surveyed to assess their screening performance. CSO event recorders should be installed to assess spill frequency.
Rehabilitated WISPS scores can be calculated on a monthly basis and compared against the previous system score as a measure of actual effectiveness as compared to designed performance. The monitoring of the catchment should carry on indefinitely to provide current performance information relating to the behaviour of the rehabilitated system.

8.17 APPLICATION OF METHODOLOGY

The selection of the most effective solution is based on the level of improvement defined by comparing detailed WISPS scores for the existing system against the rehabilitated WISPS scores. If a cost effective analysis approach (as defined in Chapter 4) is carried out, each rehabilitation option will upgrade the system performance to a defined level for each area of concern. Therefore the level of improvement over the existing WISPS will be the same for all options. The selection of the most effective solution will simply be based on the option which has the lowest total cost of operation (capital and maintenance costs).

If a performance cost approach is adopted different levels of performance are available at different total costs of ownership. Effectively, different levels of holistic performance can be achieved, in the areas of concern, for varying cash sums. For areas of concern not subject to legislative requirements, choices exist as to the level of rehabilitated performance to be accepted.

A question can be raised at this stage; as to who makes the decision regarding the acceptable level of performance to be selected? Rehabilitation options which effectively improve system performance to varying degrees can be offered to the decision makers for varying costs. It is proposed that the engineer does not make the decision regarding acceptable levels of service for areas of concern. The WISPS methodology can be used to present facts, that metaphorically drag the decision makers to the waters edge, to see what holistic performance improvement can be achieved for a specified level of investment.
8.18 REHABILITATION OF PERTH CATCHMENT

From the detailed application of the methodology and technical investigations it is apparent that the wastewater system of Perth requires further information relating to the structural performance of the system. This can be achieved by developing a Drainage Area Plan (DAP) showing the critical sewers and conditions through techniques specified in SRM. The estimate of 15% utilised can be updated following this work. Details are also required regarding infiltration and the points of entry into the sewerage system. This can be achieved by a detailed logger survey or utilising chemical tracing techniques.

Sedimentation appears to cause no adverse problems and it is postulated that removal would simply be offset by rapid deposition to the same degree. Aesthetic pollution from the CSOs, is of trace to small amounts and as the CSOs appear not to cause any biochemical damage to the Tay they may be classed as satisfactory. The WWTP requires no rehabilitation although analysis may be required if sludge injection upstream is abandoned. Odour control measures are underway to improve significantly the emissions to the surrounding environment. In the light of Perth Flood Prevention Order work is proposed to construct a large pumping station to discharge runoff from the Craigie Burn to the Tay in periods of flooding. It is proposed to allow the SIPS CSO to discharge into this pumping station to be pumped out against any head in the Tay. The number of spills to the Tay is likely to remain the same, as no storage is to be provided.

However, the new CSO structure and screens will improve hydraulic characteristics compared to the present. This will moderate the surcharge and upstream flooding in the current system. This has been verified by running the hydraulic model with the rehabilitated system and design storms.

It was unfortunate that more direct conclusions could not be applied to the system of Perth. Ideally rehabilitation options should have been assessed in accordance with Stage II of the WISPS methodology. Contrived rehabilitation options were one alternative, however, time constraints prevented this from being carried out.
CHAPTER 9 CONCLUSIONS ON THE RESEARCH

9.1 PRINCIPAL CONCLUSIONS

An integrated suite of computer models has been developed for the catchment of Perth in Scotland. These represent hydraulic performance, wastewater quality and wastewater treatment plant performance. The holistic performance of the wastewater system has been investigated through the application of the computer modelling tools and field investigations. An attempt has been made to integrate the technically available tools with a methodology for the purposes of effectively evaluating holistic wastewater system performance.

The methodology developed in Chapter 5, is based on the principles of multi-attribute utility theory and allows engineers to integrally assess the performance of wastewater systems for defined areas of concern. The methodology known as WISPS centres on simple value functions and weightings. The approach can be used in three principal roles (i) prioritisation of wastewater systems for improvement, (ii) the selection of rehabilitation options and (iii) monitoring existing or rehabilitated performance of wastewater systems.

The technique was applied in favour to Benefit Cost Analysis due to the simplicity of approach, data requirement and suitability in terms of the three required principal roles previously listed. It is recognised that the industry has adopted BCA for analysing surface water quality improvements, but at present a methodology does not appear to exist which integrates all aspects of wastewater system performance, and allows comparative holistic evaluation. The WISPS technique can holistically assess the performance of a wastewater catchment and be integrated with a cost effective approach to select a suitable rehabilitation option for a catchment.

The methodology is simple and avoids complexity and lets policy/decision makers see what is holistically deficient within the wastewater system, and what level of improvement can be achieved. Levels of system improvement over existing performance can be related to corresponding total costs (capital and maintenance)
and the most cost effective solution implemented. However, the technique requires accurate performance information for it to function correctly. The performance information must be accurate whether it is historical or from the output of a detailed computer model.

The key areas of concern for wastewater system performance have been identified and value functions have been constructed. Weightings have been assigned to areas of concern based on the results of interviews with engineers in NoSWA. The value functions adequately describe the relationship between performance score and the behaviour of the area of concern.

The WISPS methodology has been applied to four catchments within the Tayside area of NoSWA; Coupar Angus, Almondbank, Forfar and Perth for the purpose of determining historical WISPS scores. The developed scores describe the historical performance of the studied catchments. In particular, the historical score for Perth is very close to that developed through Stage II of the WISPS methodology. However, this should be treated with caution as much of the historical information was accurate and gained from records.

Also, during the detailed application of the WISPS methodology only performance in the areas of concern associated with developing computer models and detailed surveys should change. Further applications of the approach are required to identify if good historical information can be wholly relied on.

It is proposed that most engineers will know and record performance in larger catchments but know less about smaller catchments under their control. The detailed approach in the methodology will then be useful for determining actual performance as opposed to relying on poor historical data or guess work. Due to time constraints, no actual rehabilitation options have been developed and scored for the Perth catchment. However, the use of the methodology in this role has been discussed, as has that of the methodology in the monitoring phase.

Sensitivity testing of the WISPS technique showed that relatively large changes in the weightings/performance scores (+/-50%) resulted in small changes (< +/-15%)
in overall WISPS scores for the test catchment evaluated in Appendix M. This is likely to be due to the relative importance of the weightings to one another and also because of the use of a simple weighted average equation (5.1, page 124).

One particular area of concern is not dominant in terms of importance. The magnitude of influence, by an area of concern, on the WISPS score for a catchment is effectively controlled by the degree of importance afforded to it by the weighting. The larger the weighting the more influence a particular area of concern will have on the WISPS score. Rigorous sensitivity tests still require to be carried out using simulation techniques. These should be carried out in conjunction with the application of the WISPS technique to other catchments.

Through the modelling work carried out, sewer flow quality models have been shown to be of limited use to engineers. It is postulated that these models can only ever be calibrated and never truly represent the performance of the wastewater system. This is primarily due to the complex biological and chemical reactions occurring within the sewerage system, which are not as yet, fully understood.

Although at present the industry is pressing ahead in the application of sewer flow quality models, in many cases uncalibrated, this is a somewhat "head in the sand" approach. It is recognised that research into developing these tools must take place, but it is recommended that the models are not used by engineers until the fundamental problems associated with their representation of reality are adequately solved. A simpler approach utilising tools such as SIMPOL, with model calibrations against field data, maybe the preferred option when assessing pollutant spills from CSOs.

A technique based on multiple regression and historical rainfall has been applied to derive an estimate of the level of service afforded by the Perth drainage system in terms of flooding. This approach proved successful, and allowed an appreciation of real flooding performance to be derived, as opposed to traditionally relying on the application of statistical design storms. The multiple
The regression equation developed is based on relating flooding to storm peak intensity, duration and volume.

The approach has shown that it is important to evaluate and include unmodelled storage into the analysis. The flooding performance predicted by the regression model related closely to that of the sewerage system based on historical information. The regression model was developed based on the results of thirty selected storms. It is recommended that more be used if possible, and that thirty be used as a minimum for future applications.

Design storms have an important role in the sizing of new sewerage systems but do not reflect the behaviour of real rainfall. Engineers are now applying historical rainfall successfully to assess CSO performance as part of integrated studies. It is recommended that historical rainfall be applied in the attempt to assess actual flooding frequencies from sewerage systems. The flooding regression analysis used on the Perth system should be applied to other sewerage catchments with available models, as a means of further testing the approach. More investigations on relating actual flooding to that predicted by the regression approach would prove useful.

Public perception of water quality issues in the Perth catchment were carried out utilising survey techniques. These perceptions showed land derived waste to be the most influential factor when the general public considered surface water quality. The study additionally developed a modified technique to assess the aesthetic impact of sewage derived material from the Perth CSOs. Criticality scores were developed for each CSO within the Perth catchment based on the quantity of sewage derived litter present and the accessibility of the CSO to the general public.

It would have been gratifying to apply the WISPS methodology in full to a catchment in need of severe rehabilitation. This would have allowed scores to be developed for rehabilitation options and solutions consequently offered to the decision makers for perusal. Time precluded this, and the methodology as applied produced integrated performance scores, which described adequately the
performance of the considered drainage catchments, based on available historical information.

9.2 GENERAL CONCLUSIONS ON SYSTEM PERFORMANCE-PERTH

The flooding frequency in the Perth catchment appears to be acceptable to the general public and the drainage managers. Surcharging of the system is very frequent due to the ineffectiveness of the existing SIPS CSO. This is due to be rehabilitated shortly as part of the Flood Prevention Order.

Little documentation could be found relating to the structural gradings of the critical sewers in the system. It is recommended that an analysis is carried out in the near future to condense information relating to this area of concern. Sedimentation is widespread in the central area but generally is of low depth relative to pipe diameters. Infiltration has been shown to be present in large quantities. The sources of infiltration should be investigated to enable a better picture of the problem to be developed.

Aesthetic pollution from CSOs is present in trace to small amounts, although in relatively inaccessible sites. WWTP performance is excellent even with sludge loading, although this practice is clearly generating odour problems. Receiving watercourses are of good quality, although the public tended to perceive the cleanliness based on the presence and quantity of land derived waste. CSOs spill to watercourses at a frequency of six times a month on average.

9.3 RECOMMENDATIONS FOR FURTHER WORK

Any methodology must be fully applied in order to gauge the success of the approach. It is recommended that the WISPS system be applied to as many catchments within the NoSWA area as possible, subject to the availability of performance information. The methodology should be further tested in the role of rehabilitation option selection and monitoring of existing/rehabilitated catchments.
Future applications should be used to further fine tune the original weightings applied to the areas of concern. These should be the subject of a review based on a larger study group which ideally involves more engineers, water quality scientists and the general public. Surveys should also be carried out to assess the need to include or discard areas of concern. The final weightings assigned should be an aggregate of those offered by the respondents.

Comparisons should be made between weightings assigned to areas of concern by the public and those by the informed experts. It is proposed that conflict will be evident in certain areas of concern, such as odour emissions and aesthetic pollution. The value functions developed should also be fine tuned on the basis of a larger study group. On the basis of review it may be possible to develop additional value functions for new areas of concern.

The technique should be applied to other test catchments, whereby, historical and detailed WISPS scores can be evaluated and compared. This should indicate the validity of relying on historical scores as indicators of system performance and the need for rehabilitation. Ideally, the technique should be applied to a catchment requiring rehabilitation, so that the use of the approach in Stage II of the methodology can be demonstrated, tested and critically evaluated. The WISPS approach should also be appraised in the monitoring role defined in Stage III of the methodology.

It would be valuable to subdivide the areas of concern for flooding into that associated with residential, industrial, commercial and recreational areas of a catchment. Similarly, WWTP performance could be analysed on removal efficiencies relating to primary, secondary and tertiary treatment. This area of subdivision could be explored, as could the prospect of including new areas of concern, associated with sewerage systems. Subdivision may not be appropriate as the addition of sub-levels may overcomplicate the issue and ruin the simplicity of the approach. The requirement for this should become apparent through future applications of the WISPS methodology.
The methodology can be applied to potable water systems. This would require areas of concern to be developed for leakage rates, pressure consistency, reservoir performance, water quality, service interruptions and water quantity. Value functions relating performance in these areas to the scoring system would be developed through interviews with engineers and water distribution managers. Theoretically, similar work could be carried out on groundwater and solid waste management. If this idea is developed further, systems could be holistically appraised, rehabilitated and monitored in terms of key areas of concern relating to water, sewerage, groundwater and solid waste management.
Appendix A

References


46. Fraser James, (1961) *Report to the Gannochy Trust in Relation to the Drainage of the City Of Perth.*

47. Fraser James, (1964) *Sewers and Sewage Purification Works.*


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Appendix A


96. Local Government Operational Research Unit (1978) *The Economics of Pumped Drainage*, LGROU, Reading.


98. MAFF (1993) *Flood And Coastal Defence Project Appraisal-Guidance Notes*


Appendix A


122. Scottish Development Department (1977) Ministry of Housing and Local Government, Technical Committee in Storm Overflows, Storm Sewage and Disposal, Scottish Development Department, HMSO.


136. Wallingford Procedure Users Group *Hydraulic Modelling Code of Practice*.

137. Wallingford Procedure Users Group *User Note 21*.

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143. **WAA** (1988) Welsh Water Authority, QUALSOC.


Appendix B

Fundamental, Derived and Amenity Standards (*FWR 1994a*)

**Introduction**

Tabulated below are the fundamental and derived intermittent standards developed under the UPM programme for protecting aquatic life and watercourse amenity. The reader is directed to the UPM manual for a fuller description and implementation of these standards.

<table>
<thead>
<tr>
<th>DO concentrations (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return Period</td>
</tr>
<tr>
<td>1 month</td>
</tr>
<tr>
<td>3 months</td>
</tr>
<tr>
<td>1 year</td>
</tr>
</tbody>
</table>

**Table B.1 Fundamental Intermittent Standards For Dissolved Oxygen**

(concentration/duration thresholds not to be breached more frequently than shown)

These thresholds apply when un-ionised ammonia concentrations are below 0.04mg/l. At higher un-ionised ammonia concentrations the following correction factors apply.

<table>
<thead>
<tr>
<th>Un-ionised ammonia</th>
<th>Correction factor for DO</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.04 to 0.15 mg/l</td>
<td>+1.0mg/l</td>
</tr>
<tr>
<td>more than 0.15mg/l</td>
<td>+2.0mg/l</td>
</tr>
</tbody>
</table>

**Table B.2 Correction Factors For Dissolved Oxygen**

Consider the DO threshold of 4mg/l for 1 hr. This has an allowable return period of 1 month. This means that the DO concentration at any given point in the river can occasionally fall below 4mg/l for periods longer than one hour provided that
such events do not happen more often than 12 times/year on average. Furthermore, once a year the DO can be depressed below 4mg/l for over 6 hours.

<table>
<thead>
<tr>
<th>Return Period</th>
<th>1hr (mg NH₃-N/l)</th>
<th>6hrs</th>
<th>24hrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 week</td>
<td>0.1</td>
<td>0.050</td>
<td>0.010</td>
</tr>
<tr>
<td>1 month</td>
<td>0.150</td>
<td>0.075</td>
<td>0.030</td>
</tr>
<tr>
<td>3 months</td>
<td>0.225</td>
<td>0.125</td>
<td>0.050</td>
</tr>
<tr>
<td>1 year</td>
<td>0.250</td>
<td>0.150</td>
<td>0.065</td>
</tr>
</tbody>
</table>

**Table B.3 Fundamental Intermittent Standards For Un-ionised Ammonia**

(concentration/duration thresholds not to be breached more frequently than shown)

These thresholds apply when DO concentrations are above 5mg/l. At lower DO concentrations the following correction factors apply.

<table>
<thead>
<tr>
<th>Dissolved Oxygen</th>
<th>Correction factor for Un-ionised ammonia</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 to 5 mg/l</td>
<td>x 0.5</td>
</tr>
<tr>
<td>&lt; 3 mg/l</td>
<td>x 0.25</td>
</tr>
</tbody>
</table>

**Table B.4 Correction Factors For Un-ionised Ammonia**

The thresholds also assume pH is above 7 and temperature is above 5 degrees centigrade. At lower pH and temperature correction factors are applied.

Working with un-ionised ammonia and dissolved oxygen is complex and problematic and as a result work under the UPM programme (FWR 1994a) developed derived standards, shown in Table B.5, for total ammonia and BOD. These determinands are modelled by all complex packages recommended for use within the UPM manual. No modelling of the receiving watercourse is required.
Appendix B

<table>
<thead>
<tr>
<th>Mean river Slope over reach being considered (m/km)</th>
<th>BOD concentration not to be exceeded over a 6hr duration more often than once a year on average (i.e. 1yr RP) (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width/depth ratio</td>
<td></td>
</tr>
<tr>
<td>4 8 12 16</td>
<td></td>
</tr>
<tr>
<td>for rivers in this range, field data/impact</td>
<td></td>
</tr>
<tr>
<td>modelling required to assess allowable BOD</td>
<td></td>
</tr>
<tr>
<td>&lt; 0.1</td>
<td></td>
</tr>
<tr>
<td>0.2</td>
<td>10</td>
</tr>
<tr>
<td>0.3</td>
<td>11 13</td>
</tr>
<tr>
<td>0.5</td>
<td>12 14 16</td>
</tr>
<tr>
<td>0.6</td>
<td>10 13 17 19</td>
</tr>
<tr>
<td>0.7</td>
<td>12 16 19 22</td>
</tr>
<tr>
<td>0.8</td>
<td>13 18 22 25</td>
</tr>
<tr>
<td>0.9</td>
<td>14 20 25 28</td>
</tr>
<tr>
<td>1</td>
<td>16 22 27 32</td>
</tr>
<tr>
<td>1.5</td>
<td>21 24 30 34</td>
</tr>
<tr>
<td>2.0</td>
<td>26 32 39 45</td>
</tr>
<tr>
<td>2.5</td>
<td>30 39 48</td>
</tr>
<tr>
<td>3.0</td>
<td>34 45</td>
</tr>
<tr>
<td>3.5</td>
<td>37</td>
</tr>
<tr>
<td>&gt; 3.5</td>
<td></td>
</tr>
<tr>
<td>For rivers in this range, a Formula A overflow is unlikely to cause a breach of the intermittent DO standards</td>
<td></td>
</tr>
</tbody>
</table>

Table B.5 Derived Intermittent Standards For BOD

The BOD concentration for a particular spill event is calculated as follows:-

\[
\text{BOD (mg/l)} = \frac{\text{Discharge Load} + \text{River Load}}{\text{Discharge Volume} + \text{River Volume}}.
\]

\[
\text{Discharge Load (g)} = \text{Total BOD load discharged by all upstream discharges over a critical 6 hour period during the event (usually taken to be the total event load).}
\]

\[
\text{Discharge volume (m}^3\text{)} = \text{Total Discharge volume over the same period as above.}
\]

\[
\text{River Load (g) } = \text{Total BOD load in 6 hours river flow above all discharges.}
\]

\[
\text{River Volume (m}^3\text{)} = \text{River volume equivalent to 6 hours river flow.}
\]

The width/depth ratio is based on the average surface width and mid river depth during low summer flow conditions. The mean river slope is based on the change
in water surface elevation between the discharge zone and the furthest downstream point of interest (e.g. a major weir or confluence). If a tributary carrying the CSO discharge falls steeply for a short distance before entering the main river under investigation, this initial fall should be ignored in calculating the mean slope.

<table>
<thead>
<tr>
<th>pH in river after mixing</th>
<th>Total ammonia concentration not to be exceeded more often than once a year on average (i.e. 1yr RP) (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>for 1 hr</td>
</tr>
<tr>
<td>7.4</td>
<td>26</td>
</tr>
<tr>
<td>7.6</td>
<td>16</td>
</tr>
<tr>
<td>7.8</td>
<td>10</td>
</tr>
<tr>
<td>8.0</td>
<td>6.6</td>
</tr>
</tbody>
</table>

Table B.6 Derived Intermittent Standards For Total Ammonia

The total ammonia (6hr) concentration is calculated in the same way as the BOD(6hr) concentration. For rivers where the summer pH is less than 7.4, a Formula A overflow is unlikely to cause a breach of the intermittent un-ionised ammonia standards. Further standards for amenity use have been developed and are presented in Table B.7.

<table>
<thead>
<tr>
<th>Amenity Use category</th>
<th>Expected frequency of Spills</th>
<th>Emission Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Amenity</td>
<td>≥1 spill/yr</td>
<td>6mm solids separation</td>
</tr>
<tr>
<td></td>
<td>≤ to 1 spill/yr</td>
<td>10mm solids separation</td>
</tr>
<tr>
<td>Moderate Amenity</td>
<td>≥30 spill/yr</td>
<td>6mm solids separation</td>
</tr>
<tr>
<td></td>
<td>≤30 spill/yr</td>
<td>10mm solids separation</td>
</tr>
<tr>
<td>Low Amenity and Non Amenity</td>
<td>-</td>
<td>best engineering design for CSO hydraulics</td>
</tr>
</tbody>
</table>

Table B.7 Emission Standards For Protecting Amenity Use

**High Amenity**: Areas where bathing and water contact sport (immersion, e.g. wind surfing, sports canoeing) are regularly performed. Watercourse passes through formal public park or beside formal picnic site. Shellfish waters.

**Moderate Amenity**: Areas used for recreation and contact sport (non-immersion e.g. boating). Popular footpath adjacent to watercourse. Watercourse
passes through housing development or frequently used housing centre area (e.g. bridge, pedestrian area, shopping area).

**Low Amenity:** Basic Amenity use only. Casual riverside access on a limited or infrequent basis, such as a road bridge in a rural area, footpath adjacent to watercourse.

**Non Amenity:** Seldom or never used for any amenity purposes. Remote or inaccessible area.

**6mm solids separation:** Separation from the effluent, of a significant quantity of persistent material and faecal/organic solids greater than 6mm in any two dimensions. This should be applied to at least 80% of the spilled volume in a typical year, the remainder being subject to 10mm solids separation. Alternatively the hydraulic design of the 6mm solids can be based on treating 50% of the volume discharged in a 1 year return period design event.

**10mm solids separation:** Separation, from the effluent, of a significant quantity of persistent material and faecal/organic solids giving a performance equivalent to that of a 10mm bar screen.
Appendix C

Plates
Plate No.1 Ragging of South Inch Screens

Plate No.2 Rain Gauge site Burghmuir Reservoir
Plate No.3 CSO spill from Friarton Overflow

Plate No.4 Aesthetic Pollution at Bridgend
Plate No.5 CSO spill from South Inch

Plate No.6 Aesthetic Pollution downstream of Willowgate
Plate No.7 MOSQITO sampling site Moncrieffe

Plate No.8 MOSQITO sediment sampling site Bridgend
Plate No.9 MOSQUITO sampling site Rannoch Road

Plate No.10 STOAT sampling site at inlet to primary tanks
Plate No.11 STOAT sampling site at aeration tanks

Plate No.12 STOAT sampling site final effluent
Plate No.13 Dissolved oxygen meter at aeration tanks

Plate No.14 Picket Fence Thickeners at Sleepless Inch
Appendix D
Supplementary Information On Hydraulic Model

Introduction
The information presented in Table D.1 shows the extent of monitoring equipment deployed in the Perth sewerage system for the purposes of calibration and verification of the Perth hydraulic model. Verification results are presented in Appendix F.

Table D.1 Perth Flow Logger Sites 1991-1994

<table>
<thead>
<tr>
<th>Logger Site</th>
<th>Subcatchment</th>
<th>Location</th>
<th>Pipe Diameter</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0140</td>
<td>Craigie</td>
<td>Cleeve Caravan Park</td>
<td>375mm</td>
<td></td>
</tr>
<tr>
<td>0150</td>
<td>Central</td>
<td>Dry Arch, Perth Bridge</td>
<td>1310mm</td>
<td>Logger head offset to avoid sediment bed</td>
</tr>
<tr>
<td>0151</td>
<td>Central</td>
<td>Dry Arch, Perth Bridge</td>
<td>600mm</td>
<td></td>
</tr>
<tr>
<td>0160</td>
<td>Central</td>
<td>North Inch</td>
<td>900mm</td>
<td></td>
</tr>
<tr>
<td>0170</td>
<td>Rannoch</td>
<td>Goodyburn Terrace, grassed area</td>
<td>300mm</td>
<td>Partially separate system</td>
</tr>
<tr>
<td>0180</td>
<td>Rannoch</td>
<td>Rannoch Road, above Tibbermore Gardens.</td>
<td>750mm</td>
<td>Partially separate system</td>
</tr>
<tr>
<td>0190</td>
<td>Moncrieffe</td>
<td>Jewsons Access Road</td>
<td>750mm</td>
<td></td>
</tr>
<tr>
<td>0200</td>
<td>Craigie</td>
<td>Lesser South Inch</td>
<td>1050mm x 850mm</td>
<td>Egg shaped</td>
</tr>
<tr>
<td>0210</td>
<td>Central</td>
<td>Shore Road</td>
<td>1540mm</td>
<td>Logger head offset to avoid sediment bed</td>
</tr>
<tr>
<td>0220</td>
<td>Central</td>
<td>Lorry Park, Friarton</td>
<td>1200mm</td>
<td></td>
</tr>
<tr>
<td>0230</td>
<td>Central</td>
<td>Salmon Smokery, above WWTP</td>
<td>1125mm</td>
<td></td>
</tr>
<tr>
<td>0240</td>
<td>Hillyland</td>
<td>Crieff Road</td>
<td>300mm</td>
<td></td>
</tr>
<tr>
<td>0250</td>
<td>Hillyland</td>
<td>Crieff Road</td>
<td>375mm</td>
<td></td>
</tr>
<tr>
<td>0260</td>
<td>Hillyland</td>
<td>Crieff Road</td>
<td>450mm</td>
<td></td>
</tr>
<tr>
<td>Logger Site</td>
<td>Subcatchment</td>
<td>Location</td>
<td>Pipe</td>
<td>Comments</td>
</tr>
<tr>
<td>-------------</td>
<td>--------------</td>
<td>----------</td>
<td>------</td>
<td>----------</td>
</tr>
<tr>
<td>0270</td>
<td>Bridgend</td>
<td>Bridgend Interceptor sewer</td>
<td>600mm</td>
<td></td>
</tr>
<tr>
<td>0280</td>
<td>Bridgend</td>
<td>Minor leg leading to Willowgate</td>
<td>450mm</td>
<td></td>
</tr>
<tr>
<td>0290</td>
<td>North Muirton</td>
<td>Golf course North Muirton</td>
<td>750mm</td>
<td></td>
</tr>
<tr>
<td>0310</td>
<td>Tullton</td>
<td>Muirtonbank</td>
<td>600mm</td>
<td></td>
</tr>
<tr>
<td>0311</td>
<td>Tullton</td>
<td>Muirton Place</td>
<td>600mm</td>
<td></td>
</tr>
<tr>
<td>0312</td>
<td>Tullton</td>
<td>Tulloch Works</td>
<td>450mm</td>
<td></td>
</tr>
<tr>
<td>0313</td>
<td>Tullton</td>
<td>Rear of Crieff Rd Nursery School</td>
<td>375mm</td>
<td></td>
</tr>
<tr>
<td>0320</td>
<td>North Muirton</td>
<td>Grassed area adjacent to Bute Drive</td>
<td>525mm</td>
<td></td>
</tr>
<tr>
<td>0380</td>
<td>Bridgend</td>
<td>Bridgend Interceptor sewer</td>
<td>600mm</td>
<td></td>
</tr>
<tr>
<td>1001</td>
<td>Moncrieffe</td>
<td>In Pathway adjacent to Tescoes</td>
<td>375mm</td>
<td>Mosquito Site</td>
</tr>
<tr>
<td>1002</td>
<td>Tullton</td>
<td>Tulloch Works</td>
<td>375mm</td>
<td>Mosquito Site</td>
</tr>
<tr>
<td>1003</td>
<td>North Muirton</td>
<td>Inveralmond Industrial Estate</td>
<td>600mm</td>
<td>Mosquito Site</td>
</tr>
<tr>
<td>1004</td>
<td>Bridgend</td>
<td>Mansfield Place</td>
<td>375mm</td>
<td>Mosquito Site</td>
</tr>
<tr>
<td>1009</td>
<td>Moncrieffe</td>
<td>Culverted Watercourse,</td>
<td>600mm</td>
<td>Mosquito Site</td>
</tr>
<tr>
<td>1010</td>
<td>North Muirton</td>
<td>Backwash pipe from Gowans Terrace</td>
<td>300mm</td>
<td>Mosquito Site</td>
</tr>
<tr>
<td>1011</td>
<td>Rannoch</td>
<td>Rannoch Road above Tibbermore Gardens</td>
<td>750mm</td>
<td>Mosquito Site</td>
</tr>
<tr>
<td>1012</td>
<td>North Muirton</td>
<td>Grass adjacent to Bute Drive.</td>
<td>750mm</td>
<td>Mosquito Site</td>
</tr>
<tr>
<td>1015</td>
<td>Central</td>
<td>Above Friarton Pumping Station</td>
<td>1200mm</td>
<td>Mosquito Site</td>
</tr>
</tbody>
</table>
Appendix D

Introduction

During the development of the hydraulic tool many ancillaries were surveyed and represented in the model. Shown in Table D.2 are a list of these ancillaries along with their location and comments on the significance of each ancillary in terms of the hydraulic performance of the Perth system.

Table D.2 Perth Sewerage System - Modelled Ancillaries

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low-sided weir bifurcation</td>
<td>Diverts flow from Jeanfield Rd to Letham Rd. Low significance. <strong>Rannoch</strong> subcatchment.</td>
</tr>
<tr>
<td>Orifice bifurcation</td>
<td>Diverts combined flow from Tulloch works underneath Town Lade to the foul sewer on adjacent bank. <strong>Medium</strong> significance. <strong>Tullton</strong> subcatchment.</td>
</tr>
<tr>
<td>Orifice bifurcation</td>
<td>Diverts combined flow from Tullton subcatchment to outfall sewer from Rannoch/Hillyland at Crieff Road Nursery School. <strong>Medium</strong> significance. <strong>Tullton</strong> subcatchment.</td>
</tr>
<tr>
<td>Orifice overflow</td>
<td>Diverts combined flow from Bridgend subcatchment to River Tay. <strong>High</strong> significance. Mansfield Place. <strong>Bridgend</strong> subcatchment.</td>
</tr>
<tr>
<td>Orifice overflow</td>
<td>Diverts combined flow from Bridgend subcatchment to River Tay. <strong>Medium</strong> significance. Rear of Bakery, No2 Main St. <strong>Bridgend</strong> subcatchment.</td>
</tr>
<tr>
<td>Low-sided weir overflow</td>
<td>Diverts combined flow from Bridgend subcatchment to River Tay. <strong>Medium</strong> significance. Crossroads at West Bridge St/Gowrie St/East Bridge St and Main St. <strong>Bridgend</strong> subcatchment.</td>
</tr>
<tr>
<td>High-sided single weir overflow</td>
<td>Diverts combined flow from Bridgend subcatchment to River Tay. <strong>Low</strong> significance. Opposite Stanners Island, near slipway <strong>Bridgend</strong> subcatchment.</td>
</tr>
<tr>
<td>Orifice surcharge relief</td>
<td>Diverts combined flow from Craigie subcatchment around Windsor Terrace to <strong>Craigie Burn</strong>. <strong>Low</strong> significance. <strong>Craigie</strong> subcatchment.</td>
</tr>
<tr>
<td>Structure Type</td>
<td>Comments</td>
</tr>
<tr>
<td>---------------------------------------</td>
<td>--------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Orifice surcharge relief</td>
<td>Diverts combined flow from Craigie subcatchment around Windsor Terrace to Craigie Burn. Low significance. Craigie subcatchment.</td>
</tr>
<tr>
<td>Orifice surcharge relief</td>
<td>Diverts combined flow from Craigie subcatchment around Windsor Terrace to Craigie Burn. Low significance. Craigie subcatchment.</td>
</tr>
<tr>
<td>Orifice surcharge relief</td>
<td>Diverts combined flow from Craigie subcatchment around Windsor Terrace to Craigie Burn. Low significance. Craigie subcatchment.</td>
</tr>
<tr>
<td>South Inch PS</td>
<td>Diverts combined flows from Moncrieffe subcatchment to culverted watercourse. Low significance. Moncrieffe subcatchment.</td>
</tr>
<tr>
<td>Friarton PS</td>
<td>Diverts combined flow to River Tay. Central subcatchment. Very High significance.</td>
</tr>
<tr>
<td>Willowgate PS</td>
<td>Diverts combined flow from Atholl St to interceptor sewer Low Significance. Central subcatchment.</td>
</tr>
<tr>
<td>High sided single weir Bifurcation</td>
<td>2 centrifugal vertical spindle pumps. Wet well installation. Bridgend subcatchment.</td>
</tr>
<tr>
<td>Willowgate Pumping Station</td>
<td>1 No. 10HP archimedes screw.</td>
</tr>
<tr>
<td>South Inch Pumping Station</td>
<td>1 No. 25HP archimedes screw.</td>
</tr>
<tr>
<td>Friarton Pumping Station</td>
<td>1 No. 20HP archimedes screw.</td>
</tr>
<tr>
<td>Inverted Syphon</td>
<td>1 No. 60HP archimedes screw.</td>
</tr>
<tr>
<td></td>
<td>Carries flows underneath Craigie Burn. Central subcatchment.</td>
</tr>
</tbody>
</table>

Table D.2 continued Perth Sewerage System - Modelled Ancillaries
<table>
<thead>
<tr>
<th>ID</th>
<th>Date</th>
<th>Time</th>
<th>Location</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Velocity</th>
<th>Course</th>
<th>Temperature</th>
<th>Humidity</th>
<th>Wind Speed</th>
<th>Pressure</th>
<th>Rainfall</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>01/01/2023</td>
<td>08:00</td>
<td>New York</td>
<td>40.7878</td>
<td>-74.0060</td>
<td>10.2</td>
<td>326</td>
<td>23.5</td>
<td>44.6</td>
<td>15.3</td>
<td>1013.2</td>
<td>0.0000</td>
<td>Dry</td>
</tr>
<tr>
<td>002</td>
<td>01/02/2023</td>
<td>10:00</td>
<td>Chicago</td>
<td>41.8781</td>
<td>-87.6298</td>
<td>23.4</td>
<td>252</td>
<td>21.8</td>
<td>43.2</td>
<td>14.7</td>
<td>1012.8</td>
<td>0.0012</td>
<td>Wet</td>
</tr>
<tr>
<td>003</td>
<td>01/03/2023</td>
<td>12:00</td>
<td>Los Angeles</td>
<td>34.0520</td>
<td>-118.2437</td>
<td>18.3</td>
<td>279</td>
<td>28.5</td>
<td>42.1</td>
<td>14.0</td>
<td>1014.5</td>
<td>0.0000</td>
<td>Sunny</td>
</tr>
<tr>
<td>004</td>
<td>01/04/2023</td>
<td>14:00</td>
<td>Miami</td>
<td>25.7610</td>
<td>-80.1918</td>
<td>12.4</td>
<td>201</td>
<td>30.5</td>
<td>41.7</td>
<td>15.8</td>
<td>1013.1</td>
<td>0.0000</td>
<td>Rainy</td>
</tr>
<tr>
<td>005</td>
<td>01/05/2023</td>
<td>16:00</td>
<td>Houston</td>
<td>29.7604</td>
<td>-95.3699</td>
<td>9.3</td>
<td>282</td>
<td>26.4</td>
<td>40.9</td>
<td>14.3</td>
<td>1013.9</td>
<td>0.0000</td>
<td>Clear</td>
</tr>
</tbody>
</table>

**Notes:**
- **Velocity:** Miles per hour
- **Course:** Magnetic course
- **Temperature:** Degrees Fahrenheit
- **Humidity:** Percentage
- **Wind Speed:** Knots
- **Pressure:** Millibars
- **Rainfall:** Inches

*Data collected by Weather Station X.*
<table>
<thead>
<tr>
<th>Page</th>
<th>Image</th>
<th>X</th>
<th>Y</th>
<th>Width</th>
<th>Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Image</td>
<td>111</td>
<td>469</td>
<td>834</td>
<td>572</td>
</tr>
</tbody>
</table>

The text contains a table with multiple entries, each entry is a set of numbers separated by commas. The table spans several pages, with each page containing a similar layout of data entries. The table entries are not clearly transcribed due to the nature of the image.
The diameter of the conduit downstream (if any)

<table>
<thead>
<tr>
<th>Area</th>
<th>Width</th>
<th>Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>300</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>400</td>
<td>4.00</td>
<td>4.00</td>
</tr>
<tr>
<td>500</td>
<td>5.00</td>
<td>5.00</td>
</tr>
<tr>
<td>600</td>
<td>6.00</td>
<td>6.00</td>
</tr>
<tr>
<td>700</td>
<td>7.00</td>
<td>7.00</td>
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<tr>
<td>800</td>
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<tr>
<td>1000</td>
<td>10.00</td>
<td>10.00</td>
</tr>
</tbody>
</table>

**COMMENT:** The diameter of this office is set to the width of the conduit downstream (if any).
The diameter of this orifice is set to the width of the conduit downstream (if any).
Appendix E

Hydraulic Modelling Simplifications

Introduction

During construction of the hydraulic model simplifications were made. These are listed below and are split into general and subcatchment simplifications. The degree of simplification was limited as the final model (1200 pipes) required to be detailed enough to provide accurate dry weather and storm flows for the parasitic pollutant model MOSQITO.

General Simplifications

General simplifications are listed below:

1. Where pipe lengths and manholes have been omitted the storage they provide was added in by using extra manholes.

2. No allowance has been made for the storage available within domestic connections, traps and gullies.

3. Where information on certain pipe runs was not available invert levels and ground levels were interpolated. This was avoided if at all possible.

4. Floodable areas have not been entered in the model except for the subcatchment of Craigie. It is not known if these areas were estimated or were entered following a site survey. It is suspected these areas are estimated. To enter accurate floodable areas would require a detailed survey of the whole catchment. Due to staffing limitations this was not carried out.

Moncrieffe

Simplifications to the Moncrieffe subcatchment are listed below:

1. Manholes across Tesco's carpark were unlocated and thus the pipes here were joined together to form pipe 400.310.

2. The culverted water courses which are present in Moncrieffe were not modelled but flows were logged to provide input to the model.
Appendix E

Craigie

No written details are available for simplifications made to Craigie Subcatchment as the model was constructed before the present study commenced.

Tullton

Simplifications to the Tullton subcatchment are listed below:

1. Dummy pipes 21.495 and 22.480 were added at the bifurcation ancillary, present in the grounds of the Crieff Road Nursery School. This was to simplify the network for modelling purposes. The overflow pipe from the Tullton sewer to the Rannoch/Hillyland sewer was modelled. The overflow from the latter sewer, at the same ancillary, to the Tullton sewer was not modelled. This omission was based on the fact that if the water level in the Rannoch/Hillyand sewer was high enough to overflow into the Tullton sewer from the higher level bifurcation then the level would have been high enough to induce reverse flow through the Tullton overflow. This cannot be modelled by WALLRUS.

2. Infiltration was identified to be entering 21.360 from the Lade. The infiltration was small during dry weather. The magnitude of flows during storm conditions is not known but is expected to be small relative to the runoff from impermeable areas.

North Muirton

Simplifications to the North Muirton subcatchment are listed below:

1. Pipes from the North Muirton subcatchment which run through the middle of the North Inch, 1.640-1.680, were found to have buried manholes grass. Therefore, ground levels and invert levels were interpolated from available data. Backwash flow from the rapid gravity filters at Gowans Terrace, where necessary was added in as a input hydrograph to pipe 6.010.

2. Dummy pipe lengths 1.690 and 1.540 were modelled to eliminate instabilities.

3. Pipe lengths 1.140 and 4.180 were found to have negative gradients. This was possibly due to survey error and thus these were altered to interpolated positive gradients to eliminate instabilities.
Appendix E

Rannoch

Simplifications to the Rannoch subcatchment are listed below:

1. Legs omitted from the Rannoch subcatchment; Newhouse Rd, Dunsinane Dr, Moulin Crescent, Muirfield, Craiglea Rd, Muirend Rd, Birnam Crescent, Brahan Tce, Tweedsmuir Rd, Appin Tce, Kingswell Tce, Campsie Rd, Anderson Dr and Mountview Rd. These legs of the Rannoch network were omitted due to the of accurate surveyed data on ground levels and invert levels. These legs could be added in if necessary following an appropriate manhole survey, but it must be stated that the omitted pipes are of minor importance in the scale of the model, and have not affected the verification as can be seen from the presented graphs (see Rannoch verification, Appendix F).

Hillyland

Simplifications to the Hillyland subcatchment are listed below:

1. Various pipes were omitted from the Hillyland subcatchment, these are Struan Rd, Castleview, Langside Rd, Milton Rd, Tarvie Pl, part of Strathtay Rd and Fortingall Pl. These pipes were not added to the model as they were small in diameter and drained foul flows.

Bridgend

Simplifications to the Bridgend subcatchment are listed below:

1. The sewer running down Pitcullen Crescent from Pedwarden Rd to Dupplin Rd was not modelled as no information was available on invert levels.

2. Sewers in Kincarrathie Crescent, Hatton Mews, Kinnoul Hill Place, Glebe Tce, Bellwood Park and Brompton Tce were not modelled.

3. The drainage system within Potterhill Gardens was not modelled explicitly.

4. A dummy pipe was included to transfer the flows from the Willowgate pumping station over the Railway Bridge to the sewer in Tay Street. This was necessary as WALLRUS does not model rising mains explicitly.
Central

Simplifications to the Central subcatchment are listed below

1. The dual inverted syphons which convey the flow underneath the Craigie Burn between the SIPS and FPS were modelled as an equivalent sloping pipe with an on-line tank at the downstream end. This tank had an initial water level set to equal the invert level of the next downstream pipe. This ancillary was modelled in accordance with advice given in WaPUG user note 19 to enable the model to run. Conversion of the model to HYDROWORKSTM enabled the inverted syphons to be modelled in accordance with reality.

2. The SIPS and FPS overflows, were not modelled as separate structures. Instabilities were noted when this was first attempted. More flow was observed to be leaving the tanks than was entering. A decision was made to model the overflows as a integral part of the Pumping Stations. This eliminated the instabilities. However this modelling setup was not compatible with MOSQITO. MOSQITO requires every overflow to be a subsidiary outfall so pollutant loads leaving the system via the overflow can be balanced. Conversion of the model to HYDROWORKSTM enabled the overflows to be modelled in accordance with reality.

3. Where the interceptor sewer divided into three pipes downstream of the FPS this was modelled as one pipe of equivalent area and roughness. Conversion of the model to HYDROWORKSTM enabled the trifurcation to be modelled in accordance with reality.

4. The Feus Rd and Spense Crescent areas were not modelled as there were no data concerning invert and ground levels. The contributing areas associated with these were added into the model at appropriate connection points. The volume of storage associated with these unmodelled areas was added in as extra manholes.

5. Certain contributing areas within the City Centre area were assumed to have an impermeability of 90% following a site inspection.
Pipe lengths 50.260 and 50.310 were altered to positive gradients to avoid instabilities. Pipes 50.360 and 1.811 were increased in length to give accurate calculations in these pipes.
Appendix F

Hydraulic Verification

Introduction

Any sewerage system model must accurately represent the performance of the real system under wet weather. A WALLRUS model was initially used to represent the hydraulic performance of the Perth network. Presented below are the verification graphs for numerous logging locations. These graphs show the level of accuracy produced by the model in representing the Perth system. The model was converted to Hydroworks™ in the latter part of the study which allowed improved modelling of many of the complex ancillaries.

Moncrieffe

Storms used for verification for the subcatchment of Moncrieffe are shown in Table F.1.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Date of Storm</th>
<th>Peak Intensity</th>
<th>Volume</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>0190</td>
<td>12/02/92</td>
<td>6 mm/hr</td>
<td>4.0 mm</td>
<td>232 mins</td>
</tr>
<tr>
<td>07/03/92</td>
<td>6 mm/hr</td>
<td>4.6 mm</td>
<td></td>
<td>242 mins</td>
</tr>
<tr>
<td>10/03/92</td>
<td>12 mm/hr</td>
<td>2.6 mm</td>
<td></td>
<td>158 mins</td>
</tr>
<tr>
<td>1001</td>
<td>15/03/93</td>
<td>12 mm/hr</td>
<td>2.2 mm</td>
<td>74 mins</td>
</tr>
</tbody>
</table>

Table F.1 Moncrieffe Verification Storms

All fits are within the limits of accuracy given in WaPUG's "Code of Practice for the Hydraulic Modelling of Sewer Systems". The storm on 12/02/92 did not trip the logger and the results have been included as an indication of the models accuracy for this subcatchment rather than a verification.

However, it can be seen by looking at the depth graph for this event that the logged depth is essentially higher than that for the modelled flow. This is the case for all the graphs presented for Moncrieffe verification. This is due to ragging of the logger cables behind the mouse head which was substantial at sites 0190 and 1001 (more so at Site 1001). Depth verification is not presented for the event on the 10/03/92 due to ragging. The flow verification for 12/02/92 indicates a very good fit, but this is somewhat negated because the logger trip level was not reached.
Moncrieffe Verification Site 1001 Flow v Time 15/03/93

Moncrieffe Verification Site 1001 Depth v Time 15/03/93
A good fit was achieved at Site 0190 for the storm occurring on the 07/03/92. Peak flows and volumes match well while the depth suffers from ragging of the sensor head. There appears to be a time shift between the two hydrographs. This maybe caused by using rain data from the Gowans gauge, which is located some distance from the subcatchment.

The time shift was assumed to be due to the storm tracking over Perth and occurring later over Moncreiffe than was modelled. This effect can be seen in the verification graph for the storm 10/03/92. Again there is a time shift in peak flows. WALLRUS slightly overpredicts on the flow but overall the fit is good.

A good match was achieved for the storm 15/03/93 at Site 1001. This site was a MOSQITO DWF site, but use was made of storms collected to back up subcatchment verification. A good match was achieved for flows and volume but with depth affected by pronounced ragging of the logger cables behind the mouse head. A time shift has occurred but again this was assumed to be due to using rain data from a gauge a considerable distance from the subcatchment.

**Tullton**

Storms used for verification of Tullton are shown in Table F.2.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Date of Storm</th>
<th>Peak Intensity</th>
<th>Volume</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>0311</td>
<td>01/11/92</td>
<td>12 mm/hr</td>
<td>6.4 mm</td>
<td>330 mins</td>
</tr>
<tr>
<td></td>
<td>09/11/92</td>
<td>18 mm/hr</td>
<td>10.4 mm</td>
<td>526 mins</td>
</tr>
<tr>
<td></td>
<td>16/11/92</td>
<td>12 mm/hr</td>
<td>7.4 mm</td>
<td>338 mins</td>
</tr>
<tr>
<td>0312</td>
<td>01/11/92</td>
<td>12 mm/hr</td>
<td>6.4 mm</td>
<td>330 mins</td>
</tr>
<tr>
<td></td>
<td>09/11/92</td>
<td>18 mm/hr</td>
<td>10.4 mm</td>
<td>526 mins</td>
</tr>
<tr>
<td></td>
<td>16/11/92</td>
<td>12 mm/hr</td>
<td>7.4 mm</td>
<td>338 mins</td>
</tr>
<tr>
<td>0313</td>
<td>01/11/92</td>
<td>12 mm/hr</td>
<td>6.4 mm</td>
<td>330 mins</td>
</tr>
<tr>
<td></td>
<td>16/11/92</td>
<td>12 mm/hr</td>
<td>7.4 mm</td>
<td>338 mins</td>
</tr>
</tbody>
</table>

*Table F.2 Tullton Verification Storms*

Verification of this subcatchment was good. Modelled flows and depths represented logged values closely at all logger sites. At Site 0311 the model accurately predicted time to peak, peak flows and peak depths. However, all
Tullton Verification Site 0311 Flow v Time 09/11/92

Tullton Verification Site 0311 Depth v Time 09/11/92
Appendix F

graphs for this site indicated that logged flows remained higher for longer than the corresponding modelled flows. This is due to UCWI increasing during the storm and thus producing more runoff near the tail of an event as shown by the logged flows. WALLRUS unfortunately does not possess the facility to vary the UCWI and utilises a fixed user defined value at the beginning of an event. This effect is pronounced on long duration storms. The resulting volume differences are within the recommended accuracy limits for the three verification storms at Site 0311.

At Site 0312 flows and depths were both verified. The model accurately predicted flows arriving at this site during storm conditions for 01/11/92 and 16/11/92. The event occurring on the 09/11/92 did not give a good fit at Site 0312. The peak flow and peak depth were over predicted by WALLRUS. This was due to spatial variation of the rainfall event across the Tullton subcatchment. The poor fit shown for 0312 is a result of the peak rainfall intensity failing to fall on the area contributing to 0312. For the other events the modelled flows compare accurately with the logged values at Site 0312. No verification was carried out at Site 0313 for this event due to the poor fit achieved at 0312.

Problems were encountered at 0312 due to ragging and sedimentation obscuring velocity readings. The velocity at this site tended to be on the low side during DWF and thus the logger gave very little information on DWF. To combat this the mouse head was offset by 80mm which gave better results during low flows.

Sedimentation was found downstream of 0312 and this affected the depths during DWF and storms at this site. Surcharging occurred during each event used for verification and this proved difficult to reproduce accurately at first in the model. On site measurements were taken of the depth of sediment in the appropriate pipe downstream of 0312. The values recorded varied between 100mm to 150mm.

From site visits it was noted that the sedimentation in this pipe consisted of consolidated material and did not erode during storm conditions. A depth of 125mm was entered into the appropriate pipe in the model to represent the band of sediment. The depth at 0312 was also affected by the bifurcation overflow located downstream. The overflow discharges to a pipe which runs underneath
the Towns Lade and connects to the foul sewer from the Tulloch housing area. A logger was positioned at Site 0313 to verify the operation of this ancillary.

The localised flooding which occurs above 0312 is due to an interaction between sedimentation and the behaviour of the bifurcation. It was therefore important that depths be predicted accurately at 0312. Various coefficients for the continuation pipe were tried to induce the correct overflow discharge to be logged at 0313 and produce the correct depths at 0312. Depth verification at Site 0312 was achieved by using a sediment depth of 125mm downstream of the logger, a continuation pipe coefficient from the bifurcation of 0.1 and a local roughness of 30 for the pipe below 0313.

Flows were verified reasonably well at 0313 for 01/11/92 and 16/11/92 events indicating the assumptions used for the bifurcation are valid for storms of a similar nature.

Rannoch

Storms used for the verification of the Rannoch subcatchment are shown in Table F.3.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Date</th>
<th>Peak Intensity</th>
<th>Volume</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>0170</td>
<td>02/10/91</td>
<td>6 mm/hr</td>
<td>4.0 mm</td>
<td>116 mins</td>
</tr>
<tr>
<td></td>
<td>15/10/91</td>
<td>6 mm/hr</td>
<td>3.8 mm</td>
<td>158 mins</td>
</tr>
<tr>
<td>0180</td>
<td>12/11/91</td>
<td>6 mm/hr</td>
<td>3.6 mm</td>
<td>116 mins</td>
</tr>
<tr>
<td>1011</td>
<td>23/07/93</td>
<td>6 mm/hr</td>
<td>4.0 mm</td>
<td>332 mins</td>
</tr>
</tbody>
</table>

Table F.3 Rannoch Verification Storms

Verification for Rannoch was generally good considering the assumptions made regarding the impermeable areas draining from the roofs of properties. The graphs presented show a good match for flow and depth. The depth fits are affected by ragging causing the logged depths to be greater than predicted depths. This is pronounced for logger site 0170.

The trip levels used for the loggers at 0170 and 0180 were too high, and this resulted in accurate data not being available for some storms on the rising and falling limbs of logged hydrographs; due to the depth not reaching the required
Rannoch Verification Site 0170 02/10/91 Flow v Time

Rannoch Verification Site 0170 02/10/91 Depth v Time
Rannoch Verification Site 0180 12/11/91 Flow v Time

Rannoch Verification Site 0180 12/11/91 Depth v Time
Appendix F

trip level. Consequently verification is only accurate over the period where the logger was recording at 2 minute intervals. For the verification storms fits are good at peak flow and depth but suffer from the logger returning to the slow recording rate at the tail of the storms. This effect is also present at the start of the storms, where the model shows response to rainfall and the data from the logger has not reached the required trip level and gives values of flow and depth at 30 minute intervals. Linear interpolation was used between points of data recorded at 30 minute intervals to create logged hydrographs for verification.

To ensure verification for Rannoch was correct, as were the assumptions regarding the roofed areas, data were used for a storm collected at logger site 1011, a MOSQITO DWF sampling site in July 1993; some 1½ years later than the original verification. The fit for the storm on 23/07/93 is good in terms of peak depth, peak flow and volume. There is a slight difference in time to peak but this may be due to the roofs contributing very quickly to the system in reality while the model is assume some runoff is generated from road surfaces. The peak flows indicate that the contributing areas are correct.

**Bridgend**

Storms used for verification of Bridgend are shown in Table F.4.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Date</th>
<th>Peak Intensity</th>
<th>Volume</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>1004</td>
<td>29/03/93</td>
<td>6 mm/hr</td>
<td>27.8 mm</td>
<td>1146 mins</td>
</tr>
<tr>
<td></td>
<td>10/06/93</td>
<td>12 mm/hr</td>
<td>2.2 mm</td>
<td>38 mins</td>
</tr>
<tr>
<td></td>
<td>25/06/93</td>
<td>6 mm/hr</td>
<td>9.6 mm</td>
<td>608 mins</td>
</tr>
<tr>
<td>0380</td>
<td>15/03/93</td>
<td>12 mm/hr</td>
<td>2.2 mm</td>
<td>74 mins</td>
</tr>
<tr>
<td></td>
<td>29/03/93</td>
<td>6 mm/hr</td>
<td>27.8 mm</td>
<td>1146 mins</td>
</tr>
</tbody>
</table>

**Table F.4 Bridgend Verification Storms**

Problems were encountered in choosing suitable logging positions in Bridgend. This was due to a combination of pipe gradients and sedimentation. In the upper reaches of the catchment pipes were very steeply graded giving flows of high velocities and low depths and along the interceptor sewer leading to the Willowgate Pumping station deposits of sediment were found. These factors
Bridgend Verification Site 1004 25/06/93 Flow v Time

Bridgend Verification Site 1004 25/06/93 Depth v Time
Bridgend Verification Site 1004 10/06/93 Flow v Time

Bridgend Verification Site 1004 10/06/93 Depth v Time
affected the choice of logging site and initially loggers were installed at Sites 0270 and 0280. Both loggers suffered from pressure transducer drift and were removed due to inaccurate readings discovered during field calibration. A third logger was installed at Site 0380 further upstream of the Pumping Station on the interceptor sewer. This sewer was affected by sedimentation which lead to the mouse head being offset by 80mm to avoid the sediment. Storms recorded by this logger and a logger at Site 1004 were used for verification. Site 1004 was installed as part of the MOSQITO DWF sampling regime. All storms were verified for flow and depth and model predictions closely resembled logged values for both logger sites.

For the event occurring on the 29/03/93 predicted flows accurately matched logged flows for the first 600 mins. A graph is shown of the first 360 mins of this storm. After this period logged flows are higher. This may be due to WALLRUS inability to increase UCWI with time. Also the higher flows logged during the latter part of this Storm at 1004 may be originating from infiltration higher up the catchment. High base flows which are believed to be infiltration were recorded at this site. These flows ranged between 5(l/s) to 10(l/s) and were evaluated by observing the logged data between 0100hrs and 0400hrs.

This base flow varied, especially after long duration storms. This suggested along with the higher logged flows at the tail of storms that a watercourse was contributing infiltration and runoff to the logger site. Gannochy pond is located upstream from 1004 and was observed to receive runoff from an inlet pipe. This inlet pipe may be fed by the stream situated to the North of Murray Royal Hospital. This would lead to increased stream flows entering the pond during rainfall events. The flows entering the pond would be affected by hydrological factors associated with the streams catchment area. A sluice is located in the pond at the Northern edge and maintains a constant level by discharging flows to the sewer in Dupplin Rd. This excess flow appears to affect the long duration storm on the 29/03/93 more than it affects the storms on the 10/06/93 and the 25/06/93. Model predictions for the latter storms are accurate for flows and depths. Ragging affected the logged depths at this site.
Site 0380 was verified reasonably well for flow and depth for the storm occurring on the 15/03/93. This logger showed frequent surcharging during its installation period. This was believed to be caused by the local control at the overflow preceding the Willowgate pumping station. The overflow was found to have a continuation pipe partially blocked (½ to ¾ of the pipe area) by tree roots. This was reported by operatives reconstructing the pumping station (1994). The raked screen in the overflow chamber was inoperative and thus massive ragging of the screens was observed. The depth fit for the 15/03/93 was achieved by modifying the continuation pipe area to take account of the tree roots, using a weir coefficient of 0.5 to allow for the ragging, a continuation coefficient of 1.000 and raising the weir crest level by 200mm to allow for the ragging affecting the first spill level.

The storm occurring on the 29/03/93 was run to back up the above assumptions. The flow fit was reasonable but the model under predicts the peak depths. It is obvious that a better match could have been achieved for this storm by altering the overflow parameters but this was deemed to be unnecessary as the overflow and pumping station are undergoing complete renewal at the time of writing. It is proposed to upgrade the model and reverify along the section of sewer when the work is complete to investigate how the reconstruction affects surcharging.

*Craigie*

The subcatchment model of the Craigie system was constructed by TRC technicians and DIT sandwich students between April 1990 and June 1991. The model was partly verified by staff at DIT, then used to design storage and attenuation structures, to allow proposed development along the western edge of the catchment to take place. As part of the current study it was necessary to complete the verification work on this subcatchment. A logger was installed at Site 0200 on the grassed area of the South Inch in a previous logger position located during the Craigie study (Site 0100). Three storms were used for verification and are shown in Table F.5.
Appendix F

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Date</th>
<th>Peak Intensity</th>
<th>Volume</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>0200</td>
<td>02/02/92</td>
<td>6 mm/hr</td>
<td>3.1 mm</td>
<td>210 mins</td>
</tr>
<tr>
<td>03/02/92</td>
<td>6 mm/hr</td>
<td>6.4 mm</td>
<td></td>
<td>350 mins</td>
</tr>
<tr>
<td>12/02/92</td>
<td>6 mm/hr</td>
<td>4 mm</td>
<td></td>
<td>232 mins</td>
</tr>
</tbody>
</table>

Table F.5 Craigie Verification Storms

Verification graphs are presented for flows only. Site 0200 was strongly influenced by the operation of the screw pumps and by the operation of the overflow at the SIPS. Attempts were made to achieve accurate model predictions for depth at this site by applying the backwater flag to the pipes along the length of the Craigie sewer running along the South Inch (91.240 to 91.280). This produced better results but WALLRUS still under predicted peak depths for site 0200. Parameters associated with the pumping station and overflow could have been adjusted to give the build up of depth in the main interceptor which would have resulted in the required depths at 0200 being achieved. A decision was made to consider the depth verification at 0200 along with the verification for Site 0210 located upstream of the Craigie connection on the main interceptor sewer. This is discussed in Central verification.

Regarding the flow verifications at 0200, the model predictions are reasonably accurate with peak flows, time to peak and volumes being within the required limits. However, the model under predicts the peak flow for each storm suggesting a proportion of impermeable area has been omitted from the model. On inspection of the OS 1:1250 plans used for area measurement and the WALLRUS SSD file it is clear that excess permeable area has been included in the model. Pipe area boundaries have not been drawn by utilising the "10 metre rule" as stated in WaPUG user notes. Inclusion of these permeable areas effectively reduces the percentage impermeability of the contributing area and thus reduces runoff. This may be the cause of the model slightly under predicting peak storm runoffs for the Craigie subcatchment. As only a small amount of documentation exists for this model it was felt that to correct the areas would be force fitting the model for the particular verification events.
Hillyland

Verification of the Hillyland subcatchment was not achieved at logger sites 0240, 0250 and 0260. Site 0240 was installed on the road ditch pipe connecting into the Goodlyburn. This logger showed response to rainfall but it was so slight that it was unclear if the variation in the flow was due to runoff or the diurnal pattern of the expected wrong connections for small rainfall events. The rainfall over the period of installation of this logger showed that the response of the road ditch to rainfall was small. The logger response at 0240 was assumed to be a result of runoff from the Crieff Road and this area was not included in the model. This is an assumption based on logged data and observations made in the field.

A logger was installed at Site 0260 to give information on the flows in the old pipe. This logger showed the system was responding to rainfall. This is expected to originate from the roofs of the older properties along the Crieff Road. From the logged data it was apparent that the trip level was set too high, and thus no information was gained on flows and depths at 2min intervals. This information was used for indicative purposes but because of its limited nature could not be used for verification.

The most accurate information on impermeable area concerned the roofed areas of the multi-storey flats draining to the new pipe in the Crieff Road. A logger was installed at Site 0250 to collect verification data. This logger gathered flows and depths for four storms. Unfortunately two of these storms did not produce enough runoff to reach the trip level. The other two events were very high intensity events (one event reached 144mm/hr) and of very short duration. From examination of the three rain gauges it was apparent that these two latter events were spatially varied across Perth and could not be used confidently for verification purposes.

This lead to the inability to successfully verify the Hillyand subcatchment. However, the assumptions made were entered into the model. Hillyland was not verified at the logger sites above. Verification at Site 0160 was achieved. This site in the Central catchment receives flows from Rannoch, Hillyland and part of the
Town centre. The modelled and predicted flows here are accurate and lend confidence to the assumptions made for Hillyland. These graphs are presented in central verification.

**North Muirton**

The foul flows draining from North Muirton and the Inveralmond industrial estate were added in as average dry weather flows. Diurnal variation was taken into account during modelling with the MOSQITO package.

**Central**

Loggers for initial verification of the Central subcatchment were installed at two positions Site 0150 and Site 0160. Site 0150 was located within the brick interceptor underneath the Dry Arch at Perth Bridge. Site 0160 was located within the 900mm diameter pipe which connects into the Interceptor on the North Inch.

Two storms were collected for verification and these are shown in Table F.6.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Date</th>
<th>Peak Intensity</th>
<th>Volume</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>0160</td>
<td>15/10/91</td>
<td>6 mm/hr</td>
<td>6 mm</td>
<td>404 mins</td>
</tr>
<tr>
<td></td>
<td>29/10/91</td>
<td>6 mm/hr</td>
<td>10.8 mm</td>
<td>706 mins</td>
</tr>
</tbody>
</table>

**Table F.6 Central Verification Storms**

The model was verified at Site 0160 as there were problems associated with Site 0150. Uncertainties arose when verification was taking place for Site 0150. The logger head had been offset when installed because of the nature of the pipe and to avoid masking by sediment. The offset was 140mm but records were unclear and did not show where this offset was measured from; the consolidated sediment bed or from the pipe invert. Also, when manhole record cards were checked there was a difference between the survey measurements and the ones taken to construct a pipe cross section for the logged flows.

A decision was taken to not use this site for verification as any alterations made to the pipe diameter, sediment depth and offset applied made a difference to the
Central Verification Site 0160 15/10/91 to 16/10/91 Flow v Time

Central Verification Site 0160 15/10/191 to 16/10/91 Depth v Time
logged flows and affected the accuracy of verification. Thus verification was carried out for Site 0160 using the storms tabulated.

Both storms show a good match for all the required verification parameters, i.e. peak flow, peak depth, time to peak and volumes of runoff are within the required accuracy for the model. For the storm occurring on the 15/10/91 a DWF of 40l/s was used as the base flow, this is based on average values from the contributing catchments draining to Site 0160. The actual logged flow was around 20l/s and if this had been used the model would have slightly under predicted the runoff. As the flow is greater at the beginning of the storm so is the predicted depth. However, a modelled DWF of 20l/s would produce a lower base flow depth and correspondingly, a lower, but reasonably accurate peak depth for this event at Site 0160.

The second event produced a good fit for flow and depth over the first 540mins of the storm occurring on the 29/10/91. The logged flows and depths are higher after this time. This is again due to the UCWI increasing throughout the storm and producing more runoff at the logger site. WALLRUS has problems dealing with long duration storms because it cannot increase the UCWI with time and under predicts the flows at the end of long duration events.

The logger positioned at Site 0151 was not used for verification. This logger gave information on the flows entering the Central area from the North Muirton subcatchment and from the Gowans Terrace (water treatment works).

Loggers were installed at sites 0210 and 1015. 0210 was upstream of the South Inch overflow and pumping station. 1015 was installed upstream of the Friarton overflow and pumping station. Storms used for verification are shown in Table F.7.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Date</th>
<th>Peak Intensity</th>
<th>Volume</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>1015</td>
<td>25/06/93</td>
<td>6 mm/hr</td>
<td>9.8 mm</td>
<td>494 mins</td>
</tr>
<tr>
<td></td>
<td>23/07/93</td>
<td>6 mm/hr</td>
<td>3.6 mm</td>
<td>330 mins</td>
</tr>
<tr>
<td>0210</td>
<td>22/08/92</td>
<td>12 mm/hr</td>
<td>7.6 mm</td>
<td>196 mins</td>
</tr>
</tbody>
</table>

Table F.7 Central Verification Storms
Logger 0210 recorded a multitude of storms due to the length of time the instrument was installed in the sewer. Unfortunately a large percentage of storms did not cause the logger to trip. Some storms which caused the trip level to be reached were spatially varied across the catchment. As a result of this variation and the limited information from the rain gauges these events could not be used with confidence for verification.

Only one storm was used for verification. This storm was spatially varied with different peak intensities recorded at the Gowans gauge and the railway station gauge. It was apparent that most storms caused surcharging along the length of sewer above the SIPS. This was due to the condition of the screens located in the South Inch overflow chamber (See Plate 1, Appendix C).

It was apparent that the condition of the screens was severely attenuating any overflow discharge. The operation of the overflow appears to control the depths upstream of the structure. An overflow test was carried out to observe the operation of the structure. This was carried out by switching off the pumps and allowing the flow to back-up and begin to discharge over the weir. Some discharge took place immediately the weir crest level was reached. As the flow depth increased no discharge could pass through the ragging on the screens and thus the depth increased. Although this is not exactly how the structure operates under storm conditions it clearly showed the effects the ragging of the screens had upon the upstream depths in the interceptor sewer.

From observation it was clear that any overflow discharge passing the weirs would only discharge to the Tay via the flap valve if the level of the Tay was lower than the level of sewage in the overflow chamber. In the event of high levels in the Tay any spilled overflow would not leave the overflow chamber and would increase the depths at this point in the sewer. These points apply to the Friarton overflow chamber as the condition of the screens are exactly the same. These screens at the two installations are not maintained regularly because of lack of manpower. The screens were originally cleaned by automatic rakes, following failure of these rakes due to gear box shearing, they were never replaced.
Central Verification Site 1015 23/07/93 Flow v Time

Weir level = 2.272m AOD

Central Verification Site 1015 23/07/93 Depth v Time

Weir level = 2.272m AOD
The weir length on both the Friarton and South Inch overflows was reduced from 9m to 1.5m. This was to represent the effective weir length available to the flow as a result of the ragging on the screens at the weir crest. Both weirs were given coefficients of 0.07 to represent the effect of the severe ragging. Spill level at the South Inch was raised to 2.266m AOD from 1.693m AOD to allow for the ragging. Spill level at Friarton was raised to 2.272m AOD from 1.945m AOD. These alterations were done in accordance with guidance from the WaPUG User Note 27. Weir coefficients, spill levels and effective weir lengths have been input to the model based on their effects upstream of the structure and not on the overflow discharge. There was no way overflow discharge could be verified at both pumping stations due to site limitations. However, the choice of values selected seem justified under the circumstances and produce reasonably accurate verifications at 0210 and 1015.

Modelling of the River Tays effect on the flap valves was not possible for two reasons. The overflows at the two structures were included within the pump record as stated in central subcatchment. Using this record within WALLRUS means that the overflow must go to waste and not to an overflow pipe, which could have had a level hydrograph representing the Tay modelled. However, if this could have been achieved WALLRUS would not have allowed the overflow to affect the levels in the upstream sewer by backing up any overflow discharge. Two controls dictate the levels upstream. These are the operation of the screws and the operation of the flap valve. Conversion of the model to HYDROWORKS™ allowed a more accurate simulation of the dual controls operating at SIPS and FPS.

The verification shown for 0210 is adequate. Over the first 180mins the flow and depth are accurately predicted by the model. However, after this time period the logged flows are considerably higher. This may be a result of more rain falling on the catchment and not being picked up by the rain gauges. Also the surcharging predicted may be having a throttling effect upon the discharge. As only one event was available for verification it is difficult to determine the reasons as to this mismatch. Peak depths are accurately predicted by the model but logged depths
are higher over the latter part of the storm. This is possibly due to the flows not being released over the overflow and thus holding the depths in system at a higher level for longer. The original intention to consider the depths at Site 0200 as stated in Craigie verification was not done. This was due to the problems associated with achieving a good flow and depth fit at 0210.

Discharge and depth are accurately predicted for Site 1015 for both storms presented. Both verifications were carried out using an average DWF. The event occurring on the 23/07/93 shows the effects of the pumps switching on and off at the start of the simulation. Peak flow is accurately predicted for both events indicating the behaviour of the SIPS is correct. Peak predicted depth is lower than observed depths. This is due, as stated for 0210, to the effects of the overflow screens and the operation of the flap valve at the Friarton overflow.

During the data collection exercise for STOAT flows were monitored on the inlet pipe to the WWTP at Site 0020. During the period of installation of this logger only one suitable rain event could be used for verification. The profiles from each gauge proved to be similar when examined and three profiles were entered into the WALLRUS RED file. Details of the storm used for verification at Site 0020 are shown in Table F.8.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Date</th>
<th>Peak Intensity</th>
<th>Volume</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>0020</td>
<td>25/04/93</td>
<td>6 mm/hr</td>
<td>2.5 mm</td>
<td>240 mins</td>
</tr>
</tbody>
</table>

Table F.8 Central Verification Storms

The verification was reasonably accurate for flows and depths. However, the logger site was extremely close to the archimedes screws which convey the flow to the treatment works. No information on a head/discharge relationship was available for the inlet screws and thus the model has a free outfall instead of a pumping station at this point. It is clear from previous experience at the SIPS and FPS that depths upstream of screw pumps are affected by the operation of the screws. The flows in the sewer upstream of these screws will be held at certain depths depending on the behaviour of the pumps.
Appendix G
STOAT and MOSQITO Verification Graphs
BOD in the Storm Overflow - Storm

- ♦ - Predicted
- ⊙ - Measured
Ammonia in the Storm Overflow - Storm

- □ - Predicted
- ○ - Measured
Suspended Solids in the Storm Overflow - Storm

- Predicted
- Measured

Concentration mg/l

Time (hours)
Concentration mg/l

Suspended Solids in the Primary Tank Effluent - Storm

- Predicted
- Measured
Ammonia in the Primary Tank Effluent - Storm

- Predicted
- Measured
Concentration mg/l

Time (hours)

- BOD in the Primary Tank Effluent - Storm

- Predicted
- Measured
BOD in the Activated Sludge Effluent - Storm

- Predicted
- Measured
Suspended Solids in the Activated Sludge Effluent - Storm

Predicted
Measured
Dry Weather Verification - Site 1001 - TSS

Dry Weather Verification - Site 1001 - COD

Dry Weather Verification - Site 1001 - Ammonia
Dry Weather Verification - Craigie - TSS

Dry Weather Verification - Craigie - COD

Dry Weather Verification - Craigie - Ammonia
Appendix H
Public Perception Survey
Graphical Results of Questionnaire
Craigie Burn
Questions and Answers

Q1. What is the purpose of your visit to the watercourse today?

- Walking: 100%
- WalkWay: 90.0%

Q2. Do you use the watercourse for any other purpose?

- No: 100%

Q3. How often do you visit the burn?

- Everyday: 66.7%
- 2/3 Per Week: 16.7%
- 4/5 Per Week: 8.3%
- >1/Day: 8.3%

Q4. What do you find attractive about the burn?

- Nothing: 82.0%
- Wildlife/Ducks: 18.0%

Q5. How clean do you think the burn is?

- Very Dirty: 50.0%
- Dirty: 20.0%
- Neither Clean/Dirty: 10.0%
- Clean: 20.0%

Q6. On what did you base your answer to Q5?

- Clearness of Water: 18.0%
- Colour: 9.0%
- Chemical Pollution: 10.0%
- Refuse Present: 63.0%
Q7. Do you think sewage is discharged into the waterway?

- Yes: 20.0%
- (50% had seen evidence, 50% had not)
- No: 80.0%

Q8. If sewage was discharged, how much would this bother you?

- A lot: 50.0%
- Some: 40.0%
- Not At All: 10.0%

Q9. How do you think the quality of the waterways should be improved?

- Prevent Sewage Discharge: 50.0%
- Clear Vegetation: 11.0%
- Landscape Improve Path: 17.0%
- Dredging: 11.0%
Q10. In the time you have lived in Perth, how has the quality of the watercourse changed?

Q11. If the quality was upgraded, how much would you be willing to pay to use the area?

The age and sex of the interviewees were:

- Male 50%
- Female 50%
The Town Lade
Questions and Answers

Q1. What is the purpose of your visit to the watercourse today?

- Walking For Pleasure: 23.0%
- Thoroughfare: 77.0%

Q2. Do you use the watercourse for any other purpose?

- Walking: 8.0%
- Walking Dog: 8.0%
- No: 84.0%

Q3. How often do you visit the burn?

- Everyday: 15.0%
- 3/4 Per Week: 38.0%
- 1/2 Per Week: 32.0%
- >1/0Day: 15.0%
Q4. What do you find attractive about the burn?

Q5. How clean do you think the burn is?

Q6. On what did you base your answer to Q5?
Q7. Do you think sewage is discharged into the waterway?

- No: 100.0%

Q8. If sewage was discharged, how much would this bother you?

- Not At All: 17.0%
- Not Very Much: 17.0%
- Some: 25.0%
- A Lot: 41.0%

Q9. How do you think the quality of the waterways should be improved?

- Clear/Rubbish: 70.0%
- Improve Environment: 6.0%
- Landscaping: 6.0%
- Clear/Prevent Dogness: 18.0%
Q10. In the time you have lived in Perth, how has the quality of the waterway changed?

- Improved/Environmental: 44.0%
- Stayed the Same: 28.0%
- Deteriorated: 28.0%

Q11. If the quality was upgraded, how much would you be willing to pay to use the area?

- Nothing: 31.0%
- Up to £1: 31.0%
- £1 to £2: 7.0%

The age and sex of the interviewees were:

- Male: 62%
- Female: 38%

- 20-30 Yrs: 45.0%
- 30-<40 Yrs: 8.0%
- 40-=50 Yrs: 8.0%
- 50-<65 Yrs: 8.0%
- >65 Yrs: 8.0%
- <20 Yrs: 23.0%
River Tay
Questions and Answers

Q1. What is the purpose of your visit to the watercourse today?

- Walking: 25.0%
- Fishing: 17.0%
- Drawing: 8.0%
- Other: 8.0%

Q2. Do you use the watercourse for any other purpose?

- No: 36.0%
- Walking: 29.0%
- Fishing: 21.0%
- Appreciating Wildlife: 14.0%

Q3. How often do you visit the burn?

- Every Day: 27.0%
- 2 Per Week: 46.0%
- 1 Per Week: 18.0%
- 3 Per Week: 9.0%
- No: 36.0%
Q4. What do you find attractive about the burn?

Q5. How clean do you think the burn is?

Q6. On what did you base your answer to Q5?
Q7. Do you think sewage is discharged into the waterway?

Q8. If sewage was discharged, how much would this bother you?

Q9. How do you think the quality of the waterways should be improved?
Q10. In the time you have lived in Perth, how has the quality of the waterway changed?

- Improved: 18.0%
- Stayed the Same: 56.0%
- Deteriorated: 28.0%
- Don't Know: 18.0%

Q11. If the quality was upgraded, how much would you be willing to pay to use the area?

- Up to £1: 18.0%
- £1–£2: 18.0%
- £2–£3: 9.0%
- £3–£4: 9.0%
- £4–£5: 9.0%
- Over £5: 9.0%
- Nothing: 37.0%

The age and sex of the interviewees were:

Male 67%
Female 33%
Appendix J
Papers Written During Research Period

The Development of an Integrated Catchment Management Plan for the City of Perth, WaPUG, Autumn Meeting, Blackpool.


Capabilities and Limitations of Modelling in Holistic Management Practice. European Modellers Group, IMUG, 22nd November, Pembroke Hotel Blackpool.


The Optimum Management of Wastewater Systems-A Strategy for the City of Perth. 7th International Conference on Urban Storm Drainage September, Hannover, Germany.

7th International Conference on Urban Storm Drainage September, Hannover, Germany.
Sleepless Inch Waste Water Treatment Plant

Detailed below are general design data specific to the plant.

**Basic Design Data**
- Design Population: 50,000
- Dry Weather Flow: 139 l/s
- Industrial Effluent and Infiltration: 111 l/s
- Maximum Inflow: 1140 l/s
- Full Treatment to: 671 l/s
- Partial Treatment to: 433 l/s

Prior to the data collection exercise commencing it was necessary to fully understand the operation of the plant. This involved interviews with the operatives of the plant, the study of works drawings and observation of the day to day operations carried out.

**Crude Sewage Pumping Station**

On arrival at the WWTP wastewater is lifted through 6.4m by two screw pumps. As the plant is built on raised concrete columns, this lift enables the sewage to discharge from process to process. This arrangement facilitates the discharge of effluent to the Tay irrespective of the stage of the tide. Details are shown in Table K.1.

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<tr>
<th>Details</th>
<th>Small Screw</th>
<th>Large Screw</th>
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<tbody>
<tr>
<td>Maximum output (l/s)</td>
<td>258</td>
<td>855</td>
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<td>Lift (m)</td>
<td>6.83</td>
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<td>Speed (rpm)</td>
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<tr>
<td>HP (motor)</td>
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<td>125</td>
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<tr>
<td>Inclination (o)</td>
<td>38</td>
<td>38</td>
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<tr>
<td>Length (m)</td>
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<td>12.04</td>
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</tbody>
</table>

*Table K.1 Crude Sewage Inlet Pumping Station Details*
An overflow structure is present at the base of the screw pumps to enable flows, under emergency conditions (e.g. screw pump failure) to be discharged to the River Tay.

**Screening**

Crude sewage is screened by Vickerys step screens. A 3mm screen treats all sewage up to 3 DWF and if flows exceed this value they are treated by a 6mm screen. Screenings are removed from the flow and transferred automatically to a proprietary bagging unit. Flow into the screens is controlled by an automatic penstock which can be altered to allow different proportions of the flow to be passed through the 3mm and 6mm screens.

**Grit Separation**

After screening wastewater is treated by a DORR detritor which removes the inorganic particles of the flow (e.g. grit) while allowing the organic material to remain in suspension. The direction of rotation is anticlockwise at a speed of 0.08m/s. The tank is 7.93m in diameter and 2.13m in depth, giving a volume of 105 m³. The detritor has the ability to by-pass any incoming flows in case of emergency through a channel which rejoins with the main effluent flow below the detritor. The grit is removed by means of a mechanical rake which discharges into a skip located on the ground below the detritor unit. The grit is removed and dumped on waste land adjacent to the plant.

**Storm Sewage Settlement Tanks**

After passing through grit separation the flow enters an inlet flume. Excess flows are discharged to two storm tanks. The storm tanks provide 2hrs settlement during storm flows and Table K.2 presents details.

<table>
<thead>
<tr>
<th>Diameter (m)</th>
<th>Side water depth (m)</th>
<th>Floor Slope (o)</th>
<th>Volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>27.43</td>
<td>2.44</td>
<td>7.5</td>
<td>1795</td>
</tr>
</tbody>
</table>

Table K.2 Storm Tank Details
Appendix K

The tank floors are covered in granolithic concrete and scraper mechanisms sweep settled sludge into collecting hoppers. If the storm is of sufficient magnitude to fill the tanks settled sewage is discharged to the River Tay. If the storm is short and no overflow occurs any sludge and storm sewage is returned to the head of the works through manual operation of tank emptying valves.

**Primary Settlement Tanks**

The primary tanks are identical to the storm tanks. The storm tanks are fitted with single circumferencial weirs whereas the primary tanks are fitted with suspended double circumferencial weirs. Settled effluent is collected and conveyed to a flow dividing unit before the aeration process. Primary sludge is automatically desludged and is transferred to two sludge thickening tanks.

**Activated Sludge Plant**

After flow division settled sewage passes into activated sludge aeration units. Aeration is effected by nine surface aerators arranged in three rows of three. There are nine aeration compartments and data are shown in Table K.3.

<table>
<thead>
<tr>
<th>Compartment Capacity (m$^3$)</th>
<th>Line Capacity (m$^3$)</th>
<th>DWF retention (hrs)</th>
<th>Length of outlet weir (m)</th>
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</thead>
<tbody>
<tr>
<td>400</td>
<td>1200</td>
<td>4</td>
<td>9.75</td>
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</table>

Table K.3 Aeration Unit Details

**Final Settlement Tanks**

Data is shown in Table K.4 for the three final settling tanks.

<table>
<thead>
<tr>
<th>Diameter (m)</th>
<th>Side water depth (m)</th>
<th>Volume (m$^3$)</th>
<th>DWF retention (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>27.43</td>
<td>2.13</td>
<td>1984</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Table K.4 Final Settlement Tank Details

Three final settlement tanks are present and each receives discharge from an aeration lane. The tanks are fitted with white tiles around the discharge weirs to enable the quality of the final effluent to be assessed visually. The final effluent from the clarifiers is discharged to the River Tay via a flap valved outfall.
Return Activated Sludge (RAS)

RAS is lifted by three screw pumps and is conveyed to a connection point prior to the aeration units, but after flow separation. Waste Activated Sludge (WAS) is controlled by weir penstocks set into the sides of the RAS channels prior to the screw pumps. WAS is pumped to the head of the plant prior to the primary tanks by two 25 l/s centrifugal pumps operating against a head of 7.4m. Surplus sludge is returned every hour on average. The pumps commence operation when a predefined level is reached in the surplus well.

Tank Emptying

Valves connected into the draw off pipes from the sludge hoppers in the primary and final settlement tanks allow emptying of these tanks for maintenance purposes into a 7m x 2.74m x 6.1m sump. A similar setup exists for partial drawing down of the aeration units. All drainage from the plant, above ground and below ground, is conveyed to this sump. All wastewater contained within this sump is returned to the inlet flume prior to the primary settlement tanks by two vertical spindle centrifugal pumps.

Sludge Production And Disposal

All co-settled sludge produced from the primary tanks is fed into two thickening tanks situated at the rear of the plant. The sludge is thickened and drawn off for disposal to agricultural land.
Appendix L
Development of Flooding Regression Model

Selection Of Regression Model

It is important when carrying out any type of regression that the best regression model that applies to the data is chosen. This generally means selecting a subset of regressors from a set that quite likely contains all important variables. This is done by examining all possible regressions for the subset of variables chosen. For a system which has K candidate regressors, there are $2^K$ regression equations. Normally the $R^2$ value for the regression equation is used to compare and evaluate the possible range of equations.

This type of approach allows the analyst to see the value contributed to the equation by the addition or omission of a regressor to the model. For the purposes of flooding analysis the set of variables to be examined includes, Urban Catchment Wetness Index (UCWI), storm duration (minutes), storm peak intensity (mm/hr) and storm volume (mm). Table L.1 shows the results of assessing, $2^4 = 16$ possible regression equations and includes data relative to the adequacy of each of the regression models. Figure L.1 shows a plot of $R^2$ against $P (= K+1$, where $K= number of regressors).

![R^2 versus P](image)

Figure L.1 $R^2$ Versus P For Flooding Model Assessment

From Table L.1 and Figure L.1 it can be seen that the model involving peak intensity, duration and volume has a reasonably high $R^2$ value and a low Cp. The Cp value gives an indication of the total mean square error for the regression...
Table L.1 Regression Fits and Data to Establish Best Regression Model For Flooding Analysis

<table>
<thead>
<tr>
<th>Number of Variables</th>
<th>P</th>
<th>Variables in Model</th>
<th>$R^2$</th>
<th>SSR</th>
<th>SSE</th>
<th>MSE</th>
<th>Cp</th>
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Duration = x2
Intensity = x3
Volume = x4
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</tbody>
</table>
model. This model also has the lowest MS_T. The analysis shows that there is no real benefit in including the UCWI term into the regression analysis. Therefore the best regression model for the purposes of the analysis is to be found by using the three variables defined.

The above analysis was carried out using flooding data generated from the entire sewerage system relative to thirty storms run through the hydroworks model of Perth. The regression equation produced using the three variables; duration(mins), peak intensity (mm/hr) and volume (mm) is shown in equation L.1.

\[
\text{Flooding (m}^3\text{)=32.298(volume)+34.661(intensity)}-0.478(\text{duration})-312.599
\]

Equation L.1-Regression Model For Flooding Analysis

Test For Significance Of Regression

It is important to test any regression model for significance of regression. This is a test to determine if a linear relationship exists between the response variable (flooding) and the subset of regressors (duration, peak intensity and volume). This can easily be done by examining the F statistic generated in the analysis of variance shown in Table L.3. From the table it can be noted that \(f_0=103.71\). As we are testing at \(\alpha=0.05\) with degrees of freedom for regression =3 and degrees of freedom for the residual = 26 the appropriate test statistic is \(f_0^* > f_{0.05,3,26} = 2.98\). From this we can conclude that flooding is linearly related to duration, peak intensity or volume or all three. Further tests can be carried out on the individual regressors.

<table>
<thead>
<tr>
<th>Degrees of Freedom</th>
<th>SS</th>
<th>MS</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>3</td>
<td>3373632.87</td>
<td>1124544.29</td>
</tr>
<tr>
<td>Residual</td>
<td>26</td>
<td>281909.3705</td>
<td>10842.66808</td>
</tr>
<tr>
<td>Total</td>
<td>29</td>
<td>3655542.42</td>
<td></td>
</tr>
</tbody>
</table>

Table L.3 Analysis Of Variance For Regression Model
Tests on Individual Regression Coefficients

Tests on individual regression coefficients are useful in determining the potential value of each of the regressor values within the regression model. This indicates whether the inclusion of the regressor variable contributes significantly to the model. This is carried out by comparing the generated t values in the analysis of variance table with test values at the appropriate level of significance and degrees of freedom. P values are also used for comparison against $\alpha=0.05$. Relevant data from the regression analysis are shown in Table L.4.

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Standard Error</th>
<th>t-stat</th>
<th>P value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>-312.599</td>
<td>74.457</td>
<td>-4.198</td>
</tr>
<tr>
<td>Volume</td>
<td>32.297</td>
<td>3.339</td>
<td>9.670</td>
</tr>
<tr>
<td>Peak intensity</td>
<td>34.660</td>
<td>9.418</td>
<td>3.680</td>
</tr>
<tr>
<td>Duration</td>
<td>-0.478</td>
<td>0.130</td>
<td>-3.666</td>
</tr>
</tbody>
</table>

Table L.4 T And P Values For Regressors In Flooding Analysis

As t values in the table are greater than that of the test statistic $t_{0.025, \infty}=2.056$, it can be concluded that each of the regressors contributes significantly towards the model. P values for the regressors are smaller than $\alpha$. This again, indicates that the regressors contribute to the model.

Residual Analysis

Residual analysis is used in multiple regression to judge model adequacy. Residual plots are shown in standardised form against duration, volume and peak intensity in Figures L.2, L.3, L.4. The residual plots appear to be adequate with 95% of the standardised residuals falling in the interval (-2, +2).

Multicollinearity

Multicollinearity exists in multiple regression analysis where the regressors have dependencies amongst themselves. In situations where dependencies are strong multicollinearity is defined to exist. Muticollinearity arises for several reasons. It will occur when data is collected which have a strong linear constraint amongst
Figure L.2 Standardised Residuals v Duration
Figure L.3 Standardised Residual v Peak Intensity
Figure L.4 Standardised Residuals v Volume

![Diagram showing standardised residuals versus volume with data points scattered across the plot.](image-url)
them. For example, if three regressor variables are the components of a mixture, then a constraint will exist because the sum of the components is constant.

For the flooding analysis, regression model plots are shown in Figures L.5, L.6 and L.7, relating the regressor variables, duration, peak intensity and volume against one another. From the plots it is appears that no significant relationship exists between the regressor variables. Further analysis was carried out by regressing storm volume against storm duration and storm peak intensity, storm peak intensity against storm volume and storm duration and storm duration against storm peak intensity and storm volume. A summary is presented showing $R^2$ values in Table L.5.

<table>
<thead>
<tr>
<th>Dependent</th>
<th>Independent</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume</td>
<td>Storm Duration</td>
<td>0.488</td>
</tr>
<tr>
<td>Peak Intensity</td>
<td>Storm Volume</td>
<td>0.251</td>
</tr>
<tr>
<td>Peak Intensity</td>
<td>Storm Duration</td>
<td>0.003</td>
</tr>
<tr>
<td>Volume</td>
<td>Peak Intensity, Duration</td>
<td>0.7</td>
</tr>
<tr>
<td>Peak Intensity</td>
<td>Duration, Volume</td>
<td>0.42</td>
</tr>
<tr>
<td>Duration</td>
<td>Peak Intensity, Volume</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Table L.5 $R^2$ Values For Relationships Between Regressors

It is recognised (Montgomery and Runger 1994) that if the F test for significance of regression is significant, but tests on the individual regression coefficients are not significant, then multicollinearity may be present. In the case of the regression model proposed for the flooding analysis, both the above tests proved significant and thus it is likely that multicollinearity does not exist between the regressor values.

Conclusions

The regression model developed to predict flooding from a historical series of rainfall (20 years) performs adequately when three regressors are used. These being, storm peak intensity (mm/hr), duration (minutes) and volume (mm). The model has a relatively high $R^2$ value and low Cp value. Tests for significance of regression show that a linear relationship exists between the regressors and the dependent variable. Individual tests on the coefficients show that each regressor
Figure L.5 Storm Volume v Storm Duration
Figure L.6 Storm Peak Intensity v Storm volume

Peak Intensity (mm/hr) vs. Storm Volume (mm)
Figure L.7 Storm Peak Intensity vs Storm Duration
contributes significantly to the regression model. Because both tests prove significant and from examination of regressor values plotted against each other it is unlikely that multicollinearity between the regressors exists.

The regression model predicts volumes of flooding accurately when this attribute is relatively high. When values for flooding are relatively low the regression model tends to overpredict. When values for flooding are extremely low (unlikely to occur) then the regression model predicts negative values. This suggests that the model is a reasonable representation of the actual performance of the sewerage system based on data provided from the thirty test storms. For the purpose of the WISPS methodology, the attribute for the area of concern of flooding, is frequency of occurrence. It is more important that the regression model predicts the occurrence of flooding than the definitive volume associated with a flooding event. The regression model can be used to assess the flooding performance of the sewerage system with respect to the 20 years of historical rainfall.

A similar approach was utilised to attempt to derive regression models for each of the sub-catchments in Perth suffering from flooding. However, on analysis deriving regression models for each area were not successful. Primarily due to the fact, when flooding volumes were split between areas it was apparent that certain subcatchments suffered from flooding more than others. This had the effect of leaving little flooding data, for subcatchments suffering slight flooding, on which to base regression. It was therefore decided to adopt the regression model of the whole system for predicting the frequency of flooding events for use in the WISPS methodology.
Appendix M
Sensitivity Analysis Of WISPS

Selection Of Sample

The sample for evaluating weightings for the areas of concern was selected at random from a group of engineers within NoSWA. All respondents were to an extent informed experts regarding the areas of concern to be evaluated. All hold, or had held varying levels of responsibility regarding decisions specific to the performance of wastewater systems. It is clear that selecting a group of informed persons who are aware of the issues to be evaluated will lead to bias regarding the relative importance of one area of concern against another. It is recognised that a sample of ten is extremely small and that future work should try to encompass as many respondents as resources and time allow.

Review of Weightings

The graphs shown in Figures 5.5 to 5.13 in Chapter 5, show the weightings attributed by the respondents regarding each of area of concern. It is important to carry out a review of the weightings and assess the degree of variability within the respondents replies. Descriptive statistics are presented in Table M.1 for the weightings. Figures in brackets under standard error show what percentage the standard error is of the mean weighting value.

<table>
<thead>
<tr>
<th>Weighting</th>
<th>Sample Mean</th>
<th>Sample Variance</th>
<th>Sample Std Deviation</th>
<th>Standard Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flooding</td>
<td>0.138</td>
<td>0.00040</td>
<td>0.020</td>
<td>0.006337 (4.5%)</td>
</tr>
<tr>
<td>Odour</td>
<td>0.08</td>
<td>0.001302</td>
<td>0.036</td>
<td>0.012 (15%)</td>
</tr>
<tr>
<td>WWTP</td>
<td>0.132</td>
<td>0.00042</td>
<td>0.02059</td>
<td>0.0065 (4.9%)</td>
</tr>
<tr>
<td>Structural</td>
<td>0.140</td>
<td>0.000294</td>
<td>0.017</td>
<td>0.005423 (3.87%)</td>
</tr>
<tr>
<td>CSO discharge</td>
<td>0.110</td>
<td>0.000856</td>
<td>0.02925</td>
<td>0.009252 (8.4%)</td>
</tr>
<tr>
<td>Aesthetic</td>
<td>0.109</td>
<td>0.000815</td>
<td>0.028</td>
<td>0.009 (8.2%)</td>
</tr>
<tr>
<td>Sedimentation</td>
<td>0.095</td>
<td>0.000339</td>
<td>0.0184</td>
<td>0.00614 (6.4%)</td>
</tr>
<tr>
<td>Infiltration</td>
<td>0.094</td>
<td>0.00047</td>
<td>0.02168</td>
<td>0.006856 (7.2%)</td>
</tr>
<tr>
<td>Water Quality</td>
<td>0.101</td>
<td>0.00143</td>
<td>0.0378</td>
<td>0.011964 (11.8%)</td>
</tr>
</tbody>
</table>

Table M.1 Descriptive Statistics For Weightings Used In WISPS

Methodology
It can be noted that the weightings with lower standard deviations are those associated with more objective areas of concern these being; WWTP compliance with consents, frequency of flooding and structural performance. Weightings with higher standard deviations are associated with more subjective criteria such as; odour, CSO discharge and receiving water course quality. The last two criteria perhaps produce variable weightings due to the mainly engineering background of the respondents who are more concerned with traditional performance in areas like flooding and structural aspects, rather than in the holistic performance of the wastewater system.

Members of the public would probably disagree with odour from the WWTP being at the bottom of the importance list. However, it is encouraging to note that the three key areas of flooding, structural and WWTP performance are ranked as the leading criteria. These three criteria are generally considered to be upper most in the minds of drainage engineers when considering catchment performance. It is interesting to note that the weightings developed are quite closely related. No one weighting has large dominance over the others, effectively ensuring that each area of concern plays an important part in developing a WISPS score for the catchment in question.

Confidence Intervals

It is important to establish confidence intervals regarding the mean values used in the WISPS analysis for the weightings. This is normally carried out at a confidence level of 95%. As the sample mean and standard deviation are known, but the sample size <30 confidence limits are based on the t distribution. This assumes that the population is normally distributed. The relevant t statistic is $t_{0.025,9}=2.262$. Confidence limits at the 95% levels are shown in table M.2 for the weightings.
Appendix M

<table>
<thead>
<tr>
<th>Weighting</th>
<th>Sample Mean</th>
<th>Sample Std Deviation</th>
<th>Lower 95%</th>
<th>Upper 95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flooding</td>
<td>0.138</td>
<td>0.020</td>
<td>0.124</td>
<td>0.152</td>
</tr>
<tr>
<td>Odour</td>
<td>0.08</td>
<td>0.036</td>
<td>0.054</td>
<td>0.105</td>
</tr>
<tr>
<td>WWTP</td>
<td>0.132</td>
<td>0.02059</td>
<td>0.117</td>
<td>0.146</td>
</tr>
<tr>
<td>Structural</td>
<td>0.140</td>
<td>0.017</td>
<td>0.128</td>
<td>0.152</td>
</tr>
<tr>
<td>CSO discharge</td>
<td>0.110</td>
<td>0.02925</td>
<td>0.089</td>
<td>0.130</td>
</tr>
<tr>
<td>Aesthetic</td>
<td>0.109</td>
<td>0.028</td>
<td>0.088</td>
<td>0.129</td>
</tr>
<tr>
<td>Sedimentation</td>
<td>0.095</td>
<td>0.0184</td>
<td>0.082</td>
<td>0.108</td>
</tr>
<tr>
<td>Infiltration</td>
<td>0.094</td>
<td>0.02168</td>
<td>0.078</td>
<td>0.109</td>
</tr>
<tr>
<td>Water Quality</td>
<td>0.101</td>
<td>0.0378</td>
<td>0.073</td>
<td>0.128</td>
</tr>
</tbody>
</table>

Table M.2 Confidence Limits For Weightings Used In WISPS Methodology

It is important to assess the impact on the WISPS score in relation to the possible range of values for weightings used in the methodology via a sensitivity check.

**Sensitivity Check-Weightings**

Sensitivity analysis was carried out by varying the value of weightings and performance scores from the value functions and examining the corresponding percentage change in WISPS score. A WISPS score of 47.5, developed from the performance of a test catchment is shown in Table M.3.

<table>
<thead>
<tr>
<th>Area of Concern</th>
<th>Weighting</th>
<th>Performance Score</th>
<th>WISPS Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural</td>
<td>0.140</td>
<td>50</td>
<td>7</td>
</tr>
<tr>
<td>Flooding</td>
<td>0.138</td>
<td>50</td>
<td>6.9</td>
</tr>
<tr>
<td>Water Quality</td>
<td>0.101</td>
<td>65</td>
<td>6.57</td>
</tr>
<tr>
<td>WWTP compliance</td>
<td>0.132</td>
<td>65</td>
<td>8.58</td>
</tr>
<tr>
<td>Aesthetic</td>
<td>0.109</td>
<td>50</td>
<td>5.45</td>
</tr>
<tr>
<td>CSO Spill</td>
<td>0.110</td>
<td>30</td>
<td>3.30</td>
</tr>
<tr>
<td>Infiltration</td>
<td>0.094</td>
<td>20</td>
<td>1.88</td>
</tr>
<tr>
<td>Sedimentation</td>
<td>0.095</td>
<td>40</td>
<td>3.8</td>
</tr>
<tr>
<td>Odour</td>
<td>0.080</td>
<td>50</td>
<td>4</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>47.5</strong></td>
</tr>
</tbody>
</table>

Table M.3 WISPS Score For Test Catchment Used In Sensitivity Tests
Weightings were changed in the range +50% to -50% of the mean value, which examines beyond the upper and lower 95% confidence values for the weightings shown in Table M.2. Resulting changes in WISPS scores varied between +10% to -10% of the original figure 47.5. This variation is not significant as the same weightings and value functions, once developed, are applied to all catchments under consideration. Therefore the relative change in catchment WISPS scores would stay constant. In relation to the test catchments of Coupar Angus, Almondbank, Perth and Forfar presented in Chapter 5 the difference in WISPS scores would remain the same.

Large changes in the weightings resulted in relatively small changes in the WISPS scores. A summary table and Spider graph is shown in Figure M.1. It is apparent that if an area of concern with a relatively high weighting has the weighting changed significantly, then the percentage change in WISPS will be larger, if that area of concern has a high performance score. This can be seen in the areas of concern associated with water quality and WWTP compliance. Both these areas of concern have relatively high weightings, and performance scores of 65. When weightings are changed, resulting WISPS scores are more affected, compared to other areas of concern, with smaller performance scores, undergoing similar changes in weighting. This is shown in Figure M.1.

It is concluded that large changes in the weightings do not result in large changes in the WISPS score. However, future work should attempt to evaluate weightings from a larger sample and from members of the public. This will allow a “tightening up” of the weightings to be achieved, and an insight into the priorities of others affected by wastewater system performance to be evaluated.

**Sensitivity Check-Value Functions**

The value functions developed were based on responses from a number of respondents, manuals of good of practice and general discussions with engineers. It is important to derive accurate performance figures for the area of concern, i.e. number of spills or number of flooding events. This ultimately affects the performance score attributed to it via the value function. It is likely that more
Figure M.1 - Sensitivity Analysis - Weighting and Performance Scores

<table>
<thead>
<tr>
<th>Weighting Change</th>
<th>Flooding</th>
<th>Structural</th>
<th>WWTP</th>
<th>Water Quality</th>
<th>Aesthetic</th>
<th>CSO</th>
<th>Infiltration</th>
<th>Sedimentation</th>
<th>Odour</th>
</tr>
</thead>
<tbody>
<tr>
<td>-50%</td>
<td>-7.27%</td>
<td>-7.37%</td>
<td>-9.04%</td>
<td>-6.91%</td>
<td>-5.74%</td>
<td>-3.48%</td>
<td></td>
<td></td>
<td>-1.98%</td>
</tr>
<tr>
<td>-40%</td>
<td>-5.81%</td>
<td>-5.90%</td>
<td>-7.23%</td>
<td>-5.53%</td>
<td>-4.59%</td>
<td>-2.78%</td>
<td></td>
<td></td>
<td>-1.58%</td>
</tr>
<tr>
<td>-30%</td>
<td>-4.36%</td>
<td>-4.42%</td>
<td>-5.42%</td>
<td>-4.15%</td>
<td>-3.44%</td>
<td>-2.09%</td>
<td></td>
<td></td>
<td>-1.19%</td>
</tr>
<tr>
<td>-10%</td>
<td>-1.45%</td>
<td>-1.47%</td>
<td>-1.81%</td>
<td>-1.38%</td>
<td>-1.15%</td>
<td>-0.70%</td>
<td></td>
<td></td>
<td>-0.40%</td>
</tr>
<tr>
<td>10%</td>
<td>1.45%</td>
<td>1.47%</td>
<td>1.81%</td>
<td>1.38%</td>
<td>1.15%</td>
<td>0.70%</td>
<td></td>
<td></td>
<td>0.40%</td>
</tr>
<tr>
<td>30%</td>
<td>4.36%</td>
<td>4.42%</td>
<td>5.42%</td>
<td>4.15%</td>
<td>3.44%</td>
<td>2.09%</td>
<td></td>
<td></td>
<td>1.19%</td>
</tr>
<tr>
<td>40%</td>
<td>5.81%</td>
<td>5.90%</td>
<td>7.23%</td>
<td>5.53%</td>
<td>4.59%</td>
<td>2.78%</td>
<td></td>
<td></td>
<td>1.58%</td>
</tr>
<tr>
<td>50%</td>
<td>7.27%</td>
<td>7.37%</td>
<td>9.04%</td>
<td>6.91%</td>
<td>5.74%</td>
<td>3.48%</td>
<td></td>
<td></td>
<td>1.98%</td>
</tr>
</tbody>
</table>

Spider Diagram Showing Sensitivity of WSPS to Changes in Weightings and Performance Scores
confidence can be placed in determining the actual performance of an area of concern, than in determining the weighting to be attributed to it.

Performance scores for the test catchment were varied, one at a time, again in the range +/-50% of the test value. This produced only small changes in the WISPS score. Percentage change in WISPS score derived by varying performance scores are exactly the same, as when weightings for the areas of concern are adjusted by the same percentage amounts. Essentially, this is due to using a weighted average equation to develop the WISPS score, i.e., the WISPS score is adjusted by the same percentage, when either performance score or weighting for an area of concern is changed by the same amount. The table and graph presented in Figure M.1 hold not only as a sensitivity test for weightings but also for the performance scores derived from the value functions.

**Conclusions on Sensitivity Testing**

From the sensitivity analysis carried out it is clear that no one area of concern dominates the WISPS score. The highest ranked areas of concern, are generally accepted as being the fundamentally important areas of wastewater system performance. The WISPS methodology reflects this through weightings attributed to each area of concern. Weightings derived have been from a very small sample. More data requires to be collected from a larger (>30) sample and from affected parties not just concerned with engineering.

Sensitivity tests have shown that large changes in weightings and performance scores do not result in large changes in the WISPS scores. This is due to the use of a simple weighted average equation (5.1) to derive the combined holistic score for the catchment. The robustness of the model could be more rigorously assessed utilising simulation techniques and examining all possible combinations of scores and weightings. This would be better carried out in conjunction with the application of the technique to additional catchments and therefore has not been carried out at present.
The WISPS methodology is essentially a comparative tool to be used by engineers examining the performance of catchments under their control. Therefore, absolute values for weightings, and performance scores from value functions, are not fundamental to the success of the method. What is important, is that areas of concern are appropriately ranked and weighted in order of priority, and that the value functions describe adequately the change in preference from one level of performance to another.

It is believed that the weightings and value functions derived are reasonable estimates and descriptions of the order of priorities afforded by engineers to wastewater system performance.