Hydroinformatics Tools
for Planning, Design, Operation and Rehabilitation of Sewer Systems

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PREFACE

Urban streams and lakes, receiving numerous discharges of stormwater, combined sewer overflows and wastewater treatment plant effluents, represent some of the most environmentally degraded water bodies, and as such, receive high priority in the planning and implementation of environmental protection and remediation. While this is generally true in all countries, this problem is particularly pressing in the (NATO) Cooperation Partner countries, which have intensified their environmental remediation in recent years, in order to eliminate the threats to human health, economy and the entire ecosystem, caused by poor environmental conditions. Recognising high costs of such remediation and the need to ensure high returns on these investments, it is important to address the environmental protection and remediation of urban water resources in an effective and integrated way, and to seek optimal solutions. The extent and complexity of such considerations call for the development and use of modern approaches, such as those advocated in the field of hydroinformatics

The idea of hydroinformatics, which merges environmental modelling and information technology (IT), has attracted much attention in recent years, as obvious from many recent publications and meetings on this subject, including two NATO research workshops. This interest was also noticed in urban drainage and documented by special hydroinformatics sessions held at the triennial IAHR/IAWQ Urban Storm Drainage conferences in 1993 and 1996. These sessions also indicated some progress in applications of hydroinformatics to urban drainage planning and design, particularly with reference to the introduction of such tools as simulation models for drainage systems, real-time control models, GIS (Geographic Information System), and databases.

Recognising the urgent need for environmental remediation and the opportunities created by the development of various hydroinformatics tools in this field, the colleagues from the Czech Technical University in Prague suggested to hold a NATO Advanced Study Institute (ASI) on the use of hydroinformatics in urban drainage. The main objective of this Institute was to disseminate knowledge on practical use of hydroinformatics in the planning, design, operation and rehabilitation of urban drainage systems comprising the urban catchment, sewer system, wastewater treatment plants, and the receiving waters. Equally important were other objectives of this event - to foster professional contacts among the urban drainage professionals from both NATO and Cooperation Partner countries and to share participants' experience in this field.

After establishing the Institute's Organising Committee, recruiting lecturers, and receiving a NATO grant for this ASI, a detailed Institute's program was developed and presented in this course, which was attended by more than 80 participants from 20 countries. Only the formal lectures of the ASI are reflected in the proceedings that follow. Besides these lectures, there were other ways of sharing and exchanging information among the participants, including computer-based tutorials and evening ad hoc sessions, giving opportunities to "student" participants, who were all accomplished professionals in urban drainage or related fields, to present their experiences or points of view. Any use of trade, product, or firm names in this book is for descriptive purposes and does not imply endorsement by the Editors, Authors or NATO.

Besides technical findings and compilation of course notes, many other accomplishments have been achieved at this ASI, in the form of new collaborative links, professional networks, and personal friendships. Overall, spending two weeks in the Czech mountain resort town of Harrachov, while enjoying local hospitality and genuine friendship among all participants, and learning new concepts of hydroinformatics, made this ASI a memorable event. These sentiments were also confirmed by the results of an evaluation questionnaire distributed at the ASI. Even the weather cooperated; it would have been almost embarrassing to discuss urban drainage and wet-weather problems without some first-hand exposure to these phenomena, which we did get abundantly, it rained every day of the Institute. Finally, we should thank to all who helped stage this Institute, and particularly those listed in the Acknowledgements.

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The compilation of the proceedings typescript was done by Gillian Larkin, a contract employee of NWRI. All local arrangements were done by the local organising committee, headed by Evzen Zeman, with assistance from Eliska Poupova, Karolina Hazova (all from the Czech Technical University, Prague) and Zorica Todorovic (University of Belgrade). Special thanks are due to all the above contributors, and above all, to all the participants, who made this Institute a memorable and interactive learning experience for all involved.

PROCEEDINGS SUMMARY

The ASI proceedings are organised into eight chapters, each of which contains one to seven papers. A brief summary of individual chapters and papers follows.

In Chapter 1, *Introduction*, J. Marsalek introduces the *Challenges in urban drainage: environmental impacts, impact mitigation, methods of analysis and institutional issues.* Drainage impacts on receiving waters are caused by physical factors, and by the discharge of chemicals and microbiological organisms, and can be mitigated by stormwater management, and CSO control and treatment. The planning and design of such modern drainage systems requires a supportive institutional framework, and is best accomplished by means of hydroinformatics.

R. Price discusses *Business needs of urban drainage, in the context of information management* and notes that the business of draining wastewater from urban areas is increasingly information intensive. As such, the way in which information is managed is critical to the efficiency and effectiveness of the water industry. The needs of the privatised water industry in the UK are compared with those of the non-privatised industry in the USA. The key factors affecting the management of sewerage are then considered from an information management perspective.

Hydroinformatics concepts are introduced in Chapter 2, by R. Price, K.P. Holz and K. Ahmad, noting that hydroinformatics is a new topic, in which the various strands contributing to hydroinformatics are considered and interpreted in terms of the application of information technology to the management of water. Of particular concern are simulation modelling, artificial intelligence and access to a range of textual and other sign information.

Chapter 3 deals with *Urban Environmental Models*. With reference to *Model development and application*, L. Fuchs, C. Maksimovic, R.K. Price and W. Schilling describe the framework for an information system supporting the management of a water-based asset. This information system contains tools for urban drainage modelling as well as monitoring and survey systems, databases, GIS, CAD computer hardware and so on. The modelling tools are described in more detail, including the steps needed to set up a model for urban drainage. The factors considered in this process include the modelling concepts, modelling software, data, validation, calibration and verification and model uncertainties. Finally, the roles played in this process by the model developers and model users, and their interactions, are presented.

C. Maksimovic describes *Fundamentals of physically-based rainfall / runoff models*, with reference to their application. Examples of model formulations include infiltration modelling using the Green-Ampt equation and selecting overland flow solutions on the basis of the Froude number and some dimensionless geometric parameters. Such models can be calibrated with data from the existing data banks.

C. Maksimovic further described *Sensitivity analysis of physically-based rainfall / runoff models* and demonstrated such a procedure on a hypothetical catchment. The model presented (BEMUS) is used for training engineers to apply physically based models.

Model Input Data Needs and Management are discussed in Chapter 4, starting with T. Einfalt, V. Krejci and W. Schilling's discussion of *Rainfall data in urban* *hydrology*. Rainfall measurements are particularly important for hydrologic modelling and can be collected by different instruments with specific characteristics and sources of error. Therefore, a rigorous scrutiny and analysis of measured data are required. Rainfall containing large uncertainties may impact severely on modelling results.

E. Zeman, S. Vanecek and P. Ingeduld discuss the *Tools for data archiving, visualisation and analysis, applied in master drainage planning*. This process makes good use of hydroinformatics by an effective interfacing of simulation tools, retrieval of data from databases or GIS, obtaining information from monitoring systems, and evaluating, searching for and presenting the information required by decision-makers. The modelling tools are well established and have been tested world-wide. Problems caused by low accuracy, poor consistency and/or lack of data are notorious. Comprehensive applications of the aforementioned tools should accelerate the process of better planning of sewer systems.

Application of GIS in urban drainage are discussed by C. Maksimovic, including the specific problems of matching storm drainage simulation models with data sources and commercial GIS packages. Detailed descriptions of data needs and the development of interfaces needed are given for several catchments.

Chapter 5 is devoted to *Modelling Urban Drainage System Components*. In the first paper, L. Fuchs describes the *Hydrologic modelling of urban catchments*. The presentation of theoretical concepts starts with hydrologic abstractions, including the initial abstraction, infiltration and evaporation. In a general lumped approach, overland flow hydrographs can be modelled by physically based models, unit hydrograph, or reservoir models. Finally, some major urban hydrology models, which are widely used in Europe, are compared with respect to the principal hydrologic concepts.

V. Havlik reviewed *Modelling sewer hydraulics*. The governing equations for unsteady open-channel flow in sewers and surcharge flow models are discussed, including the associated numerical solutions. Special attention is paid to initial and boundary conditions. Numerical solutions are presented for a combined sewer overflow side weir and compared to the measurements obtained in physical models and in the field.

J. Delleur describes *Modelling quality of urban runoff*, presenting first some runoff quality data collected under the U.S. Nationwide Urban Runoff Program. A brief overview of mathematical descriptions of basic water quality processes focuses on the one-dimensional advective-diffusion equation, first and second order decay processes and the Streeter-Phelps equation. A description of runoff quality modelling in the U.S. EPA Storm Water Management Model (SWMM) is presented, including the pollutant buildup and washoff. Other approaches to pollutant accumulation and washoff are also mentioned. Criteria and expert systems for model calibration and a derived probability distribution approach are discussed.

Water quality modelling in sewer networks is reviewed by J. Delleur, starting with the basic equations for flow and pollutant routing, as well as for sediment scouring and deposition in sewers. Contaminant routing can be also simulated by the TRANSPORT block of the SWMM, which uses a kinematic wave approximation for flow routing; however, the EXTRAN module of SWMM, that uses the full unsteady flow equations, does not perform contaminant routing. An adaptation for quality routing that is not part of the original model is given. Attention is given to the current research on non-cohesive and cohesive sediment transport in sewers. The MOUSE model is presented briefly. A stochastic transfer function approach is presented to estimate the suspended solids concentrations, given the flow rate and water temperature.

Optimisation models for urban runoff control planning are discussed by J. Li and B. Adams. For planning stage evaluations of runoff control alternatives, efficient screening and optimisation models are needed. For this purpose, analytical probabilistic models have been developed and used to estimate the average annual percent of runoff volume controlled and the average annual number of overflows from a storage-treatment system in an urban catchment. An equation of control performance is thus established between the statistics of rainfall characteristics, catchment parameters, and the control system storage and treatment rate. Using a constrained cost optimisation model, the least-cost combination of storage and treatment systems which can achieve a particular level of control is obtained.

Modelling wastewater treatment plants is described by R. Szetela, who states that the existing models are capable of simulating all the major processes occurring in municipal wastewater treatment plants. The dynamic nature of these models accounts for unsteady conditions which occur in plants as a result of continually changing hydraulic and pollutant loads. Design of new plants, upgrading as well as analysis of alternative operational strategies and control of existing wastewater treatment plants are facilitated and greatly enhanced by computer simulation. Many potentially feasible solutions may be evaluated quickly and relatively inexpensively.

Modelling receiving waters, in the form of urban rivers, is discussed by P.Ingeduld and E. Zeman. River channels in urban areas usually contain various structures, which may have far reaching, and often adverse, effects on the surrounding areas. For example flood protection dykes and embankments built along the rivers in cities increase the magnitude (amplitudes, steepness) of flood waves. Diversion structures and weirs change the morphological equilibrium of the river. The environmental effects have to be considered when designing sewer or riverine hydraulic structures, because these structures control the behaviour of the drainage system within their domain. In most cases, the cause-effect relationships between the drainage structures and the receiving waters are so complex that only deterministic models can be applied to study the current state or design alternatives and their impact on the environment.

Operation and Rehabilitation of Sewer Systems are discussed in Chapter 6. First, J. Marsalek and W. Schilling address *Operation of sewer systems*. Urban drainage infrastructures represent large investments which have to be properly operated and maintained. The development of an operational plan starts with defining the system performance parameters with reference to discharges, the degree of treatment, and acceptable frequencies and durations of service interruptions. To meet and sustain this performance, day-to-day operation rules are developed as well as a maintenance plan. Maintenance is based on regular inspections, which are used to plan both corrective and preventive maintenance. Modern operation of sewer systems uses real-time control to minimise the drainage business risk, which is defined as the cost of system operation and damages.

Sewerage rehabilitation is discussed by J. Delleur. Three types of sewer failure or malfunction are recognised: 1) structural, 2) hydraulic and 3) environmental. Structural failures can be detected by visual inspection, closed circuit TV, and remote sensing methods. Hydraulic failures are usually related to insufficient discharge capacity and should be analysed by simulation models. Water quality violations in receiving streams, caused by discharges of stormwater and CSOs, are typical environmental failures. The principal rehabilitation methods are reviewed. A multi-attributed model that selects the segments to be rehabilitated and the rehabilitation methods to be used, under specified maximum construction costs, is presented. A chance constraint model and the British sewer rehabilitation program are summarised.

Chapter 7, by K. Ahmad and R. Price, focuses on *Safe Hydroinformatics*. Hydroinformatics systems should be applied with safety in mind. In particular, there is an urgent need for a safe environment in which users can reach reliable decisions. A decision support system is therefore needed to provide a suitable framework to encourage safety. Such a framework is developed for application of simulation modelling software within a hydroinformatics system.

Closing Chapter 8 deals with *Integrated Urban Water Management*, within an urban area and the whole river basin. *Integrated urban drainage management* is discussed by V. Krejci, P. Krebs and W. Schilling. Future planning and cost-effective operation of urban drainage systems can be accomplished only by adopting an integrated approach. Based on several case studies from Switzerland, urban drainage processes are described, the associated problems are identified, and the methods for selecting the site-specific, problem-targeted remedial measures are given. In an integrated approach, the urban area, sewer system, waste water treatment plant, the receiving waters and groundwater are considered as interconnected elements of one system. The integrated approach requires interdisciplinary cooperation, in order to find technical, economic and ecological optima in urban water management.

C. Maksimovic and Z. Todorovic considered *Urban drainage as a part of river basin management*. There are many interactions between the urban catchments and the river basins, in which those catchments are located. Consequently, an integrated approach to flood protection and pollution control is needed. Two types of urban floods (i.e. those caused by the receiving water course and those caused by local storms) are analysed in terms of both structural and non-structural management measures. For this purpose, new technologies and problem solving tools are introduced.

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CHALLENGES IN URBAN DRAINAGE

Environmental Impacts, Impact Mitigation, Methods of Analysis and Institutional Issues

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1. Introduction

Urbanisation leads to concentration of population in relatively small areas and this process causes many environmental impacts on water resources and increases the population demands on such services as water supply, flood protection, drainage, wastewater disposal, and water-based recreation. The drainage systems, which have evolved through history, represent compromises between ecological needs, human well-being, technical abilities and availability of resources. Only in recent years has the depletion and degradation of urban water resources led to the advocacy of a sustainable urban water system, characterised by lower water consumption, preservation of natural drainage, reduced generation of wastewater through water reuse and recycling, advanced water pollution control, and preservation and/or enhancement of the receiving water ecosystem.

Historically, the three main components of urban water systems, sewers, wastewater treatment plants (WTPs) and receiving waters, were addressed separately, and their interactions were to some extent disregarded. Such an approach cannot produce cost-effective solutions with reference to the protection of receiving waters and is untenable. Consequently, an integrated approach to water management, sometimes also referred to as an ecosystem approach, has evolved. In this approach, the interdependency and interactions among the principal system components are fully recognised and used in the development of solutions to water management problems. The beneficial use of the receiving waters, including natural functions, in-stream uses and withdrawals, is the driving force dictating the level of control of flows and pollutants conveyed by urban drainage, including wastewater treatment.

The primary purpose of this overview is to review the current state of art in urban drainage, with emphasis on drainage impacts on the receiving waters, means of impact mitigation, and implementation of management programs, all with reference to the main subject of this Advanced Study Institute, hydroinformatics.

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2. Drainage as Part of the Urban Water System

Urban drainage is provided for prevention of flooding, reduced inconvenience due to surface water ponding, alleviation of health hazards, and improved aesthetics, and has been traditionally based on a steady expansion of the drainage infrastructure without consideration of the impacts of drainage discharges on receiving waters or optimisation of the performance of the integrated urban water system.

Two types of urban drainage systems have evolved, combined and separate systems. In the combined system, both surface runoff and municipal wastewater are conveyed in a single pipe. Dry weather flow is transported to the sewage treatment plant and treated. In wet weather, as the runoff inflow into the combined sewers increases, the capacity of the collection system and of the treatment plant would be exceeded and the excess flows are allowed to escape into the receiving waters as combined sewer overflows (CSOs). In the separate sewer system, surface runoff is transported by storm sewers and discharged into the receiving waters, and municipal wastewaters are transported by sanitary sewers to the sewage treatment plant and treated prior to discharge into the receiving waters. Both drainage systems exist in many variations.

2.1. COMBINED SEWER SYSTEMS

Large populations are served by combined sewers and current policies are to maintain and upgrade the existing combined systems, quite often in the context of municipality-wide integrated pollution control. This generally involves control of the CSO discharges to provide effective treatment and minimise the pollution impact on receiving waters.

Spills of combined sewage occur at the CSO structures, whose primary functions are to provide hydraulic control on the system, restrict the flow to the downstream sewers to a design value (setting), prevent flooding by providing a relief overflow, generally to the nearest watercourse, and retain sewage pollutants for treatment. Common design of such structures include leaping and side weirs, stilling ponds, and vortex and swirl concentrator separators (Marsalek *et al.* 1993). Field studies of the pollution retention performance of CSO structures highlight the difficulties associated with the collection of representative data of good quality. Total efficiency values for different chamber geometries indicate that for total suspended solids, BOD, COD and ammonia, the efficiency was, at best, only slightly better than the value of flow split. Often large proportions of first flush pollutants were spilled from the CSO structure (Marsalek *et al.* 1993).

The pollutants discharged from CSOs include gross solids, coarse and suspended solids with attached pollutants or bacteria, and dissolved pollutants. Sewer solids (sediments) cause numerous problems in drainage operation, including loss of hydraulic capacity, concentration and transport of pollutants, a source of septicity usually accompanied by gas and corrosive acidity production, and a risk of washout into the receiving waters or overloading at the wastewater treatment plant (Ashley and Verbanck

1996). Furthermore, many treatment or control options applied to CSOs result in sediment settling and accumulation (e.g., in CSO tanks) and the settled sediments require proper removal and disposal.

Analysis and management of sewer sediment problems is complex, because of the heterogeneous nature of such sediments, which are sometimes classified into a number of categories characterised by widely varying properties. Each category may cause different types of problems; while the bulk of sediment appears to be grit-like particles readily depositing in sewers and interfering with their hydraulic capacity, smaller particles with high organic content are efficient vectors for transport of trace organic contaminants (Ashley and Verbanck 1996).

The sediments deposited in sewers store pollutants, which are released during storm events as a result of sediment erosion and movement. Studies of combined sewage quality indicate that the time of the day, the antecedent dry weather conditions, the magnitude and pollutant characteristics of the dry weather and the storm flows, together with the characteristics of the sewer system and the layout and size of the catchment area, all influence the temporal variability in the concentration and load of the pollutants (Geiger 1984). The hydraulic and pollutant separation performance of CSO structures is therefore critical for the retention of these pollutants in the sewer system. In practice, it is common to relate such performance to the settling velocity distribution of the particles (Marsalek *et al.* 1993).

With reference to the integrated pollution control and CSO impacts on receiving waters, the magnitude and frequency of pollution loads discharged through CSOs have to be quantified, including the CSO volume, pollutant concentrations, load rate and frequency of overflow. To achieve environmental quality objectives one must consider both the acute effects of bacterial pollution, toxic substances and oxygen depletion in intermittent overflows, as well as the longer term, perhaps annual, load of accumulating pollutants such as nutrients, metals and persistent trace organic contaminants (Harremoes 1990).

2.2. SEPARATE SEWER SYSTEMS

The separate sewer system comprises two sets of sewers – sanitary sewers conveying municipal sewage and storm sewers conveying stormwater. Conceptually, the main advantage of the separate system is the separation of two wastewater streams – highly polluted sanitary sewage and less polluted stormwater. For that reason, in recent urban developments in North America, the separate systems have been used exclusively. In practice, the full separation of the two wastewater streams is not always achieved and this may lead to uncontrolled discharges of sanitary sewage to the receiving waters. Thus some separate sewers function almost as combined sewers. This occurs, for example, in the case of leaky sanitary sewers, whose capacity is exceeded in wet weather by excessive infiltration of groundwater and by inflow. Consequently, such sewers surcharge, cause flooding of basements and bypassing of sewage at the wastewater treatment plant.

Modern design of storm drainage systems is based on the major/minor drainage concept and water quality/environmental protection objectives. The minor drainage includes underground sewers and small open channels, and provides local convenience and prevention of water ponding. It is typically designed for short return periods, from 1 to 10 years. The major drainage system conveys flood flows through urban areas and includes large sewers, the natural drainage system, swales, streets and other overland routes. This system is generally designed for return periods from 50 to 100 years.

Storm sewers discharge stormwater directly into nearby receiving waters. Before stormwater management was introduced, older storm sewers were designed to drain quickly high runoff flows from urban areas, without any considerations of runoff quantity and quality impacts on the receiving waters. Modern storm drainage systems employ stormwater management to reduce runoff impacts by reducing runoff volumes, distributing runoff hydrographs over longer time periods and enhancing the quality of stormwater. These concepts have been successfully applied in new developments, where space and funding are available for stormwater management measures; applications in existing, older areas are much more challenging.

2.3. INTERACTIONS AMONG URBAN WATER SYSTEM COMPONENTS

Interactions between storm sewer discharges and the receiving waters are particularly strong and related to the impacts of urbanisation on the hydrologic cycle. Discharges of high stormwater volumes and flows, carrying significant loads of various pollutants and sediment, lead to flooding, morphological and habitat changes, degradation of water quality and biological communities, impairment of water uses and other adverse effects in the receiving waters. Associated reductions in low flows are also harmful, because of reduced groundwater recharge, lowered groundwater tables and reduced baseflows in streams and rivers, with concomitant reductions in self-purification and the dilution of polluted influents (Marsalek *et al.* 1993).

Inflow of municipal sewage into separate storm sewers contributes to the pollution of stormwater, and the inflow of stormwater into sanitary sewers increases the flow rates, which may exceed the WTP capacity and lead to sewage bypasses. Other connections between sanitary and storm sewers and the groundwater are caused by infiltration of groundwater (undesirably increasing flow rates in both types of sewers) and wastewater exfiltration leading to the pollution of groundwater.

High inflows of stormwater to WTPs (via combined sewers or leaky sanitary sewers) produce hydraulic and pollution shocks on the treatment plants which affect especially the nitrification and denitrification processes of biological treatment by shortening the reaction time and reducing the return sludge flow. In addition, the biomass is diminished as sludge is flushed into the final clarifier. Furthermore, the biomass reactions are rather sensitive to fluctuations of concentration, temperature and pH-value. All these factors can lead to reduced treatment efficiencies and increased discharge of pollutants in the treated effluent into the receiving waters (Harremoes *et al.* 1993a).

In combined sewer systems, the interactions among the three major components are stronger than in separate systems. In dry weather, the combined system functions like the separate sanitary sewers; the only flow generated is municipal sewage which is transported to the WTP for treatment. In wet weather, surface runoff enters combined sewers, the capacity of the system is exceeded, and overflows are either discharged directly into the receiving waters (as CSOs), or enter CSO control facilities, many of which interact with the WTP. The CSO discharges are characterised by high loads of gross solids, biodegradable organic matter, nutrients, ammonia, and fecal bacteria. Their impacts on the receiving waters include oxygen depletion, eutrophication and increased productivity, toxicity caused mostly by ammonia, and fecal pollution (Lijklema *et al.* 1993).

3. Impacts on Receiving Waters

Urban impacts on receiving waters are caused by discharges of stormwater, CSOs and WTP effluents, and depend on both the characteristics of the catchment producing such effluents (with respect to stormwater, CSO and WTP effluent quantity and quality), and the characteristics of the receiving waters. In receiving waters, physical, physico-chemical, biochemical and biological processes are important. Thus, the actual impacts must be evaluated in terms of specific characteristics of each site, including physical habitat changes, water quality changes, sediment and toxic pollutant impacts, impacts on biological communities, groundwater impacts, and public health impacts caused by discharges of fecal pollution, particularly during wet weather (Ellis and Hvitved-Jacobsen 1996). Because of the complexity of wet-weather impacts, it is no longer adequate to assess them only by collection of discrete data required by individual disciplines, but rather to undertake biological monitoring which integrates effects of physical, chemical and biological factors. In this assessment, both temporal and spatial scales are important; temporal scales should reflect the nature of acute or cumulative impacts and spatial scales should correspond to the extent of impacts on the receiving waters (Lijklema *et al.* 1993).

Acute impacts are felt instantaneously and represent those caused by flow (flooding), biodegradable matter (impact on dissolved oxygen levels), toxic chemicals (acute toxicity) and fecal bacteria (impacts on recreation). For pollutants causing acute impacts, concentrations and the frequency and duration of occurrence of pollutant levels are of interest. Transport dynamics in the receiving waters, including effluent mixing and dispersion, and pollutant decay, are important phenomena influencing the resulting ambient concentrations in the receiving waters. The frequency of acute impacts is related to the frequency of rain and snowmelt events, as determined from local climatic data. The duration of such impacts exceeds the duration of rainfall/runoff and snowmelt events by the duration of after-effects (i.e., the persistence of the wet-weather conditions). Such extended durations may vary from a few hours, for small discharges to well-flushed or large stable receiving water systems, to one or more days in water bodies with limited circulation.

Cumulative impacts generally result from a gradual build-up of pollutants in the receiving waters and become apparent only after such accumulations exceed some critical threshold value. Examples of such impacts are those exerted by nutrients and toxicants released from accumulated sediment (Harremoes 1988). For pollutants causing cumulative impacts, fine time scale dynamics is not important and the main interest consists in loads integrated over extended time periods. The significance of temporal scales is reflected by water quality criteria for intermittent discharges, particularly for dissolved oxygen and its depletion by CSOs. Such criteria include return period of the impact causing event, duration of the impact causing discharge, and the constituent concentration (House *et al.* 1993).

Characteristics of the drainage area and the receiving waters determine the spatial scales of stormwater and CSO impacts. Stormwater outfalls are greatly dispersed throughout the urban areas, with hundreds of outfalls even in medium-sized communities. CSO outfalls are more consolidated to fewer locations, and further efforts to reduce the number of overflow points are continuing. Pollutant transport in the receiving waters further increases the spatial scope of stormwater and CSO impacts.

Municipal effluents are discharged into the receiving waters, which range from creeks, rivers, or lakes of various sizes, to estuaries and oceans. The stormwater and CSO impacts are most serious in small urban creeks, in which the dilution of stormwater and CSO discharges is minimal. These small streams can be severely damaged by the impacts of elevated runoff discharges, and inputs of thermal energy, chemicals, and fecal bacteria and pathogens. In small streams, the morphology may change dramatically and these changes contribute to habitat destruction. Both acute impacts (flows, bacteria, oxygen levels) and cumulative impacts (morphological changes, accumulation of toxicants) are important.

In rivers, the mixing and dispersion of stormwater and CSO pollutants are important processes reducing the pollutant concentrations outside of the mixing zones. Stormwater and CSO impacts are generally less important than in creeks, because of large input dilution and the stream self-purification capacity. The impacts on lakes and reservoirs depend on the size of such water bodies. The most impacted are small impoundments in urban areas, particularly by fecal bacteria, nutrients, and contaminated sediment. Influx of sediments also destroys the habitat. In large lakes or oceans, stormwater and CSO discharges typically impact only on the near-shore waters in the vicinity of urban areas.

A summary of spatial and temporal domains of various wet-weather effluent impacts is given in Fig. 1 (Lijklema *et al.* 1989); a detailed discussion of individual types of impacts follows in Sections 3.1–3.4.

3.1. IMPACTS CAUSED BY PHYSICAL FACTORS

Flow impacts – progressing urbanisation leads to increased surface runoff volumes and runoff peak flows. Environmental effects include flooding, sediment erosion and deposition, habitat washout (Borchardt and Statzner 1990), and combined morphological changes (Schueler 1987). Some flow impacts are instantaneous (i.e., flooding, washout), others are long-term and cumulative (morphological changes and the concomitant loss of habitat). Ecological impacts include those on food web, critical species and ecosystem development; fishing is the most affected primary beneficial water use (Lijklema *et al.* 1993).

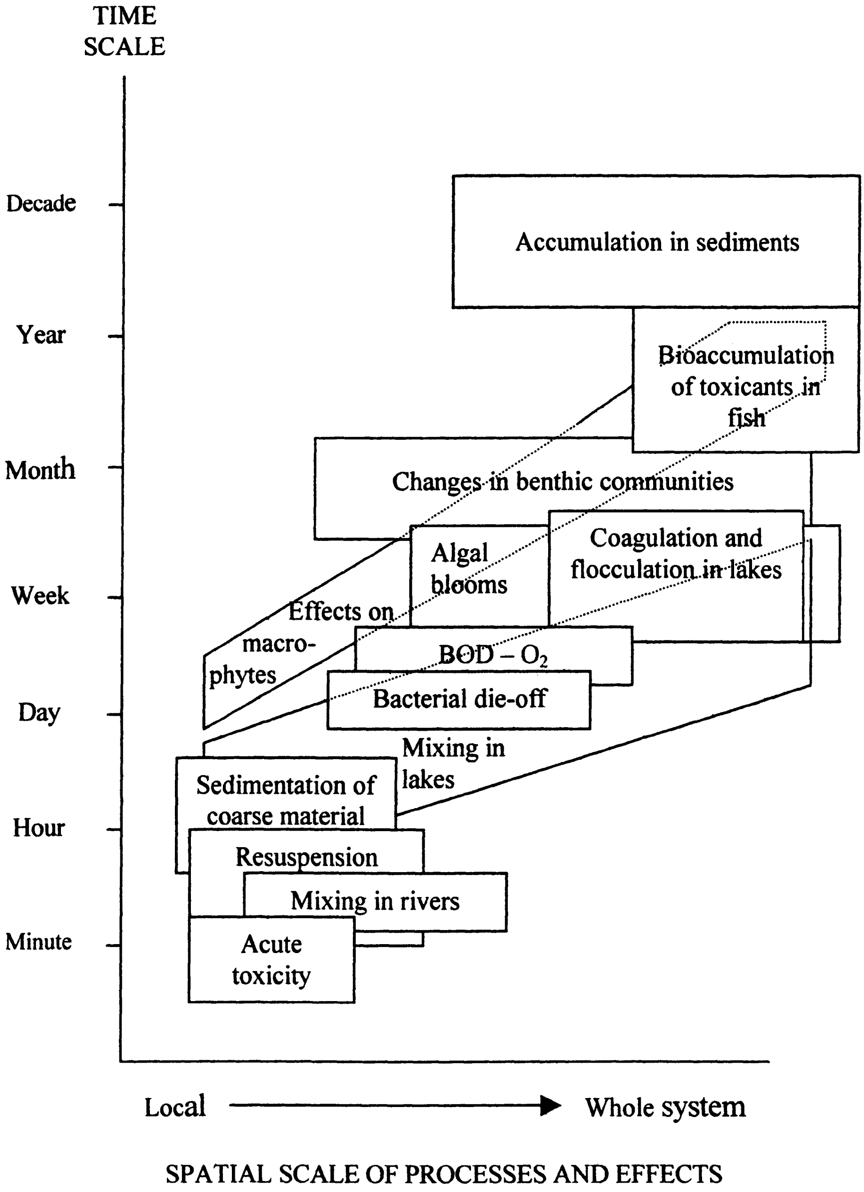

Figure 1. Relationship between the rates of processes and spatial and temporal scale of effects (after Lijklema *et al.* 1989)

Erosion and increased concentrations of suspended solids – rainfall and runoff cause erosion in urban areas and increased concentrations of suspended solids in the receiving waters. Large increases in erosion rates result from two factors – the stripping of natural protective vegetative covers from the soil surface during construction and increased runoff flows, which cause scouring in unlined channels and also provide increased capacity for transport of eroded material to the downstream areas (Horner *et al.* 1994). Sediment yields may increase more than 100 times during catchment urbanisation (Wolman and Schick 1962), but would subside significantly after completion of the urban development, and establishment and consolidation of surface cover. Even in well-established urban areas, soil erosion can increase as a result of redevelopment or break down of the vegetative soil cover (e.g., by seasonal die-off of vegetation). Thus, high soil erosion in urban areas is a transient process, which should be mitigated by erosion and sediment control programs.

Ecological damages are caused by excessive erosion, which wears away habitats and expands the channel width and depth, sometimes by rapid downcutting or channel incision (Booth 1990). Suspended solids cause a number of direct and indirect environmental impacts, including the abrasion of fish gills and other sensitive tissues; reduced visibility for catching food and avoiding predators; transport of pollutants; loss of riparian vegetation with the concomitant loss of shade and refuge; loss of protective qualities of large woody debris (Horner *et al.* 1994); reduced sunlight penetration (interference with photosynthesis); blanketing of gravel substrates where fish spawn, rear their young, and where algal and invertebrate food sources live; and, filling up the pools where fish feed, take refuge from predators and rest.

Significant erosion impacts can be caused by a single large rainfall/runoff event, but long-term morphological impacts are generally more important. Ecological impacts include those related to critical species, and dispersal and migration; and, practically all beneficial water uses are affected (water supply, bathing, recreation, fishing, industrial water supply and irrigation; Lijklema *et al.* 1993).

Temperature rise and species succession – urban sources of heat (e.g., hot pavements) may increase the temperature of urban runoff by as much as 10°C, compared to runoff from undeveloped areas (Schueler 1987). This thermal enhancement is particularly noticeable during the periods of low flows. As the catchment development progresses and thermal enhancement of runoff takes place, the original cold-water fishery may be succeeded by a warm-water fishery. Similarly, thermal runoff enhancement can cause algal succession from cold-water species (mainly diatoms) to warm-water filamentous green and blue-green species, as well as severe impacts on cold-water invertebrates (Galli 1991). Ecological impacts of thermal enhancement include those related to energy dynamics, food web, genetic diversity, and dispersal and migration. The most impacted beneficial water use is primarily fishing (Lijklema *et al.* 1993).

Chlorides – large quantities of chlorides are conveyed by runoff from, and snowmelt in, urban areas, as a consequence of road salting. The primary physical environmental effects of elevated chloride levels include high discharges of dissolved solids to, and densimetric stratification of, the receiving waters. Densimetric stratification inhibits vertical mixing and transport of oxygenated water to the bottom layers. The time scale is instantaneous, and ecological impacts include those on food web, genetic diversity and ecosystem development. The affected beneficial water uses include water supply, fishing, and irrigation (Lijklema *et al.* 1993).

3.2. IMPACTS CAUSED BY CHEMICALS

Urban wet-weather effluents generally contain low levels of chemical contaminants, which may cause biological damage in two forms – by chronic impacts resulting from cumulative water quality stress, and by pollutant accumulation in aquatic sediment and the resulting impacts on the organism that inhabit or spend considerable time in or on the streambed or reservoir/lake bottom (Horner *et al.* 1994). Several types of chemical impacts are discussed below.

Dissolved oxygen (DO) depletion – reduction in DO and the concomitant biomass accumulation are typically caused by discharges of oxygen demanding substances, characterised by biochemical oxygen demand (BOD), chemical oxygen demand (COD) and ammonia. Oxygen demanding substances are conveyed in relatively high concentrations by CSOs (Harremoes 1988); stormwater sources are much less important. Environmental impacts occur on two time scales – short-term impacts are caused by dissolved BOD/COD and ammonia, and intermediate-term impacts are caused by the sediment oxygen demand (Hvitved-Jacobsen 1982). These impacts affect the receiving waters ecology and water uses; ecological impacts include those on biodiversity and critical species; the affected water uses include water supply, bathing, fishing and industrial water supply (Lijklema *et al.* 1993).

Nutrient enrichment and eutrophication of receiving waters is typically caused by total nitrogen and phosphorus found in both CSOs and stormwater. Increased nutrient loadings may cause lake eutrophication characterised by an overall increase of algal biomass, and changes in the composition of algal community from one-celled diatoms to filamentous green forms, followed by blue-green forms. Eutrophication degrades lake ecosystems in a number of ways, including reduced food supplies to herbivores, reduced water clarity, and at the end of the bloom, algal decomposition which causes high oxygen demands leading to oxygen deficiency, particularly in the bottom layers. These effects manifest themselves over longer time periods, a season or longer (Harremoes 1988). Ecological impacts include those on energy dynamics, food web, critical species, and ecosystem development. The affected beneficial water uses include water supply, bathing, recreation, fishing, industrial water supply, and irrigation (Lijklema *et al.* 1993).

Toxicity – toxic impacts may be caused by elevated levels of ammonia, chlorides, metals, hydrocarbons (particularly polycyclic aromatic hydrocarbons, PAHs), and trace organic contaminants (including pesticides) in stormwater (Hall and Anderson 1988; Dutka *et al.* 1994a, b) and CSOs (Lijklema *et al.* 1993). Depending on circumstances, these impacts can be either acute or cumulative. The toxicity of urban runoff is measured by various bioassays, but conjunctive determination of causes (i.e., pollutants and their forms) is lagging behind.

Furthermore, laboratory bioassays do not reproduce well the field conditions, and therefore, may underestimate or overestimate the field toxicity, particularly when dealing with intermittent exposure. Chronic bioassays should produce a better indication of biological effects, but they are expensive and unsuitable for on-line monitoring (Ellis *et al.* 1995).

To overcome some of the above problems, Dutka (1988) proposed to apply a battery of bioassays to water, sediment, sediment pore water and sediment solvent extracts. The use of stormwater and CSO sediments has a number of advantages – such sediments seem to provide a fairly steady, integrated record of the rapidly varying quality of the overlying water column, and consequently, the toxicity assessment of benthic sediment is gaining on importance. Recent applications of this approach indicated toxicity of sediment and pore water from four stormwater ponds; ammonia and pesticides were identified as potential sources (Dutka *et al.* 1994a, b).

Alternative approaches use biological early warning systems, applied in conjunction with various forms of aquatic life. For example, caged rainbow trout was used to monitor the impact of CSO pollution events by Seager and Abrahams (1990) and showed rapid and significant respiratory response to physicochemical changes in water quality, with normal breathing patterns not being restored until nearly 24 h after the event.

Ecological impacts of toxicants include those on food web, biodiversity, and critical species, with reference to ammonia and trace organic contaminants; in the case of metals, such a list could be further expanded for ecosystem development. In short term, the only beneficial water use significantly impacted is fishing (Lijklema *et al.* 1993); in long term, the receiving water ecosystem is downgraded.

3.3. MICROBIOLOGICAL POLLUTION IMPACTS

The microbiological pollution impacts on human health and biomass are associated with both CSOs and stormwater. The effects on public health are mostly related to swimming beaches, the effects on biomass include contamination of shellfish and closure of harvesting areas.

Public health – both stormwater and CSOs convey high loads of fecal bacteria, which are typically described by counts and fluxes of indicator bacteria, such as *Escherichia coli* or fecal coliform. The actual health risk depends on the nature of recreational activities – the highest being for swimmers and the smallest for those engaged in wading. While the determination of microbial pollution in the receiving waters is a routine task, the potential public health risks are not well understood (lack of epidemiological data). Furthermore, these effects manifest themselves instantaneously, though their measurement (involving laboratory incubation) introduces time delays of about 24 hours into the process of detection (Marsalek *et al.* 1994).

Recognising that typical concentrations of *E. coli* in CSOs may reach up to 10^7 EC/L and in stormwater up to 10^6 EC/L, these sources can cause bacterial contamination and exceedance of the recreational water quality guidelines in the receiving waters. Such exceedances occur during wet weather and usually persists for a significant time period afterwards, often 24 to 48 hours after the end of storm, depending on bacteria dieoff and transport in the receiving waters. Many beaches in urban areas are frequently closed during and immediately after rainfall events, because of fecal bacteria contamination caused by stormwater and CSOs (Dutka and Marsalek 1993; Marsalek *et al.* 1994).

Ecological impacts of microbiological pollution include those on energy dynamics, food web, and ecosystem development. The impacted water uses include water supply, bathing, and fishing (Lijklema *et al.* 1993).

3.4. COMBINED IMPACTS

Stormwater and CSO discharges cause numerous biological impacts through combination of three factors – habitat destruction, thermal enhancement, and pollutant discharge. In field situations, it is often difficult to separate the impacts of individual factors, particularly where they manifest themselves as long-term cumulative impacts. Typical biological effects, observed in connection with urban runoff and CSOs, include adverse impacts on food web, biodiversity, critical species, genetic diversity, dispersal and migration, and ecosystem development. An improved understanding of the integrated biological effects can be obtained by the biological community assessment, conducted in conjunction with the assessment of physical, chemical and biological factors (Horner *et al.* 1994). Such factors are shown schematically in Fig. 2 (after Yoder, 1989).

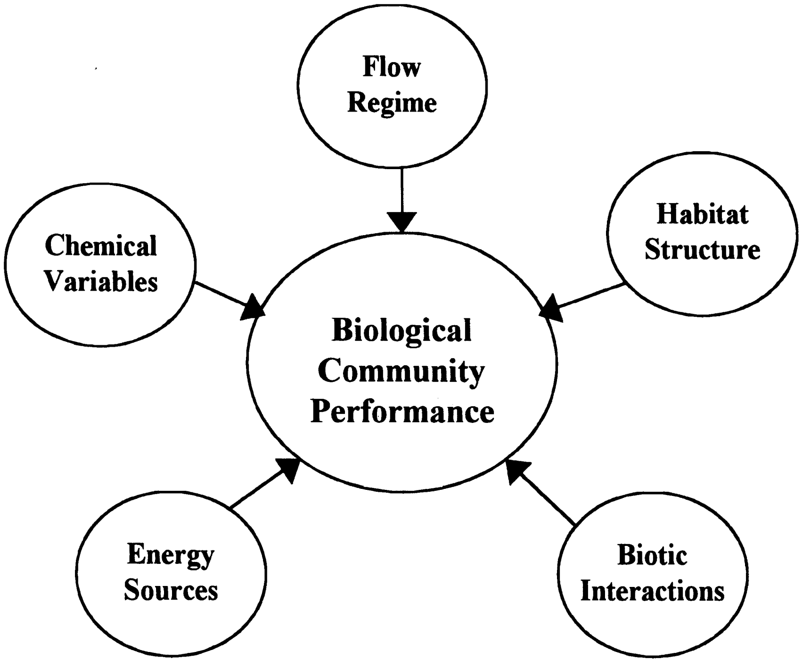

Figure 2. Physical, chemical and biological factors influencing biological community performance in surface waters (after Yoder, 1989)

The complexity of processes in receiving waters and time varying impacts necessitate the assessment of field conditions by computer modelling. Several well-tested commercial packages exist on the market and are used in practice for modelling rainfall/runoff processes, pollutant export, mitigation of impacts by treatment and control, and physico-chemical and biological processes in receiving waters. While significant advances have been made in ecotoxicological modelling (Chapra, 1997; Thomann *et al.*, 1992), the existing equilibrium, steady-state models will require further development for applications to highly dynamic impacts of stormwater and CSOs on biota (Ellis and Hvitved-Jacobsen 1996). Further advancement in the understanding of stormwater and CSO impacts will require considerations of the catchment, drainage, groundwater, treatment plant and receiving waters as one entity, and planning sustainable integrated development and ecological enhancement of urban streams and corridors (Ellis and Hvitved-Jacobsen 1996).

4. Impact Mitigation: Control Measures and Treatment

Concerns about the ecological and human health impacts of stormwater and CSOs led to the development of various control and treatment schemes, which are designed for events with certain frequencies of occurrence. The design capacity of such facilities will be exceeded during events of greater magnitudes, because it would be uneconomical to design for very low frequencies of occurrence (catastrophic events), with limited utilisation during long inter-event periods. Thus, the selection of the design return period is a compromise between the costs of protection and the costs of damages, and reflects an acceptable level of risk. Similarly, any guidelines or standards for stormwater and CSO control should be probabilistically based, specifying the acceptable probability of compliance (House *et al.* 1993).

Recognising that each urban setting is unique in terms of sources of stormwater and CSOs, their characteristics, the existing infrastructure, and the receiving waters (type, quality, beneficial water uses), the solutions to stormwater and CSO problems are also unique and should be designed to fit the local conditions. In general, the control and treatment measures are arranged in the treatment train which refers to a system of various control and treatment options, arranged in series and designed to provide the desired effects (Marsalek *et al.* 1992; Schueler 1987). The need for combining various measures also follows from the fact that typical control measures and treatment processes perform selectively in removal of pollutants or their fractions. The mitigation of stormwater impacts is provided by stormwater management, the abatement of CSO pollution is achieved by CSO control and treatment.

4.1. STORMWATER MANAGEMENT

Best management practices (BMPs) attempt to mitigate the impacts caused by progressing urbanisation including increased discharges and volumes of runoff, and increased production and export of pollutants from urban catchments. Most common BMPs include the following practices (Azzout *et al.* 1994; MOEE 1994; Schueler 1987; Urbonas 1994): non-structural measures, lot-level measures, grass filters and swales, infiltration facilities, porous pavement, water quality inlets, oil/grit separators, filters, stormwater management ponds, and constructed wetlands. Brief descriptions of individual measures follow.

Non-structural measures – generally refer to policy, regulation, and public awareness and education measures serving to reduce stormwater generation and enhance its quality at the source. Specific measures include land use management (preservation of natural drainage, minimisation of impervious areas), management of household materials (minimise potential for entry of chemicals and other materials into stormwater), management of pollutant build-up (street sweeping, sewer system cleaning and maintenance), and sewer infiltration management (by using proper construction materials or rehabilitating ageing systems) (Lawrence *et al.* 1996).

Lot-level measures are implemented on site and represent mostly source controls. Such measures include enhanced rooftop detention, flow restrictions at catch basins to enhance local storage/detention, reduced lot grading to slow down runoff flow and enhance infiltration, redirecting roof leader discharges to ponding areas or soakaway pits, and sump pumping of foundation drains (Azzout *et al.* 1994; Geiger and Dreiseitl 1995; MOEE 1994).

Grass filters and swales – runoff passing through grass filter strips or swales is treated by such processes as biosorption and filtration, and also subject to enhanced infiltration. Filter strips require certain lengths (> 15 m) to be effective; swales also help to reduce the speed of runoff (Azzout *et al.* 1994; Schueler 1987).

Infiltration practices – are used in urban areas, with combined or separate sewer systems, in various types, including wells (pits), trenches, basins, and perforated pipes, catch basins, inlets, and manholes (Azzout *et al.* 1994; Geiger and Dreiseitl 1995; MOEE 1994; Schueler 1987; Urbonas 1994). All these structures reduce the volume of runoff by allowing it to infiltrate into the ground and recharging ground water. Infiltrating water also conveys some pollutants, thus reducing their export. In general, infiltration structures can be very cost effective, but their more widespread use is impeded by concerns about groundwater contamination, lack of design guidance, and concerns about maintenance and design life (Mikkelsen *et al.* 1996). With reference to groundwater contamination, the chemicals of concern include heavy metals, polycyclic aromatic hydrocarbons (PAHs), simple aromatic compounds, polychlorinated biphenyls (PCBs), pesticides, chlorides and pathogens. Mikkelsen *et al.* (1996) concluded that the risk of groundwater contamination by hydrophobic chemicals included in this list was low, because they would be immobilised in the soils adjacent to infiltration facilities. However, highly soluble chemicals (e.g., chlorides) would be transported into groundwater aquifers.

Porous pavement – represents another infiltration measure for reducing surface runoff (Geiger and Dreiseitl 1995; Urbonas 1994). In the Canadian climate, the use of this measure is discouraged (MOEE 1994), but extensive applications were reported in France, particularly with subsurface gravel-filled storage used to redistribute flows and enhance the stormwater quality (Azzout *et al.* 1994).

Water quality inlets – were originally developed as small three-chamber storage tanks installed at inlets to the sewer system. They provide some stormwater treatment by sedimentation and skimming of floatables and hydrocarbons. Problems with the original three-chamber designs were reported, particularly the washout of deposited materials during severe storms (Schueler 1987).

Oil/grit separators – function similarly as water quality inlets, but can be also installed in-line at locations further downstream from inlets. A number of commercial designs are available on the market and some of these indicate good potential for removing coarse solids (sand) and containing spills of free oil, and thereby providing effective stormwater pre-treatment (Chocat 1997; Geiger and Dreiseitl 1995; MOEE 1994).

Filters – stormwater sand filters were introduced in the USA, with reasonable success in some locations (Urbonas 1994). They are effective in removing pollutants, but efforts must be made to reduce the risk of their clogging by providing stormwater pretreatment and backwashing (Schueler 1987). Biofilters (i.e., filters with a coarse medium suitable for growing biofilm on granular surfaces) were also tested for stormwater treatment and showed some promise (Anderson *et al.* 1997).

Stormwater management ponds – are used commonly in urban areas, often as wet ponds with a permanent pool of water and emergent vegetation. Such ponds provide flow control (reduction of flow peaks) and stormwater quality enhancement by sedimentation (removing sand, and some silt and clay), adsorption by active sediments, and interception and uptake by emergent and submergent plants, algae and fauna (Lawrence *et al.* 1996; Marsalek *et al.* 1992). Besides water quality benefits, ponds also provide aesthetic and recreational amenities, which make them desirable for siting in parks. Among their disadvantages, one could name the thermal enhancement of water passing through ponds (Schueler 1987). The current research issues in studies of ponds include flow circulation in ponds (including wind effects and thermal stratification), turbulent and flocculent settling, exchanges of chemicals between the active sediment and the overlying water column, development of pond ecosystems, and effects of ponds on wildlife, including contaminant uptake.

Constructed wetlands – provide stormwater treatment by various processes, including filtration, infiltration and biosorption, and remove both particulate and dissolved pollutants (Reed *et al.* 1993; Rochfort *et al.* 1997). Wetlands can be used as stand alone facilities, or in combination with other BMPs, such as stormwater ponds. The most widely used are reed beds which trap sediment, nutrients, bacteria and toxins, and also promote oxygen recovery (Ellis 1993). Effective removal of nutrients may require regular harvesting. Problems associated with this BMP include thermal enhancement, seasonal variations in performance (caused by temperature changes and the life cycle of plants) and complicated maintenance (MOEE 1994).

The treatment train applied in stormwater management does not end at the drainage outfall, but continues into the receiving waters, where it takes advantage of the self-purification capacity of receiving waters. Such an approach requires maintenance of the natural character of receiving waters, in which aquatic plants, organisms and microorganisms, and benthic sediment play important roles. Where such elements have been disrupted by stream training or urban impacts, renewal of the natural character and the self-purification capacity of urban streams, by returning them to the state resembling their pre-development condition, is required (Ellis 1995).

From the literature on BMPs (Azzout *et al.* 1994; Geiger and Dreiseitl 1995; Lawrence *et al.* 1996; MOEE 1994; Schueler 1987; WEF and ASCE 1992), it appears that no single BMP offers a panacea for stormwater pollution. Instead, individual BMPs should be considered as part of the treatment train, which starts in the catchment, continues in the collection system and ends with in-stream measures. The sustainability of BMPs has to be ensured through proper operation, design modifications (where required to meet the original objectives) and maintenance (MOEE 1994). Finally, even though well-designed BMPs provide many benefits, they must be also recognised as wastewater treatment facilities that may impact on the environment and wildlife by contaminant entry into the food chain (Ellis and Marsalek 1996).

4.2. CSO CONTROL AND TREATMENT

The need to control CSO pollution ranks high on the list of environmental objectives in many countries. Perhaps the greatest innovation in this field is the integrated management approach to CSO control considering the catchment drainage (including the collection system), wastewater treatment plant and the receiving waters. In fact, only this integrated approach to pollution control offers a true assessment of the effectiveness of individual system components and a basis for their optimal design. Harremoes and Rauch (1996). demonstrated that in terms of cumulative impacts caused by nitrogen discharges, WTP dominates and CSOs play a minor role. For acute detrimental impacts, e.g., oxygen depletion, both WTP and CSO discharges are equally important. In this case, CSO storage tanks were found to be marginally effective in acute pollution reduction, because of increased demands on the settler volume at the WTP.

In the overall analysis of CSO control, several components are considered – source controls, storage (in-line and off-line), treatment (both central and satellite), and in-stream measures. Source controls address both reduced influx of stormwater, and controls of dry weather flow (storage prior-to storms, reduced pollutant fluxes through regular maintenance etc.). Since CSOs are caused by excessive inflows of stormwater into the sewer system, any measure discussed in Section 4.1. for reducing generation of stormwater would also help to abate CSOs. Such helpful measures include all lot-level measures, infiltration measures (pits, trenches, basins, porous structures) and porous pavements.

<u>CSO storage</u> – storage capacity can be created in a number of ways: by maximising the utilisation of storage available in the existing system (e.g., through centrally controlled operation of dynamic flow regulators in real time – Schilling *et al.* 1989), as newly constructed on-line or off-line storage (on-line storage includes oversized pipes or tanks; off-line storage includes underground storage tanks or storage and conveyance tunnels), or even

in the receiving waters (flow balancing systems created by suspending plastic curtains from floating pontoons, in a protected embayment in the receiving waters) (WPCF 1989). Some storage facilities are designed for improved CSO treatment by sedimentation, which can be further enhanced by installing inclined plates. Stored flows are returned to the wastewater treatment plant, which must be redesigned/upgraded for these increased flow volumes. Without such an upgrading, the plant may become overloaded, its treatment effectiveness impaired and the benefits of storage would be defeated (Harremoes and Rauch 1996).

<u>CSO treatment</u> – takes place either at the central plant, together with municipal sewage, or may be done in satellite plants dedicated to this purpose. Treatment technologies are available to achieve almost any level of CSO treatment, but proper cost/benefit considerations are crucial for achieving the optimal level of abatement, with given fiscal constrains. Demands on treatment capacities can be reduced by balancing inflows by means of storage (Marsalek *et al.* 1993). From the maintenance point of view, the operators (municipalities) prefer relatively simple treatment systems, with more or less automatic operation and minimal maintenance requirements.

The issues related to the central wastewater treatment and handling wet-weather flows are addressed in Chapter 5. However, to avoid long wastewater transport and central WTP upgrades, satellite treatment facilities have been proposed, some as simple as overflow separator structures designed to retain as much of the CSO pollutant loads within the sewer system as possible. These separators typically use centrifugal forces or secondary currents for solids separation and several vortex separator designs, with or without chemically aided separation, and/or with flotation, seem to provide the best pollutant separation and retention in the sewer system.

In other cases, treatment processes are implemented at satellite wet-weather flow treatment facilities. Various processes have been proposed or implemented for the treatment of CSOs, including plain, inclined-plate and chemically aided settling (Delporte *et al.* 1995), screening, filtration, dissolved air flotation, degritting with a chemical stage for pre-treatment and flotation reactor (Pfeifer and Hahn 1993), and chemically aided settling with microsand ballasting (Le Poder and Binot 1995). Furthermore, the treated effluents may have to be disinfected, either by conventional chlorination (sometimes followed by dechlorination), or by UV irradiation (WPCF 1989). From the municipal point of view, concerns have been expressed about operating the satellite facilities in addition to the central treatment plant, which would put strain on manpower and increase maintenance needs.

The most cost-effective CSO abatement schemes deal with the entire urban area (and all system components) and combine various source controls, storage and treatment measures, while allowing various degrees of control and treatment, depending on the event frequency of occurrence (Marsalek *et al.* 1993). More frequent events should be fully contained and treated; less frequent events may be still fully or partly contained and treated to a lower degree, and finally, the infrequent events may cause overflows of untreated sewage, but of greatly reduced volumes.

The complexity of sewer systems, and the dynamics of flow, storage, loads and treatment processes, make it particularly desirable to control the sewerage / treatment / receiving water systems in real time. Real time control (RTC) was found particularly useful in systems with operational problems varying in type, space and time, and with some idle capacity (Schilling *et al.* 1996). The best developed types of RTC are those for wastewater quantity and the associated modelling. The remaining challenges include RTC of quality of wastewater and receiving waters, and further improvement in the reliability of hardware. It was suggested that in typical wastewater systems with no control (i.e., control by gravity only), approximately 50% of the system capacity remained unused during wet weather, and by applying RTC, about half of this potential could be realised (Schilling *et al.* 1996). For further discussion of RTC of sewer systems, see Chapter 6.

5. Analysis of Urban Drainage Processes

The complexity, temporal and spatial detail of urban drainage processes and their management are such that sophisticated methods of analysis are required and have to be applied using special tools. An overview of the methods of analysis and analytical tools follows.

5.1. CATCHMENT PROCESSES AND SEWER TRANSPORT

The understanding of rainfall/runoff processes is fundamental for urban drainage management and serves for estimating flows and pollutant transport in various parts of the drainage system. Pathways of runoff with entrained materials and pollutants in the urban environment are quite different from those in rural areas and, consequently, urban hydrology has evolved as a special discipline. In this connection, O'Loughlin *et al.* (1996) suggested that the theoretical development of rainfall/runoff models has reached the state of maturity and further progress will be of incremental nature. This level of model development is reflected in proprietary and non-proprietary modelling packages, which are widely used in the current drainage practice (Marsalek *et al.* 1993). Apart from these large well-established modelling packages, research is continuing on the development of new research models of specific processes; for example, storm washoff of solids from impervious surfaces (Deletic *et al.*, 1997). Also, there is a tendency to adopt some features of new research models as optional subroutines in larger packages.

Innovative trends in this field include application of models in the realm of hydroinformatics, i.e., in conjunction with special databases featuring storage of information in GIS or similar spatially oriented tools. The quality and robustness of such information are of utmost importance, as further discussed in Chapter 4. Other innovations include the use of radar measured rainfall data (including moving rain storms), efficient routing of pressurised flow in a computing environment with full graphical support, consideration of management options (storage and treatment facilities), inter-active, dynamic control required in real-time control studies, and realistic modelling of sediment transport and flow quality (O'Loughlin *et al.* 1996).

In spite of the ever-increasing model sophistication, there are obvious problems in the modelling practice, such as lack of local short-term rainfall inputs, calibration data and guidance for selection of model parameters, and widespread underestimation of modelling uncertainties. This state of modelling practice led to calls for less complex stochastic modelling, which would recognise uncertainties in both the model inputs and the modelling results (Harremoes *et al.* 1993b).

Although the typical modelling packages include water quality subroutines, difficulties with further refinements of quality modelling and the lack of supporting data led to a widespread use of alternative methodologies, including the use of databases, unit area pollutant loads, and statistical modelling of observed data (Marsalek 1991). These approaches recognise the temporal nature of stormwater impacts, acute and cumulative. For acute impacts, extreme concentrations and their durations are of interest and, where feasible, may be produced more reliably by statistical interpretation of field measurements than by modelling, whose accuracy in prediction of extreme concentrations is questionable.

In the assessment of cumulative impacts, long-term loadings are of interest and can be determined by modelling or from the existing databases. A major problem in addressing cumulative impacts of stormwater and CSOs is the production and transport of sediments and adsorbed pollutants, particularly when dealing with combined sewers with high variations of flows and solids carrying capacity (Ashley and Verbanck 1996). One of the best stormwater quality databases is that produced under the U.S. Nationwide Urban Runoff Program (U.S. EPA 1983). In this program, the concept of event-mean concentrations, used in conjunction with runoff volume estimates to produce loadings, has been found useful. The main advantage of the database information is its robustness.

5.2. TREATMENT AND WATER QUALITY ENHANCMENT

As discussed in Section 4, many remedial measures are applied in urban drainage systems to treat urban wastewaters or to enhance their quality by management practices. Much progress has been achieved in modelling such processes, particularly with reference to the conventional municipal wastewater treatment. For example, the multi-purpose modelling system for simulation, optimisation and control of wastewater treatment plants, also known as the General Purpose Simulator (GPS), contains a library with more than 30 different steady-state and dynamic treatment models, which can be used to simulate processes in various wastewater treatment plants (Patry and Takacs 1990). Further descriptions of such models can be found in Chapter 5, and the conjunctive use of this class of models with sewer network models was described by (Harremoes and Rauch 1996).

The situation is less satisfactory with respect to BMPs for enhancement of stormwater quality or the treatment of CSOs. In particular, the experience with such processes or measures is rather limited and the information available in the literature is often incomplete or too much site specific. Some of the CSO treatment processes described in Section 4.2 are still in an experimental stage and further developmental work is needed. Consequently, the modelling of these processes is still lagging behind, though the general framework for their implementation in simulation exists, for example in the form of the earlier mentioned GPS system (Gall *et al.* 1997).

Some aspects of stormwater quality enhancement are addressed in the existing urban drainage models, which are described in Chapter 5. In particular, Huber (1996) pointed out that the following BMP options could be simulated with the SWMM model:

storage and associated treatment, screening and filtration, chemical treatment and chlorination, wetlands (only as a storage device), source controls, street cleaning, overland flow infiltration (no quality), maintenance (sewer flushing), illicit connection removal, inlet constrictions, and some aspects of real time control. However, it should be noted that for most of these options, only some processes can be simulated (usually hydrologic or hydraulic processes) and others have to be described by empirical equations obtained from other sources (for example removal equations for treatment processes). Furthermore, the existing models of stormwater storage and settling generally do not include flocculent settling, or removal of contaminants associated with specific types of solid aggregates.

5.3. RECEIVING WATERS PROTECTION

Recognising that urban drainage design should start with the protection of the receiving waters, specific ecosystem or water quality objectives are needed for designing drainage projects or remedial measures in the existing areas. However, the appropriate water quality objectives are difficult to define. Attempts to impose end-of-pipe water quality standards are rarely acceptable, because they neglect the linkage between drainage effluent quality and the environmental state, uses and self-purification capacity of the receiving waters. Thus, there is more interest in defining drainage water quality objectives with reference to water quality in the receiving waters, quite often outside of the mixing zone. This significantly complicates the analysis. For example, in new urban developments in Ontario, Canada, the most severe restriction on drainage design appears to be the requirement to protect fish and preserve fish habitat, with water temperature and stream morphology implications often being more restrictive than water chemistry considerations.

In the ecosystem approach, water quality objectives for the receiving stream, which are typically derived from the traditional water uses, are expanded for ecological protection and enhancement of the stream, thus further increasing the expectations on control of runoff quality as well as quantity. Consequently, in this approach, the water quality objectives for urban runoff will continue to be driven by water quality conditions in the receiving water, as established by stakeholders for individual water bodies.

Functional, disciplinary, spatial and temporal integration required in comprehensive drainage studies calls for the use of mathematical modelling (Ellis and Marsalek 1996). The advanced modelling practice continues to be dominated by a limited number of well-supported and continuously updated modelling packages incorporating some aspects of hydroinformatics and merging environmental modelling with information technology, as discussed elsewhere in these proceedings (Chapters 3, 4 and 5). A wide range of options available in some of these tools allows to address comprehensive drainage/environmental systems, including the collection system, management and control schemes, WTPs and receiving waters. Expert system supports and RTC simulation modules are also available. The collection and processing of physiographical data for drainage modelling and presentation of results are simplified by the use of Geographic Information Systems (GIS). Finally, while development of detailed models will continue to attract research interest in practically all aspects of urban drainage, integrated water quality planning seems to place emphasis elsewhere – on modelling simplifications, which focus on critical contaminants (state variables), reduce requirements on input data, and provide quick results with some assessment of uncertainties (Lijklema *et al.* 1993).

6. Policies, Regulations and Institutional Aspects

Technical and scientific progress in dealing with urban drainage problems is well recognised and documented in the literature. However, the ability to apply these new findings to the fullest depends on adoption of progressive environmental policies and regulations, and establishment of suitable institutional frameworks (Tyson *et al.* 1993). A brief overview of the underlying issues follows.

Although the ultimate goal of environmental policies is the use related protection of the receiving waters, some of the existing pollution control policies are still sectoral in their application. In other words, they may target specific sources (e.g., industries), but fail to promote an integrated approach needed to achieve the desired objectives. Progressive policies should start with source controls, focusing on hazardous substances. With control of hazardous substances, other policy aspects should deal with controlling non-hazardous materials by means of control, treatment and safe disposal (Tyson *et al.* 1993). Furthermore, these policies need to be based on environmental economics, in which all aspects of environmental actions are examined to determine the true environmental costs. Only this approach allows for proper assessment of management options and selection of the most effective option (Tyson *et al.* 1993).

A successful implementation of stormwater management and CSO control is enhanced by progressive environmental programs, regulations and laws. Examples of such regulatory tools from several countries were examined by Roesner and Rowney (1996) who noted that the variation of regulatory stance among the countries studied was greater than variation in opinions and approaches of the technical community, and interpreted this observation as implying that regulatory aspects of stormwater management are rather uncertain, and in some cases, may even impede effective environmental practices. Among the trends noted were attempts to regulate according to a comprehensive assessment including impacts (Environmental Quality Objectives), according to discharge without regard to site specific impacts (Uniform Emission Standards), according to the available technology without regard to its specific effectiveness, and with and without consideration of economic means to implement these regulations. While the approaches based on Environmental Quality Objectives (EQOs) are appropriate for degradable constituents in municipal effluents, in the case of persistent, toxic or bioaccumulative substances, a precautionary, technology-based approaches were recommended by Tyson *et al.* (1993).

There are many different institutional frameworks for delivering urban drainage services. A three-level approach is fairly common, with general policies determined at the national or territorial government level, planning and enforcement done at the regional level, and daily operation conducted at the municipal (local government) level. Two recent significant institutional changes are worth of emphasising – privatisation and public involvement. There is a world-wide trend to privatise delivery of water services, including water supply, drainage and wastewater treatment. While the responsibilities at the higher institutional levels remain largely unchanged, daily operations of drainage and treatment systems are taken over by private companies.

The second important change is increasing public involvement. The planning of water pollution control is no longer conducted just by professional experts, but involves all stakeholders, including various citizen groups. This approach requires to involve all parties at an early stage and inform the public about the environmental benefits and costs of pollution control projects. Public involvement is ensured by such action as public meetings, open houses, tours of similar facilities, neighbourhood walks, visual displays of proposed changes and distribution of printed materials. This approach greatly increases public acceptance of proposed projects, which is essential for successful implementation (Marsalek *et al.* 1992).

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BUSINESS NEEDS OF URBAN DRAINAGE, IN THE CONTEXT OF INFORMATION MANAGEMENT

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1. Introduction

Pundits are forecasting the emergence of an 'information' society. The European Commission has stated that the 'dawning of multi-media world (sound – text – image) represents a radical change comparable with first industrial revolution' (EC White Paper: Growth, Competitiveness, Employment; 1994). Increasingly companies from all sectors of industry and business are becoming profoundly aware that the way in which they handle their corporate information will be crucial to their survival and future prosperity.

It is questionable whether industry and business is generating the demand for improved information technology facilities or whether companies are being forced to climb on the 'band wagon'. What is obvious is that there is a rapid growth in cable networks, with a number of large business corporations developing their own information highways (Intranet). Many people are now reputed to be teleworking. With ready access to Internet and World Wide Web and improvements in security using these networks and facilities there is ready access to databases and information for a wide range of purposes. Many of the new benefits are shown in areas such as preventative health care and home medicine for elderly, opening up new possibilities for improving the quality of life for the average citizen. So far as business is concerned, management quality, speed of information and time to market are key factors for competitiveness. The new IT facilities make these factors possible for most companies.

The EC is determined to support these enhanced facilities through what it calls a 'common information area' consisting of

- information in electronic form,
- hardware, components and software to process the information,
- physical infrastructure including terrestrial cable, radio communication networks and satellites,
- basic telecommunication services, particularly electronic mail, file transfer, interactive access to databases, etc.

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- applications that are user-friendly with structured information
- users who are trained and aware

Such an information area provides a ready framework for the implementation of hydroinformatics systems to manage better the aquatic environment. Our concern is with its implementation for urban drainage. To explore this we need to appreciate three factors. The first concerns the way in which urban drainage as a utility is managed. The second factor is the extent to which advances in information technology are being taken up in the industry. Finally, there are the potential applications of information technology that would bring sizeable improvements to the management of drainage assets. These factors can be appreciated by first looking at the *business* level of management rather than the technical. This is not to say that the technical is unimportant, but the significance of the technical is better appreciated within the context of the business aspects. For convenience two alternative models of managing urban drainage utilities are considered, namely in the UK and USA. What emerges is the view that hydroinformatics is as much concerned with the process of managing sewerage as it is with the function of the associated informatics tools that may be used in the management process. A more rigorous definition of hydroinformatics is expounded by addressing developments in computing and networking hardware and in the management of information.

2. Urban drainage in the UK

Urban drainage in the UK is now the responsibility of a privatised water industry. Before 1973 sewerage as an activity was undertaken by each local authority. But the growing recognition of the need to bring together catchment and river management, water supply, treatment, and drainage led to the amalgamation of these various services in England and Wales within ten Regional Water Authorities. This initiative prompted a national appraisal of the services and their associated supply and wastewater assets. In time, however, it was apparent that there was a need for considerable ongoing investment in the assets to maintain satisfactory levels of service. Part of the reason for this is that the UK was one of the first countries in modern times to invest heavily in sewerage. With the advent of the Industrial Revolution in the UK there was a growing need for better sewerage to combat growth in cholera and other infectious diseases. The wealth generated by industry enabled considerable investment in underground piped sewerage during the second half of the nineteenth century. Consequently there are now a large number of sewers that are more than a century old. Fortunately, many of these sewers were oversized for what was actually needed at the time. Problems of flooding were tackled reactively, which meant that the networks were adapted and extended on the basis of local criteria rather than in terms of general performance. Many sewers have failed structurally due to ageing. The UK can therefore be regarded as paying the penalty of being one of the first countries to provide extensive sewerage for its more densely populated urban areas. There is little doubt that lack of adequate investment has exacerbated the problem, leaving the water service providers with the need for sizeable investment to maintain and improve their assets. One of the primary reasons for privatisation was to attract investment from the private rather than the public sector.

Another contributory reason to the privatisation of the water industry in the UK was some confusion over the role inherent within the previous Water Authorities to regulate themselves, particularly with regard to water quality impact from combined sewer overflows and treatment works on receiving waters. This led to the setting up of the 10 Water Service public limited companies (plc) in 1989 covering the whole of England and Wales with the exception of x private Water Companies responsible for supplying water to x% of the population. The Water Service plcs were established to take responsibility for water supply, distribution, treatment, sewerage and waste disposal. Responsibilities for receiving waters passed to a new body called the National Rivers Authority. This organisation remained under the UK Department of the Environment and became one of the regulators for the Water Service plcs. Because of the need for the Water Service plcs to attract private investment for capital expenditure on their assets, the plcs became profit distributing companies. In turn, for the plcs to be attractive to investors it is necessary for them to keep their share prices high. One of the many factors affecting the share price is the level of customer satisfaction with the services they provide. It is therefore understandable that each of the Water Service plcs has invested heavily in corporate software to help manage the customer services function. In practice the Water Service plcs have a monopoly over the provision of piped water and the treatment and disposal of wastewater, other than for the x private Water Companies. Therefore the regulators have the responsibility to safeguard the interests of the government, the customers, and the environment. As part of the political developments there are now discussions taking place to decide how competition may be introduced to water supply service. The idea is for a number of suppliers to put water into the supply networks that may continue to be owned and operated by the existing Water Service plcs. The suppliers would make separate agreements with consumers for the provision of water. This type of arrangement is also being considered in the gas industry and has been implemented in the electricity industry for many years. Time will show whether the water industry can accommodate this form of competition for water supply.

The significance of water supply and drainage to society means that the corresponding services are backed up by extensive government legislation. There have been a number of Water Acts down the years, particularly in establishing the significant administrative changes that have come about. The 1989 Water Act setting up the privatised Water Service plcs has now been almost entirely replaced by the Water Industry Act 1991, the Water Resources Act 1991, the Statutory Water Companies Act 1991, the Land Drainage Act 1991, and the Water Consolidation Act 1991. A recent Environment Act has brought about changes in the way that the regulatory powers of the National Rivers Authority (NRA) are exercised. This has resulted in the demise of the NRA and the establishment of the Environment Agency. The Agency has absorbed the former NRA along with the regulatory responsibilities of Her Majesty's Inspectorate of Pollution which looked after the quality of potable water and waste disposal, among other things. The Environment Agency has wide ranging powers not only for water but also for air and noise.

Whereas the UK has been developing its own structure for the water industry the actual structure is affected by membership of the European Union. Indeed, dominant drivers for UK legislation are EC Directives. So far as urban drainage is concerned the key EC Directive is the Urban Wastewater Treatment Directive. This Directive has a primary emphasis on protecting the environment, particularly sensitive receiving waters. To do this, urban communities throughout the EC have to provide themselves with sewerage and wastewater treatment facilities by certain dates depending on the size of the communities and the nature of the receiving waters. See Fig 1. Additionally, other Directives such as the Bathing Waters Directive add weight to the need to improve treatment facilities throughout Europe. See Fig 2.

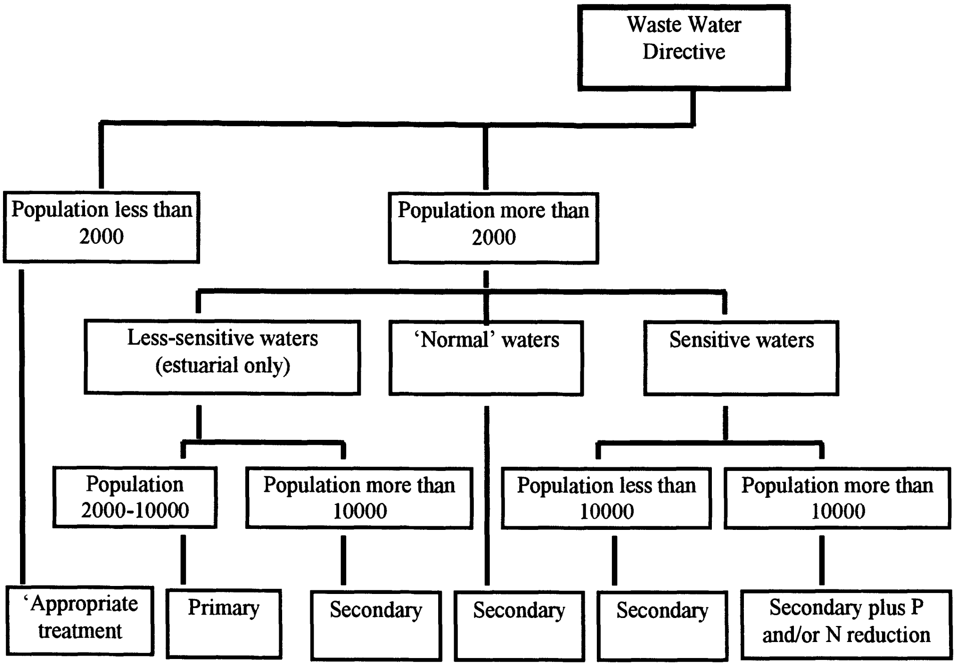

Figure 1. Levels of treatment required for estuarial and inland waters under the Waste Water Treatment Directive.

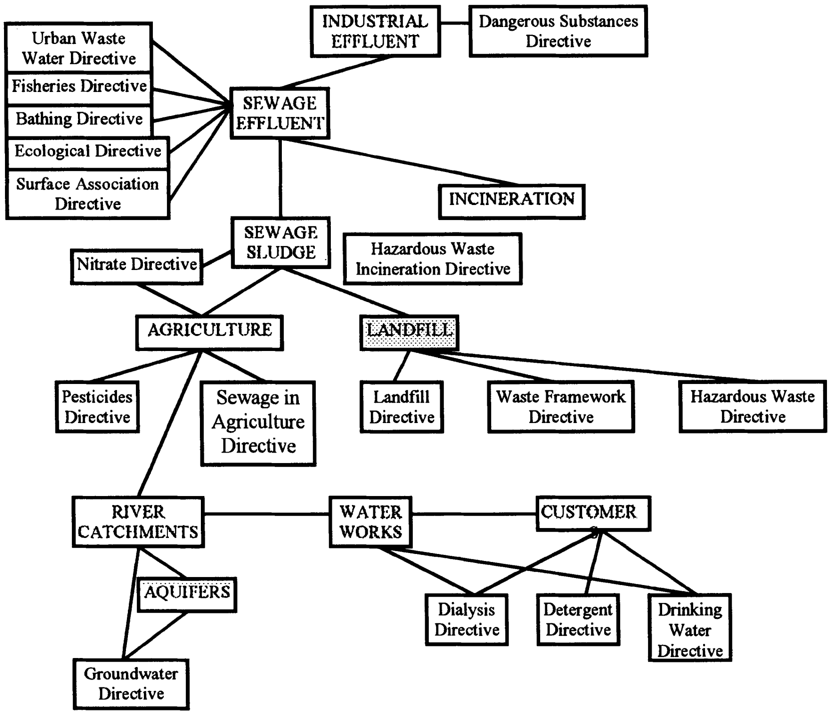

Figure 2. The EC Directive Cycle.

In any country there are a number of stakeholders in water. These include the public at large, local and national government, amenity and pressure groups, water supply, sewerage and treatment companies, waste disposal organisations, agriculture, domestic, business and industrial customers for water supply and drainage, shareholders in private companies, and so on. It is important that all of these stakeholders ensure that proper use is made of water as a sustainable resource. The opportunities for abuse of the aquatic environment are however considerable. Every member of society has some interest in what is done to the aquatic environment. Therefore there is a strong need for regulation of those who have any significant impact on that environment. The UK government has set up a range of regulators to ensure that those organisations working within the aquatic environment are responsible about what they do and seek at all times to protect and preserve the environment as a sustainable resource. Mention has already been made of the National Rivers Authority. So far as the Water Service plcs are concerned, the regulators besides the NRA include the Secretaries of State (Environment, Wales), the Office of Water (OFWAT), the Customer Services Committees, the Drinking Water Inspectorate, Her Majesty's Inspectorate of Pollution (also in the Environment Agency), the Ministry of Agriculture Fisheries and Food, the Monopolies and Mergers Commission, and the Office of Fair Trading; see Figure 3.

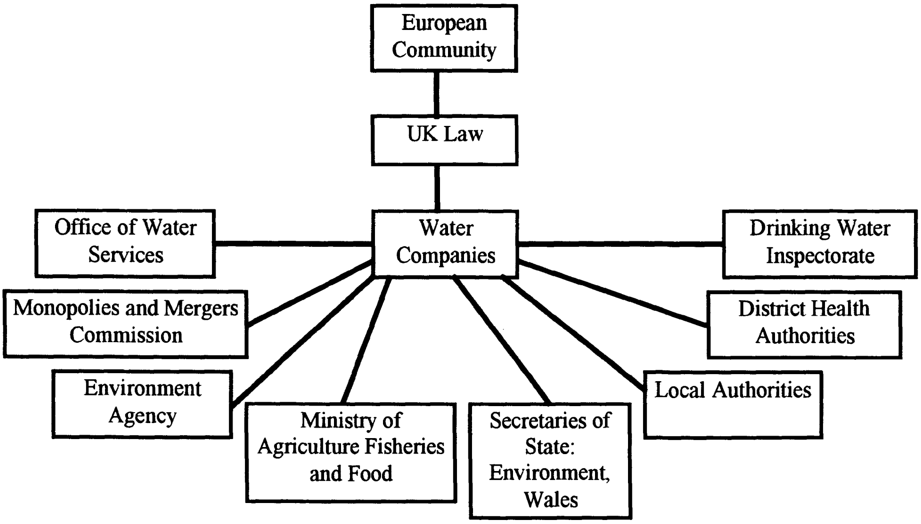

Figure 3. Regulation in England and Wales.

The Water Service plcs are to abide by particular *codes of practice* in relation to their customers and the community. These define environmental and recreational duties, good agricultural practice, exercise by water companies of their works powers on private land, customer relations with the Customer Service Committees, disconnections, and leakage.

Because of the responsibilities of the Water Service plcs they have been required by the government to produce *Asset Management Plans* for water supply, treatment and sewerage – in two phases – AMP1 (1987) and AMP2 (1993). These plans are used as a basis of justifying investment. The emphasis is on minimising costs and ensuring that the regulators are satisfied. Stress is therefore given to minimising the risk of failing to satisfy the regulators. Besides concern about satisfying the regulators the Water Service plcs also have to improve services to customers and provide adequate return to shareholders.

OFWAT is now insisting on much more rigorous 'ring-fencing' of the Water Companies to prevent profits from the core businesses being used for dubious adventures in diversification. Therefore the utility companies have to be innovative in their use of technology, that is, providing *serviceability with minimum capital,* to consider all engineering options in exclusively financial terms, to grapple with issues of *risk* in that they cannot control external factors, and to rely on loan capital and mergers to fund diversification rather than their core business of supplying water supply and drainage services to their immediate customers.

The view of the business stock market has been that the UK Water Service plcs have been over-manned and inefficient. This has lead to considerable cost-cutting exercises by the plcs to maintain profitability and attractiveness to the corporate investors. The severity of these cost-cutting exercises has lead to links with other utilities (gas, electricity, heat, CO2, telecommunications – cable TV, cellular telephone). The assumption is that additional savings can be made in improving the multi-functionality of merged utilities in terms of communications with the customer (billing, information, etc.), construction and maintenance (co-ordination between teams digging up roads, sharing resources, etc.), and so on. North West Water has merged with Norweb supplying electricity in the north-west region of England to form United Utilities and Welsh Water has bought South Wales Electricity to become Hyder. Privatisation is also leading to mergers or take-overs between the different water companies, not necessarily from the UK; for example, Lyonnaise des Eaux has bought Northumbrian Water.

Increasingly the Water Companies are investing heavily in IT as a means of improving their efficiency and profitability. Interestingly, North West Water have bucked the trend of recent years and reverted to a central mainframe having tried distributed client-server networks. There is also ongoing investment into GIS and other technologies that appear to offer considerable savings.

The situation regarding sewerage and wastewater management in the UK has changed considerably following privatisation. Although there is growing concern about the impact of CSOs on the natural environment, flooding from combined or separate sewers is still a major problem. Typical facts that have to be considered by the industry are

- 20% of UK sewers were built before 1900
- 50% of UK sewers are more than 60 years old
- Risk of failure of 50% of UK sewers over the next 50 years is very high
- Cost of replacing 50 % of UK sewers over the next 50 years is the order of £450M/annum
- 20% of failures account for about 90% of expenditure

Capital costs to solve the problems are very large. Somehow the Water Companies have to minimise investment to achieve serviceability. Tables giving UK capital expenditure by service, inventory of the main assets, total household bills for water and sewerage services, % population served by sewage treatment for different EC countries, and sewage connections and volumes for different EC countries are adapted from *Water Facts* (1995) and given below.

By way of comparison it is helpful to compare statistics for the European Union on sewage connections and volumes; see Table 1 below.

TABLE 1. Main assets of the UK Water Service plcs 1993/94

	Anglian	Dwr Cymru	North-umbria	North West	Severn Trent	Southern	South West	Thames	Wessex	Yorkshire
Sewage Treatment Works										
primary	29	216	88	146	62	36	24	2	22	69
secondary	682	519	283	422	673	315	445	283	220	464
tertiary	361	109	12	50	277	16	9	86	108	97
Sewers (km)	29,000	16245	13409	30060	51413	21791	7490	83474	13942	24139
Sewage pumping stations	3969	1430	486	1475	2461	1905	648	2286	1178	1253
Storm overflows	2534	2500	1334	2337	2162	933	1032	2617	1002	1845
Sea and estuarial outfalls	36	129	33	62	0	49	-	0	-	-

TABLE 2. Capital expenditure of the UK Water Service plcs by service 1993/94 (£M)

	Anglian	Dwr Cymru	North-umbria	North West	Severn Trent	Southern	South West	Thames	Wessex	York-shire	Total
Sewerage	43.2	26.9	7.9	47.7	71.2	44.9	22.5	77.0	28.6	34.9	404.8
Sewage treatment	91.9	46.4	38.5	107.3	108.7	70.4	107.6	71.9	41.6	36.5	720.8
Sewage management	31.5	1.9	4.4	51.6	25.3	5.7	10.5	27.2	7.8	5.4	171.3
Total	371.5	180.5	70.1	423.6	434.2	152.2	204.9	390.8	115.1	220.7	2563.6

3. Urban drainage in the USA

The drainage industry in the USA is somewhat different to that in the UK. To begin with, the sewerage and drainage networks are publicly owned, primarily by the municipalities or other local organisations. The design and even the operation may be contracted out to private companies, some of whom may be locally owned while others may be offices of nation-wide organisations. The Environmental Protection Agency (EPA) has a long history of protecting the hugely varied environment in the USA. If a state proves that it will administer the Federal law then the EPA will yield the primary enforcement function to the corresponding State organisation responsible for environmental protection. The actual arrangement varies between the EPA regions and from state to state. For example in Texas the state environmental organisation parallels the EPA but the EPA issues enforcement actions against individual cities or industry owners for violations. An important success of EPA in terms of modelling is that it has sponsored the most widely used model in the history of urban drainage, namely the Storm Water Management Model, developed at the end of the 1960s.

TABLE 3. Sewerage and sewage treatment by population 1993/94

	Population connected to sewer		**Level of treatment**				**Connected to sea outfalls**	
	000s	% of residents connected	preliminary or none %	primary %	secondary %	tertiary %	short %	long %
Anglian	5184	91	15	8	52	25	-	15
Dwr Cymru	2922	93	45	5	48	2	12	33
Northumbrian	2554	98	33	38	27	2	15	8
North West	6753	98	8	22	67	3	2	4
Severn Trent	8226	98	0	0	68	32	0	0
Southern	4118	95	33	12	50	5	32	22
South West	1358	89	35	14	33	18	33	3
Thames	11529	98	2	0	85	13	0	0
Wessex	2342	93	10	26	51	13	5	5
Yorkshire	4684	99	9	0	87	4	0	3
Total	49670	94	12	9	65	13		

TABLE 4. Sewage connections and volumes

	% population served by sewage works	**% connected to sewers**	**Total number of sewage works**	**Number of sewage works serving >100000**
Belgium	25	58	292	13
Denmark	92	94	1805	18
France	50	65	7805	
Germany (West)	86	91	8456	137
Greece	10	40	26	2
Ireland	25	66	530	2
Italy			3783	102
Luxembourg	76	96	324	1
Netherlands	88	92	485	64
Portugal	37	38	166	
Spain	43	80	1595	
UK	83	96	7645	102
England and Wales	83	96	6524	

Besides the EPA a number of other US Departments have an interest in urban drainage, including US Department of Agriculture, US Department of the Interior, Federal Highway administration, US Army Corps of Engineers, National Oceanic and Atmospheric Administration.

As with other developed countries there is an extremely well-developed infrastructure of legislation, administration, planning, capital improvements, operation and maintenance, regulation, monitoring and evaluation, education programs, technical assistance, good science and funding both on a state and national basis. The state has statutory authority to set up on a local basis the necessary bodies to fund urban storm and waste water utilities. Some such utilities are semi-autonomous separate organisations, others are part of public works, streets and drainage, or even parks and recreation. Typically, planning covers a 20 year period broken down into 5-year sub-programmes. Some capital improvements are designed by multi-disciplinary, in-house teams, though external consultants are often used. Operation and maintenance is often viewed as requiring public ownership because of the interaction with the landscaping and environmental maintenance. Regulation requires the interpretation of complex rules originating from local, state and national legislation. Monitoring and evaluation by administrative departments are seen as vital to implement regulatory procedures properly. Similarly, education programmes are important to ensure that the public, namely the end-customers, is properly informed. This is partly because the USA local government system puts more technical and planning choices directly to the public than in, say, the UK. In addition, there is strong regulatory emphasis on procedures for new development, at the design, design review and construction inspection levels.

Major legislation in recent times stems from the Clean Water Act 1972. This has important consequences for different national programmes, including Near Coastal Waters, State Wetlands Programme, Wetlands Protection Programme, Assessment and Watershed Protection Support, Water Pollution Control, Water Quality Management Planning Programs by states, Water Quality Standards and Implementation Plans, Water Quality Information and Guidelines, Water Quality Inventory, Clean Lakes Programme, Non-point Source Program implementation, National Estuary Program, National Pollutant Discharge Elimination System (NPDES), and permits for dredged or fill material.

NPDES permits were originally focused on discharges through sanitary sewer overflows (SSOs). The Water Quality Act 1987 contains an expansion of the NPDES permits to include stormwater from different sources: industrial discharges (heavy/light manufacturing, light industries, recycling, transportation, construction projects > 5 acres, steam electric power stations, hazardous waste treatment, storage and disposal, mining/oil and gas), discharges from separate storm sewer serving populations > 250,000 (large) or > 100,000 (medium), and discharges determined to violate water quality standards. Final rules for the regulation of stormwater discharges were published in November 1990. All cities with populations greater than 100,000 were obliged to apply for a permit to discharge. NPDES permits for combined sewer systems were coupled with the permits for the corresponding wastewater treatment plants. An average cost of applying for a stormwater permit has been estimated as $761,000.

The National Combined Sewer Overflow (CSO) Control Policy was published by EPA in April 1994. This policy requires all communities responsible for CSOs to implement Nine Minimum Controls (see Table 5). These controls have been developed in recognition that combined sewer systems are site-specific in nature and there is considerable variability of receiving water conditions and impacts. Because of the large capital investments made in improving the performance standards of the CSOs the

Association of Metropolitan Sewerage Agencies has published a report on Performance Measures for the National CSO Control Program. In this way it is intended that the environmental benefits of CSO control programs can be identified and quantified for assessment by the communities. A number of supporting guidance documents have been published by EPA during 1995.

Recent thinking about sanitary sewer overflows (SSOs) in the USA has focused on human contact with receiving waters or in high risk situations (e.g., house basements) rather than how many times a year SSOs cause a water quality violation. This change in emphasis is more concerned with human health and aesthetic impacts rather than DO and ammonia-nitrogen. Relevant determinands include faecal coliform count and floatables.

The various EPA policies for SOs, SSOs, and CSOs are consistent with EPA's five strategic goals for water resources:

- Protection and enhancement of public health
- Protection and enhancement of ecosystems
- Attainment of uses designated by states and tribes
- Improvement of ambient conditions
- Reduction in pollution loadings

Each of these goals has consequences for the management of urban storm and wastewater drainage directly. The design, operation and performance assessment of overflows, not only on an individual basis but also in terms of the cumulative effect on a watershed basis, is therefore an important programme. Increasingly, environmental measures are being implemented as a means of quantifying and tracking the physical, chemical and biological integrity of the waters of the USA. Such measures must be adopted by those working with urban drainage systems in order to improve their design and operation.

Undergirding the various publications on wastewater and stormwater management emerging in recent years is an insistence on using *best management practices* (BMPs).

It is worthwhile noting that the US government has an open policy as regards legislation, and in particular on information relating to wastewater and stormwater management. See, for example, the pages put up by USEPA on World Wide Web: **http://www.ehsg.saic.com/pipes**.

TABLE 5. Summary of the Nine Minimum Controls

1. **Proper operation and regular maintenance programs for sewer system and the CSOs.** This control should consist of a program that clearly establishes operation, maintenance, and inspection procedures to ensure that a combined sewer system and treatment facility will function in a way to maximise treatment of combined sewage and still comply with NPDES permit limitations.
2. **Maximum use of the collection system for storage.** This control consists of making relatively simple modifications to the combined sewer system to enable the system to store wet weather flows until downstream sewers and treatment facilities can handle them.
3. **Review and modification of pretreatment requirements to ensure that CSO impacts are minimised.** The objective of this control is to minimise the impacts of discharges into combined sewer systems from non-domestic sources during wet weather events, and to minimise CSO occurrences by modifying inspection, reporting, and oversight procedures within an approved treatment program.
4. **Maximisation of flow to the publicly owned treatment works for treatment.** This control entails simple modifications to the combined sewer system and treatment plant to enable as much wet weather flow as possible to reach the plant.
5. **Elimination of CSOs during dry weather.** This control includes any measures taken to ensure that the combined sewer system does not overflow during dry weather conditions.
6. **Control of solid and floatable material in CSOs.** This control is intended to control, if not to eliminate, visible floatables and solids using relatively simple measures including baffles, screens, racks, booms and skimmer vessels.
7. **Pollution prevention programs to reduce contaminants in CSOs.** This control is intended to keep contaminants from entering the combined sewer system and to prevent subsequent discharge to receiving waters through street cleaning, public education, solid waste collection and recycling.
8. **Public notification to ensure that the public receives adequate notification of CSO occurrences and CSO impacts.** The intent of this control is to inform the public of the location of outfalls, the actual occurrence of CSOs, the possible health and environmental effects of CSOs, and the recreational or commercial activities curtailed as a result of CSOs.
9. **Monitoring to effectively characterise CSO impacts and the efficacy of CSO controls.** This control involves the visual inspections and other simple methods to determine the occurrence and apparent impacts of CSOs.

4. Key factors affecting the management of sewerage

The analysis of the way in which sewerage is managed in the UK and the USA leads to the identification of a number of key factors.

4.1. LEGISLATIVE AND REGULATIVE DRIVERS

There is little doubt that sewerage is regarded by the public at large, at least when they think about it, as providing society with an important means of removing domestic and industrial wastewater and stormwater from urban areas. Wastewater sewerage is perceived as being important because health hazarding or unpleasant waste is removed from the immediate vicinity of residential areas, the work place and public spaces. There is also a strong aversion to storm water flooding, whether it damages property or delays traffic. Often sewage flooding on the streets or of basements will result in complaints by the public to the responsible authority. Such complaints are not made invariably because individuals fear the implications for their property values. The reduction in the frequency of surface flooding remains a major driver in the UK for capital investment in improving sewerage. It is not however the primary driver which instead is concern for the environment. This is highlighted by national legislation in response to the EC Wastewater and Treatment Directive and the way in which USA legislation is built on the 1972 Clean Water Act. The EC Directive is driven by the perceived importance of protecting the environment, and particularly sensitive receiving waters. Treatment works and sewerage networks have to be constructed to meet the standards required for discharges from CSOs and the works. The emphasis is on the protection of the receiving waters from long term deterioration. There is also a need for short-term protection from levels of oxygen depletion that may lead to fish kills through discharges from CSOs or treatment works or of accidental spills such as of agricultural or industrial chemicals.

The concerns of legislation in the USA over the last 25 years are similar. The details are somewhat different because of the ownership of the assets and because there is greater reliance on separate storm and wastewater sewerage in the USA. Private ownership of the Water Service providers in the UK and the present monopoly within their regions means that customers are more remote from the decision making within the plcs and have more reason to emphasise the 'us-and-them' nature in any dispute involving a failure of the services provided. Interestingly however, there is a growing interest in the USA of the UK privatised model.

The focus on the environment is sustained by the public through various pressure groups such as Friends of the Earth, and by the support of scientific evidence. Awareness is converted into political pressure that then results in legislation to protect local, national and international waters. The implementation of that legislation is through regulation. The regulatory bodies differ between countries, although interestingly, the European Union and the UK, as well as the USA, all now have Environment Agencies that have powers to protect the environment. As shown by the experience of the UK there are a number of needs for regulation. Urban flooding, for example, is dealt with by OFWAT and not by the Environment Agency. Again, the Monopolies Commission has a watching brief to ensure that adequate competition to supply services is maintained in the market place.

4.2. SOCIETAL RESOURCES AND URBANISATION

If concern for the environment is now the primary driver for the provision of sewerage and treatment services, and if legislation provides both for the set-up of appropriate organisations to facilitate the service and to regulate its provision, then there remain a number of scientific issues that are being urgently addressed in response to these drivers. Society does not have infinite resources to spend on improving and running sewerage and treatment services to eliminate the impact on the environment, desirable as this may be. Besides the lack of scientific knowledge and understanding about how to achieve such a goal (which may be impossible anyway, depending on what are defined as acceptable standards) there is also the recognition that there are insufficient resources to make the goal possible. As a result there has to be compromise. Society has to do the best it can with finite resources. This means that there needs to be an equitable distribution of resources to satisfy different needs. Consequently, there is an iterative process between society represented by its politicians who are supported by their legislators and the water industry under managers who are advised by their scientists, engineers, and economists. Having decided on the principles of what *should* be done then it is necessary to deduce what *can* be done with the limited resources available. As with other service industries there is a continuous drive to provide a better service to the customers and the public at large using less resources. This generates the requirements to develop improved ways of using resources, better management practices, and innovative solutions for traditional problems.

The urgency for these developments is also driven by considerable concern about the growth in the human population of the planet. It is anticipated that by the year 2020 the world population will be about 7 billion of which 15% will be in rural areas and 85% in urban towns and cities. Most of the growth in population is in the urban areas while the population in rural areas remains steady. The effect on the urban areas is critical. There is urgent need for innovative plans to respond to the rapid growth that will take place in many cities and to make the necessary resources available. Demands on water supply and drainage will become ever more acute as time goes on.

4.3. URBAN DRAINAGE AS A HOLISTIC PROCESS

Whereas even 20 years ago urban drainage was commonly treated in isolation from natural catchment drainage or impact on other receiving waters there is now a growing awareness of the need for a *holistic* view of urban drainage, particularly in the context of the whole catchment (or watershed), or of the receiving lake or coastal waters. In this respect Europe is somewhat behind North America, though theory on both sides of the Atlantic has reached a similar point. This is possibly because there is a greater readiness to listen to and learn from the experience of engineers in other countries, even if the cultural ways of doing things vary. This latter point is important. Although the physics, chemistry and biology of processes associated with water can be assumed to be strictly universal, some of the processes and implementation of systems to manage water and even the theories to describe those processes are peculiar to each country, or even to separate states as in North America. For example, each EU member country has at least some minor variants in the way that pipes are sized or drainage systems laid out. The UK has used the Lloyd Davies method for design for much of this century whereas France uses the Caquot formula. Italy designs for percentage fullness whereas other countries are content with pipe full flow conditions. Similarly, many of the sewers in the Netherlands operate under surcharge due to the flat terrain. The design and operation of such sewers will differ from those in a traditionally more steep terrain. Storm drainage in South East Asian countries is primarily through open drains. Canada and Australia design their storm drainage systems to include both minor (below ground) and major (above ground) networks.

4.4. NEED FOR MODELLING

Any analysis of the combined collection-treatment-receiving water system must now depend on modelling, to some degree or other, of the integrated system. Simulation engines are well recognised in the UK as tools for the routine analysis of urban drainage networks. There are also several alternative software packages for the analysis of treatment processes and for water quality impact in rivers or coastal waters.

The original impetus to simulation engine development came in the 1960s with the first commercial computers. The result was the RRL method, already too complex to do by hand. Subsequently, WASSP was developed in the 1970s as a powerful mainframe system. Then in the early 1980s when the IBM personal computer took the computer market by storm, MicroWASSP, was developed closely followed by products such as MOUSE, WALLRUS and SPIDA. These were precursors to more sophisticated models such as HydroWorks DM for drainage networks, STOAT for treatment process modelling, and MIKE11 for river impact. Each package includes water quality modelling and runs on PCs under the latest interactive graphics facilities provided by a Windows environment.

The drive towards treating urban drainage holistically has come about partly because of the technical ability to model the different phases of the whole process, namely the collection, treatment and impact phases. We are now able to model deterministically the details of each phase with a fair degree of accuracy and reliability. Perhaps the simulation of collection systems has received more attention because of the complicated nature and wide geographical spread of some networks and the fact that the collection system is the driver for the other two phases. The modelling of treatment works has generally been more basic, and any difficult aspects are confined to the details of individual treatment processes. What is demanding the attention of researchers and model software developers is the need to model water quality as well as quantity through each of the three main processes. The confidence that can be placed in the modelling of water quality is an order of magnitude smaller than the confidence that can be put into water quantity. Therefore, whether modelling the water quality of the collection system, the treatment works or the receiving waters, the same problems of inaccuracy and lack of confidence in the results emerge.

Part of the difficulty in modelling water quality is the lack of knowledge of the chemical and biological processes, and of understanding some of the physical processes, particularly in relation to sediment transport. This lack of knowledge remains despite large investment in data collection and analysis. Following many years of data collection for water quality in urban drainage networks some researchers have come to the conclusion that even with good data for water quality it is virtually impossible to identify the underlying physical laws for sediment and pollutant transport in pipes. This reflects the emphasis in the USA of the site-specific nature of sewerage, particularly of combined sewer systems, and the variability of receiving water conditions and impacts. Even if modelling can be done with some degree of precision it is argued that any system designed and implemented using simulation models should be regarded as suspect until it has been thoroughly monitored to determine the performance of the installed system.

4.5. ENGINEERING PROCEDURES

The actual process of designing and operating sewerage systems can be described by engineering procedures. Even a casual look at the history of urban drainage in the UK and the USA shows that engineering procedures have been a significant agent of change in the Water Industry. For example, in the UK there have been three distinct engineering procedures advocated for design and rehabilitation of sewerage systems during the last 20 years. After the Wallingford Procedure for the design of new networks was introduced in 1981, it was closely followed by the Sewerage Rehabilitation Manual (SRM) in 1984. A recognised deficiency in the SRM was its minimalist account of water quality in affecting rehabilitation. This was subsequently rectified in the publication of the Urban Pollution Management (UPM) method in 1994. The advances in understanding and knowledge of the operation of drainage networks embodied in these procedures contributed to the development of National Guidelines for AMP (2)/Periodic Review. The three procedures reveal a growing ambition to provide cost-effective solutions to a basic problem, namely that of constructing and maintaining sewers to provide adequate levels of service. The key issue for the SRM was the rehabilitation of existing sewers to reduce the impact of surface flooding. But short term polluted discharges from combined sewer overflows (CSOs) can adversely affect the receiving waters, whether a river or coastal waters with associated bathing beaches. The focus therefore shifted to what the receiving waters can tolerate, based on the conclusion that rehabilitation of the sewerage system should depend on the receiving waters impact. The solution became one of upgrading the *integrated* system, that is, of the treatment works as well as the sewerage network. A holistic view of the whole wastewater management process had become established.

The development of engineering procedures and simulation models has depended obviously on the growth in computing and pressure from the political and legislative drivers. There is also a strong mutual dependence between the procedures and the models. For example, although the later procedures were developed to address particular issues of regulatory concern, the procedures could not be implemented without the simulation models. As the models have improved in power and capability, so more advanced solutions have been made possible.

The interdependence between standards/guidelines, procedures and modelling in the urban drainage area is a feature of sewerage management in the UK. As yet, other applications, such as to water distribution, river basin management and coastal waters management, do not appear to have the well-developed interdependence between models and procedures. There is however a discernible trend towards achieving a mutually complementary role between models and procedures for topics such as catchment management, coastline management, and bathing beach protection.

The UK engineering procedures for urban drainage and their associated software have been widely accepted in the industry for a number of reasons. They had the backing of both government and industry. The procedures were developed as a consensus between contractors and the working groups consisting of representatives from a wide range of client organisations. Then the incorporated simulation engines or models underwent extensive trials within the industry. Following their launch, both procedures and models were supported by an advisory or user group that brought together the expertise that accumulated to individuals and organisations through the use of the models in engineering practice. The procedures and models were sustained partly through the perceived need for them in the industry as engineers found they could produce more reliable designs. Managers also realised that the models enabled them to look at innovative solutions that could not have been analysed previously. Similarly, the industry appreciated the status of using such sophisticated procedures and computational models.

Typically the procedures are defined in terms of tasks and task structures along with varying levels of information. Therefore the procedures contain knowledge accumulated by the developers. In this sense the procedures, defined in paper-based manuals, are repositories of their knowledge. The wide-spread adoption of the procedures and methods by the UK Water Industry has meant that additional knowledge has 'come to presence' for engineers involved in implementing the procedures and the associated models. This extra knowledge resides with the individuals and their organisations. Some organisations have refined the original procedures for their own internal use. Some make the refined procedures freely available. Others are conscious of the commercial implications and only permit them to be used internally. Fortunately, a national user group, that is, the Wastewater Planning User Group (formerly the Wallingford Procedure User Group), emerged with the first of the three procedures and has since evolved to incorporate the others. This group has ensured that there is on-going dissemination of the emergent knowledge to the wider community so that the industry as a whole has benefited.

The earlier procedures, namely the Wallingford Procedure and the SRM, were developed primarily by the contractor organisations and the industry with significant government funding. At that stage the industry was largely government controlled. Similarly, the UPM procedure was initiated when the water service and river management functions were under the same government organisations. The procedure was refined, however, after privatisation. The timing of the development ensured a joint effort between, not only the contractors and the plcs, but also the NRA in the position of regulator. Fortunately, poacher and gamekeeper continued to work together to maximise the benefits to each, taking account of the constraints that both were under. With the recognition of the principle that the 'polluter pays' both the NRA and the plcs see the need for good science. There is also the added involvement of the plcs in doing their own monitoring for compliance.

The scope of the engineering procedures is wide and far reaching. Each procedure is defined by a domain of knowledge that covers acquisition of asset and performance data, project requirements and constraints, and relevant legislation and standards. Subsequently the data is archived and analysed ready for use in hydraulic modelling, structural analysis, and so on. Once the decision is made to use a hydraulic model then the business of building the model and confirming its performance against recorded data from the real world asset is pursued until the model is 'fit for purpose', that is, capable of being used to find answers to specific problems. Subsequently the model becomes a 'virtual laboratory' for solving any number of a range of applications.

We will consider the nature of these applications below. It is sufficient here to draw attention to the nested domains of knowledge that are needed when using the model and procedure. The outer embracing domain is that concerned with implementing the procedure. Some of this knowledge is in hard copy format as a manual. Other, more comprehensive knowledge is acquired, archived and analysed by the organisation as its staff gain experience in using the procedure. This knowledge domain is complemented by particular knowledge that 'comes to presence' for the engineers working with the procedure on the given network. The inner domain of knowledge applies to the model: how it is built reliably, calibrated or confirmed, and operated. Again, some of the knowledge for doing these tasks is defined as part of the overall engineering procedure, augmented by manuals accompanying the software. There is also knowledge won by engineers from using the software to model a range of different networks. Again, this inner knowledge domain is complemented by particular knowledge that also 'comes to presence' for the engineers as they build, calibrate or confirm, and operate the particular model.

There are, of course, other inner domains of knowledge to do with other tasks within the overall engineering procedure. In this sense the procedure should be seen as a knowledge sub-domain of a larger domain to do with the overall management of a network.

An illustration of the nested domains of knowledge associated with particular procedures and tasks appropriate for urban drainage management is shown in Fig. 4. Most of these tasks require significant skills for them to be carried out efficiently and cost-effectively. Such skills are not acquired in the classroom; rather they are hard won through years of experience and insight. Hydroinformatics systems, such as we are discussing here, provide a framework within which such skills and experience can be captured and harnessed as decision support systems accompanied by other relevant informatics tools.

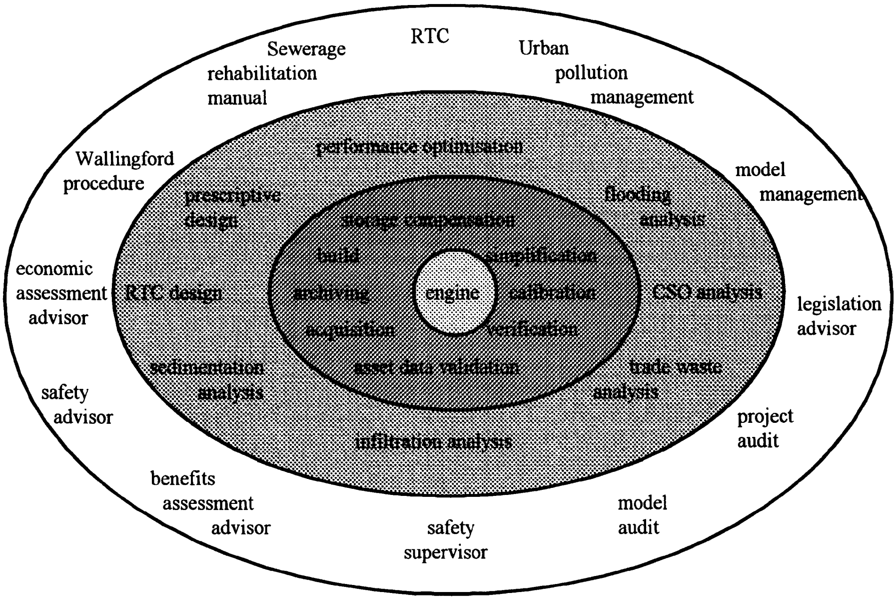

Figure 4. Range of tasks associated with a simulation engine for urban drainage analysis.

4.6. STOCHASTIC NATURE OF SYSTEM INPUTS

Another trend that is shown in the modelling, but also appears in the general management of sewerage, is the growing recognition of the stochastic nature of storm and other events. This has meant, for example in modelling CSO performance that engineers are now using time series rainfall input rather than design storms. Engineers are looking to multiple simulations using events from time series rainfall to generate results that can, in turn, be analysed statistically to derive the frequency of operation of overflows or of surface flooding. In the past, engineers went to extreme lengths to develop design storms in terms of frequency of rainfall depth, duration, profile and antecedent wetness, such as in the UK. The derivation of these design storms depended on a sophisticated analysis that involved the selection of the right values from the statistical distributions for the four key parameters. These design storms are crucial for the prescriptive design of, say, new networks. However, there are considerable difficulties in actual application for analysis. One of the main difficulties is that it is often assumed that the return period of water level at any point in the network is the same as the return period of the flow. This assumption cannot be guaranteed even if the right sampling of values from the four distributions for the parameters has been made. The use of time series rainfall can be interpreted as an acknowledgement that the performance of a sewerage network is stochastic and therefore there is a need to take into account the *risk* of failure, that is, of overflow discharge or surface flooding. The concept of risk is slowly becoming more accepted and even seen as necessary if sewerage assets are to be managed in a better manner. The need for risk analysis is highlighted further by the considerable uncertainty in measurements, model construction, user consistency, and so on. The use of sensitivity, uncertainty and risk analysis is explored in more detail in Chapter 3.

4.7. RISK AT THE BUSINESS LEVEL

Besides the importance of determining the technical risk associated with each process, it is being recognised that risk at the business level is also an important tool. There are considerable problems of investment in sewerage primarily because of the uncertainties in data, methods and external forces. At management level there is the need to know how best to invest limited capital in a competing number of sewerage improvements. There are substantial risks that management has to take into account. Many managers are paid to make decisions on scant data. Hydroinformatics offers considerable opportunity to integrate simple decision support tools involving commercial risk with more detailed analysis tools to evaluate technical risk.

4.8. COMBINED VS SEPARATE SEWERAGE

Those countries in the vanguard of sewer construction, such as the UK, adopted a combined sewerage approach in which both waste and storm water were collected together and transported to the nearest water course. Eventually treatment works were introduced to treat at least the dry weather flows. This meant that excess sewage had to continue to be discharged to adjacent watercourses through what are now known as combined sewer overflows (CSOs). The water quality problems generated by combined sewer systems lead engineers to distinguish between waste and storm water. Separate systems were built with storm water discharged directly to watercourses and wastewater carried to treatment. The value of having some storm water in a wastewater network to assist in flushing the network has been accepted in the development of partially separate systems. These systems may have the roofs of houses discharging to the waste water system while the remaining storm water runoff is collected in a separate system. There appear at first sight to be obvious benefits in separating storm and wastewater collection. Separate systems have however their own difficulties. A wastewater system is prone to inflow from illegal connections and infiltration from groundwater through cracks in the pipes. These increase the flows in the network during or after rainfall events, necessitating the need for overflows. Elimination of inflow and infiltration (I&I) is expensive and generally incomplete. I&I increase over time such that after a period of years system performance deteriorates to previous levels. Similarly, it is well known that some separate storm water systems convey highly polluted material from roads and paved surfaces generally. The extent is such that some engineers advocate the treating of even storm runoff. Although a number of studies have been done to identify the reasons for and against combined or separate systems and to quantify the benefits and costs there has been no definite conclusion for one type of system or the other. So the fundamental problems to do with sanitary sewer overflows (SSOs), stormwater overflows (SOs) and

CSOs, inflow and infiltration to separate wastewater systems, flooding in urban areas, flows to treatment, discharges from treatment, effects on receiving waters, river basin management, lakes, and coastal waters continue to remain with engineers. Again we are back to the two acknowledgements that urban drainage is a highly site-specific process and the impacts on receiving waters are very variable.

4.9. IMPACT ON RECEIVING WATERS

The impact on receiving waters of urban wastewater is a particularly important issue now for many developed countries. Part of the difficulty in dealing with discharges from CSOs or treatment works to streams is the complex nature of the chemical and biological interactions that take place, not only involving dissolved pollutants but also those pollutants attached to suspended and deposited sediments. The streambed is usually an important location for plant and animal life that is very sensitive to the prevailing water quality. Unfortunately the stream carries with it the results of runoff from the whole of the catchment upstream. Therefore any long term planning analysis of the impact of CSOs and treatments works discharge on a stream must take into account the management of the river basin as a whole. Certainly, if planners are interested in the consequences of removing CSOs using an interceptor sewer and opening up new CSOs, resetting existing CSOs, relocating a treatment works, etc., such consequences would need to be appraised in the light of how the whole river basin may respond. There may of course be several urban sewerage catchments within a river basin. The management options for such a basin would need careful modelling in order to identify the best option.

Discharges to coastal waters and estuaries are seen to have direct implications for health, whether affecting bathing beaches or getting into the food chain such as shellfish. Primary concerns are for compliance with the EC Shellfish Waters and Dangerous Substances Directives, trade wastes, sewage debris, international obligations on plastic materials and problems of smell. Discharges must meet initial dilution criteria.

4.10. REAL TIME CONTROL

A growing trend in the application of technology to sewerage is real time control (RTC). This is seen as the most advanced technology for improving the performance of sewerage systems. Whereas engineers will generally agree to design their sewerage systems to drain under gravity with only passive controls there is a growing recognition that RTC is a viable option where the performance of the existing network has been optimised. Many sewerage engineers are reluctant to consider RTC because of the higher risk of failure of a system with RTC. Improvements in sensor, regulator and controller technology have meant however that systems can perform with sufficient reliability to be competitive with other options such as installing larger tanks or bigger pumping stations. The design and operation of such systems may well be a major contribution by hydroinformatics to urban drainage management in the future.

5. Hydroinformatics for urban drainage

It has already been argued that engineering procedures with embedded models have enabled these models to be used widely throughout the UK Water Industry, despite their sophistication. In this sense the Industry is adopting some of the technologies of the information revolution. There are however requirements such as to be more efficient in planning and design, to produce rehabilitation schemes at lower capital costs, or to introduce real time control schemes that are economical to implement and operate, that place greater demands on engineers to take advantage of the new IT technologies. At the heart of these requirements is the need for better management of information. Urban hydroinformatics is the response to such a need.

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HYDROINFORMATICS CONCEPTS

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1. Introduction

The first chapter has established the vital importance of urban drainage to human communities. Increasingly the management of urban storm and wastewater is being viewed as a significant *business* process, even more so with the privatisation of the water services industry as in the UK. The legislative and regulative requirements made on the industry by society to ensure efficient service to customers and the effective protection of the natural environment are cultivating a holistic view of managing the urban storm and waste water process. Such a view is supported by the availability of sophisticated modelling tools that enable engineers to model the integrated storm and waste water management process from 'source to sink', that is, from rainfall and domestic and industrial waste to the impact on receiving waters. Additionally, demands for efficiency and effectiveness are encouraging innovative ways of reducing flows at source, controlling flows in real time, rehabilitating existing assets, and improving the overall system performance. Information technology, that is, the technology incorporating computing, telecommunications and microelectronics technologies, is viewed by the water industry as a means of improving efficiency and facilitating decision making. The adoption of information technology within the water industry is therefore growing rapidly, albeit in a piecemeal manner, and driven more by available technology than by user-needs. This is particularly true of the so-called *advanced information technologies*, including software engineering, artificial intelligence and human factors computing. The user need for an integrated approach to managing water as a business process and as a resource for all requires a new conceptual framework. It is claimed that *hydroinformatics* provides such a framework wherein advances in information technology can be harnessed to the advantage of those who have a stake in the aquatic environment. This claim is examined in the light of how water scientists and engineers have adapted particular advances made in mechanics, geography, biology, civil engineering, and especially information technology.

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2. Hybrid Disciplines of the Aquatic Environment

The evolution of all civilisations, whether ancient or modern, has depended crucially on how they harnessed their aquatic environment. Access to fresh water and the drainage of wastewater have been almost of as much concern to urban inhabitants as their interest in esoteric sciences and exotic art forms. This is not to say that a drawing showing the design of an urban drainage network has the same aesthetic value as a painting of Leonardo da Vinci or Picasso; nor that the drainage network has the momentous impact of, say, the Iron Bridge or the V2 rocket. The life and well being of the artists, scientists and engineers did, however, depend vitally on whether or not these people and their host communities had easy access to clean drinking water and their waste water was removed effectively.

This has meant that a number of self-contained yet interacting sub-disciplines of science and engineering, each dealing with the sustenance of the aquatic environment, have been established in recent centuries. Some have roots in physics, others in chemistry and biology, and yet others are branches of engineering (see Table 1).

TABLE 1. Some of the branches of science and engineering concerned with water (dates are taken from the Oxford English Dictionary)

Interdisciplinary Subject	Concerned with
Fluid dynamics	the study of forces exerted on fluids and the motion that results from those forces.
Fluid mechanics	the study of fluids at rest and in motion.
Glaciology	the study of glaciers and their effect on landscapes.
Hydraulics (1671)	the physical science and technology of the static and dynamic behaviour of fluids
Hydrodynamics (1779)	the study of incompressible fluids and their interaction with their boundaries.
Hydrography (1559)	the study, determination, analysis and publication of the conditions of sea, rivers and lakes
Hydrology (1762)	the study of water, including rain, snow and water on the Earth's surface
Hydromechanics (1851)	the study of mechanics of liquids especially in relation to its application to mechanical contrivances
Hydrostatics (1660)	the study of fluids at rest and under pressure
Limnology (1893)	The study of fresh water lakes
Public Health Engineering	The application of sanitary measures and monitoring of environmental hazards
Sewage/Drainage Engineering	deals with (a) system of sewers and (b) with the removal of waste materials from the system

These subjects continue to thrive in that they are taught in our education systems. Each subject has its experts, its rites of passage including protocols of experience validation, and its own special language. This language is identified by specialist vocabulary or terminology together with a less visible grammar. The knowledge concerning a given subject is disseminated through textbooks, journals, popular science literature, newspaper articles, handbooks, manuals and so forth. More recently, information technology has had a particular impact on this dissemination of knowledge through distributed systems such as the Internet and desktop publishing systems.

Today, almost all of the water-related science and engineering disciplines, whether theoretical, empirical or experimental, have, to a greater or lesser extent, incorporated hardware and software computer systems in their methodologies. Methods have been refined, and in some cases abandoned, due to the advent of, say, real time computers or large and easily retrievable data stores; sophisticated mathematical techniques have replaced 'dimensional analysis' and 'empirically obtained' equations; software tools and even robots have now been added to the tool sets of water scientists and engineers.

Mathematical and statistical methods and techniques were adapted for managing the aquatic environment in two ways. Firstly, Isaac Newton's adoption of Leibniz's calculus led to the development of mathematical models in hydrodynamics, and terrestrial and celestial mechanics. Subsequently, applied mathematics has shaped and changed engineering practice with models applied to a range of topics from one dimensional flow in idealised open channels to wind-water interaction over ocean surfaces. Secondly, mathematical and statistical methods formed the basis for computer simulation. The Newtonian models are mathematically very satisfying but they consist of complex systems of equations which can only be solved analytically if quite drastic simplifying assumptions are made about the *real world* behaviour of water. Computer simulation provides the means of avoiding some of the more serious simplifying assumptions even though the numerical difference equations are still approximations of the analytical integro-differential equations. It also enables the computation of apparently intractable statistical distributions rather than interpolating them crudely from 'books' of (pre-computed) tables.

Almost all of the subjects mentioned in Table 1 have benefited from computer simulation models. The numerical algorithms for such models were formulated soon after the Second World War by well-motivated and mathematically literate engineers and scientists. Once commercial computers became available the algorithms were coded by the same group of people who mastered the intricacies of programming main frame computers. Indeed, they were the only people who could instantiate models based on their software systems. Those engineers responsible for implementing the results of the models had to take on trust what the 'experts' produced. This situation changed dramatically in the early 1980s with the introduction of the personal computer. There were two contrasting consequences. The first was that practising engineers were now able to access the simulation modelling techniques directly rather than rely on the specialist developer. The model became more a tool than an end in itself. End users escaped the control of their computer centres and were able to arrange their own working environment. Secondly, with the extension of the range of people using modelling tools their development became more and more the responsibility of teams of mathematicians, hydraulic engineers <u>and</u> software engineers. The way was open for the emergence of a commercial market for such tools. Once in the hands of the professional software engineers the tools came to incorporate more sophisticated information technology techniques with corresponding improvements in user-friendliness and integration with other systems.

For the hydraulic engineer the primary value of a computer, whether a work station or personal computer, is as a machine that can process extensive numerical operations very rapidly. Such operations can, however, generate huge amounts of data. Whereas disc storage technologies have developed to the stage where the archiving of such data is not a problem, the analysis and extraction of information from the data is much more difficult. For example, simple listings of the data and results are no longer adequate. Instead the user may need high powered graphics processors and 2D and 3D drawing and manipulation software complete with light sources, shadowing and the ability to generate video sequences to make better sense of the data more efficiently and to draw appropriate conclusions. Many of these facilities are now available commercially, greatly facilitating the scope of what the engineer can achieve. However, rapid access and analysis of particular data types such as time series data require specific data structures that have not yet been exploited commercially. In this case users have to resort to the development of their own software; see, for example, Grauer *et al.* (1996).

Besides the raw number crunching power of today's computers the potential of such machines for communicating is becoming even more significant. In particular, the advent of easy-to-use network communications has had a number of far reaching consequences. For example, external links enable software and data to be imported and provide access to control processors. The client-server concept allows sharing of computational resources between remote computers. Event driven functionality facilitates improved working processes between remote partners. Multi-media support and advanced document management lead to better communication of information. Finally, the development of Internet has opened up radically new ways of sharing and communicating information.

Hydroinformatics is the integration of the various traditional disciplines associated with understanding and managing the aquatic environment using information technology as the common integrating factor. As such, hydroinformatics is emerging as a discipline in its own right.

3. Hydroinformatics and Computational Modelling

The strongest roots of hydroinformatics are in computational hydraulics which is itself a hybrid discipline that has emerged within the last 50 years. Once hydraulic modelling had been given a secure foundation it was only natural to turn attention to the more difficult problems of modelling the associated chemical and biological processes in the aquatic environment. In each case it is necessary to generate numerical solutions of certain differential equations subject to particular boundary conditions using computers. Although some of the processes can be well defined, others are not well understood. Additionally, most of the equations include one or more unmeasureable parameters that have to be derived statistically through calibration. Stationary potential flow in groundwater and for free surface flows were some of the first problems to be solved using solutions of the Laplace equation. These were followed by analysis of hyperbolic equations for time dependent flows using characteristics and later finite differences. Knowledge of the pressure and flow fields within a system led to the modelling of transport processes using solutions of parabolic equations that could be coupled or uncoupled to the flow calculations. Whereas the modelling of many water quality determinands can be regarded as having a negligible interaction with the flow, salinity and temperature are related by density to the pressure and therefore feed back to the flow calculations. Sediment transport and resulting bathymetry changes are generally small for large bodies of water such that direct coupling of the processes is not necessary. This is not necessarily the case in sewers, for example, where changing bed forms can have a significant interaction with the flow. With growing societal concern for the natural environment there is an increasing need to include chemical and biological components with the flow simulations. Computational hydraulics is therefore being subsumed into computational 'aquatics' where the interest is more in the interaction of chemical and biological systems with water in motion.

The parametric relationships used with the differential equations to describe the physical, chemical and biological processes have either been established in the laboratory or have to be calibrated from field observations. Each relationship holds for a range of conditions that must be respected. No relationship should be used outside the conditions for which it was derived without taking note of the exception.

Numerical analysis techniques in simulation modelling have ranged from finite differences to finite element and Eulerian to Lagrangian. Explicit, characteristic and implicit finite difference techniques have been dominant in the search for accurate and efficient methods of computation. Care is needed to preserve conservation of various properties inherent to the governing equations and to minimise numerical errors consequent on the numerical approximation. Considerable research is still proceeding into numerical analysis techniques for commercial use to solve problems such as simultaneous treatment of sub- and super-critical flows as well as free-surface and pressurised flows in sewer systems, for example. In the drive for speed and efficiency there is a need for methods that can accommodate spatially as well as temporally varying time steps, and for parallel processing algorithms that can work efficiently on clustered PCs or work stations.

Besides the numerical solution of the equations there is a requirement to identify parameters in the equations such that computed predictions can achieve a best fit with observed data. This can be done by 'trial and error': iteratively running the program and adapting the parameters. Operational research techniques can, however, be used to computerise these manual strategies. An objective function defined, say, in terms of the square of the difference between the computed and observed data can be minimised using techniques such as steepest gradient or genetic algorithms. There are also problems of initial value estimates for time marching problems and the assimilation of observed data within the calculation domain using some form of inverse modelling or iterative matching technique.

Control of systems such as sewer networks introduces another dimension to simulation modelling. There is often a need to optimise performance by adjusting weirs and gates to minimise the frequency of flooding or the effects of polluted discharges. Again, an analysis based on minimising appropriate objective functions can be adopted for off-line control. On-line control demands very powerful numerical computations of the basic equations for forecasting unless other, simpler methods can be adopted.

The spatial boundary conditions for the differential equations consist largely of the system attributes such as topography including areas, terrain, land use and so on. These data originate with surveys that result in maps, charts and plans. Information from these intermediary formats can be extracted by digitising or scanning and then used by geometric modelling tools incorporating computer-aided design (CAD) and geographical information systems (GIS) in a form suitable for the differential equations.

Temporal boundary conditions can involve long time series of information at a few spatial points. Like both the spatial and temporal data there can be a need to manage gaps in the data sets. There are a number of suitable interpolation and statistical techniques: cluster analysis, discriminant methods, multiple regressions, Fourier analysis, and Kalman filters.

Reliable answers from simulation modelling are required under a range of conditions. Models have to be confirmed with respect to physics, chemistry and biology. They have to be checked for different scenarios encompassing a wide range of different conditions. They need to be tested for their ability to make correct and accurate diagnoses. Comparisons should be made with benchmark data taken from analytical and real life situations. What is sought are the confirmation of simplifications in the differential equations and of the valid ranges and magnitudes of parameters for specific situations. Evaluation of the results should be supported by interaction with intelligent graphic processors accompanied by appropriate statistical analysis.

Reference has been made to advances in computer science, telecommunications, and cheap, efficient micro-electronic devices, that is, information technology and ancillary *enabling technologies*. The latter refer to computing techniques and methods, like software engineering, artificial intelligence, and graphics programming, amongst others. Their influence has not only affected the direction of computational modelling but has also highlighted the complementary tools and techniques of data management. These trends are symptomatic of the emergence of hydroinformatics as a discipline that embraces them.

4. Hydroinformatics and Data Management

Decision making in water management is heavily dependent on data acquisition, archiving and analysis. Traditionally, drainage engineers have collected large amounts of data such as above ground and asset surveys, monitored rainfall and system performance in terms of rainfall, discharges, sediment and quality, recorded flooding and pollution reports, produced structural surveys, and so on. These data sets are used to identify design and operating deficiencies, accommodate catchment development, plan engineering works, and improve operational performance. Increasingly, the perceived value of such data and the need to archive and analyse it quickly and efficiently is leading to computer-based management of the data from acquisition to application in decision making. At the heart of such management is the recognition that all data is spatial as well as temporal and can therefore be handled within a geographical information system (GIS).

A GIS is a specialised database management system used in such studies as map-making, land and resource management, demographic research, environmental analysis, market research, and so on. In particular, a GIS provides its users with information about objects and features in their graphical context. Such information can be displayed and analysed within the GIS. The system can be used to interrogate spatial relationships between different kinds of objects and to investigate spatial what-if? questions using in-built functions for computing changes to the locations or characteristics of geographically-contextualised features and objects.

Modellers, planners and engineers now have access to GIS databases that may cover an entire city. Besides the many uses of a GIS for managing urban infrastructure, housing, and transport for example, engineers can view asset and performance data for a water distribution or drainage networks in a conurbation, retrieve information about underground water resources, and so on. The emergence of privatised water utilities in the UK, coupled with strong environmental pressure groups across Europe and a proactive UK Environment Agency, has led to the development of a number of water-related GISs.

Any decision made on the basis of historic real world data alone, whatever manipulation is done within a GIS, is somewhat limited in that it is based on data for the system as it is in the present or as it was in the past. Besides the inherent uncertainties in the original survey data including those due to the purpose and method of collection, the historic data has limited value for prediction purposes. In particular, the historic data is usually inadequate to provide answers to general what-if? questions on system performance. For example, if there is a need to design major improvements to a drainage network then the historic data is insufficient to provide a basis for design in that the data cannot be used to predict what will happen to the designed system in the future. This is where computational modelling comes into its own.

Hydrologists, such as Ball (1994) and Djokic *et al.* (1996), have argued the case for GIS as an integration tool for the hydrologic modelling. The authors point to a 'kind of operational synergy' that may be exploited by a GIS providing services to hydrologic models. For example, the model could be provided with spatial data from a GIS. The model, after performing hydraulic or hydrologic computations can return the computed values to the GIS for further spatial analysis. Djokic *et al.* (1996) also suggest that a GIS can be used as an interface development tool for modelling in that 'hydrologic models often have antiquated user interfaces that can be replaced by user-friendly interfaces developed using GIS tools'. This synergistic interaction between a GIS and hydrologic model depends crucially on whether data (files) can be exchanged between the two systems.

Decision making is therefore heavily dependent on the complementary relationship between historic data and modelling. Although hydroinformatics has its roots in simulation modelling it is as much concerned with decision making based on historic data collected from the *real world* as it is on results from models. What is more, it is concerned with how information technology can improve the acquisition, archiving and analysis of data, integrated with modelling, to provide better decision making at every level of management.

5. Hydroinformatics and Information Technology

Information technology is a term that was coined at the beginning of the 1970s; see Green (1991). It was conceived as a popular generality to cover the emergent disciplines of *computer science, microelectronics and telecommunications* addressing the production, storage and transmission of a wide range of information in ways that are said to revolutionise contemporary society. A contemporaneous discipline or term is *informatics*, sometimes used in continental Europe to refer to the production, storage and transmission of scientific and technical documentation.

Computer science is the study of computers, their underlying principles and use. Essentially, it is a systematic body of knowledge with a foundation of theory that helps to deal with the practical problems of design and construction of useful systems within constraints of time and budget. The practical orientation has motivated many to argue that it is as much a branch of engineering as it is a science. Computer science has also been defined as 'the study with the aid of computers, of computable processes and structures'; see Walker (1995). For most people, therefore, computer science is about programming languages and operating systems. More informed people will suggest that computer scientists work on compilers (programs that convert programming language instructions onto the language of binary digits), the analysis of user requirements, and data base management systems. In particular, computer science can be divided into four major areas of activity: *software engineering, hardware systems, theoretical computer science, and applications and uses.*

Software engineering encompasses programming languages, operating systems and other topics mentioned above. But the remit of the subject is much broader. Software systems should be treated as engineered artefacts that can be planned, produced, delivered, maintained and decommissioned within time and within budget. Thus, programming for a software engineer ideally should include systematically eliciting user requirements and generating abstract designs for them, and then writing a program that is as error free as possible and that may ideally be proven to be mathematically correct.

Hardware systems include hardware design, testing and architecture. Theoretical computer scientists study aspects of logic, mathematics (including set theory) and statistics with a view to developing frameworks for program language design, or for proving programs formally correct. Applications of computer science cover broader aspects like scientific computing (defined as processor intensive computations with low volumes of data) and business computing (data intensive but with lower processing demands) on the one hand and off-line computing and real-time computing on the other. Increasingly, computer scientists have been discussing the social, economic, ethical and political impact of the uses of computers. Well established branches of computer science include information retrieval and cybernetics. Table 2 shows some of the key developments in computer science in the last three decades:

TABLE 2. Key developments in computer science

Key Area	Exemplars	1950-70	1970s
Programming	Languages/ Compilers	Procedural (FORTRAN, Basic, C)	Declarative (Prolog) Functional (LISP) Object-oriented (C++)
Information Structures	Types	Constants, variables, arrays, records	Linear lists, trees, graphs, sets
Applications Software	Enterprise oriented	Scientific, business	Educational, recreational
	Task oriented	Tabulation	word processing; spreadsheets; diary management
Systems software	Operation Systems	Hardware specific, e.g., IBMOS	UNIX, MS Dos
Systems Analysis and Design	Analysis/ design	Diagrammatic	Analytic

The umbrella term *computing techniques* covers a range of topics, which happen to be very relevant to hydroinformatics, including *graphics, simulation, artificial intelligence, and neural networks.* These very active branches of computer science, together with computer-aided software engineering, enable the users of computer systems to best exploit the underlying information technology.

Microelectronics is defined as the technology of constructing and utilising complex electronic circuits and devices in extremely small packages using advanced manufacturing techniques. (Tele-)communications deal with the transmission and reception of data-carrying signals, usually between two widely separated points within local area or wide area networks either through terrestrial circuits, including fibre optic cables, or through communication satellites.

The advent of information technology, that is, of low-cost and easy-to-use computer-controlled communications systems and remote computing based on ever faster and smaller 'micro chips', led to a number of opportunities exploiting another set of technologies namely the enabling technologies. Consider, for instance, the programming bottleneck. Anecdotal evidence suggests that whilst during one decade hardware productivity increased 100-fold, the productivity of a programmer only increased by a factor of two at the most. In a similar vein, we hear of large software projects costing twice as much as budgeted, systems delivered late or never at all, and containing so many bugs that often the cost of bug-fixing exceeds the cost of the original investment. Concern about these problems has lead to the adaptation of engineering practices and the evolution of scientific practices for defining, designing, developing and maintaining software systems; that is, software engineering has emerged as a specific discipline. Computer-aided software engineering (CASE) tools have helped in the development of well-engineered artefacts designed to meet end-user requirements and produced within time and budgetary constraints. The use of such tools has increased software productivity and assisted with quality control. Software engineering also covers topics as diverse as software re-use, software reliability and safety, software documentation, and formal methods for specification and design of software systems. Computer-aided software engineering, an enabling technology, is therefore a subdiscipline of software engineering. The emphasis is on developing systems that will help to automate as much as possible the various phases of software development.

Developments in software engineering and telecommunications have helped to solve a number of problems in wastewater management, from the design of sewer networks to real time control of systems. Awareness of the significance of software engineering led to the establishment of commercial software houses such as Wallingford Software within HR Wallingford which is an international engineering hydraulics laboratory. Other similar laboratories such as Danish Hydraulics Institute and Delft Hydraulics have strengthened their computing departments. Each of these organisations now describes its programs as software *products* with version control, debugging protocols, alpha and beta testing procedures, delivery protocols, hot-line support, user groups and so forth. Most of this organisational change can be attributed to developments in software engineering with the emphasis on the production of quality-controlled software systems within time and budgetary constraints.

In effect, the software developers and managers in hydraulic, hydrological and water resources management computing were amongst the vanguard of hydroinformatics in that these workers adapted techniques of software engineering for problems related to the management of the aquatic environment. Consequently, they raised the status of their simulation modelling programs from research laboratory prototypes, informally specified, casually designed, to engineered products that take cognisance of their users, are formally designed and are generally well documented. Today, products such as HydroWorks from Wallingford Software and MOUSE and MIKE11 from Danish Hydraulic Institute have achieved the standard of commercial office systems and sell in hundreds of copies worldwide.

A particular feature of such products is the extensive use of the latest advances in graphical user interfaces. Alternative input devices include windows, icons, menus and printing devices (WIMPS) together with colour and facilities for resizing, etc. The windows typically contain textual and graphical information with slide bars, radio buttons, video controls, check boxes and pick-lists. Sophisticated data visualisation techniques such as contour mapping and hidden line removal have been applied in computational fluid dynamics, heat transfer engineering and computer-aided engineering to assist in comprehending energy and mass flows to and from various sources and sinks. The value of these data visualisation techniques can be seen in the display of results by the Systeme Hydrologique Europeen; see Abbott *et al.* (1986) and Abbott (1991b). Abbott (1993) also describes a more comprehensive use of GUI technology and data visualisation in ROSA for the real time control of urban drainage systems. The control panel of ROSA can be used to open a number of windows, some showing level and discharge sensor readings, to monitor the consequences of pumps and weirs (which are also displayed in windows).

The incorporation of major advances in human computer interfaces in computer systems used for solving problems related to the aquatic environment, especially in the management of urban drainage, also signals the emergence of hydroinformatics, that is, the systematic adaptation of IT for solving water related problems. The next step may well be the regular use of virtual reality systems that enable users to have a yet more profound insight into the behaviour of the aquatic environment.

Despite the increase in programmer productivity, it can be argued that most currently available programs encrypt knowledge, perform repetitive computation and work on large databases. Human beings, however, use heuristics. These are the opposite of algorithms in that heuristics do not guarantee a correct or even any solution. Humans also tend to make inferences about missing data, and appear able to compute on the basis of incomplete or partial data. Above all, humans 'represent' their knowledge. This involves the use of knowledge primitives, and a variety of abstractions, theories and formalisms, to efficiently store and retrieve knowledge, particularly knowledge gathered from experience. Knowledge base expert systems (KBES) offer new possibilities of replicating some of these human thought processes. The purpose is to make knowledge explicit rather than storing such vital information in data structures that effectively encrypt knowledge (see Table 3).

TABLE 3. Comparing and contrasting a conventional program and a KBES

	Conventional Program	**KBES**
Represented entity	data	knowledge
Reasoning	algorithmic and repetitive	algorithmic and inferential
Effectively manipulates	large databases	large knowledge bases
State of knowledge	encrypted	represented

Artificially intelligent programs will help to improve the utilisation of existing developments in computing through the explicit representation of heuristic knowledge, the provision of reasoning strategies for searching on this explicitly represented knowledge, the inclusion of explanation facilities (for explaining why a certain input is required), and the adoption of good visualisation tools. Such features are essential to hydroinformatics if it is to facilitate better, more informed decision making.

6. Hydroinformatics and Artificial Intelligence

Artificial intelligence (AI) is a recently coined term (c. 1956). It refers to a branch of computer science that aims to study human and animal intelligence through the construction of computer programs. These artificially intelligent programs are used for solving problems in narrow domains or are used for evaluating theories from psychology, sociology and biology about human intelligence. Among the better known artificial intelligent programs are the so-called knowledge-based expert systems, natural language processing systems that can process written and spoken human language and mimic humans, and computer vision systems that can 'see' images, scenes and pictures in a manner like humans and animals. More adventurous authors talk about machines with artificial consciousness and systems with artificial morality! A more recent addition to the literature in computer science is that of distributed artificial intelligence, that is, artificial intelligent programs acting as individual agents either autonomously or under the control of a 'supervisor'. Each agent has its own 'mission statement' encoded inside it and interacts with other similar agents through human notions such as co-operation, negotiation, conflict, collaboration and adversity. Such agent programs are also referred to as intelligent agents. For an introduction to AI see Winston (1992), Rich and Knight (1991) or refer to handbooks by Barr and Feigenbaum (1981, 1982), Cohen and Feigenbaum (1982) and Barr *et al.* (1989) or to encyclopaedias such as Shapiro (1992).

Knowledge-based expert systems (KBES), variously referred to as knowledge-based systems or just expert systems, have had more impact on engineering sciences research than any other branch of AI. A KBES comprises a knowledge base and a reasoning engine. The knowledge base is a systematically organised collection of IF-THEN type rules and a set of specialist facts about a domain. The reasoning engine selects the appropriate rules and facts pre-stored in the knowledge base for solving well defined problems in the domain, given input through the user interface. The end result of the reasoning process is displayed through the user interface together with any explanation of the KBES's input requirement and justification of its output. This is in contrast to the operation of a database management system such as ACCESS or ORACLE where the database only contains facts and the reasoning engine is merely a retrieval algorithm. The IF-THEN type rules, usually acquired from domain experts, can be classified in several ways. For example, causality rules relate causes with effects and vice versa; patronymy rules relate parts to the whole and the whole to parts; and material rules relate materials to artefacts. In addition to the rule base, a knowledge based expert system also has a fact base that contains key facts about its domain. For example, the fact base may include a taxonomy of structures in a structural design expert system or the properties of pipes, manholes, pumps, tanks and overflows for designing or controlling waste water networks.

Knowledge based expert systems have been developed for a number of engineering sciences including civil, chemical, mechanical, electronic and aerospace to name but a few. Within civil engineering, by and large, KBES have focused on problems of structural analysis and design, and the related areas of construction management (Dym and Levitt 1991), earthquake engineering and geotechnical engineering. There have been, however, a number of research applications in water sciences and engineering, in particular to the rehabilitation and design of sewerage networks and water distribution networks. As in many other fields of engineering very few of these research systems in civil, water and public health engineering have been transferred to real world operational systems. This may largely be due to the fact that, like simulation modelling in the 1960s, AI in general and KBES in particular are technologies which have yet to gain the confidence of the engineering community. Expert systems have nevertheless shown the way in acquiring and validating experience-based knowledge.

Broadly speaking there are two major categories of KBES designed to solve particular problems. In the first category there are systems that interpret data resulting, for example, from other programs, sensors, databases and files for the purpose of either monitoring, diagnosing, predicting or controlling plant, machinery or other programs. The second category of KBESs assist in the construction of artefacts such as networks, assemblies or medical drugs that involve components which have to satisfy spatial and temporal constraints. KBESs designed to construct artefacts include systems that can help in engineering design, such as configuration and planning, specification of constraints and assemblage of artefacts.

HYDRO was one of the first expert systems used in water science and engineering; see Waterman (1986) and Reboh *et al.* (1982). It acted as an intelligent front-end for the Stanford Watershed Model, particularly the program HPSF that simulated the physical processes by which precipitation is distributed throughout a watershed. HPSF uses information about soil types, vegetation, land use and geology. HYDRO assisted its users to estimate values of associated parameters and could cope with uncertainty through the use of Bayesian statistics.

Over the last 20 years the University of Surrey, HR Wallingford and Wallingford Software, in close co-operation with water utilities, consulting engineers and universities in the UK and the European Union have developed a number of knowledge-based expert systems for managing aspects of the aquatic environment. During the 1980s the focus of developments at Surrey was targeted at the introduction of KBES technology to the water industry in the UK. The first KBES was an advisor for the use of the Wallingford Storm Sewer Package (WASSP), called the WASSP Intelligent Front End (WIFE). This system was incorporated into a Sewerage Rehabilitation Planning Expert System (SERPES) to assist with the development of a WASSP simulation model for an urban drainage network; see Ahmad *et al.* (1987). SERPES helped build the model, execute it and use the results to identify the most cost-effective upgrading options to deal with performance problems. It was based on the UK Water Research Centre's (WRc) Sewerage Rehabilitation Planning Procedure (1986). Another expert system developed in the same period was the Water Distribution Network Control Expert System (WADNES); see Ahmad *et al.* (1988). The intention here was to demonstrate how KBES technology could assist a distribution network control-room operator on the event of an emergency, such as pump failure in the distribution network. WADNES functioned by comparing the status of a given network such as pressures and flows with templates that identified various emergency situations and were generated by the KBES previously. The system was based on the operations of a control room in Poole, Dorset, England. Both KBESs were built by the University of Surrey as a project sponsored partly by a 17 member consortium that included UK water organisations, engineering consultancies and the UK Government's Department of the Environment; see Walker (1988).

The University of Surrey also developed a program for building expert systems, namely the Water industry Expert Systems Environment (WIESSE). This was used for both SERPES and WADNES; see Hornsby *et al.* (1987). WIESSE was subsequently used to build two more expert systems: Q2X to predict algae growth in storage reservoirs (Ahmad 1989), and an expert system for inspecting and maintaining offshore structures; see Ahmad *et al.* (1990).

The development of these various expert systems was motivated by the need to introduce expert systems to the water industry. In each case the principal deliverable was a prototype system. The above mentioned expert systems projects were, however, different to other projects of a similar kind in three important aspects. Firstly, the principal objective was to stress the efficacy of the expert systems technology in acquiring experiential knowledge. WADNES and the offshore expert system were developed for domains where there was little or no knowledge of operational control or inspection and maintenance protocols. SERPES demonstrated through its knowledge base of 800 rules how a well documented strategy could be put into action. Secondly, each of these systems was built using a well established software engineering methodology, that is, the 'waterfall' model; see Birrell and Ould (1985). This model recommends that user requirements be collected and analysed, a detailed software design based on the requirements specification is prepared with appropriate test plans, and the software is implemented, documented and tested. Thirdly, each system integrated well known simulation models that were available commercially.

The expert system development methodology evolved by the University of Surrey and HR Wallingford is shown in Fig 1. This approach to expert system development is integrative; that is, an information environment consisting of KBES, simulation engines and databases is put together following a systematic analysis of user requirements, followed by the design and implementation of those requirements. Such an approach forms a basis for developing hydroinformatics systems.

7. Hydroinformatics and Digital Libraries

Subsequent to the KBES developments above, co-operation between the University of Surrey, HR Wallingford and Wallingford Software led to three major projects. The first was an information environment developed for the UK Environment Agency (then the UK National Rivers Authority) for water resources management (1990-1994). The second project centred on an information environment for disseminating safety-related advice through the various stages of sewerage rehabilitation planning (1993-1996). Finally, the third project involved the development of the Urban Drainage Modelling Intelligent Assistant (UDMIA). This was a 'toolkit' designed to provide guidance, instruction and support for training on key topics in network modelling, especially urban drainage. UDMIA was developed jointly by the University of Surrey and Vrije Universiteit Brussel.

The first of the three projects was focused on the development of an Expert Licensing Information environment, ELSIE, that could help in issuing licences for the abstraction of water from rivers and from groundwater resources; see Ahmad *et al.* (1994) and Ahmad and Griffin (1995). ELSIE was developed with the co-operation of four regional offices of the UK National Water Authority (now the Environment Agency). It could advise on whether or not an abstraction licence could be issued, remind the user on licences pending, calculate drawdown on resources and compute water requirements of crops. An innovative part of the system was its access to a digital library of legal hypertexts including the UK Water Resources Act (1991) and Wisdom's Law of Watercourses (1990) which is an interpretive text of case law prior to the Act. The rules in ELSIE were cross-referenced to the relevant parts of the texts with a help facility to provide users direct access to the texts as required.

The second project concerned the safe design of networks using information systems (SAFE-DIS) and was sponsored by the UK EPSRC and the UK Department of Trade and Industry under the Safety-Critical Software Programme. The SAFE-DIS prototype, developed in conjunction with four UK water companies (Thames, North West, Severn Trent, and Yorkshire) together with Montgomery Watson plc, animates the behaviour of an experienced engineer carrying out tasks in sewerage rehabilitation design. These tasks are selected from the four phases of WRc's Sewerage Rehabilitation Manual (SRM) procedure. The objective is to guide a less experienced engineer through the execution of the tasks. During each task the SAFE-DIS prototype offers advice and

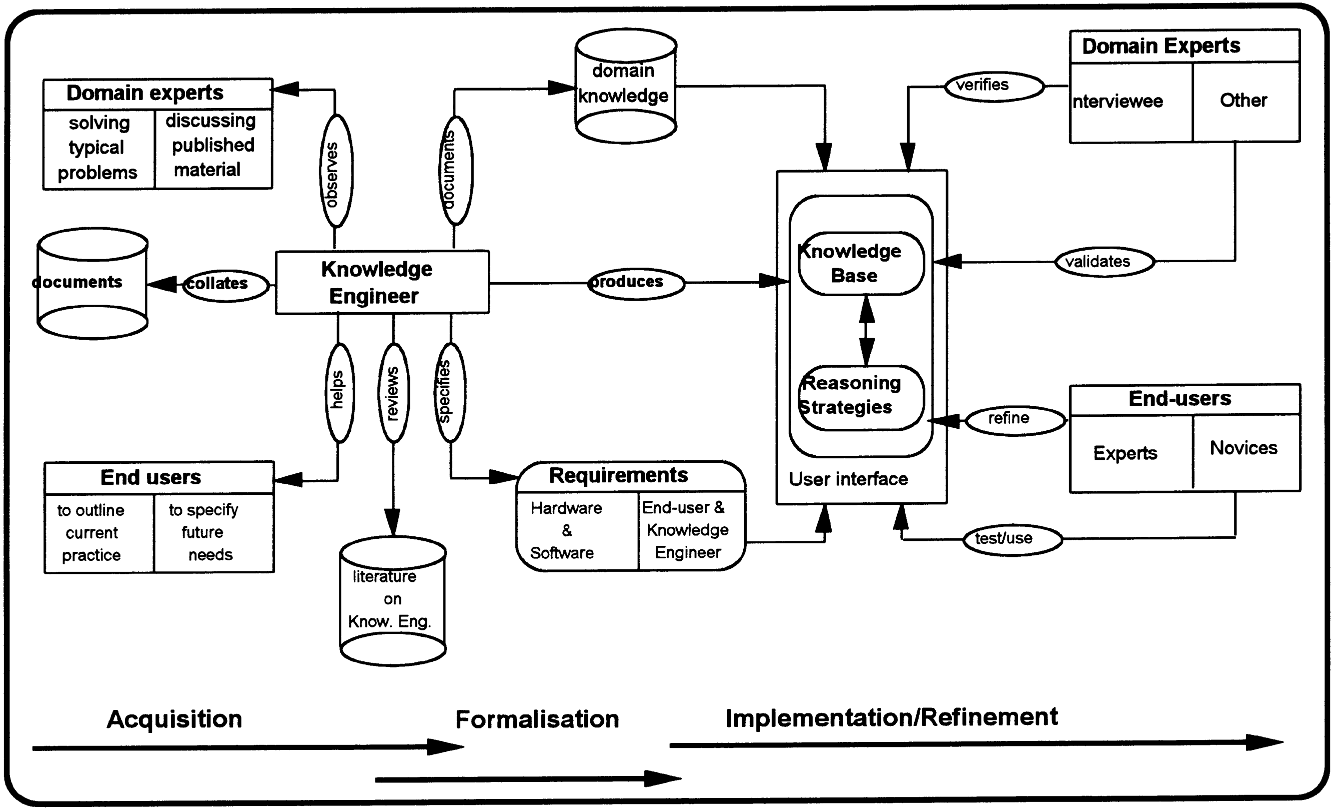

Figure 1. Expert system development methodology.

displays safety warnings. These warnings were extracted from extensive interviews with sewerage rehabilitation experts from the participating companies. Transcripts of each interview formed the basis of a digital library that also included other sewerage rehabilitation planning texts, legal documents and user experience. In addition, SAFE-DIS includes a chronology of tasks performed and decisions made by the user as a basis for an audit, links to the Internet and text analysis software systems, and interfaces to simulation modelling system for sewerage networks, namely HydroWorks, and risk analysis software; see Ahmad (1997).

It now appears that a hydroinformatics system that provides intelligent advice or control should be able to perform the following functions:

TABLE 4. Functions performed and software required for a hydroinformatics system

Function		**Software System**
Access	Rules and heuristics	Knowledge-base Systems
	Electronic documents	Full-text &Hyper-text Management Systems
	Network-specific data	Geographical Information & Data-base Management Systems
Model	Complex Networks	Network Simulation Systems
Audit	Modelling Effectiveness	Workflow tracking Systems & Report-generating Systems
Sensitivity	Reliability Analysis	Risk Analysis Systems

8. Hydroinformatics and Knowledge-based Simulation Systems

Knowledge-based expert systems are now being used to build prototypes that model the behaviour of systems for which it is not easy to find analytical solutions, but where solutions can be found in terms of 'rules of thumb' or heuristics. This adopts the view that the 'real world' can be replicated through a judicious synergy of analytical algorithms and heuristics. AI researchers such as Round (1989) suggest that we should investigate the 'potential of integrating knowledge-based techniques with simulation for the purpose of predicting complex system behaviour'. The 'integrated' systems will be called knowledge-based simulation systems. These are at the heart of hydroinformatics.

Round defines three kinds of integration: sequential, parallel and intelligent front-end.

AI systems began life as all-purpose problem-solving algorithms and graduated to domain-specific knowledge bases. Simulation systems started as custom-built domain specific systems and progressed to become general purpose modelling environments. Both neglected the societal and linguistic aspect of human knowledge and relied on one expert and a highly encrypted representation of knowledge. More recent systems are beginning to deal with these issues and thereby contributing to the emergence of hydroinformatics.

TABLE 5. Round's (1989) classification of knowledge-based simulation systems

System Type	Information Flow	Examples
Sequential Integrated	Knowledge-base component → **Results** → Simulation Component	HYDRO → **Select Parameters** → HPSF (Waterman (1986:263)). SERPES → **Select Parameters** → WASSP (Ahmad *et al.* (1988)).
	Simulation Component → **Results** → Knowledge-base component	Drawdown Calculations → **Drawdown** → W-RAISA (Ahmad and Griffin (1993)) Drawdown Calculations → **Drawdown** → ELSIE (Ahmad *et al.* (1995))
Parallel Integrated	Knowledge-base Component ←→ **Results** ←→ Simulation Component	Drilling Rule-base → **Modify Parameter** → Tolerance Calculation (Brown *et al.* (1985)). Drilling Rule-base ← **Sample** ← Tolerance Calculation
Intelligent Front-Ends for Numerical Simulation Models	Domain data-base → Knowledge Base **Build & Execute** → Simulation Components	Ecology data-base → Encode user description as a Mathematical Model → **Build & Execute** → Mathematical Simulations in FORTRAN and C++ (Meutzelfeldt *et al.* (1986))

9. Hydroinformatics and the Sub-symbolic Paradigms

As Abbott (1991) points out, 'a very large part of AI is currently taken up with *symbolic processing*'; that is, with facts and rules in symbolic form that are manipulated by explicit logical reasoning to deduce conclusions, also in symbolic form. Systems based on such symbolic processing can only accept new concepts if those concepts have already been anticipated and logically consistent connections made to accommodate them. What is also needed are systems that can absorb new concepts and knowledge without having made prior connections, that is, systems that can learn. Such systems are identified as being *sub-symbolic.* The sub-symbolic paradigm that is most readily recognised at present is neural networking. Neural nets are endowed with unique attributes that are ideal for dealing with change. In particular they can help establish complex non-linear relationships between inputs and outputs.

There are two broad types of learning algorithm: supervised and unsupervised. Neural nets using supervised learning algorithms rely explicitly on an 'external teacher' to guide the learning process, whereas unsupervised neural nets have no such external tutor. Neural nets based on supervised learning algorithms are useful when finding subsets of vectors that provide the best linear, and sometimes non-linear, approximations to the input vector. Such nets have been used in adaptive control and for mapping text to speech. Nets based on unsupervised learning algorithm are used especially in tasks that require detection of features such as pattern recognition, motor control and language development. An important variant of neural nets is recurrent networks; that is, networks with feedback loops. These may involve the use of unit delay elements which result in a non-linear dynamic behaviour. Recurrent networks show considerable promise in time series analysis and real-time linear adaptive prediction of non-stationary signals.

Hall and Minns (1993), Minns (1995), Mason *et al.* (1996), and Mason *et al.* (1997) have used supervised neural nets for modelling the relationship between the depth of rain falling on a natural or urban catchment and the magnitude of the recorded or predicted runoff from the catchment for a given set of antecedent conditions. Neural nets using multi-layer perceptrons and radial basis functions have been trained to learn such relationships for a range of catchments exhibiting linear to highly non-linear responses. For a review of the ways in which neural nets can contribute to urban drainage modelling see Loke *et al.* (1996)

Sub-symbolic paradigms, such as neural networking, open up within hydroinformatics radically new ways of modelling, especially of ecological processes, and of exploring social interactions within hydroinformatics systems.

10. Hydroinformatics and Document Management

The chief output of an engineering study involving computational modelling is normally a technical document containing a description of the problem, how it was solved and the conclusions. This document becomes the record of the project. Communication of the information is done in the shared language of the client and contractor. As a document it evolves during the project in parallel with the progress made. This and associated documents are best communicated over a network linking various agents. The generation of such documents and the incorporation of data, results and animations from the modelling are themselves part of the working process and are therefore included within the hydroinformatics system. Acceptance of such documents in a non-linear hypertext form rather than a paper-based linear form will be another radical step forward. With these and other important documents in electronic form a considerable range of opportunities is opened up, such as automatic tools for generating hypertext links within and between documents based on automatic identification of technical terms, animation of tables, figures and equations, text summarisation, and document identification, categorisation and routing.

In examining the dynamics of the information life-cycle it is apparent that information flows between a wide variety of stakeholders in urban drainage. These include surveyors, scientists, engineers, managers, operators, maintenance gangs, administrators, lawyers, economists, financial experts, regulators, the public, pressure groups, emergency services, other utility services, and so on. Each person understands the information they receive or transmit according to their experience, training, perspective etc. Communication is, of course, helped if each person or group understands, appreciates and participates in a common discourse for urban drainage. In using natural language as the primary medium of communication between the people involved one of the major problems affecting understanding is the terminology that is used. This is because terminology is linked to the way terms are coined and used, which in turn depends on the world-view adopted by the individual. Inevitably, in widely dispersed discourse there are going to be people with different world-views attempting to communicate which means that the opportunities for interrupting the information flow are considerable.

Hydroinformatics practitioners need access to a range of information in the different categories. They therefore require an information system that not only provides access to the typical simulation and modelling engines in current use and the associated databases, but also to a variety of textual information, such as relevant legislation, standards and learned papers, technical manuals dealing with both quality and quantity issues, and a number of other documents essential to the pursuance of the work of the practitioner. Similarly, any such system should have document generation capabilities to convey conclusions and information from one user to another.

This introduces the need for hydroinformatics practitioners to communicate with others who are not as fluent with the use of specialist terms in hydroinformatics, and indeed, to communicate with others who may not be as fluent in the practitioner's own language(s). It is essential therefore that such communication be as unambiguous as possible. Documentation and translation experts have always insisted that, whenever possible, specialist communication within a specialism or across specialisms must use only standardised terminology. Standardised terminology is generally prepared and distributed by national or international standards organisations or by the learned body of the specialism. The terminology collection is usually available as paper-based dictionaries, but increasingly these collections are also being made available as computer-based terminology databases.

Specialist communication involves the use of standardised terminology, that is, the set of words that were coined or adapted by specialists for communicating knowledge of the specialist community and have come to be accepted by the community. These words, single or multi-words, are said to comprise the terminology of a specialist domain and in themselves are referred to as *specialist terms* or just *terms*. They are generally rooted in the language of the specialist who coined them, though sometimes classical languages such as Greek and Latin are used to construct the terms. No matter what the origin of a given term, the specialist generally adapts it to his or her own language. Thus the term may have a Latin origin but the specialist may well use a plural suffix or an adjectival affix that is based on the rules of grammar that he or she is used to. This will appear to be a travesty to the language purist, but language is not the province of the purist but is ever changing in response to usage. Terms may be coined in conversation between experts or dreamt up by a practitioner to convey succinctly a concept or idea. Generally, however, they are created, or at least one becomes aware of their existence, in the texts of the given specialism. Interestingly, hydroinfomatics is a young discipline and is essentially synergistic in that it adapts techniques and technologies from other disciplines. Therefore its terminology is also adapted from other specialisms and is still in a state of flux. There is a growing corpus of texts, though it can be argued that as yet there are not enough texts that are distinctive of the subject. Time will show whether a clearly recognisable terminology for hydroinformatics will emerge. The possibilities are that it will, simply because of the following that it has attracted.

A systematic collection of terms is called a terminology database. Sager (1990) has defined such a database as 'a collection, stored in a computer, of special language vocabularies, standardised terms and phrases, together with the information required for their identification. The terminology database can be used as a mono- or multi-lingual dictionary for direct consultation, a basis for dictionary production, a control instrument for consistency of usage and term creation and an ancillary tool in information and documentation'.

A terminology database can contain any number of terms, whether as few as 200 terms or as many as 250,000. A specialist dictionary is a good basis for a terminology database. Such dictionaries vary in the number of entries and extent of coverage. Some, for instance, are just glossaries of terms or long lists in one or more languages. Others, found as addenda to specialist textbooks, may be very specific and contain the names of terms together with their definitions. There are encyclopaedic dictionaries that augment the name and definition with grammatical category information, references, acronyms, synonyms and some commentary. These paper-based specialist dictionaries usually incorporate between 3-5 attributes per term.

In a terminology database each term is stored in a so-called record format that records the attributes of a term. Record formats include between 25 to 60 attributes that usually include the name of the term, its definition, grammatical category data, relationships to other terms, acronyms, deprecated form of a term, foreign language equivalents, notes on how to use the term, and so on. Table 6 contains examples of specialist dictionaries and terminology databases.

The management of terminology is not trivial and spans a range of interdependent phases, including: organising documentation and capturing data from running text (acquisition and elaboration); storing, editing, maintaining and updating data using various data structures (representation); and publication (dissemination). All of these phases may be carried out electronically, given the appropriate tools. The management of terminology may be viewed as an infrastructure need for translation, technical writing, and knowledge acquisition for expert systems. An integrated text and terminology management system, called System Quirk has been developed to provide these facilities; see Holmes-Higgin and Ahmad (1992) and Ahmad and Holmes-Higgin (1995). The system can be used to

- access terms from a terminology data bank or term bank
- enter terms in a term bank
- modify terms in a term bank
- create new terminology data banks
- customise terminology data banks according to the needs of an organisation

System Quirk has also been used to manage general language lexical databases.

TABLE 6. Details of some well-known specialist dictionaries and terminology data banks

Title (Year of Pub.)	No. of Terms
Specialist Dictionaries	
The Oxford Dictionary of Computing (1996)	6000
The Cambridge Illustrated Thesaurus of Computing Terms (1984)	10000
Multilingual Terminology Data banks	
University of Surrey's Catalytic Converter Term bank (Eng., German, Spanish) (1992)	12000
Canadian Government's NORMA-TERM (French-English; Tech)	47000
Terms of the Environment (the US Environmental Protection Agency, 1996)	updated regularly

There are very few large-scale terminology databases in water engineering, hydroinformatics or any related area. There is an urgent need to think about creating terminology databases, particularly for hydroinformatics. Such a move will be valuable for minimal-error communication within information systems, in teaching, in research, in business and in communicating with lay people.

The University of Surrey and Wallingford Software have been involved in creating terminology databases in urban hydrology and urban drainage for more than three years. The present terminology database comprises 2000 terms in English, Dutch, French and German. These terms have been fully elaborated and validated with the help of British and Belgian experts under a COMETT project; see Fulford *et al.*(1992). An example of a record format for a typical term in urban hydrology is shown below. The urban hydrology and drainage terminology database is freely available on the World Wide Web at the following address: http//csps03.mcs.surrey.ac.uk/cgi-win/bazaar.exe. In addition, a number of terminology sources on the World Wide Web are becoming available:

TABLE 7. Terminology sources

Subject	Repository	World Wide Web Address
General Purpose	Term Bazaar	http://www.surrey.ac.uk/MCS/AI/pointer/bazaar.html
	Eurodicautom	http://www.uni-frankfurt.de/~felix/eurodictautom.html
	The Devil's Dictionary	http://www.cs.uit.no/~frankrl/Devil/dd_.htm
	Dictionary of Roadie Slang	http://searider.jpl.nasa.gov/~gms/text/slang.html
Engineering and Technology	NASA Terminology Collection	http://www.sti.nasa.gov/nasa-thesaurus.html
	Free On-Line Dictionary of Computing	http://wombat.doc.ic.ac.uk/
	Software Engineering Glossary	http://dxsting.cern.ch/sting/glossary-intro.html
Leisure	Dan's Poker Dictionary	http://www.universe.digex.net/~kimberg/pokerdict.html
Food and Drink	Whisky Glossary	http://www.dcs.ed.ac.uk/staff/jhb/whisky/glossary.html
Flora and Fauna	Vascular Plants glossary	http://155.187.10.12/glossary/glossary.html
	Aquarium glossary	http://www.actwin.com/fish/glossary.html
Commerce	Real Estate and Mortgage Glossary	http://www.homebuyer.com/realestate/common.dir/glossary.html
	Credit, Financial and Legal Glossary	http://www.teleport.com/~richh/glossary.html

Hypertext has been very useful for *marking up* text. The marking up involves identifying key terms in a computer file and indexing them such that a hypertext program will recognise the indices. A user may browse through the computer file and when he or she sees the marked up term the user can, by merely pointing at the highlighted term, end up on another page of the computer file where the term is used again. More importantly, the user can ask for the definition of the term whilst browsing through the computer file.

The marked up terms can also be used in the creation of the contents table and document index, and to link one computer file containing a text to another computer file containing similar terminology. In marking up a computer file for hypertext browsing the keywords are usually identified by manual methods and the definitions are seldom entered. A terminology database will be very useful in that the database can be used to identify the terms. In addition, a program can be developed to read the terms of a terminology database and use this list to mark up terms in the document automatically.

A project, ELSIE, aimed at demonstrating the efficacy of knowledge-based expert systems in issuing licences for abstracting water from rivers and other underground sources has already been mentioned above. This project used a digital library of texts comprising just under 400,000 words; see Table 8. System Quirk was used to extract a number of terms automatically from these texts together with some of the definitions. Table 9 shows a selection of 'candidate terms' extracted by the System.

These terms, together with a number of single word and multi-word terms were extracted and used to mark up the largest of the texts automatically, namely The UK Water Resources Act (HMSO 1991) and a commentary on case law before the Act: Wisdom's Law of Watercourses (Wisdom 1970); see Table 8. These texts can be browsed independently on an IBM-PC compatible system. This means that while a licensing officer is dealing with an abstraction licensing application, he or she can quickly browse through the relevant legislation and the commentary to clarify minor points of law, such as the exact definition of a term, the section number of a certain part of the Act, and so on. The definitions of key candidate terms in Table 9 were extracted automatically from the texts using System Quirk.

The analysis of the texts identified in Table 8 resulted in the creation of a terminology database for water resources. This database was used by the knowledge engineers in the project, who were computer scientists rather than water resources experts, to build the knowledge base for ELSIE.

TABLE 8. Description of a 'digital' library for building the ELSIE project. The first three texts were obtained as computer files, and last two texts were created using a word processor.

Text Type	Title	Length (in words)
Legislation Text	Statutory Instruments	843
Official Documentation	Technical Officer's Report (Abstraction etc.)	8793
Interview Transcripts (4)	Ground/Surface water, Impoundments, Licensing Procedures	34667
Legislation Text	Water Resources Act 1991	127977
Learned Book	Wisdom's Law of Watercourses	223960
TOTAL		396240

TABLE 9. Candidate terms extracted by System Quirk from the four interview transcripts

Candidate Term	Candidate Term	Candidate Term
abstract water	aquifer area	groundwater abstraction
contaminated waters	aquifer parameters	groundwater abstraction licences
fossil waters	aquifer properties	groundwater areas
intercept water	confined aquifer	groundwater catchment
irrigation water	fissured aquifer	groundwater catchment boundaries

Systematically organised terminology was used by the ELSIE project team to document knowledge of experts and relevant legal knowledge. The hypertext marked-up legislation and associated documents were the spin-offs of a planned terminology management exercise. A similar approach has been adopted in a more recent project, Safe-DIS: Safe Design (of urban drainage networks) using Information Systems; see Ahmad (1995, 1997) and Chapter 7. The information system in Safe-DIS has facilities for text analysis and management and has access to an updated version of the urban drainage database mentioned above. These text handling facilities are seen as very important if hydroinformatics systems are to be accessed as much by non-numeric specialists such as sociologists, lawyers and decision makers as engineers, scientists and technicians.

11. Hydroinformatics, Distributed Information Systems and the Internet

Pundits have been forecasting the emergence of an 'information' society for some time. The European Commission has stated that the 'dawning of multi-media world (sound - text - image) represents a radical change comparable with the first industrial revolution' (EC White Paper: Growth, Competitiveness, Employment, 1994). Increasingly companies from all sectors of industry and business are becoming profoundly aware that the way in which they handle their corporate information will be crucial to their survival and future prosperity.

It is questionable whether industry and business is generating the demand for improved IT facilities or whether companies are being forced to climb on the 'band wagon'. What is obvious is that there is a rapid growth in cable networks, with a number of large business corporations developing their own information highways (Intranets). Many people are now reputed to be teleworking. With ready access to Internet and World Wide Web and improvements in security using these networks and facilities there is a greatly improved access to databases and information for a wide variety of purposes. Many of the new benefits are shown in areas such as preventative health care and home medicine for the elderly, opening up new possibilities for improving the quality of life for the average citizen. So far as business is concerned, management quality and speed of information are key factors for competitiveness. The new IT facilities make these factors possible for most companies.

The EC is determined to support these enhanced facilities through what it calls a 'common information area' comprising

- information in electronic form
- hardware, components and software to process the information
- physical infrastructure including terrestrial cable, radio communication networks and satellites
- basic telecommunication services, particularly electronic mail, file transfer, interactive access to databases, etc
- applications that are user-friendly with structured information
- users who are trained and aware

Such an information society provides a *reason d'etre* for the implementation of hydroinformatics systems to manage the aquatic environment better.

The Internet, in particular, is a global information link that, despite its informal organisation, links a large number of computer systems within the world's academic, research and increasingly business centres. It serves primarily to connect other networks on which programs and data may be available. The Internet provides services for transferring files, electronic mail and remote access to computers. Its high level services include the hypermedia document access system, the World Wide Web (WWW), and video-conferencing facilities.

Internet servers, that is, computer systems connected through the Internet, are being used to perform a number of services for those interested in the aquatic environment. These include, for example, software exchange and distribution, software sales and modelling services, and dissemination of academic and practical information.

Software is widely available in the Internet. For example, US-based Environmental Hydrosystems makes available at no cost a number of public domain and proprietary ground water, porous media and environment software and related information through WWW. Similarly the commercial laboratories such as Danish Hydraulic Institute, HR Wallingford (and Wallingford Software), Delft Hydraulics, and the Canadian Hydraulics Centre (Ottawa) advertise their software and in-house expertise in general hydraulic, coastal and river engineering problems, water distribution, ecological modelling and pollution monitoring and prevention also through World Wide Web.

The Canadian Hydraulics Centre advertises its 'framework for Hydroinformatics Management Systems - HYFO' (1996). HYFO comprises a physical and numerical modelling system, geo-spatial analysis and modelling systems, and knowledge-based decision support systems. The framework can be used to address problems ranging from coastal, river and estuary, port and harbour, and waterway management. It can also be interfaced to socio-economic databases.

WWW is used extensively by individuals and organisations for disseminating information related to the aquatic environment. Publishers of books and learned journals use the Web for publicity purposes whereby they make available the abstracts of current journal articles and contents of books in hydrology and water resources management. Some learned journals are now also available electronically and usually cost less than or equal to the price of their sister paper versions. A number of individuals and organisations make their papers and technical reports available through WWW: the Universities of Alberta (Canada) and New South Wales (Australia) are amongst those educational and research organisations that make their papers on hydraulics, urban hydrology and related subjects available on the Web.

The US Geological Survey sponsors a water network focusing mainly on groundwater resources. The Survey has sponsored a water resources website at the University of Southern Illinois through the good offices of the US Universities Council on Water Resources. The US Government's Department of Energy also sponsors a hydrology website at http://terrassa.pnl.gov:2080/EESC/resourcelist/hydrology.html. The University of Alberta maintains a server from which one can 'jump' to a number of water related university departments in North America and the European Union together with a number of research laboratories (http://maligne.civil.ualberta.ca/home.html). There are complementary geography servers (http://www.frw.ruu.nl/nicego.html) and computing and telecommunications servers. Some WWW sites are run by government agencies involved in sustaining the physical environment. The US Environment Protection Agency uses the WWW to release information about the legislative aspects of its programme. There is an entire text of key water-related laws available on the EPA website (http://www.epa.gov) together with interpretation of these laws by each of the states in the USA. The EPA also releases guidelines, technical notes related to the aquatic environment through its website; (there are semi-government and private sector organisations within the UK that sell the UK-related water laws on CD-ROM).

As an example of a hydroinformatics server see http://helios.emse.fr /~ankerrech/rrechen.html. This server has a multilingual front-end (French, German, English and Spanish) with a number of hyperlinks, that is, links to other servers catalogued under the following categories:

TABLE 10. Hydroinformatics server

Aquatic Environment & related topic	Information Technology	Enabling Information Technology	Others
Hydrology	Informatics	AI	Bibliographic sources
Hydrogeology	Numerical Methods	GIS	Addresses
Urban Hydrology		Internet	Key Events
Models in Hydroinformatics			

This table shows, in fact, the metamorphosis of hydroinformatics through an interaction of subjects related to the aquatic environment with IT and the enabling technologies.

12. The Emergence of the Term 'Hydroinformatics'

To date, the seminal text on hydroinformatics is the book with that title by Abbott (1991) though the term was coined by Abbott some years before. Gradually more and more researchers and practitioners have been using the term and its use has helped to fill out its meaning. For some of the definitions see Table 11:

TABLE 11. Some definitions of the term 'hydroinformatics'

Abbott (1991)	is the coming to presence in water technology of a new way of functioning of the numbers; it is a new way of creativity, of formation, of production, of activity in relation to the waters, realised through the intervention of the numbers; it is a new place where truth happens, between the waters and the numbers'.
Belleudy, Darlet and Sauveget (1994)	'is a domain of interaction of Information and Communication Technologies, Hydraulics, Computational Hydraulics and Environmental Technologies..... 'is a technology applied to make or maintain our environment acceptable, development sustainable, water quality better, aquatic ecosystems protected.
Larsen and Gavranovic (1994)	'handling the flow of information and knowledge within hydrology and hydraulics in all its aspects'.
Viera, Warren and Lingaard-Jorgensen (1994)	'combines and utilises a large number of different tools and techniques in order to fulfil its generic task of providing and processing information'.
Rodenhuis (1994)	'encompasses the whole range of information processing tools, and knowledge encapsulation tools used to transform information about the aquatic domain into useful answers for hydraulic problems'.
Abbott (1996)	'is the study of this union of human and artificial agents directed to meeting mankind's responsibilities, both towards itself and towards the rest of the water-based economy of nature, and these inseparably'.

In order for hydroinformatics to develop as a distinct discipline it has to keep generating the term 'hydroinformatics'. As it does so the nature of hydroinformatics will change and its unique emphasis will evolve. What is becoming clearer is that hydroinformatics is now rarely concerned with the elaboration of exclusively technical systems: it is most seriously concerned with exploring the development of complex socio-technical systems. It is also apparent that hydroinformatics may be expected to discuss particular processes in, say, ecology in ways that go against tradition, such as recreating the mental world of organisms, as agents, by using such knowledge representation schemes as classifier systems, and so introducing learning and evolutionary capabilities.

13. Resume and End Notes

This chapter has attempted to establish the nature of hydroinformatics by looking at the threads that connect it to other more traditional disciplines, with IT providing the primary integrating factor. Stress has been placed on the need for better communication within a computerised hydroinformatics system and with the people who use it.

In summary, hydroinformatics is about better management of the aquatic environment through appropriate adoption and integration of information technology. The roots of hydroinformatics are with computer-based simulation modelling of complex water-based systems such as drainage and river networks, estuaries and coastal waters. Such models are needed because it is impossible to appreciate the complex behaviour of the flows and associated physical, chemical and biological processes from data alone.

Hydroinformatics is also about better management of the *asset*s associated with the aquatic environment, that is, with the drainage network infrastructure, the river channels, flood plains and structures, and the estuarial and coastal zones. Here IT is used to manage all relevant physical, chemical, biological, sociological, economic, legislative (etc) data from acquisition through to archiving and analysis to application. Because most data is spatial as well as temporal GIS becomes a valuable tool for managing the data and extracting information for decision making.

Information technology is essential to hydroinformatics in that it deals with information as a resource within a specialist enterprise. For the creation, replenishment and usage of any resource one needs to acquire raw material, process it on demand and in a timely manner, and use the processed material responsibly and reliably. Furthermore, information management helps in understanding when to discard erroneous raw material or obsolete processed data.

The use of IT in water science and engineering is in part motivated by a need to manage *changing* information. An understanding of change is often gained directly by regularly acquiring data and interpreting the differences found. Such an understanding is complemented by the insights gained from physical or mathematical modelling of water flows and associated chemical and biological processes within particular assets.

A danger with present computational modelling tools is the tendency for some users to treat them as 'black boxes'. Such users, through pressures of time and convenience, run the risk of failing to be aware of the situations in which the software can be applied properly, of the need for a rigorous appreciation of the reliability of the input data, and of how to interpret the results correctly. The design of software interfaces is crucial in this respect. Advisory and supervisory facilities can help both to warn and educate the user who is prepared to receive appropriate advice and assistance. An example of these and other facilities is given in Chapter 7. The argument is that hydroinformatics is concerned with the reliability of decision making and therefore with a safe use of models and data.

The emergence of hydroinformatics has been motivated by the observation and experience that the simulation modelling of complex physical, chemical and biological phenomena depends to a large degree on six major considerations:

- understanding the underlying principles governing the phenomena
- abstracting the generic from the specific and the strategic from the situational; using conceptual idealisations, such as point mass, incompressible fluids, energy conservation and entropy decrease, to deal with the real world
- reducing a complex artefact or process to a set of mathematical equations
- reducing the real-world data to the input requirements of a model
- interpreting the results of a simulation
- discarding some results and applying others

The effective solution of problems in water science and engineering involves not only questions of accuracy of quantity and quality but also such issues as safety, cost-effectiveness, and compliance with legislation. Hydroinformatics is therefore as much concerned with textual and graphical information of all sorts as with numeric data from the real world and simulation models.

But probably the most radical aspects of hydroinformatics lie in new ways of working with data from the real world and in opportunities for new ways of communicating. For example, many of the chemical and biological processes occurring in the aquatic environment are extremely complex. Deterministic modelling struggles to accommodate them. Instead, sub-symbolic paradigms such as neural networking offer considerable scope for 'letting the numbers speak for themselves' and for generating new knowledge. Similarly, the computer is now as much, if not more, a device for communication as it is for number crunching. Facilities for communication open up new possibilities for the complementary integration of human and machine 'agents'. This is particularly significant in view of the fact that present-day computers are logically unable to replicate all of the relevant aspects of the human reasoning process; see Penrose (1996) for an argument founded upon Goedel's theorem. Hydroinformatics, therefore, has the responsibility to explore the whole range of socio-technology issues raised by the burgeoning progress of IT when addressing the management of the aquatic environment.

These and other opportunities raised by innovative information technologies will radically alter the perspectives of hydroinformatics in the future.

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MODEL DEVELOPMENT AND APPLICATION

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1. Introduction

When discussing the modelling of urban drainage it is easy to focus on the software tools and how they should be used to good effect. The danger with such an approach is that it ignores the context in which the modelling is done. In particular, it ignores the management information system. Therefore, before addressing the urban drainage modelling process we examine first the context in which it is set.

The essential element in any management information system for storm and wastewater networks is people. A given drainage organisation has a management structure that is designed to achieve particular goals in managing one or more assets. Such a structure should enable efficient and effective acquisition, archiving, analysis and application of information. To this end the organisation will have a number of existing modules to support the information management and decision making. These modules may include monitoring and survey systems, databases, data analysis, modelling software, GIS, CAD, computer hardware, communication networks, manuals, legislation and cost data, and so on.

It is helpful to identify the key categories of components in an information system. There are four broad categories:

- acquisition and feedback
- archiving and analysis
- modelling
- decision support

Consider these in turn. The first category, namely acquisition and feedback, covers the group of components that have direct interaction with the 'real' world and with the physical asset contained by it. (Remember, the real world is more than just the asset; it includes the whole context of the organisation and the wider society). The principal functions of these components are to acquire information and knowledge and

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to feed back instructions to the asset or to individuals or groups whose prime function is to engineer changes in the asset. Physical asset survey, catchment survey, flow monitoring data, water quality sampling, laboratory analysis, and CCTV images are all examples of data acquisition. There are, however, many more ways in which information is acquired. These include customer reports of flooding, comments by the maintenance crews on the state of the structures, the latest legislation and standards reports, costing information, even how the company's shares are performing on the stock market (if it is profit distributing). Feedback is characterised by instructions such as to make engineering alterations to the asset, switch on pumps, adjust weirs, and remove sediment. It is also characterised by reporting to regulators, providing information to customers, working with pressure groups and the public.

Most information is difficult and expensive to obtain. It is therefore valuable in its own right and should be properly archived and analysed. Examples include the physical survey of the asset and the catchment, the historic performance data in terms of flooding reports and flow surveys, legislation and standards, discharge consent licences, CSO compliance records, and catchment planning information. All of the associated information needs to be archived in a form that is readily accessible by a range of staff with a number of different purposes, roles and tasks. When received, the information and data are normally in a raw form. The archiving process, therefore, often has associated with it analyses involving integration or synthesis with other data. There may be the need to provide more extensive analysis to generate derived data for particular purposes.

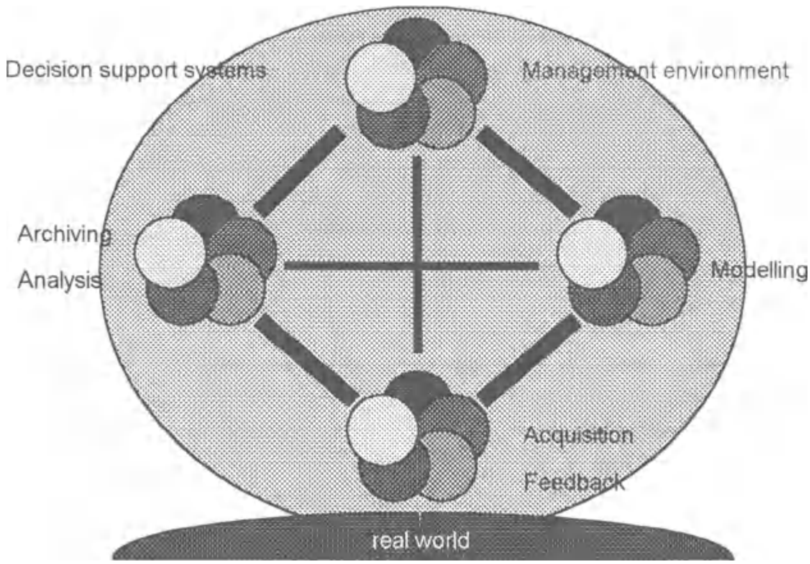

Figure 1. Framework for an information system to manage a water-based asset.

The third category is that of modelling. Data acquired directly from the real world and subsequently archived can only represent what has happened to date in the asset, recording a previous state of the asset, or describing the conditions by which it was affected. Some analysis can be done to try and predict statistical trends for what might happen, but, generally, extrapolation of such data to provide future predictions is limited. Nevertheless, decision making requires information on how the performance of the asset would be affected if particular events or circumstances occur. Without such information, decision making has to bear in mind the large risks that ensue. Any design therefore has to have sizeable, expensive and may be inappropriate safety factors. The deterministic simulation model provides an environment in which the necessary information for making better decisions with lower risk can be generated. A well-confirmed model, that is, a model whose behaviour accurately reproduces the performance of the real world asset for a range of particular recorded events, enables the engineer to predict what effect changes to the physical asset and its operation have on its performance. The model therefore becomes both a data *and* a knowledge generator. It can provide data on such details as the levels of service provided by the CSOs or the locations and degrees of flooding in the catchment. It can also give the engineer insight and knowledge about how and why the asset functions as it does, or would do if changes were made. This latter point is very important in that many engineers do not know how their complex drainage systems actually work. The mechanism for knowledge to emerge would include extensive use of graphics images by which large amounts of data can be readily assimilated.

The fourth category is that of decision support for management or business functions. In wastewater management there are a number of such functions, including planning and design, construction management, operations, maintenance, discharge consent and compliance for CSOs and treatment works, trade waste consent and compliance, liaison with the regulators and planning authorities, and so on. Most organisations have internal procedures to cover these functions, and many of these will usually be based on nationally agreed procedures. Where such procedures exist, then decision support systems can be introduced to make the implementation of such procedures more efficient and provide a dynamic environment to improve and enhance them for greater productivity, reduced risk, and better communication.

These categories of components are conceived primarily to reflect the working practices of an existing organisation. Twenty years ago almost all of these were supported by paper-based procedures. Acquired data would have been logged in hard copy. There would have been a filing system for paper containing text and plans. Engineering calculations would have been done by hand using slide rules or calculators. The management procedures would have been supported by hard-copy reference manuals. People responsible for different functions would have communicated and negotiated with their staff and other managers by word of mouth. Information and data would be gleaned for decision making, knowledge would emerge or 'come to presence' for those involved in making the organisation work.

What we have now are informatics tools and models to assist different staff in the organisation to carry out their tasks more efficiently and cost-effectively. Indeed the data acquisition, archiving and analysis systems, the simulation models, and the intelligent decision support systems, provide dramatic improvements in capability as well as carrying out some tasks previously done by people more efficiently. Indeed, the last ten years have seen a quantum leap in computing power, software development and communications hardware which in turn have generated the considerable potential for the new information systems that are now emerging.

2. Process modelling

One way of describing the complex process of urban drainage management is through process modelling. This incorporates concepts such as workflow but is inherently flexible as a system and can include changes while the system is in operation. Process modelling is concerned with a human agent's behaviour at the interface with, say, a computer and with the flow and transformation of data within an information system. It is even more concerned, however, with the behaviour characterising the interaction between human or machine agents.

The primary components of a process model are

- agent: a human or machine actor who performs a process element
- role: a coherent set of process elements to be assigned to an agent as a unit of functional responsibility
- artifact: a product created or modified by the enactment of a process element (Curtis *et al.* 1992).

A process can now be defined as 'one or more agents acting in defined roles to enact the process steps that collectively accomplish the goals for which the process was designed'. Similarly, 'a process model is an abstract description of an actual or proposed process that represents selected process elements that are considered important to the purpose of the model and can be enacted by a human or a machine'.

Process modelling is therefore made up of a number of separate processes, each of which involves one or more roles, with each role having several tasks to perform. The integration of sub-processes, roles and tasks involves communications and interactions between the roles with the initiation of a task dependent on the completion of one or more other tasks. So, for example, the overall process can be described in terms of a series of Role Activity Diagrams (RADs) (Ould 1995) which form the basis of the process modelling. The objective is to build a computer support system around the RADs such that staff can perform their tasks in communication with each other and supported by the system in giving timely access to databases, knowledge bases, documents, tools, models, etc.

It is important to recognise that 'process-driven environments will experience a gradual evolution in use rather than a revolution because they cannot be used effectively beyond the level of process definition that an organisation has been able to institutionalise' (Curtis *et al.* 1992).

3. Intelligent agents

Another way of looking at the information process is through 'intelligent' agents. Instead of focusing on a tightly defined process to solve management problems, the concept of intelligent agents minimises the notion of 'process' and concentrates on a loosely-coupled group of problem solving agents. These agents have the attributes of autonomy, communication, co-operation, negotiation, social law and co-ordination/control. There are many different types of agents: we are largely concerned with benevolent agents that work co-operatively towards a specific goal. They communicate not simply by message passing but also by propagating constraints. Consequently, a simulation of a particular system can be viewed as the interaction between agents that can help with the input of data, that are knowledgeable about the simulation model and its implementation, and that assist with the interpretation of the output data. The simulation process then resolves into a set of interactions between agents with specific roles. The agents interact by adapting their communication to solve the problems that arise. Therefore, the actual process followed will possibly be different for alternative problems (Selvaratnam and Ahmad 1995).

4. Computational modelling

4.1. GENERATIONS OF MODELLING

Abbott (1991) has identified five generations of computational hydraulic modelling:

1. algorithms for simple calculations
2. bespoke software generated within a modelling group (Sogreah 1956 onwards)
3. standardised modelling system (DHI 1970 onwards)
4. off-the-shelf modelling software including engine, interface and associated tools running on PCs and workstations (1983 onwards)
5. simulation engine, interfaces and associated tools with intelligent decision support facilities (about 1990 onwards)

These generations are comparatively easy to trace. They are heavily dependent on the growth in the power of computing hardware. There is also a strong dependence on software engineering techniques, including programming languages, interface development, and more recently databases and knowledge-based systems. Note that the fifth generation models have decision support facilities directly related to the modelling process. It is possible that we are seeing the emergence of a sixth generation in which the fifth generation models are seen *in the context of* the processes inherent to information systems as described above and which include decision support for business as well as technical processes. That is, models lose their present focus of attention, much as many car drivers may now never see the engine of their car. In what remains, our focus is on the engine within a fifth generation modelling tool. That is, there will be associated with the engine, technical decision support involving knowledge-based or expert systems. The nature of the engine is our primary concern.

4.2. REPLICATION OF THE REAL WORLD

The purpose of having a computer model for a sewerage asset is to generate information and knowledge for decision making. Therefore, the model should be able to replicate the real world. This means that the computational model should reproduce the behaviour of sewage within the asset with reasonable accuracy. In this way the results and insights gained from the model can hopefully be used with confidence in the management of the asset under the present conditions and in design of its rehabilitation and upgrade in order to meet the future needs. What level of confidence can be put in the results from a model is, however, a contentious issue. There are many factors that influence the reliability of the model itself. Here we address the issues surrounding the purpose and use of the model.

4.3. DETERMINISTIC AND STOCHASTIC MODELS

One approach to modelling could be to use statistical analysis of data. Such an approach would be very attractive to engineers who are used to working with data collected directly from the asset and intuitively tend to trust such data more than the results generated by a deterministic model. There are, however, difficulties in working with data models that strictly have no physical basis and therefore have limited capacity to forecast for events or circumstances outside the range of the original data. There is also the problem that such data models will generally have insufficient data to generate the detailed information required for many of the problems encountered in managing the asset. It should also be born in mind that, due to rapid urbanisation, the catchment characteristics may have been changed during the period of data acquisition. A much more attractive approach therefore is to adopt deterministic modelling techniques. These models are deterministic in the sense that they are based on numerical solutions of mathematical equations derived from information and knowledge of the various topological, land use and other characteristics of the catchment to be modelled, and of the physical, chemical and biological sub-processes (and, in the more advanced models, even economic, social and legislative 'sub-processes'). The generic, theoretical knowledge of the flow of (incompressible) fluids, the transport of pollutants, the chemical interactions between pollutants and the sewage, and the behaviour of biological entities within the asset under the physical and chemical environment, are all used to build a model that can simulate the performance of the asset under different present and future input conditions.

One should be aware, however, of the fact that models have to be applied for the conditions for which they have been developed. For example, the models developed and verified for modelling the rainfall-runoff under temperate climate conditions cannot be directly applied to the modelling runoff under snow melt conditions unless the appropriate modules for modelling the local physical processes are included; see Milina (1997). The adoption of a deterministic approach means that the uncertainty in modelling is reduced, and therefore the reliability is increased. This is due to a more realistic "mapping" of the real world and a better description of the system's elements.

4.4. SCOPE AND MAPPING

Which 'what-if?' questions can be addressed by a particular model depends on the scope and mapping of the model with reality. The scope is important because a model that only reproduces the physics of the flow cannot be used to answer questions about the biological processes. Similarly, if a model has been developed primarily for wastewater systems it cannot necessarily be used for storm water drainage. Careful attention, therefore, needs to be given to what sub-processes are included in a given model and how they are implemented in order to cover various climate, seasonal and similar variations. This is considered in detail below. The scope of a model will of course be driven by or determine the appropriateness of the particular application. So it will be important to establish that the model can be used for certain purposes such as design, analysis, forecasting, simulation, and so on. Further assumptions may be needed in order to cover the effects of different climates, seasonal variations of climates etc.

The mapping of the model with the physical asset is important because of the problem of interpreting the results from the model in the context of the asset. Normally this problem is addressed by having a one-to-one mapping between identified objects that make up the physical asset, such as conduits, junction manholes and ancillary structures (pumps, overflows, gates, outfalls, etc.), and the corresponding objects, namely links, nodes and structure links, as conceived in the model. The nature of the correspondence then makes the interpretation of the detailed behaviour in the model, and therefore in the physical asset, easier to appreciate and to understand. Again, the scope of the model has implications for the nature of the mapping. For example, rainfall-runoff from a large sewered sub-area may be modelled using a lumped conceptual sub-model. Such a model could be useful in the analysis of interactions of storm drainage system with treatment plants, CSOs activities and other "end of the pipe" problems, however they cannot be used to give information on the detailed behaviour of flows within the sub-area sewers. If this is needed, the sewers in the sub-area would have to be included as separate links in the deterministic model. Similarly, the mapping may only be between the monitored nodes of the asset and the model with conceptual links connecting the model nodes. Such a model may have the notion of storage for the conceptual links, much like the reservoir-type models. These 'simplified' models are advocated for simulation of long term input records, whether of rainfall or wastewater inflows, to predict the frequency of overflows or flows to treatment. There is considerable research going on into the production of neural network-type models to connect the monitored nodes in the asset; see, for example, Nouh (1996). These models may 'learn' from the deterministic model that contains a more detailed description of the real world network, or have 'learnt' from a number of previous networks and/or models.

4.5. MODEL AS A VIRTUAL LABORATORY

A deterministic drainage model that has the appropriate scope and necessary mapping with the real world asset is a valuable tool for decision making. However, it is probably more important to regard the model as providing a virtual laboratory in which experiments can be conducted. Such a laboratory becomes a 'place' in which the engineer can explore what happens or might happen in his real world asset, without having to wait for the right circumstances to occur. (Admittedly, the engineer may have the opportunity to make some experiments within the asset, but these will be extremely limited). Although there is no substitute for first hand experience of the physical asset for those who are responsible for operating and maintaining the asset, the right model can provide an environment in which the operatives can experience what might happen and therefore generate additional insight and understanding. Knowledge can be said to 'come to presence' through 'playing' with the model either in the analysis of full-scale problems or in running the educational versions for the sensitivity analysis. The value of this feature is highlighted by the difficulty that any engineer has in deducing how their system behaves. On being given a visualisation of the whole network the interrelationships between different parts of the system can be identified and detailed analysis of flows at any point can be made.

4.6. MODEL ASSUMPTIONS AND APPROXIMATIONS

By definition a model can only be an approximation to reality. There is little illusion that the processes involved in urban drainage modelling are extremely complex, let alone the representation of the boundary conditions in terms of the physical geometry of the asset, the above-ground catchment (for storm water drainage), the rainfall, and extraneous inflows such as the upstream inlets, in-catchment locations and downstream outfalls. Assumptions have to be made to circumvent ignorance of details of the processes involved. Approximations are introduced to make a modelling process tractable. Simplifications are made to accommodate limited computing power or to reduce the need to collect further data.

5. **Model development**

5.1. MODELLING PROCESSES

A modelling process for urban storm and waste drainage integrates a number of sub-processes. The main sub-processes include

- rainfall
- rainfall-runoff
- surface loading of pollutants
- rainfall-wash-off
- wastewater inflow
- groundwater infiltration and exfiltration
- network hydraulics

- hydraulics of ancillary structures
- surface flooding and overland flow
- sediment and pollutant transport
- interactions with other urban water assets

Details of the modelling of these processes are given in other chapters of this book. Each of these processes can be modelled in detail by models that are more or less sophisticated. Before focusing on the different approaches, attention will be given to the to scope of the model.

5.2. MODELLING CONCEPTS

As mentioned in Section 4, the level of complexity in modelling of an urban rainfall-runoff process in general depends on the objective of the modelling. From an engineering point of view, differences between a general master plan for a big community and the design of storm water pipes of a small sub-catchment are logical and self- explanatory. In general, one can distinguish between the following features of the model:

- Objective of the model (what is it used for)
- Level of complexity (which processes can be modelled in what detail)

Different objectives of a model are possible. A general difference is between a model designed (or used, or needed) for general master planning or for design of smaller pipes. The first objective can be thought of as catchment development or strategic planning. There is also the need to distinguish between models used for operational management purposes or as training tools. Operational management tools are more focused on significant results needed for the operation of sewer systems such as critical water levels at specific points of the systems whereas training tools are needed to illustrate and understand the model (either the processes or the model philosophy).

The need for development and application of modules of various levels of complexity is highlighted. Highly sophisticated models based on more realistic assumptions of the physical processes involved may be justified for detailed analysis of the rehabilitation of an existing network, analysis of the potential benefits of the introduction of source control concept, and so on. They require, however, a substantial amount of data on the physical characteristics of the catchment. These data are not always readily available. There is also the problem of building in a proper post-processor that will ease the problem of presenting the complex results of simulation in an easily understand form.

5.3. MODELLING SOFTWARE

Once the general concept of a model is clearly defined, the question arises: which modelling software should be used for the specific application? The development of software for a specific application is impossibly expensive and, in general, not necessary. This is more the field of scientific research and development, where new approaches are conceived and tested. More practical applications mostly rely on existing and available software.

The art of selecting an appropriate modelling tool from the existing stock has to be learned on the basis of the specific criteria such as:

- Are all the sub-processes that are needed for the particular application included in the package?
- Does the concept of the software fulfil the modelling requirements for the project?
- Is an extension of the software solution possible by incorporating additional processes, etc?
- Are the input and output data sufficient and do they fulfil the users' requirements?
- Is the software available for the appropriate operation system?
- Is training and support for the software solution and the modelling concept available?
- Is its cost affordable?

5.4. MODELLING DATA

Data are the essential component of each model. They can be grouped into two broader groups such as hard data that do not change in the given configuration (these are the data of the sewer system, terrain, land use, special structures, etc.) and the soft data like rainfall data and model parameters (Fuchs et al. 1993).

In principle, all the details of a sewer system (as it is) will not necessarily be used for the simulation within the model. The reasons are mainly given by the modelling concept and the objective of the study. The complexity of a sewer system can be adjusted (simplified or presented in a different form) when constructing the model while maintaining a good representation of its behaviour. Some means of adjusting the data include:

a. Pipes and branches can be removed from the model by assigning the separate contributing areas to one (conceptual) pipe representing the whole area. This will normally be done for all sewers smaller than a certain size. The size depends on the level of detail required; for example, replacement of the pipes of the diameter smaller than 250 mm is usual in a detailed model. This can go up to 500 mm in a coarse model used, for example, for master planning.

b. Consecutive pipe lengths can be combined into one length. This reduces the number of pipes in the model without reducing the total length of the system. The pipes should obviously be of similar size, roughness and gradient.

c. Pipes or structures can be combined into some other structure with the same equivalent hydraulic behaviour in order to simplify the representation of complex situations. An example might be where two parallel pipes of the same size and gradient are represented by one single pipe with an equivalent hydraulic capacity. Other examples are the representation of complex inlet and outlet arrangements for storage tanks and pumping stations by a single device, or the reduction of a complex urban drainage systems into an approach which can be used for operational purposes.

All forms of simplification tend to reduce the volume of storage available in the model when the system is surcharged and close to flooding. It is therefore important to make allowance for this by including extra storage volume otherwise the amount of flooding will be over-predicted.

Ancillary structures such as overflow chambers or pumping stations are built up from nodes and links representing the individual components of the structure.

On-line storage tanks can be modelled as a large manhole chamber. The flow control on the outlet can be represented by a weir, orifice or other control link. Open storage ponds can be modelled in a similar manner. Off-line tanks and ponds can be represented in similar fashion, and connected to the on-line node by overflow and return links.

A simple overflow consisting of a continuation link with no discharge control and an overflow link at a high level with no discharge control can be modelled simply by connecting the two links to the same node with no special conditions defined. A discharge control can be specified at the continuation link as an orifice, weir or other control link. The overflow can similarly be controlled with any type of control link. Other overflow links can be added at different levels to represent complex structures. The overflow links should all be able to flow in reverse when there is high downstream level.

A simple pumping station can be modelled as a storage volume at a node with one or more pump control links discharging into a common node. More complex stations can be modelled as well. There can be one or more overflows with flow controls. The situation commonly found in combined systems where dry weather flow pumps and pumps used for storms discharge to different locations can be modelled by two groups of pump control links discharging to two different links.

5.5. COMMON SOURCES OF ERRORS

Because errors frequently occur in data all data items should be checked carefully. It is often found that for pipes and nodes there can be an error either in the elevations of ground or in the invert level. Ancillary structures often show an inaccuracy for elevations or for pumping stations in switching levels. Another significant influence on the simulated results is the pervious and impervious areas connected to the drainage system. Because for many reasons these data cannot be checked in detail and are changing in time, a significant range of error may be generated. A significant reduction of this error can be achieved through calibration and verification.

With the introduction of GIS facilities to the modelling exercise care should be taken to check that the real and fictitious geometrical features of the information entered in the various layers of a GIS are correct. It is also important to ensure that the formats used are compatible with the ones prescribed. A trial plotting or some other visual display of data is recommended.

Once the model has been constructed and before any calibration starts the model should be tested to prove its stability under a range of flow conditions. It is preferable to test for at least three cases including the lowest and highest flows which are likely to be used with the model and also the most rapidly varying flows. These are the flow conditions that are likely to cause instabilities in the calculations. The testing should use a high level of results output and the results should be carefully inspected for any signs of instability or inconsistency.

If any problems are found then the way in which the system is represented may have to be changed. This process is made much easier when the pipe or node causing the instability is identified automatically.

5.6. VALIDATION, CALIBRATION, VERIFICATION AND EXTRAPOLATION

Inevitably the number and nature of the assumptions above mean that considerable attention has to be given to estimating their significance and ensuring that the assumptions are reasonable. Ideally there needs to be assurance that the resulting modelling software is at least reliable and can be used with confidence, even if the way the software is used is not as the author(s) intended. The validation of modelling software is well known to be extremely difficult and can only be done in strict terms for very simple software. Consequently, other ways have been suggested to at least substantiate the claims made for the modelling software. In particular, the European Hydraulics Laboratories have put forward a paper-based approach that provides evidence to substantiate the claims. The principle of the approach recognises the dependent sequence of steps in developing a modelling software product. One should distinguish between the actions taken by the software developer and the user. The software developer has to make sure that the software is implemented according to the original specification (validation), whereas the user instantiates models for one or more particular catchments using the software. The latter will perform a number of different benchmark tests on the resulting models before the software is used 'in anger'. The tests performed by the user may also include repetition of part of the original validation tests. Where necessary, feed back by the user to the developer can lead to improvement of the software. On the other hand, as shown in the Figure 2, in order to run successful simulations on his test case after creation of the model geometry the user normally performs the following tasks:

a) **Calibration:** to bring the results of a simulation in agreement with a selected set of measured values by adjusting the model parameters
b) **Verification:** to make sure that the parameters obtained in calibration provide predictions that are consistent with a set of measured values other than those used in calibration
c) **Extrapolation:** to simulate the performance of the system with a changed network geometry, catchment characteristics etc. in the absence of data obtained by measurements.

The validation sequence performed by the model developer can be said to consist of the following steps:

- identify the appropriate physical, chemical or biological processes
- reduce these process to their essential features
- conceive a structure to map modelling objects to the physical asset
- generate analytical equations or expressions to interpret the processes
- develop numerical algorithms to interpret the analytical equations and expressions
- produce a method for solving the integrated set of numerical algorithms

- generate the software to interpret the numerical algorithms and solution method within the modelling structure
- confirm the validity of the assumptions at each step using well-defined bench mark tests

Ideally the evidence substantiating the claims made is presented at each step. It is particularly important that the assumptions are explicated clearly and that their validity is confirmed through testing and demonstration. Particular types of question will be raised at each step. For example, the one-dimensional Saint-Venant equations for gradually varying flow are a pair of first order, non-linear hyperbolic equations. The derivation of these equations, whether from energy or momentum principles, leaves open the definition of certain coefficients. More importantly perhaps, there are a number of ways generating numerical equations using different finite difference schemes to interpret the analytical equations. The choice between the alternative ways of formulating the numerical equations is dependent on the handling of well known problems of numerical instability, accuracy and consistency, minimisation of numerical dispersion, maintenance of the same algorithm for different physical flow states (free-surface vs. pressurised flow, sub- vs. super-critical flow), treatment of low flow and dry bed conditions, preservation of volume conservation, and so on.

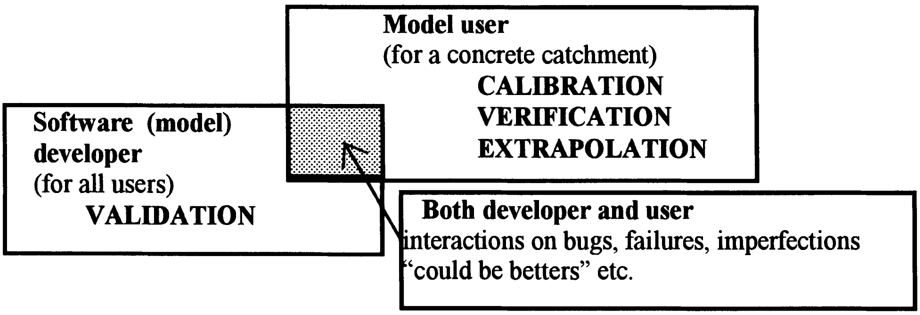

Figure 2. The roles of model developer and user in development and application of the software package and the areas of interaction and feedback.

As another example, there is the issue of robustness and reliability of the modelling software. It is very frustrating for users to experience the failure of software due to a violation such as a 'stack overflow', or being trapped in an infinite 'loop'. Careful programming and software engineering procedures can avoid many of these deficiencies, though it is well known that no significant software system is entirely free of 'bugs'. This highlights the fact that the results of any computer modelling exercise are subject to risk of failure or inadequacy.

Calibration is an optimisation or trial-and-error procedure in which a set of model parameters are obtained that minimise the difference between the measured and simulated values of the quantities being modelled (for example runoff hydrographs, pollutographs etc.). In event-based modelling the minimum difference should be recorded for both the runoff volume and peak flow. (Sometimes time-to-peak and an integral-square-error are included as well). In each case the minimum difference should be at least in a range comparable with twice the possible measurement error. This procedure should be followed for a set of different rainfall-runoff events. The possibility exists that the procedure could produce different sets of model parameters. In this case another optimisation procedure is needed to result in a single set of parameters. Alternatively, the concept of the model should be checked to ensure that all processes are included. In the case of continuous modelling a continuous series of rainfall events, that includes dry weather processes, is simulated and appropriate sets of "goodness of fit" are used.

Verification is the art of showing that a model does not disagree with the real system. It is therefore not the same as calibration, which is the art of adjusting a model to make it agree with the perceived performance of the system. The difference is best illustrated by considering what happens if the model does not agree with the performance of the real system. In verification we re-check the input data until we identify the errors and correct them. In calibration we arbitrarily change one parameter of the model until we get agreement for the conditions which we have observed.

Verification can never show that the model does agree with the real system because that would require checks on all possible flow conditions. However, we must check the operation of the model under as many different flow conditions as we can or as seem to reflect the range of critical situations. All available information should be used for verification. This can include:

- Reports of flooding during heavy storms
- Flow records at treatment works and pumping stations
- Marks of surcharge levels in sewers and manholes
- Continuous or ad hoc measurements of flows and depths made in the sewers.

It is important to note that records can be at different time scales:

- The maximum flow or level in an event.
- Continuous measurements at short time steps.

Each of these sources and types of data have their advantages and disadvantages.

Flood reports are notoriously unreliable. Many people do not report flooding for various reasons, and reports should always be backed up by asking residents in areas where flooding has not been reported whether any has in fact occurred. Flooding reports will often be due to temporary factors such as blocked gullies or sewers and may never recur.

There is another difficulty in comparing flood reports with the results of a model. This is because the flooding may not cause a problem at the point where it first occurs, but may run off across the ground surface to a low point where it does cause a problem. Unless the model includes a detailed representation of the ground surface and the possible flood routes this will not be reproduced correctly by the model.

If the large storm causing flooding occurred some years ago it may be difficult or impossible to get adequate rainfall data to use with the model. However, flood reports are useful because they enable the model to be tested at the point of failure, that is, under the conditions we want to study. They also have the advantage that they are available already before we carry out additional data collection for other forms of verification. At the present level of model development there is an apparent lack of a reliable tool (modules in the existing urban drainage models) that can combine the detailed digital elevation model with an advanced dynamic simulation of flooding in both cases surcharging of buried closed conduits or overflowing of the open channel.

A lot of data useful for verification can be obtained while surveying the manholes of the system by observing the marks on the manhole which show the highest water level. The disadvantages of this type of data is that there is no knowledge when the mark was made or in what type of storm. The data can best be compared with results from a synthetic storm of, say, a five-year return period which may be typical of the high water levels in the system. However, it is possible that the marks were caused by temporary blockage or maintenance work, and so they can be misleading.

Verification of models with measurements of flow and depth in the sewers is one of the most important parts of modelling an existing drainage system. This is normally done by an intensive survey using many monitoring points for a short period of about two months. However, the alternative of a few monitoring points for a longer period of time can also be used, especially in areas where storms do not occur frequently.

The number of monitors to be used depends on the purpose of the model and the level of complexity to which it has been constructed. For an initial planning model it may be sufficient to have monitors only on the major outflow points from the catchment. For a detailed study it is necessary to confirm that the model represents conditions throughout the system correctly and monitors will be required at many locations within the system.

If the survey is for only a short period then it must be remembered that there is unlikely to be a large storm during the survey period. The location of the monitors must therefore be planned carefully in order to get the best results from what storms do occur. To achieve this it is best if the model is constructed before the survey is planned so that the response of the system to typical small storms can be studied. For example, it is not usual to put monitors on overflows for a short-term survey unless there is a high probability of the overflows operating.

There are now several different types of flow monitors available with different characteristics and there is considerable experience in many countries of using these techniques for short-term flow measurement. Additional monitoring should also be carried out on the operation of important structures such as detention tanks and pumping stations during the survey so as to confirm the way in which they are modelled, that is, to calibrate the model on their characteristics.

5.7. MODEL UNCERTAINTIES

Every modelling has a certain range of uncertainties that are caused by inaccurate data as well as by the model which simplifies the complex hydrological and hydraulic behaviour.

Sources of uncertainties are for example the inappropriate mathematical structure of the model, the schematisation level of the data, the model parameters, the input data, initial state variables, numerical problems and manipulation and handling problems. To quantify the uncertainty we can do an analytical analysis like a first order analysis or a statistical estimation or numerical analysis such as a Monte Carlo analysis (Schilling and Fuchs 1986).

Examples show that the range of uncertainties or errors in a simple quantity simulation is highly dependent on the errors of the input data. If not processed properly, the data needed for the hydrological simulation like impervious and pervious areas could create the biggest errors. Maksimovic *et al.* (1994) present ways of using GIS to handle the processing and matching of elevation and network data with simulation packages to reduce the range of uncertainty. The full potential of GIS has still to be mastered by both model developers and users. But even with exact data a highly sophisticated model still produces a range of uncertainty when, for example, simulating flow over a weir under a complex hydraulic situation.

6. Applications

A simulation model of a sewerage network depends on the modelling software and the input data. There are several tasks that the modelling practitioner may be responsible for, including:

- Model purpose
  - system records assessment
  - asset data collection and validation
  - above-ground data collection and validation
  - catchment planning data collection and validation
  - event data collection and validation
  - historical records upgrade
  - system records upgrade
  - critical sewers identification
  - known performance problems assessment
  - flow survey management
- Model construction
  - asset data assembly
  - event data assembly
  - network simplification
  - storage compensation
  - runoff parameter identification
- Model development
  - asset data confirmation (verification)
  - sensitivity analysis
  - runoff calibration
  - wash-off calibration
  - prototype testing
- Model application
  - base performance analysis
  - infiltration and inflow analysis
  - performance optimisation
  - RTC design analysis
  - trade waste analysis
  - overflows analysis
  - flooding analysis
  - regulation to treatment works
  - sedimentation analysis
  - prescriptive conduit design and analysis (system's rehabilitation and upgrade)
  - prescriptive storage design and analysis.

A selection of these tasks can be brought together in a procedure. It should be noticed that, regardless of the level of model's sophistication, there will always be a need for the user to interpret both raw data and the results of simulation, and to reflect on how the problems can and should be solved. Details on approaches to analysis, design and operational management of a system are given elsewhere in this book.

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FUNDAMENTALS OF PHYSICALLY-BASED RAINFALL / RUNOFF MODELS[1]

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1. Introduction

Rapid urbanisation in many cities is not always accompanied by the proper development of infrastructure facilities. Extensive urban construction increases the impervious area, thus increasing the fraction of rainfall converted into direct runoff. The final result is the increase of both runoff volume and peak discharge, leading to property damage and inconvenience to urban inhabitants. Solving the problem of flooding in urban areas has two aspects: reduction of flood risk and damage, and control of pollution caused by urban floods. To cope with these in an efficient way, as a part of an integrated solution, it is necessary to have appropriate tools for analysis of physical processes involved, and for design and operational management of the storm drainage infrastructure system. Although the modelling of rainfall-runoff in urban areas is based on similar approaches as in natural catchments, the principal difference is caused by the details needed to model the underlying processes correctly. Also, there are many man-made structures in urban areas that either obstruct or facilitate runoff, and they have to be taken into account in detailed modelling.

Thus refined, physically based, realistically calibrated and robust modelling is required. The lumped or black-box models, that oversimplify the process, are unsuitable for this environment as soon as we have to tackle the problem of network design, analysis of retention facilities, flood pattern analysis, etc. Therefore, advanced models with high level of discretisation and realistic presentation of temporal and spatial distributions of both rainfall and runoff have been developed and are being used for various tasks.

The hydroinformatic tools developed recently can facilitate and raise the reliability of the analysis.

To carry out these tasks, the simulation models have to be supported by a relevant data base describing topography and land use of surface layers, the underground sewer network, structures, soil characteristics, model parameters etc. In formation of the appropriate related databases, in addition to manual data preparation and handling, GIS-based data preparation systems have a number of merits. One of the

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possible approaches in matching simulation packages with sources of data by means of GIS or CAD is introduced in Maksimovic *et al.* (1995) and Prodanovic *et al.* (1995), and results of its application are presented in this paper.

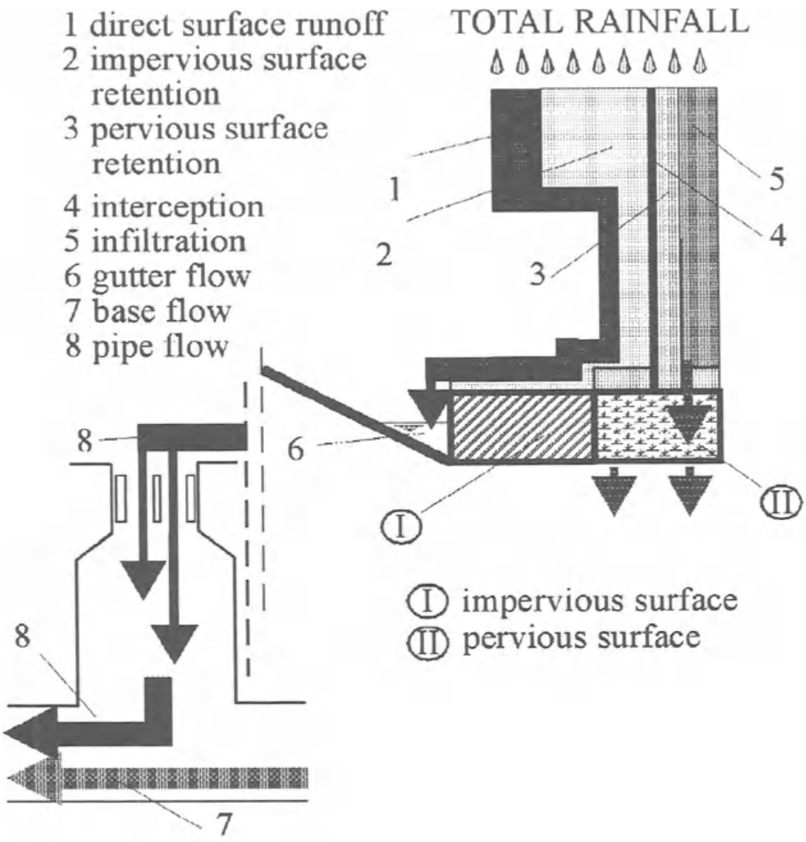

Figure 1. Balance of rainwater in urban areas.

2. Rainwater Balance in Urban Areas

For urban areas, the storms with high rainfall intensities and short duration are of particular interest because they tend to cause higher runoff peaks and, therefore, can cause more harm in urban areas. The incoming rainwater quantity is subdivided into the following elements of the water balance shown in Fig. 1: (1) direct surface runoff, (2) surface retention on impervious areas, (3) surface retention on pervious areas, (4) interception, and (5) infiltration.

Before reaching the receiving waters, the surface runoff flows along the streets: (6) gutter flow, (8) through inlets and together with base flow (7), it flows through the system of underground pipes, ancillary structures, retention facilities, overflows etc. For the sake of simplicity and to logically follow the physical process, most of the current advanced urban drainage models analyse the surface flow, which generates a series of input hydrographs to the inlets, separately from the flow through the system of pipes or open channels. Modelling of surface runoff is performed by discretisation of the catchment into smaller units (subcatchments), such as shown in Fig. 2. Traditionally, it has been assumed that these two phases are separate, that there is no feedback from the underground system once the surface runoff reaches the underground pipes. However, these two runoff components can be treated in an interactive way, i.e., by enabling mutual interchange of water mass between the two phases (e.g., Djordjevic et al. 1992). In order to model the surface flooding and complex flow pattern caused by this kind of interaction, it becomes almost inevitable to rely on the information provided by GIS subroutines.

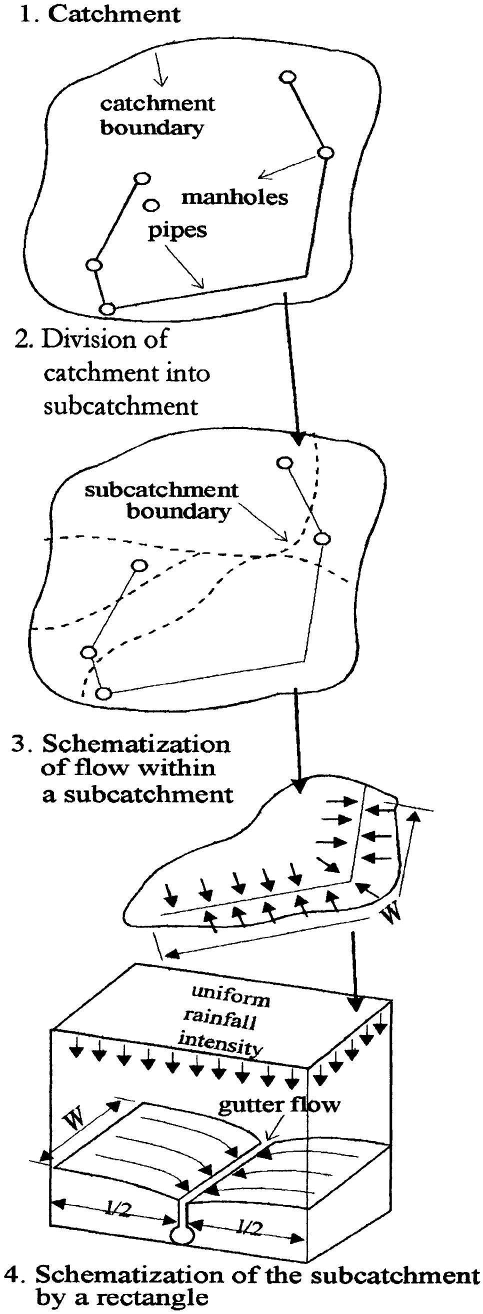

Figure 2. Discretisation of a catchment into smaller units, subcatchments, and their schematisation.

2.1. GENERAL PRINCIPLES

Physically based urban runoff models can be decomposed into several submodels which comprise the following processes (Fig. 3):

- transformation of total rainfall into effective rainfall within a subcatchment (accounting for infiltration and depression storage),
- transformation of effective rainfall over a subcatchment into discharge at the inlet to the manhole (overland flow and gutter flow),
- transformation of the hydrographs in the sewer system (sewer flow).

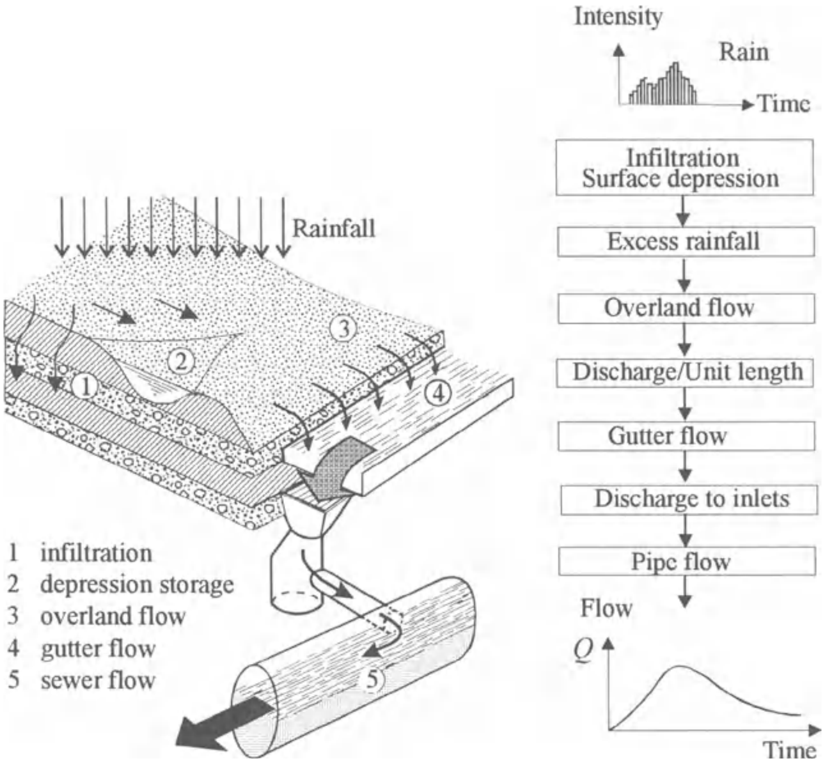

Figure 3. **Transformation of rainfall into output hydrograph.**

In most cases, these processes can be analysed independently (computation of one process is completed before starting the computation of another). The exception can be, for example, a case of heavily surcharged sewer system in which water is released back to the streets.

Basic equations of the submodels are derived from the fundamental laws of hydraulics (conservation of mass and momentum or energy). Since the geometry of surface and subsurface elements of any catchment is too complex, these equations cannot be solved for original catchment geometry and structures.

For that reason, the general principles usually adopted in the development of submodels are the following: - the catchment is decomposed into a number of control volumes called subcatchments (Fig. 3); - each subcatchment is replaced with its idealised equivalent surface, usually of rectangular shape (Fig. 3); - three-dimensional equations are replaced by a set of one-dimensional equations for infiltration, overland flow, gutter flow and sewer flow; - some terms in the momentum equation are neglected; - equations for surface and subsurface flow are solved for uniform rainfall distribution over idealised subcatchments to compute input hydrographs to the sewer system; - equations for free surface and possibly pressurised flow are solved for the sewer system to compute the hydrograph for each pipe.

Following these principles, the submodels for each process included in urban runoff models will be analysed briefly in the following sections.

2.2. RAINFALL ABSTRACTION

The part of rainfall which does not reach the sewer network is called a rainfall abstraction. Rainfall abstractions are caused by such processes as interception, wetting, evapotranspiration, infiltration and depression storage. Among them, infiltration and evapotranspiration are the most important for permeable surfaces, and depression storage for impermeable surfaces. Interception and wetting are rarely analysed independently, but simply are added to depression storage.

2.3. INFILTRATION

The infiltration process is well described by equations of mass and momentum conservation for unsaturated porous media flow (where the second one is approximated by Darcy's law). One-dimensional mathematical models of these laws usually suffice for a realistic description of these processes. In most urban drainage models, infiltration in the unsaturated zone is not linked to ground water flow in the saturated zone of natural catchments. The exception can be the case when infiltration is increased artificially to decrease the peak flow in sewers, and ground water quality problems may arise.

The basic equations for modelling infiltration processes are:
mass conservation law:

$$\frac{\partial \theta}{\partial t}+\frac{\partial w}{\partial z}=0 \tag{1}$$

momentum conservation law (Darcy's law):

$$w=-D(\theta)\frac{\partial \theta}{\partial z}+K(\theta) \tag{2}$$

where:

θ	- moisture content
w	- vertical component of filtration velocity
z	- vertical coordinate axis (positive downwards)
t	- time
$D(\theta)$	- soil diffusivity
$K(\theta)$	- unsaturated soil conductivity

Soil diffusivity $D(\theta)$ is defined as:

$$D(\theta)=K(\theta)\frac{dh_C(\theta)}{d\theta} \tag{3}$$

where:

$h_c(\theta)$ - capillary potential of the soil

Equations (1) and (2) can be written as a single second-order equation (known as Richards equation):

$$\frac{\partial\theta}{\partial t}+\frac{dK}{d\theta}\frac{\partial\theta}{\partial z}=\frac{\partial}{\partial z}\left(D\frac{\partial\theta}{\partial z}\right) \tag{4}$$

The conditions for solving Richards equation for the analysis of infiltration are the following:
initial conditions:

$$\theta\ (z,t_0)=\theta_i\ (z) \tag{5}$$

boundary conditions:
upper:

$$\frac{\partial\theta}{\partial z}(0,t)=\frac{i(t)-K(\theta)}{D(\theta)} \qquad 0\le t\le t_p \tag{6}$$

$$\theta\ (0,t)=\theta_R \qquad\qquad t>t_p$$

lower:

$$\theta\ (-\infty,t)=\theta_R \tag{7}$$

where:

$\theta_i(z)$	- initial distribution of moisture content
$i(t)$	- rainfall intensity
θ_S	- moisture content at saturation (porosity)
θ_R	- residual moisture content
t_p	- ponding time (time when saturation at $z = 0$ occurs)

Typical solutions of Richards equation are given in Fig. 4.

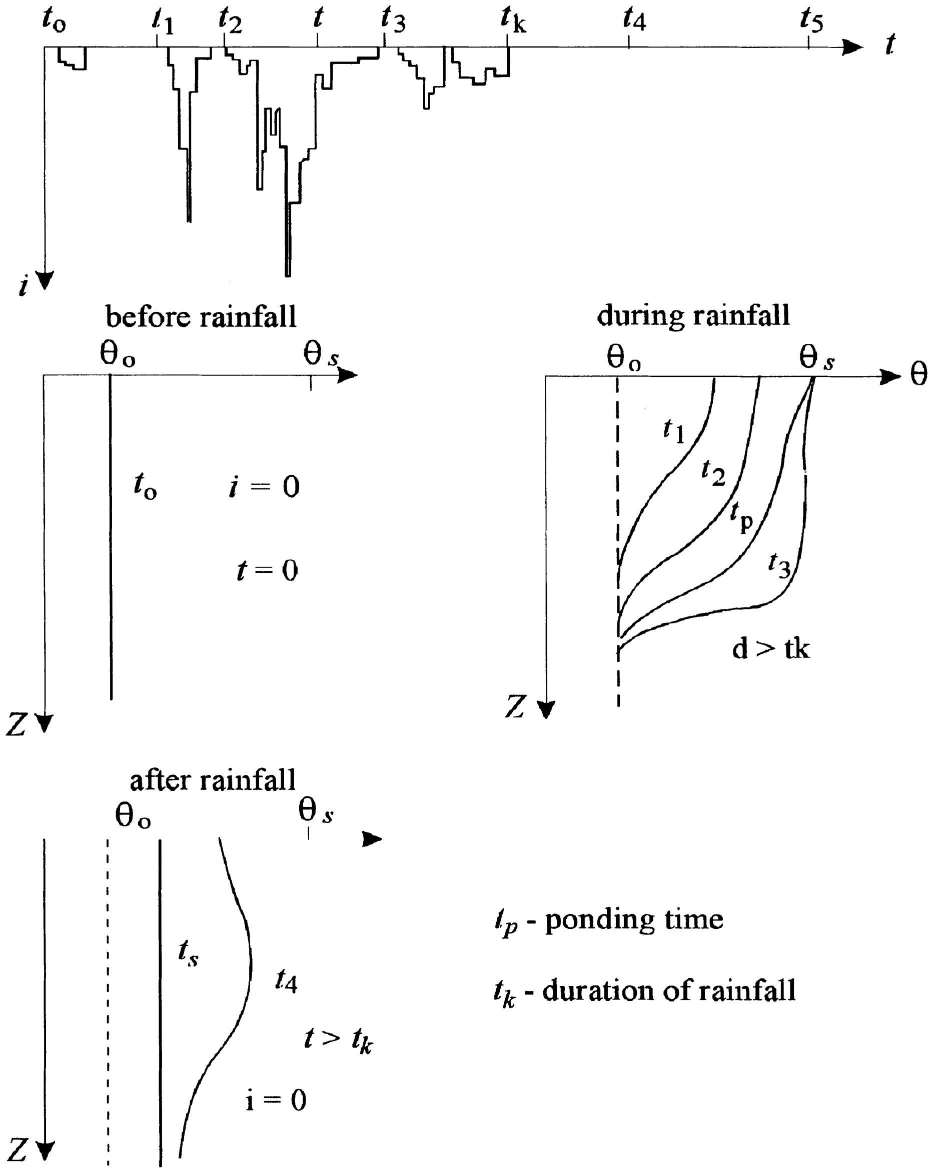

Figure 4. Typical solutions of Richards equation.

Analysis of infiltration by solving Richards equation is usually limited primarily due to the lack of information on areal distribution of submodel parameters, especially of the functions $h_C(\theta)$ and $K(\theta)$, and also due to occasional difficulties associated with numerical solutions.

Consequently, simpler submodels have been developed. Among them, in the category of physically based models, one of the best known is that of Green and Ampt (1911) with modifications introduced by Mein and Larson (1973).

The basic idea is to formulate the submodel in terms of extreme values of moisture content values (θ_R, θ_S), representative capillary potential ($\tilde{h}_C$) and saturated hydraulic conductivity (K_S).

The solution is obtained by introducing average height of soil moisture profile (H), defined as (Fig. 5):

$$H(t)=\frac{1}{\theta_S-\theta_R}\int_0^{z(t)}(\theta-\theta_R)\partial z \tag{8}$$

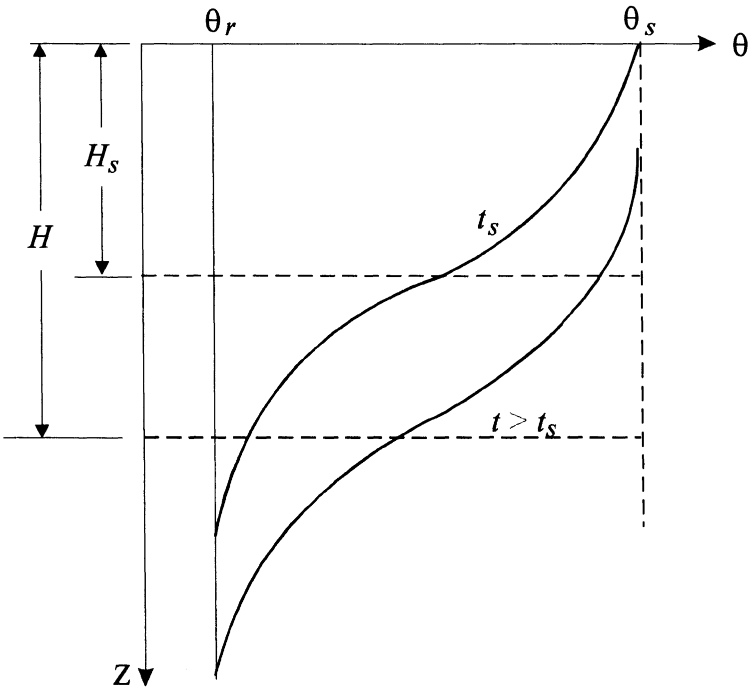

Figure 5. Soil moisture profiles and their schematisation by an average height.

Then equations (1) and (2) can be simplified to yield:
mass conservation law:

$$\frac{\partial H}{\partial t}=\frac{w_0}{\theta_S-\theta_R} \tag{9}$$

momentum conservation law (Darcy's law):

$$w_0 = K_S \frac{\tilde{h}_C + H}{H} \tag{10}$$

where:

w_0 - vertical component of filtration velocity at surface ($z = 0$)

After substitution of eq. (10) in eq. (9), one obtains:

$$\frac{\partial H}{\partial t} = \frac{K_S}{\theta_S - \theta_R} \frac{\tilde{h}_C + H}{H} \tag{11}$$

This is an ordinary first order differential equation which has the analytical solution:

$$H = H_S + \tilde{h}_C \, l_n \frac{|\tilde{h}_C| + H}{|\tilde{h}_C| + H_S} + \frac{K_S}{\theta_S - \theta_R}(t - t_p) \tag{12}$$

where:

H_S - average height of soil moisture profile at ponding time (t_p)

Ponding time (t_p) is determined from the integral form of eq. (1), to which definition (8) is applied, resulting in the following equation:

$$(\theta_z - \theta_R) \frac{\tilde{h}_C K_S}{i(t_p) - K_S} = \int_0^{t_p} i \, \partial t \tag{13}$$

This equation can be transformed into a non-linear algebraic equation with (t_p) as unknown and solved by an appropriate numerical method. H_S (t) is then computed from eq. (10) noting that $w_0 = i(t_p)$.

After that, eq. (12) is solved for H(t), where $t > t_p$ and for $w_0(t)$ from eq. (10).

Comparing the solution described above with the solution of Richards equation for several typical soils (Fig. 6), Mein and Larson (1973) found good agreement for infiltrated volumes represented by the height H (Fig.7) and infiltration velocity (Fig.8).

Infiltration submodels described above contain four parameters (K_S, θ_S, $\tilde{h}_C$, θ_R). The first two should be measured for each particular site while the other two, in the absence of better information, can be estimated from the following empirical relations:

$$\tilde{h}_C = 0.35 \frac{\sigma}{\rho} \sqrt{\frac{\theta_S}{g v K_S}} \tag{14}$$

where:

σ	- surface tension at the interface between water and air
ρ	- water density
ν	- kinematic viscosity,

and

$$\theta_R = \theta_S \left(0.51 + 0.35 \log K_S + 0.06 \log^2 K_S\right) \tag{15}$$

(applicable in interval $-7 \leq \log K_S \leq -3$)

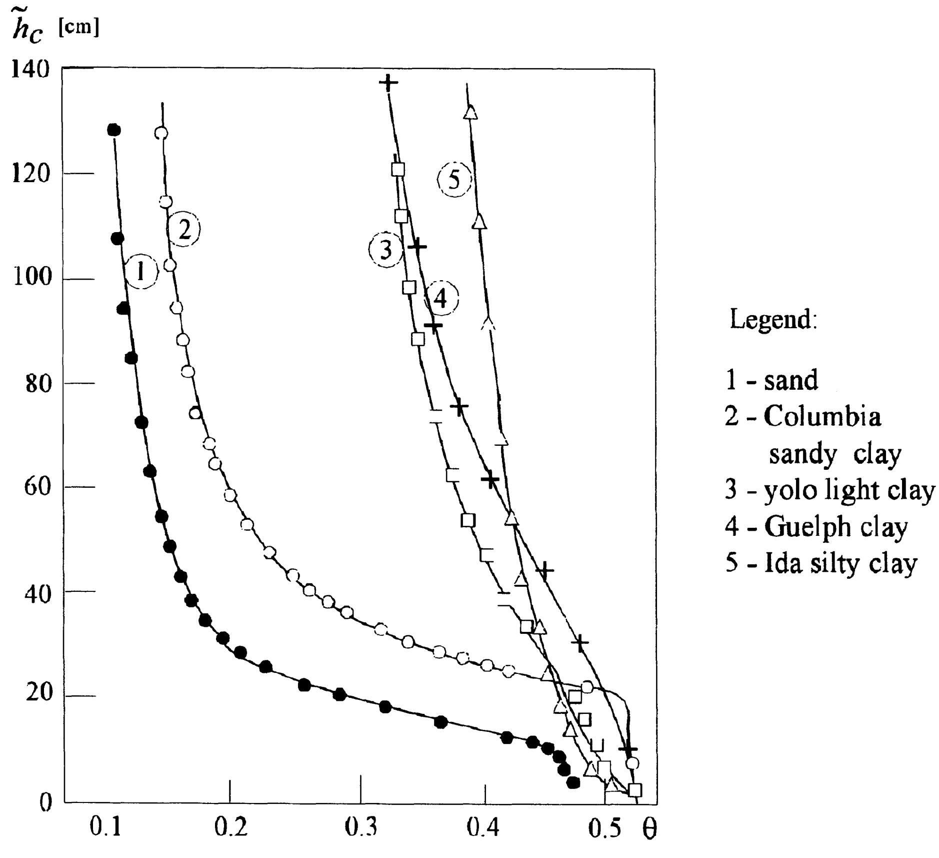

Figure 6. Variation of the representation capillary potential for several typical soils.

Finally, one should note the importance of antecedent moisture content before the rainfall. If each rainfall event is analysed separately assuming dry soil ($\theta(z) = \theta_R$), the runoff could be considerably underestimated if the other rainfall event occurred shortly before (Fig. 9). For that reason, antecedent moisture content should be taken into account either empirically or computed with continuous models by coupling infiltration and evapotranspiration processes (James and Robinson 1986).

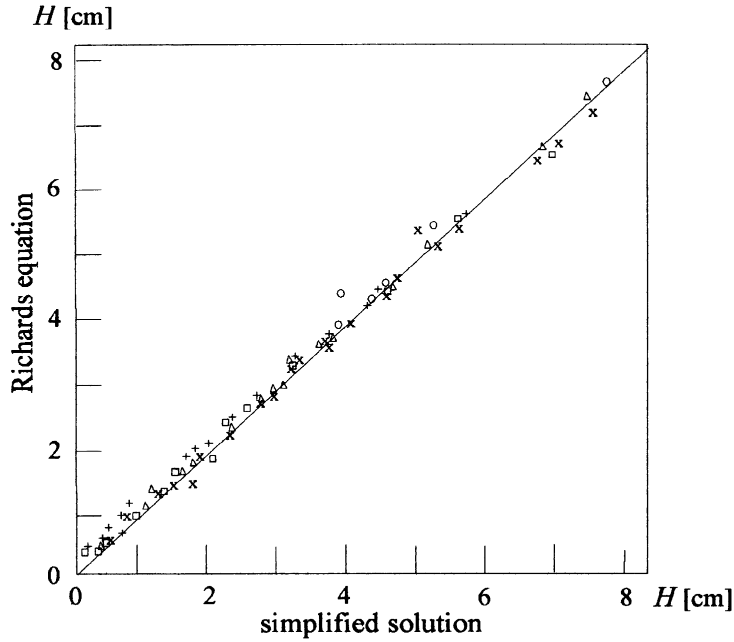

Figure 7. Comparison of infiltration volumes.

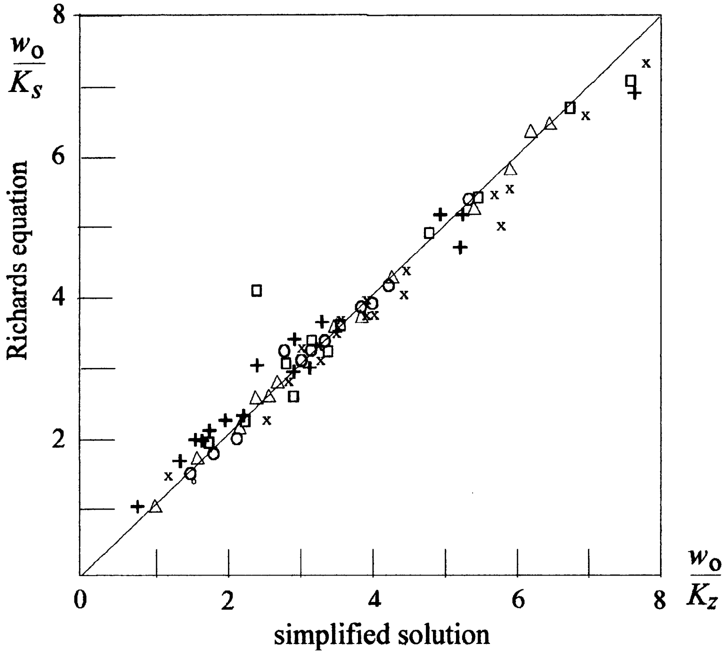

Figure 8. Comparison of infiltration volume.

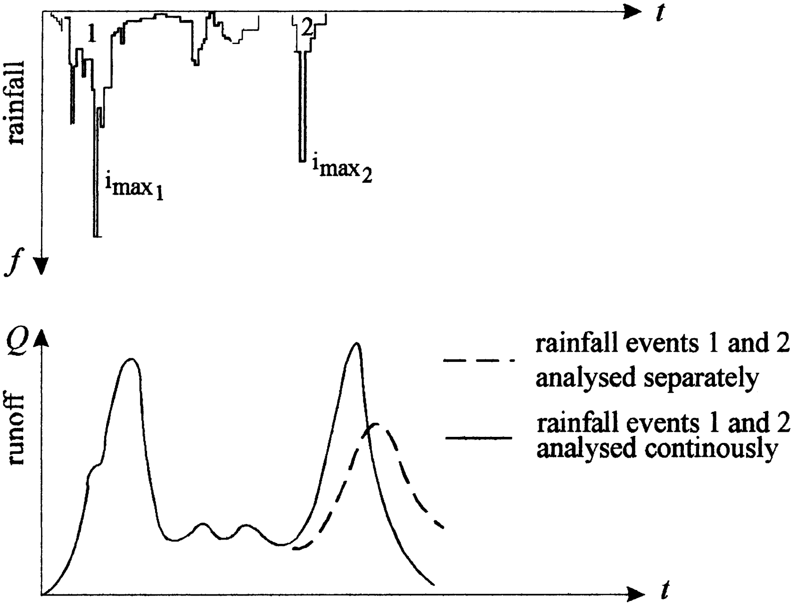

Figure 9. Effect of antecedent moisture content on peak runoff of two successive storms.

2.4. DEPRESSION STORAGE

Depression storage represents water retained in puddles and other depressions in the pavement or the soil surface (Fig. 10).

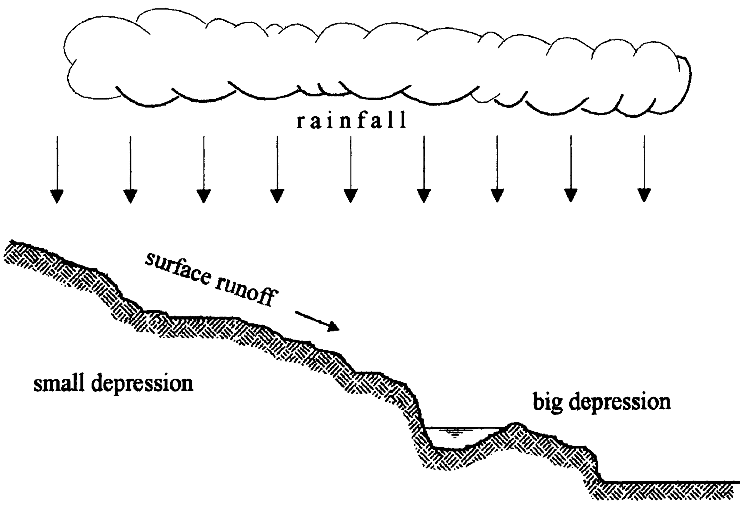

Figure 10. Surface depression.

Although depression storage is considerably higher on pervious surfaces, for runoff analysis it is more important on impervious surfaces, where it represents the only abstraction. The special importance of depression storage is in analysis of low runoff rates associated with quality problems (Pratt *et al.* 1984), while it is of less importance for high runoff rates.

In almost all urban runoff models, the effect of depression storage is computed by the semi-empirical model of Linsley *et al.* (1994). Although (from the point of view of hydraulics) this model can be subjected to verification by comparison with physically based models for overland flow, which would take into account the influence of depression storage, the practical value of such an exercise is questionable.

The equation used to compute the rainfall abstraction due to depression storage is:

$$h_e(t) = h(t) - h_d\left(1 - e^{-h/h_d}\right) \tag{16}$$

Rainfall abstraction due to depression storage can be computed by differentiating (16) as:

$$r_d(t) = i(t)e^{-h/h_d} \tag{17}$$

where:

h	- rainfall depth from the beginning of rainfall to time t
h_e	- effective rainfall depth
h_d	- surface depression parameter

Typical empirical values of surface depression parameter (depth) are (Pecher 1969):

very smooth impermeable surface	0.4 - 0.6mm
smooth impervious surface	0.7 - 0.9mm
bare soil, rare vegetation, grass	0.8 - 2.7mm
soil with abundant vegetation	2.7 - 4.2mm

Analysis of surface depression on impervious areas in some Swedish and U.K. catchments has shown the relation between surface detention parameter (h_d) and surface slope (S_0) (Fig. 11).

The following empirical relation was derived, based on the data from Fig. 11 (Jacobsen 1980):

$$h_d = 0.65 - 6.S_0 \tag{18}$$

Finally, surface abstractions, infiltration and surface retention, are usually considered separately as:

$$i_e(t) = i(t) - w_0(t) - r_d(t) \tag{19}$$

where:

$i_e(t)$ - effective rainfall intensity to be used subsequently for surface runoff computation.

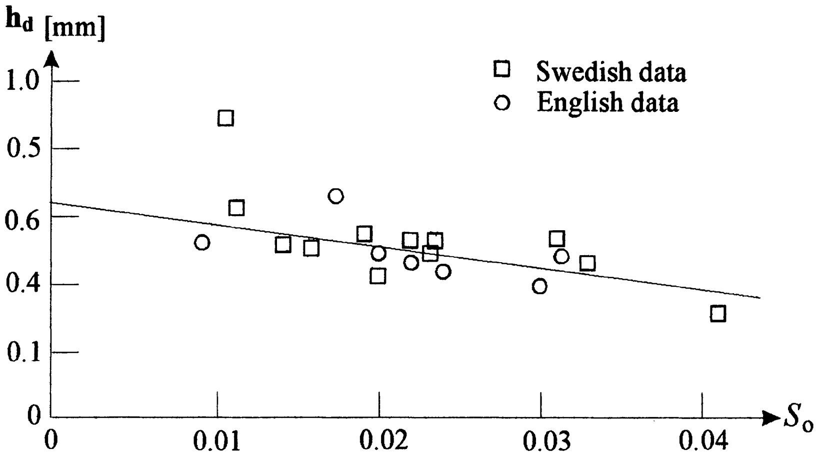

Figure 11. The effect of slope (S_0) on surface depression.

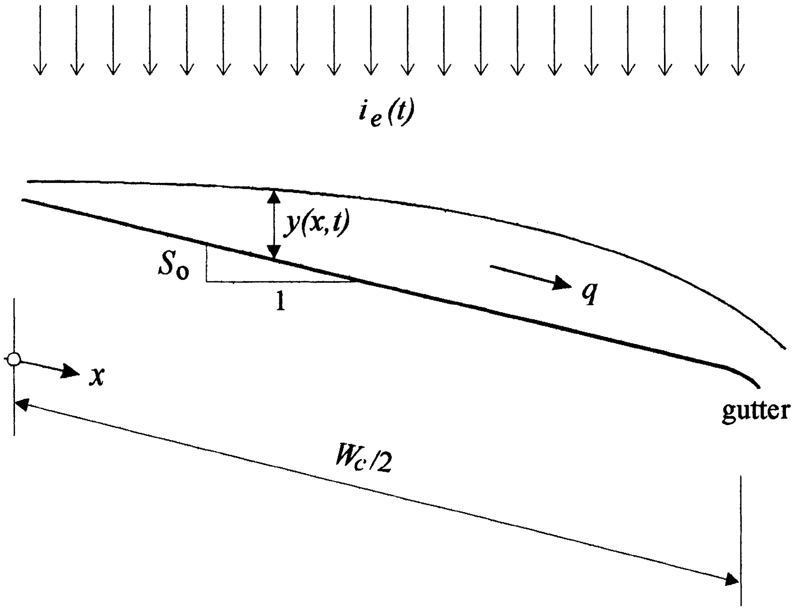

Figure 12. Schematisation of overland flow.

3. Surface Flow

Effective rainfall serves as the input to surface flow analysis over idealised subcatchments (see Fig. 12). The output from overland flow is surface runoff which is used as input to sewer flow analysis. In most urban runoff models, the submodel for surface flow computation is based on the following assumptions:

- two-dimensional (depth averaged) overland flow over the catchment can be analysed as one-dimensional flow,

- flow from parts of a subcatchment characterised by conditions different from those used in computation of effective rainfall (pervious, impervious, roofs, etc.) can be analysed separately and then synthesised to form a unique lateral discharge, which is used as input to gutter flow analysis.

3.1. OVERLAND FLOW

Basic equations for overland flow are written for the rectangular channel of unit width as:

mass conservation law:

$$\frac{\partial y}{\partial t}+\frac{\partial q}{\partial x}=i_e \tag{20}$$

momentum conservation law:

$$\frac{\partial q}{\partial t}+\frac{\partial q^2/y}{\partial x}+gy\left(\frac{\partial y}{\partial x}-S_0\right)+\frac{\tau_b+\tau_R}{\rho}=0 \tag{21}$$

where (see Fig. 12):

- y - water depth
- q - unit discharge
- i_e - effective rainfall
- S_0 - surface slope
- τ_b - bottom shear stress
- τ_R - additional shear stress due to the impact of rainfall drops

3.1.1. *Possibilities for simplification of basic equations*

Although there are no theoretical or practical problems in numerically solving eqs. (20) and (21) (see e.g., Cunge *et al.* 1980), there is a considerable interest to simplify the solution procedure, provided that the simplified solutions are proven to be more efficient and no less accurate, at least from the practical point of view.

The simplification of basic equations can be done by omitting some terms in the momentum equation (see Table 1).

TABLE 1. The possible simplification of dynamic equations

Name of the submodel	Retained terms in momentum equations				
	$\frac{\partial q}{\partial t}$	$\frac{\partial q^2/y}{\partial x}$	$gy\frac{\partial y}{\partial x}$	gyS_0	$(\tau_b+\tau_R)/\rho$
full dynamic	+	+	+	+	+
gravitational	+	+	+	+	+
convective dynamic	-	+	+	+	+
noninertial (diffusive)	-	-	+	+	+
kinematic	-	-	-	+	+

For analysis of overland flow, only the full dynamic, noninertial and kinematic models are of interest. It was shown by numerical experiments that the criteria for applicability of these submodels can be given in terms of two dimensionless numbers, F* and G* (Fig. 13).

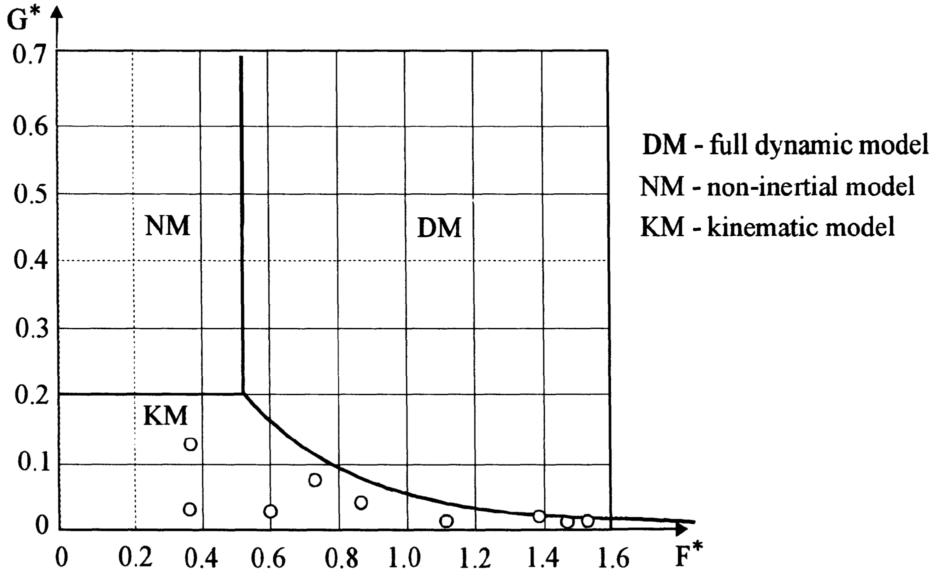

Figure 13. Regions of applicability of different submodels for computation of overland flow.

The definition of dimensionless numbers F^* - the Froude number; G^* - a geometric number are:

$$F^* = \frac{q_0}{y_0\sqrt{g y_0}} \tag{22}$$

$$G^* = \frac{y_0}{0.5 w_C S_0} \tag{23}$$

where the reference quantities (denoted with subscript 0) are:

$$q_0 = 0.5 w_C i_e \tag{24}$$

y_0 = normal depth for discharge

These numbers were evaluated for q_0 from the experimental field catchments in the [UDM] Data Bank and plotted in Fig. 14.

It can be seen that the kinematic submodel would suffice for practical purposes in most of the cases analysed.

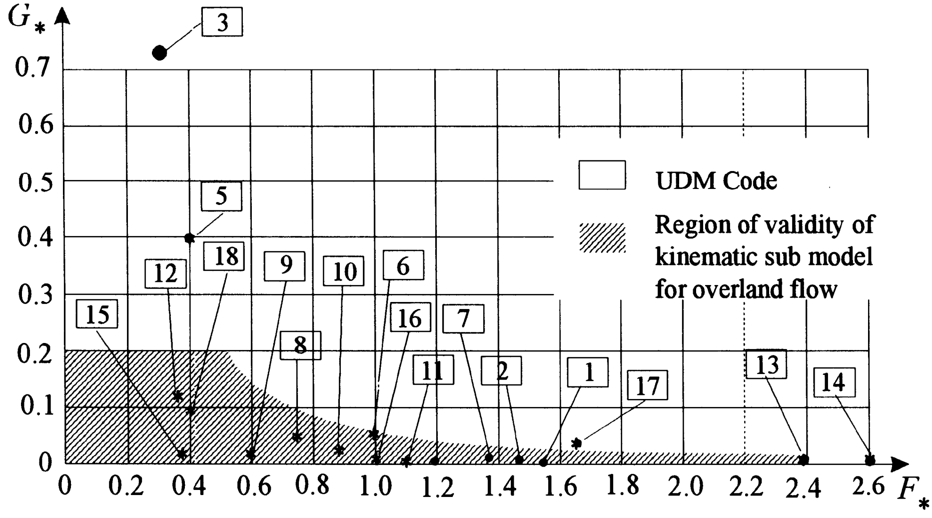

Figure 14. Applicability of the criteria for kinematic submodel for UDM Bank experimental catchments.

3.1.2. *Flow resistance formulae*

Bottom shear stress (τ_b) in momentum eq. (21) can be, by dimensional analysis, related to flow discharge and depth as:

$$\tau_b = \rho \frac{C\tau_b}{2}\left(\frac{q}{y}\right)^2 \tag{25}$$

where:

$C\tau_b$ - friction coefficient

The friction coefficient is a function of the Reynolds number as given in Fig. 15 for laminar and transitional regime (turbulent regime is an exception in overland flow as will be shown later).

Good agreement between the theory and the measurements was obtained only for smooth surfaces. For rough surfaces, which are practically the only ones of interest in urban drainage models, the agreement is less satisfactory (Fig. 15b and 15c).

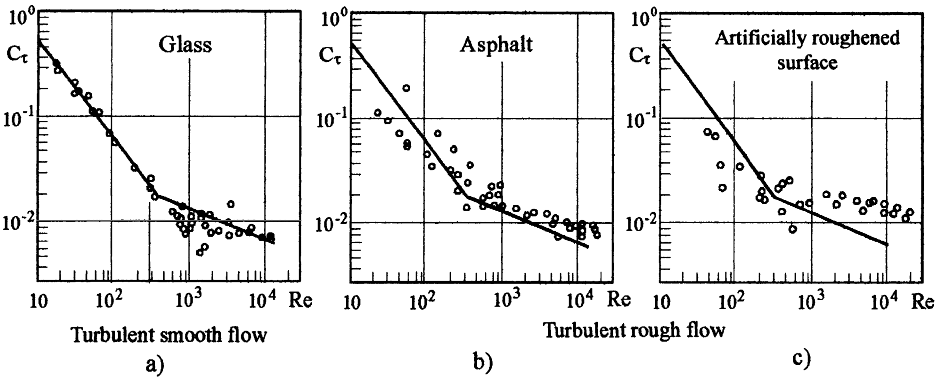

Figure 15. Relationship between friction coefficient and Reynolds number for shear flow over (a) glass, (b) asphalt and (c) artificially roughened surfaces.

It is very difficult to separate the analysis of bottom shear stress (τ_b) and additional shear stress due to the impact of rainfall drops (τ_R) either theoretically or experimentally. For that reason these two stresses are added together and analysed as a single term:

$$\tau_e = \tau_b + \tau_R \tag{26}$$

By analogy with eq. (25), one can define this effective shear stress (τ_e) as:

$$\tau_e = \rho \frac{C\tau_e}{2} \left(\frac{q}{y} \right)^2 \tag{27}$$

where:

$C\tau_e$ - effective overland flow friction coefficient

The experimental relation between $C\tau_e$ and Reynolds number for rough surface (asphalt) is given in Fig. 15b.

On the basis of the extensive analysis of Izzard's data, Radojkovic and Maksimovic (1987) suggested the following formula for computation of effective friction coefficient ($C\tau_e$):

$$C\tau_e = \begin{cases} \dfrac{C_1 + D_1 i_e^{D_2}}{\mathrm{Re}} & \mathrm{Re} \le \mathrm{Re}_1 \\ \dfrac{C_2 + (1 + D_3)}{\mathrm{Re}^{C_3}} & \mathrm{Re} > \mathrm{Re}_1 \end{cases} \tag{28}$$

where Re_1 is the limit value between laminar and transitional regime and C_1, C_2, C_3, D_1, and D_2, are dimensionless constants (C_2 has value that requires input of i_e in metres per second). Recommended values of these constants for typical impervious surfaces (asphalt) and pervious surfaces (grass) are given below in Table 2; these values were obtained by comparison of measured and computed runoff for 59 experiments stored in the UDM Data Base. An example is given in Fig. 16.

TABLE 2.

Constant	Type of surface	
	Impervious	Pervious
C_1	6	3.103
C_2	0.12	0
C_2	0.28	1
D_1	1.6 x 10^5	0
D_2	1	1
D_3	0.08	0

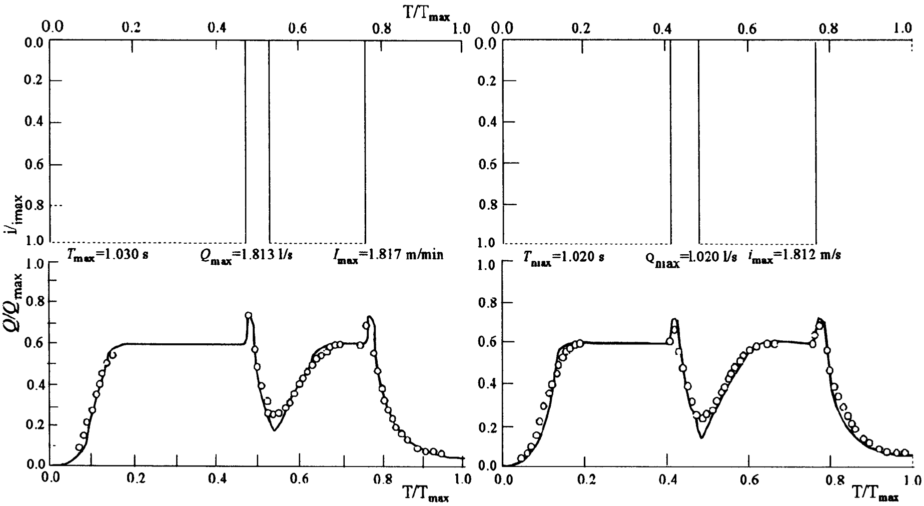

Figure 16. Comparison of measured and computed runoff from impervious rough surface with constants from Table 2.

If the computation of overland flow is done with $C\tau_e$ determined as described above, no parameter is required as input. Of course the values of constants can be changed on the basis of analysis of other experiments for other types of surfaces.

3.1.3. *Solution of Kinematic Submodel Equations*

Following the foregoing analysis, the momentum conservation law can be expressed as:

$$gyS_0 = \frac{C\tau_e}{2}\left(\frac{q}{y}\right)^2 \tag{29}$$

with $C\tau_e$ defined by eq. (28).

Equations (20) and (29) can be solved for surface runoff either by the methods of computational hydraulics (such as Preismann four point implicit scheme, see Cunge *et al.* 1980) or by simpler single element solutions (without spatial discretisation of computational domain), which are not amenable to using the methods originally developed in hydrology (such as nonlinear reservoir). One of the simpler methods is described later.

Equation (29) can be written as:

$$q = \alpha\, y^{\beta} \tag{30}$$

where α and β are parameters which can be easily related to those appearing in eq. (28) and (29). After integration of mass conservation equation (20), assuming the flow profile to be parabolic, and substituting q via eq. (30), the resulting equation is:

$$\frac{\partial h_w}{\partial t} = \frac{3}{2}\left(i_e - \frac{\alpha\, h_w^{\beta}}{0.5 w_C} \right) \tag{31}$$

where:

h_w - flow depth at the end of overland flow half width ($x = 0.5w_C$)

This is a first order ordinary differential equation which can be easily solved numerically by some well-known methods (improved Euler, Runge-Kutta).

By numerical experiments it was shown that this simple kinematic submodel solution is comparable to the more accurate ones (four point implicit scheme) if the time of concentration (T_C) is less than half of duration of rainfall (T_K) (Radojkovic and Maksimovic, 1987).

The analysis of the data from the UDM Data Bank revealed that this condition is fulfilled for all the catchments presented in Table 3, in which the previously introduced dimensionless numbers are also included for completeness.

TABLE 3. Dimensionless numbers for the catchments from the UDM Data Bank

Catchment code		Name of the catchment	F_x	G_x	Re	T_C / T_K
1	CAO1	Malvern	1.586	0.005	588	0.134
2	CAO2	East York	1.509	0.011	2403	0.157
3	USO1	Pompano Beach	0.319	0.732	751	0.115
4	USO2	Sample Road	0.732	0.250	1440	0.032
5	USO3	Fort Lauderdale	0.404	0.406	1222	0.022
6	USO4	Kings Creek	1.006	0.058	1373	0.058
7	USO5	Gray Haven	1.401	0.015	1568	0.034
8	AUO1	Vine Street	0.776	0.060	855	0.020
9	DKO1	Munkerisparken	0.604	0.026	170	0.045
10	FRO1	Livry Gargan	0.878	0.023	444	0.306
11	GBO1	Clifton Grove	1.120	0.013	206	0.104
12	GBO2	St. Marks	0.373	0.131	266	0.100
13	HUO1	Miskolc	2.487	0.005	1851	0.251
14	ITO1	Luzzi	2.689	0.000	114	0.053
15	SEO1	Porsoberg	0.371	0.030	101	0.049
16	SEO2	Klostergarden	0.136	0.007	256	0.050
17	YUO1	Miljakovac	0.678	0.041	458	0.046
18	JAO1	Shirakawa	0.396	0.110	342	0.117

One should also note that because the Reynolds number does not exceed 2500, it is not justified to apply the Manning formula for computation of friction coefficient as done in most of urban runoff models in current use (at least not with the values of Manning coefficient corresponding to those given in the text books on open channel hydraulics).

3.2. GUTTER FLOW

The analysis of gutter flow is very similar to that of overland flow and will not be repeated in these lecture notes. The basic difference in the development of this submodel is that the mass and momentum conservation laws have to be written for a triangular cross section. One should also note that, as Reynolds numbers are one order of magnitude greater that for overland flow, direct application of flow resistance formulae given in classical open channel hydraulics is less restricted.

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SENSITIVITY ANALYSIS OF PHYSICALLY-BASED RAINFALL / RUNOFF MODELS

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1. Introduction

The aim of this section is to increase the awareness of the actual physical processes occurring on the catchment surface and of the capabilities of models to represent them. Some users of advanced simulation models have experience with those approaches which had been used before computers were introduced. Those approaches are characterised by many oversimplifications and the use of fixed (constant) model parameters and fixed input conditions. Examples of elements of these approaches are such concepts as the "design storm" of a prescribed shape, unit hydrograph, time of concentration, or fixed runoff coefficients, which are built into conceptual models, but will not be dealt with here. When the users are not aware of the model capabilities, advantages of modern computer programs in running several alternative solutions are often not fully exploited. In the application of a software package in the analysis and design of storm drainage networks, one of the important steps is the training of users in understanding the physical background of the rainfall/runoff processes and the hypotheses built into the model, as well as in mastering the features of a particular model in performing the sensitivity analysis.

It is important to distinguish between such two separate aspects as:

1. analysis of the model capabilities and its sensitivity to changes of input parameters, conditions in the catchment and in the sewer network, and
2. validation of model parameters for a particular catchment, in which the rainfall / runoff data were obtained by measurements and the characteristics of this catchment and its sewer network were obtained by field surveying and are represented by the appropriate input files (through manual digitising, application of a Geographical Information System, GIS, or by automatic surveying) (Fuchs et al. 1994).

This section focuses on the first aspect of the problem, whereas the second aspect, with a discussion of modelling errors, is described elsewhere, for example in Fuchs (1990). In a hypothetical catchment, the rainfall and runoff measurements are not available. An educational tool, developed for a hypothetical catchment using the BEMUS model, can be used for preparatory training of the users of a particular model, in order to raise their awareness of the range of possible combinations of input

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conditions, catchment characteristics, and model parameters, which may generate unfavourable conditions in the drainage network. To demonstrate misleading and wrong conclusions that the user may draw when working with a limited number of options, this WINDOWS based program enables experimenting with the following options:

A. Shape factor for subcatchment and flow direction
B. Subcatchment and pipe slope
C. Source control
D. Surface retention
E. Soil characteristics
F. Rainfall depth and duration (for a given recurrence interval)
G. Shape of the input hyetograph
H. Roughness parameters for surface runoff and pipes.

It is assumed that, after mastering the effects of variations of these fundamental characteristics and model parameters on runoff simulations, the use of advanced models in full scale studies will be more productive.

2. Surface runoff - What is important and what can be neglected?

Careful analysis of the rainfall runoff models currently in use indicates that several questionable assumptions have been made in development.

One of the objectives of working with a hypothetical catchment is to allow model users to judge the importance of particular catchment or input characteristic with respect to the final result (output hydrograph). To conduct sensitivity analysis, several examples of these assumptions will be discussed.

2.1. CONTRIBUTION FROM PERVIOUS AREAS NEGLECTED

This assumption, which is applied in several models, is very questionable. It is valid for very low rainfall depths when the initial infiltration potential of the soil is not surpassed. However, when the storm of an average or high rainfall depth is applied as an input, the contribution of pervious soils is shown to be significant even in the case of event modelling, with low initial soil moisture. By investigating the soil characteristics (soil conductivity) in a range of physically realistic limits, it can be seen that this assumption is acceptable only for very permeable soils. More specifically, the contribution from the pervious areas can be neglected only in very permeable soils in which even the top layer has very high conductivity (sand and gravel). On the contrary, pervious areas with low conductivity generate high runoff volumes. The effect of soil characteristics on surface runoff (runoff coefficient) is shown in Fig. 1. In this figure the results of runoff simulation from a pervious area of 1 ha are presented. The soil porosity was kept constant, but its conductivity was varied.

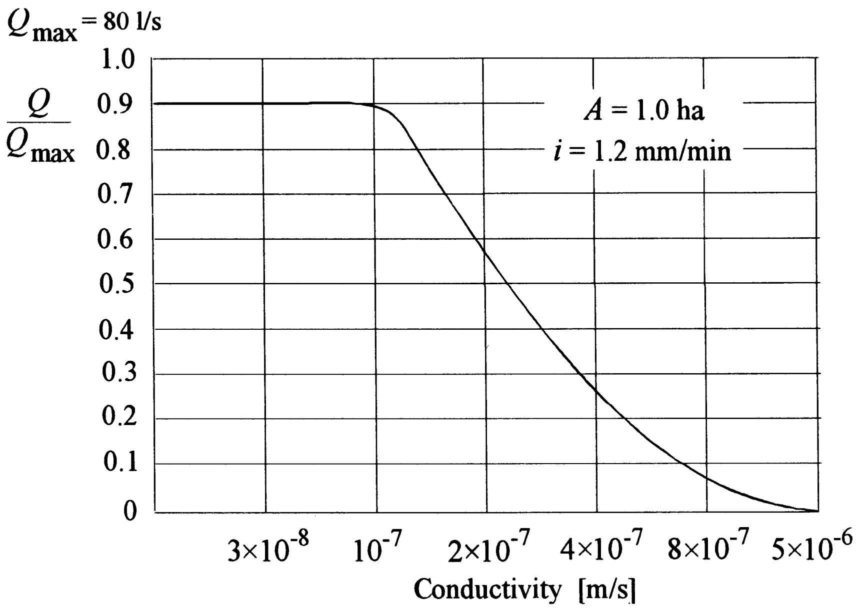

Figure 1. Effect of soil conductivity on surface runoff from a one-hectare pervious area.

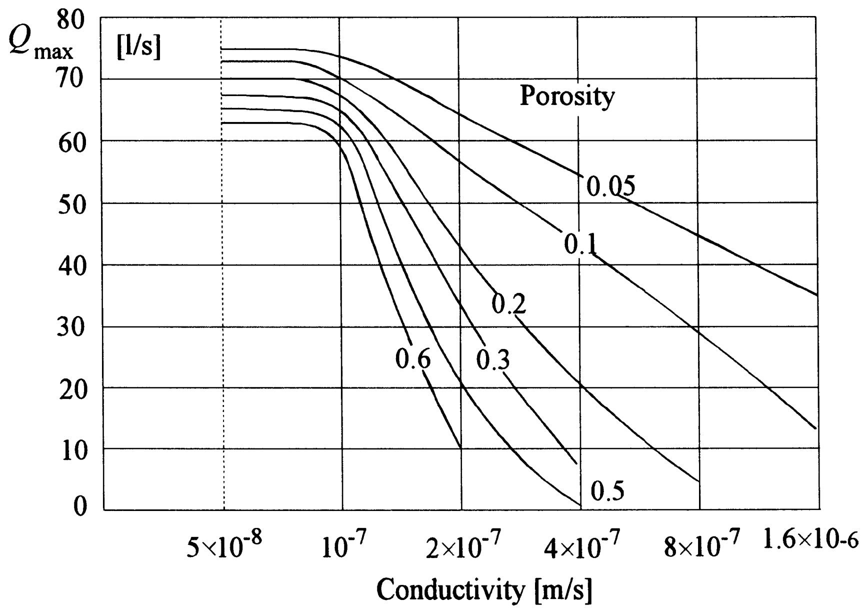

Figure 2. Effect of soil porosity on surface runoff.

It can be seen that the contribution of pervious areas varies, as described by runoff coefficients ranging from close to zero to almost one. Within the range of soil conductivity from 1 to 8 x 10^{-7} m/s, the soil conductivity strongly affects runoff (this can be demonstrated by running options E1 through E9 of the hypothetical catchment). For the same range of conductivity, soil porosity has been varied between 0.05 and 0.5, and the range of corresponding solutions is shown in Fig. 2 (Maksimovic and Vukmirovic 1992).

2.2. CONTRIBUTIONS OF IMPERVIOUS AREAS ARE SPECIFIED ONLY BY THE AREA IMPERVIOUSNESS (%)

This assumption implies that all impervious areas are directly connected to the sewer network or to the street gutter (curb), and it is only valid when these conditions are actually met (case A in Fig. 3), though this is often not the case.

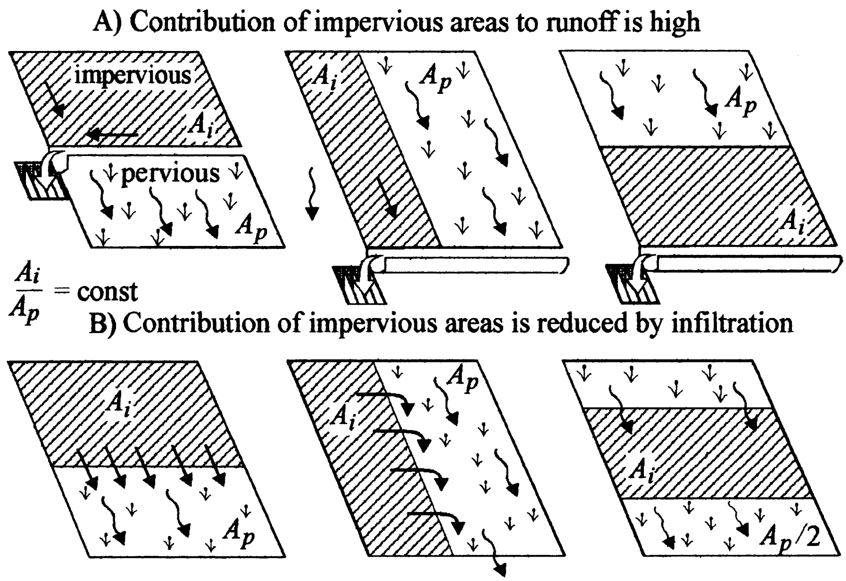

Figure 3. Different ways of draining impervious areas affect runoff.

Many combinations of pervious and impervious areas exist (some of them are shown in case B in Fig. 3), in which the direct runoff from impervious areas is significantly reduced as it flows over the pervious areas before entering the gutter or inlet.

Therefore, it is extremely important to distinguish between at least these two types of drainage of impervious areas, including the drainage of roofs. In several models, roofs are assumed to be drained directly to the sewers. In recent years, efforts were made to introduce automatic - computerised mapping by GIS, digital image processing, automatic surveying, etc. (Fuchs *et al.* 1994). It is essential to note here that these methods should include descriptions of contributions of pervious and impervious areas, in order to ensure reliable modelling.

Several other assumptions, built into the model, also have to be examined in a similar way.

3. Which is "The Best Model"

This question is often asked by the participants of training courses, once several models were introduced to them. The participants of some workshops were asked to play inexperienced users of a particular model by assessing model parameters. The rating of being the "best" was offered, although insufficient information on catchment characteristics was given. In some cases, more reliable data could have been provided if attention had been paid to what is needed for reliable physically based modelling. There are numerous examples of seeking "the best" without attempting to provide sufficient information on physical processes on which the model is based and on the characteristics of catchment and subcatchments (Niemczynowicz and Sevruk 1990, Jacobsen *et al.* 1991).

A lot of conclusions on particular models are based upon biased judgements because of such reasons as:

a. the user had no experience with other models
b. the user does not make additional effort in learning about model's physical background
c. model developers do not provide sufficient information on the assumptions and weak points of their product
d. lack of reliable measurements of different phases of the flow process prevent the intercomparison of the modules for infiltration, surface runoff pipe flow, etc.

The intercomparison is therefore meaningful only for specific modules (such as infiltration, surface runoff, etc.) and less for the models as the whole. Some of the first trials and results obtained by the application of the VP EXPERT as a system shell were presented by Fuchs *et al.* (1990).

4. The Features of the Hypothetical Catchment

For the analysis of runoff from an urbanised area, a catchment of a regular (rectangular) geometry was hypothesised. The area consists of 4 blocks. Each block is further subdivided into individual parcels - 6 per block. Total area of the catchment is 4.08 ha. It consists of 41 % streets, 6 % other impervious areas (such as sidewalks, etc), 9 % roofs and 44 % pervious area. The layout of the hypothetical catchment is shown in Fig. 4. Although the percentage and characteristics of pervious and impervious areas are chosen such as described above they can be changed. Additional features such as a retention pond, overflow etc., can be added.

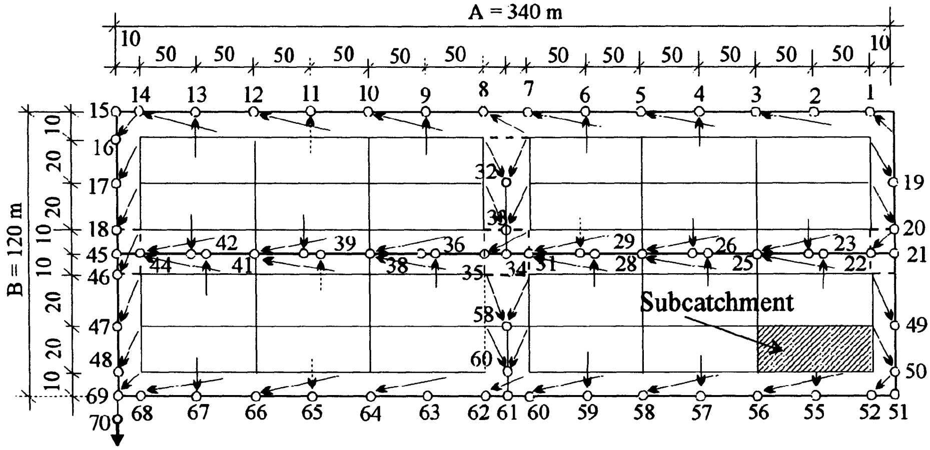

Figure 4. Layout of the hypothetical catchment.

Within the present version of a WINDOWS based program named EBEMUS (Educational Belgrade Model of Urban Sewers) serving to perform the sensitivity analysis, the options shown in Tables 1-3 and Figs. 5 and 6 can be modelled:

TABLE 1. Flow direction and shape factor of subcatchment

Option	L/W
A1	0.4 (as shown on Fig. 5)
A2	2.5 (as shown on Fig. 5)

TABLE 2. Slopes of subcatchment and pipes

Option	Slope (%)
B1	0.5
B2	1.0
B3	1.5

Figure 5. Subcatchment surface runoff flow directions and relevant shape factors.

TABLE 3. Source control in the subcatchments (parcel)

Option	
C1	As shown in Fig. 6 - case C1 (no control)
C2	Disconnect roofs drains (downspouts) from the network and allow the water from roofs to run over pervious areas
C3	Dispose roof water into infiltration trench
C4	Dispose water from roofs and impervious areas on pervious ones

The options C1 through C4 are shown in Fig. 6.

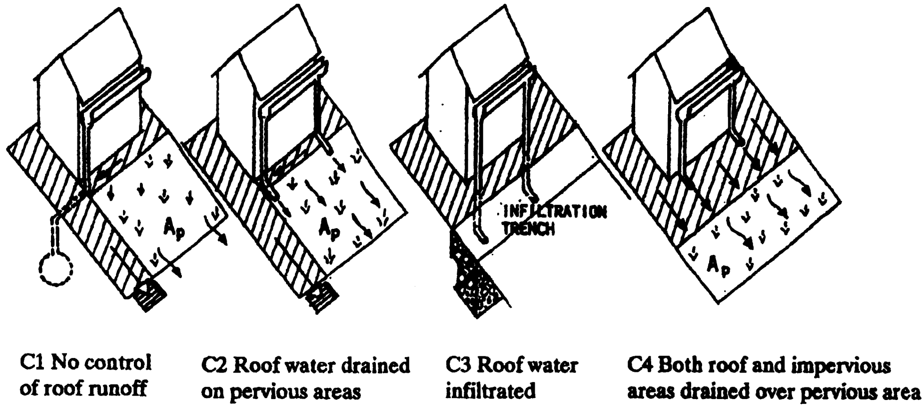

Figure 6. Source control in subcatchments.

Some of the results of simulation which show the effect of the source control measures are given in Fig. 7. In four simulations, several options were kept constant (A1, B2, D1, E1, F4, G1, H1) whereas only source controls described by the options C1 through C4 were varied.

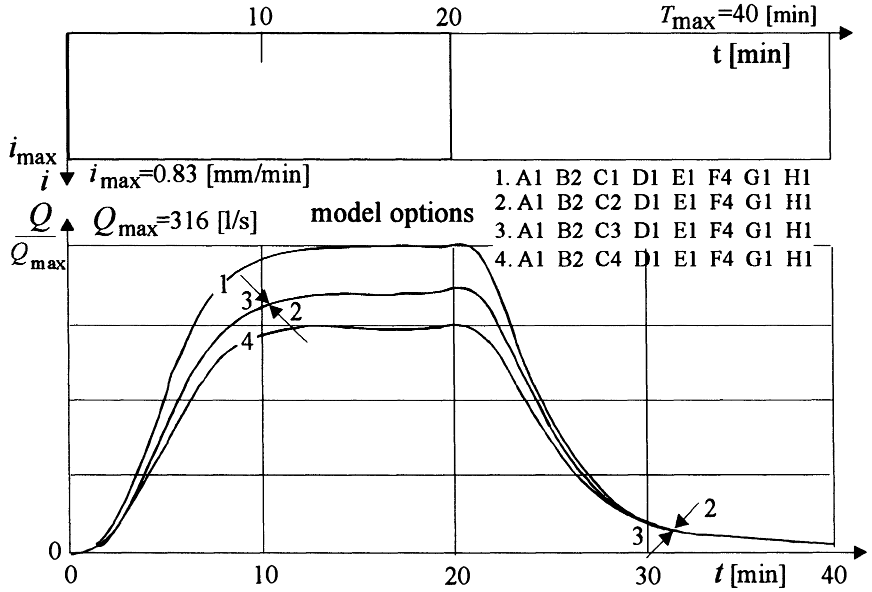

Figure 7. Output hydrographs showing the effect of source control measures.

TABLE 4. Surface retention

Option	Retention (mm) for impervious areas	Retention (mm) for pervious areas
D1	0.6	2
D2	0.6	4
D3	0.6	8
D4	0.1	4
D5	0.8	4
D6	1.5	4

TABLE 5. Soil characteristics

Option	Conductivity (m/sec)	Porosity (-)
E1	1×10^{-6}	0.2
E2	5×10^{-7}	0.2
E3	1×10^{-7}	0.2
E4	1×10^{-6}	0.4
E5	5×10^{-7}	0.4
E6	1×10^{-7}	0.4
E7	1×10^{-6}	0.6
E8	5×10^{-7}	0.6
E9	1×10^{-7}	0.6

The possible shapes of input hyetographs (storm profiles) used in the analysis of this effect on the runoff hydrograph are shown in Fig. 8. The shape varies between the so called "block" rain (option G1) and various forms of temporarily varied rainfall intensities. Both rainfall intensity (left side) and mass curves (right side) are shown.

TABLE 6. Rainfall durations and depths.

Option	Duration (min)	Rainfall depth (mm)
F1	5	8.5
F2	10	11.0
F3	15	14.5
F4	20	16.5
F5	30	18.5

In addition to the above characteristics, the roughness can be described by the options listed in Table 7.

TABLE 7. Roughness parameters.

	Equivalent Manning's roughness parameter ($m^{-1/3}sec$)			
Option	pervious area	impervious area	gutters	pipes
H1	0.200	0.016	0.013	0.013
H2	0.210	0.018	0.015	0.014
H3	0.220	0.020	0.017	0.015

Similar to the analysis of the effect of source control measures (Fig. 7), the effect of the change of any other option can be easily simulated and results made available immediately. For example, the effect of rainfall duration for a set of 5 discrete values (F1 through F5 - Table 6), and a set of constant options A1, B2, C1, D1, E1, C2, H1, is presented in Fig. 9.

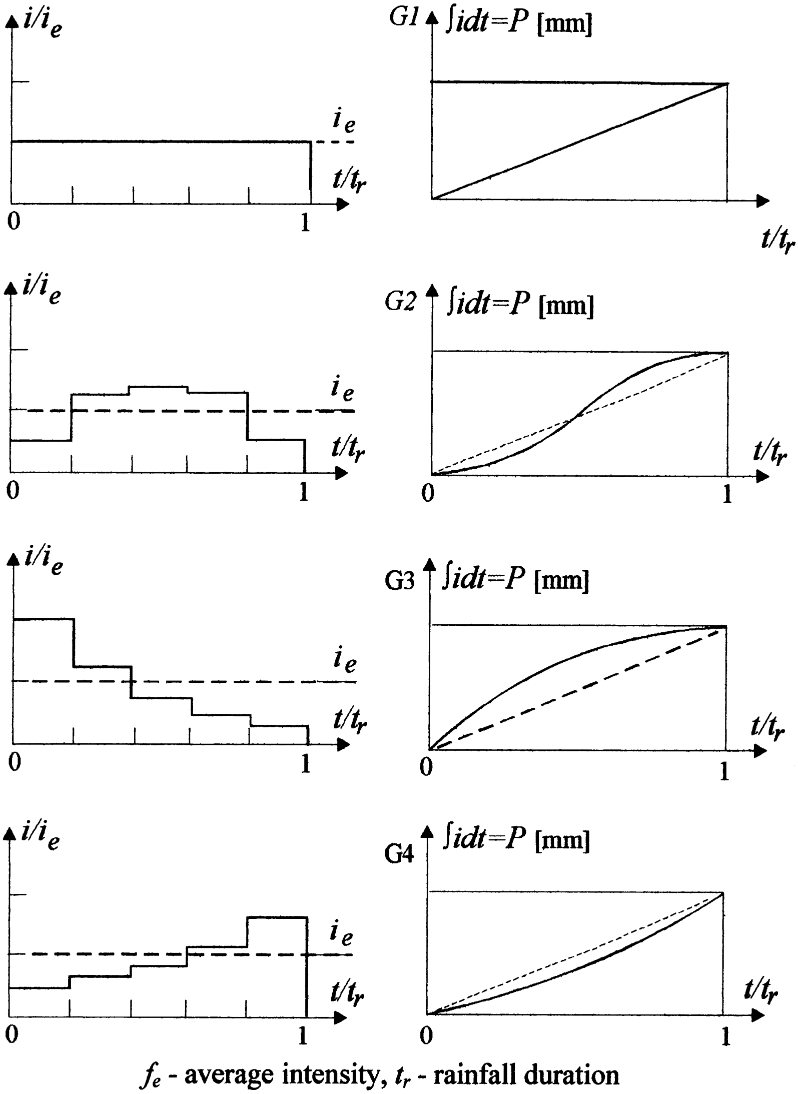

Figure 8. Hyetograph shapes G1 through G4.

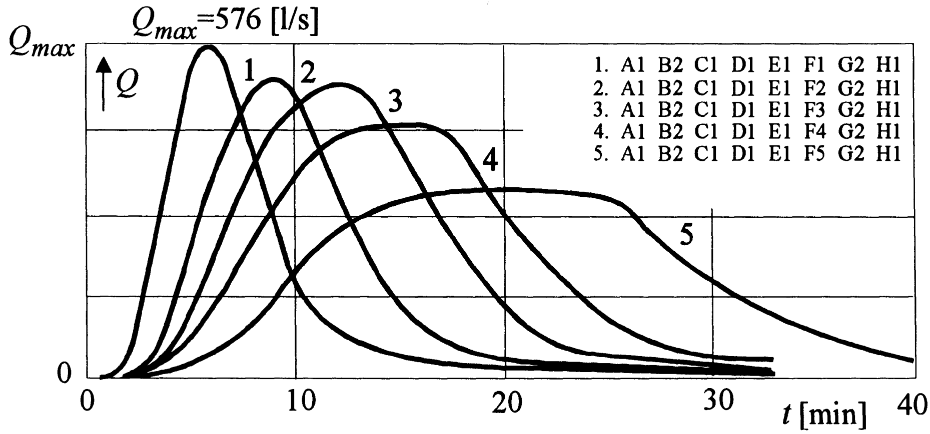

Figure 9. Effect of the rainfall duration on the output hydrograph.

A selected set of catchment data, model, model parameters and input options were put on one diskette, which can be run under a WINDOWS environment. A large number of combinations of options can be performed.

For the computation of runoff, the BEMUS (Belgrade Model of Urban Sewers) simulation model was used. The description of the model was given in Radojkovic and Maksimovic (1984, 1987). After completion of the sensitivity analysis, trainees are advised to proceed with full-scale applications in catchments, for which measured rainfall and runoff data are available.

5. Conclusions

The hypothetical catchment presented in this section, in combination with a simulation model, can be used for the preparatory phase of training users of storm drainage models. It enables the user to run a certain number of options and to gain a sense of the importance and effects of catchment characteristics and model input parameters with respect to runoff. It is an educational tool which should not be confused with a simulation model which is used for calibration and validation of model parameters, and for which rainfall and runoff measurements are needed. For the latter purpose, data are available from the UDM (Urban Drainage Modelling) data bank, the selected set of which is available in Urban Drainage Catchments (Maksimovic and Radojkovic 1980) and from the UDM Italiana (Calomino *et al.* 1995; UDM Italiana 1995). The first time this tool (the DOS version) was used was at the training course organised for Italian engineers in Palermo (IRTCUD 1991).

6. Acknowledgments

The development of the educational tool - hypothetical catchment - has been performed within two international projects:

1. COMETT1 (EC): Expert system for computer aided design and renovation of sewer networks, and
2. UNESCO/UNDP project RER/87/020: INTERNET - Scientific and Technical Information Network, Output 1.1. Urban Drainage and two national projects:
    1. Project T164 financed by Research Fund of Serbia, and
    2. Project 233/88 financed by OZNB - Research Union of Belgrade.

The final setting of the program and data was accomplished by Mr Mihailo Draskovic, and text processing and drafting was done by Mr Vladimir Jankovic, both from the Institute of Hydraulic Engineering, Faculty of Civil Engineering, Belgrade, Yugoslavia. The WINDOWS version has been adapted by Mr. Mario Lucin.

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RAINFALL DATA IN URBAN HYDROLOGY

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1. Introduction

The tasks for engineers and hydrologists, addressing the problems of the urban environment, have gradually evolved during the last several decades. While initially these tasks were concerned with establishing a suitable urban drainage system, using the available knowledge but without deep understanding of such terms as "receiving water quality", "integrated view of the urban hydrological cycle", or "hydroinformatics", today's engineers are faced with very detailed questions to answer, on the basis of the progress in knowledge, technology, and awareness.

The shift of working objectives in urban hydrology will be demonstrated by a number of examples.

Today, in many cities, urban drainage systems already exist. Therefore, the initial investment was made a long time ago, system ageing is a predominant concern, and the question is not "how to design the system?", but "how can we renew the system in a cost-effective way?". Furthermore, the main focus of civil engineering is slightly shifting from planning and design tasks towards operation of the system, its maintenance, and optimisation. The reasons for building urban drainage were sanitary problems - the population had to be protected against the health effects of its wastewater. In the meantime, the public health problems have been solved in most parts of the world and the protection of the environment has become the driving force of urban drainage design philosophy.

Much development has also been noted in the technology field. While in former times engineers were limited by the available computing power in solving their problems, the main limitations nowadays are the availability of data and knowledge. This is due to the fact that the concern has shifted from pure hydrology and hydraulics, related to the sewer system only, towards multidisciplinary integrated approaches to pollution and toxicity reduction. Simulation models are the tools of choice needed to protect the receiving waters in a comprehensive way by site specific measures. The use of uniform measures in an end-of-pipe pollution control strategy, or use of the rational method, no longer represents the state-of-the-art. One major consequence is that new

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approaches require a new kind of information on rainfall: problem and site specific rainfall data. Existing IDF-curves are no longer sufficient for many applications.

2. Short Overview of the Rainfall Process

2.1. CONCEPTUAL OVERVIEW

Satellite and radar images have contributed to the understanding of the rainfall process as a spatial-temporal process, involving a displacement component and a dynamic component. In simple terms, one could speak about an advective process (movement of rainfields) and a convective process (shape and intensity modification of rainfields) being superimposed. The physical reality is much more complex.

Depending on the type of rainfall (and the meteorological situation), rainfields are organised in different ways. The smallest entity is a single convective cell (e.g. a thunderstorm cell) which covers an area of a few square kilometres. Larger entities are the supercell, multicellular rainfields (e.g. squall lines), or large relatively homogeneous rainfields or rainbands (Browning 1985). Associated with the size of a rainfall structure is its lifetime. Table 1 shows that the larger the rainfall structure, the longer its lifetime. For rainfall measurements, this determines the temporal and spatial measurement requirements: if convective cells are to be observed, the temporal resolution has to be very short (1 minute is desirable), and the spatial density of measurement points has to be very high (1 value per square kilometre is desirable). When other rainfall structures are of importance, the requirements are less rigorous.

TABLE 1. Characteristics of some important precipitation systems (after Denoeux, 1989)

Name	Size	Life time	Shape	Meteorological origin
single cell	several km^2	several dozen minutes	symmetrical	convection
super cell	several dozen km^2	one hour or more	symmetrical	convection with wind shear
multicellular thunderstorm	several dozens to several hundreds km^2	several hours	symmetrical	convection with wind shear
squall line	several dozens to several hundreds km^2	several hours	oriented	convection with wind shear
narrow rain band	several km x several hundred km	several hours	elongated	warm belt with an anabatic cold front
wide rain band	several dozen km x several hundred km	several hours	elongated	warm belt of a perturbation

2.2. TEMPORAL VARIABILITY

Rainfall is a process which is highly variable in space and time. When measuring rainfall at a point, the rainfall intensity is highly dynamic, regardless of the time resolution. Even at a time scale of seconds, the intensities fluctuate substantially from one time step to the next. Therefore, care has to be taken to define the appropriate measurement time step because longer time steps smooth out the dynamic nature of rainfall. Aggregation of minute rainfall data to five-minute data may result in decreasing intensities by a factor of 80 % (Figure 1).

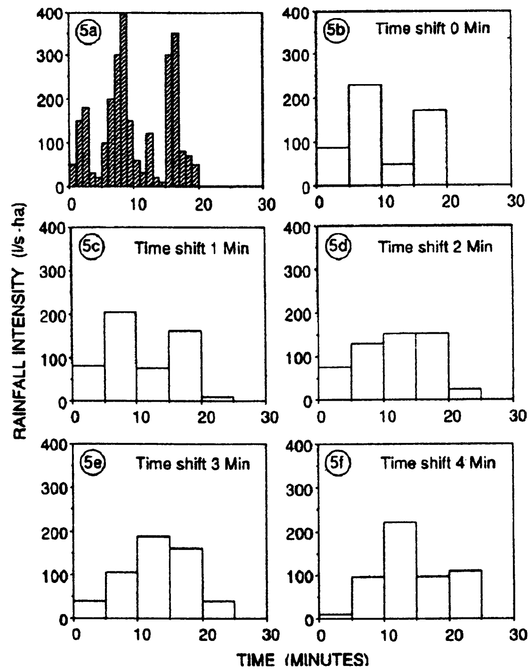

Figure 1. Influence of temporal resolution on peak intensities (after Gujer and Krejci, 1988).

The temporal variability of rainfall is most often perceived from a stationary viewpoint in space, e.g., a raingauge. This is the so called Euclidian viewpoint. However, from a Lagrangian viewpoint, the rainfall may be static, nevertheless leading to a high temporal variability at a point (see Figure 2).

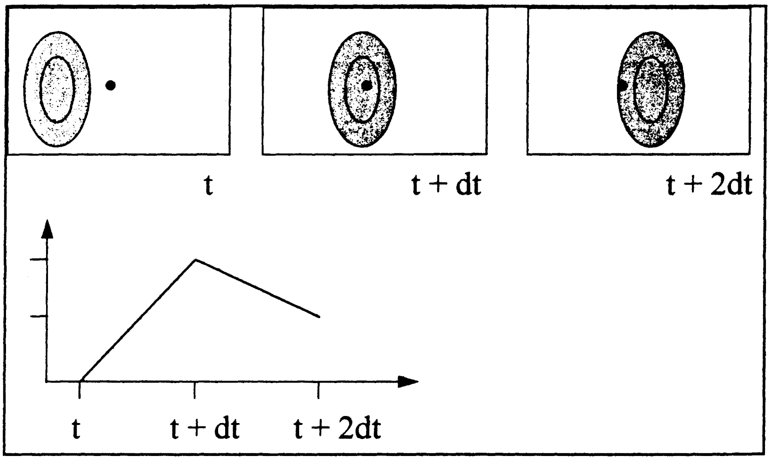

Figure 2. Point measurement of spatial rainfall.

2.3. SPATIAL VARIABILITY

Rainfall often is spatially very heterogeneous. Particularly in summer, recordings of very high intensities in the vicinity of very low ones may occur in convectively dominated events. In Figure 3, the distance from station 201 (0 mm in 5 hours) to station 102 (49.8 mm in 5 hours) is approximately 10 kilometres. In Figure 4, the four stations are inside a catchment of 50 km^2.

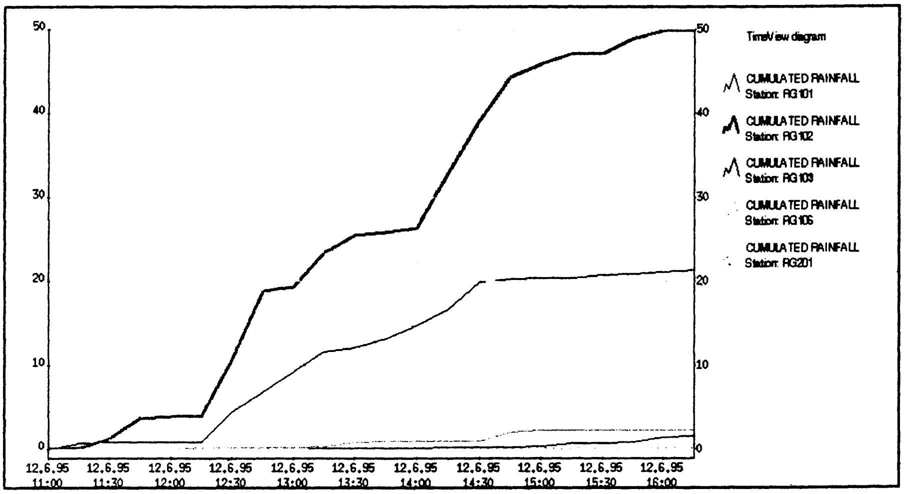

Figure 3. Rainfall event of 12 June 1995 at two stations in Gothenburg (Sweden).

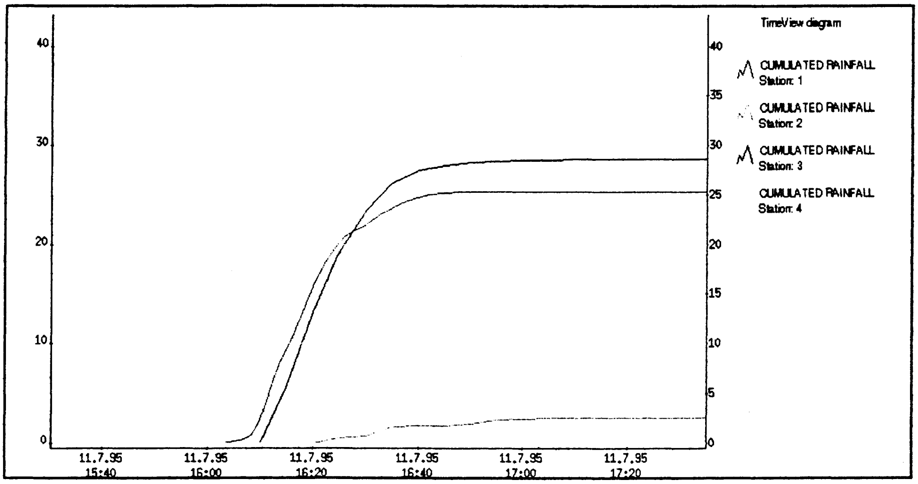

Figure 4. Rainfall event of 11 July 1995 at four stations in a catchment of 50 km^2.

A good means of getting an appreciation of the spatial variability of rainfall is remote sensing by radar or satellite. Raingauges as point measurement devices are lacking the spatial coverage of a region; even a raingauge network can only be used to approximate the extent of spatial rainfall variability.

In winter, rainfall tends to be dominated by large rainfields driven by the advective component leading to uniform rainfall in space and in time. However, passing cold fronts may also provoke very heavy rainfall intensities in winter.

2.4. DISCUSSION

As it was demonstrated above, the variability of rainfall depends on the structure of the rainfield producing rainfall in space and time. Important features are

- travelling speed of the rainfield
- size of the rainfield
- mean intensity of the rainfield
- convective activity inside the rainfield

3. Rainfall Measurement and Instrumentation

Figure 5 demonstrates the different steps that rainfall data may undergo from measurement to their use in an urban hydrology application. The traditional means of measurement is the analog recording of point rainfall by the Hellmann type raingauge. However, digital recording by tipping-bucket, drop counter, or drop size meter is becoming more common.

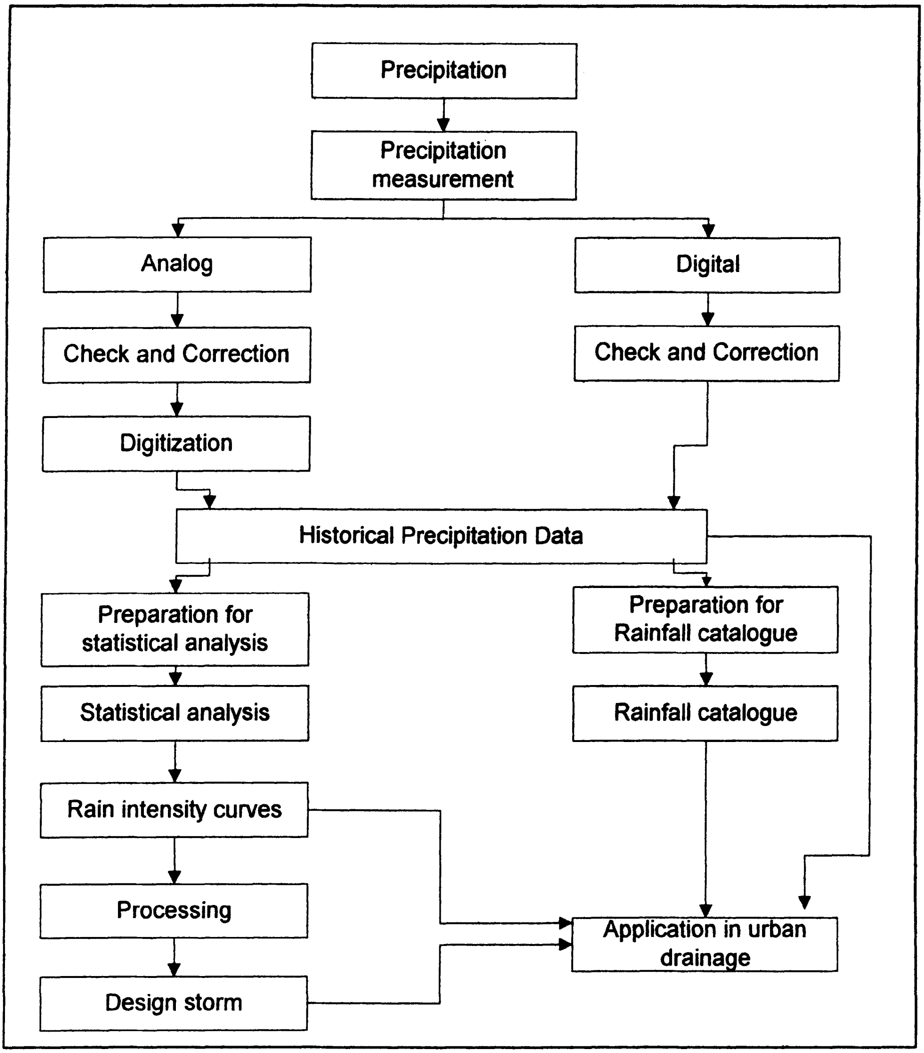

Figure 5. Overview of different kinds of rain data for practical applications in urban hydrology.

Further treatment of rainfall data differs depending on the intended application - it varies from statistical analysis for design storm construction to complete use of the whole continuous time series for long term simulation.

Reviews of rainfall measurement devices were reported by many authors. Very recent and exhaustive discussions of rainfall measurement in the urban domain were reported by the Swiss Water Pollution Control Association (Krejci et al., 1990; VSA, 1994) and by the World Meteorological Organisation (WMO, 1996). Only the most important aspects of rainfall measurement instrumentation will be presented in this section.

The objective of rainfall measurement in the hydrological context is obtaining information on the amount of water falling over an (hydrologically significant) area. For this purpose, no direct measurement method is known. Therefore, point measurements of local precipitation, at the points considered to be representative of their surrounding area, represent the standard method used for the last hundred years. The temporal resolution in general is either "daily" (one reading of precipitation per day) or "continuous" i.e. a continuous recording of the precipitation on-site. Recently, indirect volumetric measurement by means of weather radar has gained in importance.

The following sections provide an overview of different types of measurement devices for point and areal (volumetric) rainfall.

3.1. POINT RAINFALL MEASUREMENT DEVICES

Traditional point measurement devices consist of a collector funnel of a certain surface area, a precipitation measurement system and a storage system (Figure 6).

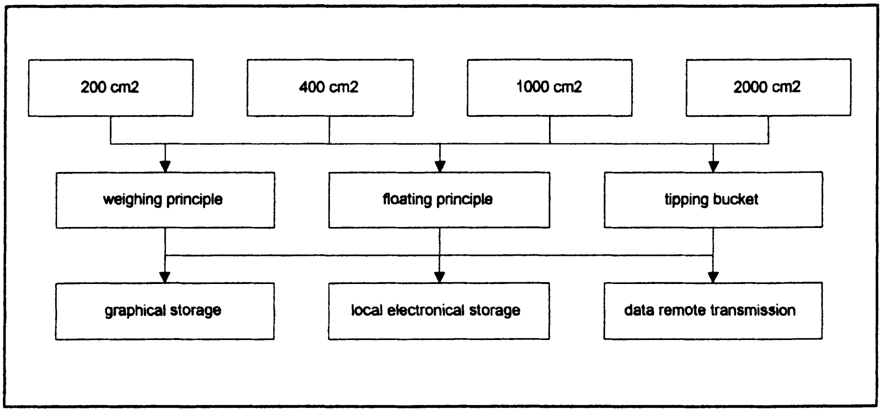

Figure 6. Point precipitation measurement systems (adopted from VSA, 1994).

3.1.1. *Collector Surface Areas*

In practical applications, collector areas of 200, 400, 1000, and 2000 cm^2 are used for precipitation measurement. Most frequently, an area of 200 cm^2 is used, which corresponds to the surface area of the Hellmann type raingauge. Larger collection areas are most often used either in experimental raingauges or in gauges serving special purposes.

3.1.2. *Measurement Devices*

Float Type Raingauge. The float type raingauge collects water through a funnel in a container. In this container, there is a float which is linked to a pen recording the depth of water in the container, usually on paper attached to a rotating drum (Figure 7). Each time the rainwater container is filled, it is emptied through a siphon. Thus, the pen records increasing levels when it is raining, no level change when the weather is dry, and a drop from maximum depth to zero when the container is emptied. Since the drum on which the paper chart is affixed rotates with a steady speed, the recording on paper can be linked to time. The higher the rotation speed, the better the temporal resolution of the recorded data. The achievable time resolution ranges from about 15 minutes for a rotation speed of 2.29 mm/hr to about 2 minutes for a rotation speed of 20 mm/hr.

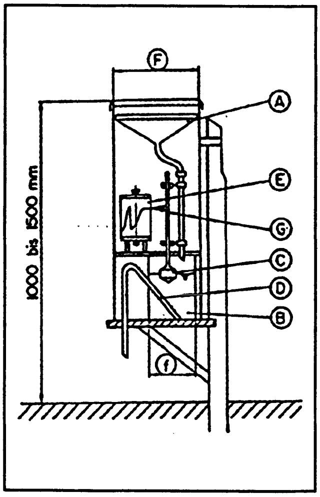

Figure 7. Raingauge with float measurement device.

Tipping-Bucket Raingauge. The tipping-bucket raingauge collects small amounts of rainwater in a balanced bucket consisting of two compartments (Figure 8). Once one of the compartments has been filled, the bucket tips over, empties the first compartment and starts filling the second one. Each tip represents a certain amount of water and usually is recorded electronically together with the exact time of the tip. Crucial points with this measurement device are the calibration of the tipping-bucket and the loss of water during each tip while the bucket is rotating. Furthermore, extremely light rain below the volumetric resolution of the raingauge (i.e. 0.1 to 0.2 mm) cannot be measured.

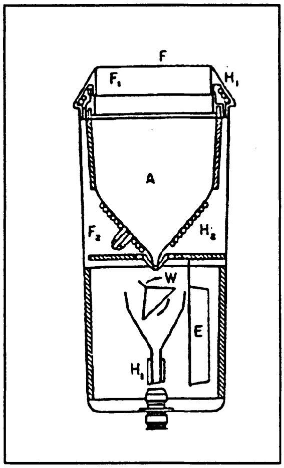

Figure 8. Tipping-bucket raingauge.

Weighing Raingauge. The weighing raingauge collects the precipitation and measures its weight. The result can be digitally recorded and the time resolution may be in the order of several seconds. One major advantage of this device is its capability of also measuring solid precipitation (snow, hail). However, experience with this gauge is very limited and sometimes contradictory.

Drop Counter Raingauge. In the drop counting raingauge the collected water passes through a drop forming device where a well defined quantity of water is released each time it is "full". Each drop is electronically recorded and gives a certain amount of rainfall. Thus, the volumetric resolution is very high, but the accuracy for high intensities is limited. The quality of data from drop counting gauges depends on the drop forming device; whether it is capable of producing homogenous drop sizes under various conditions (high / low intensities, high / low temperatures, high / low humidity, etc.).

Optical Raingauge. The optical raingauge records the particles passing between a light source (e.g. infrared light) and a receiver. In this manner, liquid and solid precipitation can be detected, the optical signal has to be processed and can be digitally recorded. However, experience is still very limited and capital and operation (electrical energy) costs are high.

Disdrometer. The disdrometer is an acoustic measurement device which measures the drop size as well as rainfall volume. Presently, it is not yet used routinely, even though it has been used in research projects for the last twenty years. Main drawbacks are the sensitivity to noise and high investment and maintenance costs.

Discussion. The most frequently used raingauge is the float type gauge, which has been in use for the longest time, i.e., since the last century. However, currently the tipping-bucket raingauges are taking over the position of the float gauge in many places. Their advantages include the feasibility of digital recording and the vast experience with their operation. The other types have not yet been widely tested in routine operation.

3.1.3. *Data Storage Devices*

Pen and Paper (Mechanical Storage). A recording pen is mechanically driven by the float, the weighing device, or the tipping bucket and writes at a certain position on paper, which is moving at a constant speed. The paper movement is facilitated by a moving drum or a spool mechanism transporting paper from a first spool with blank paper towards a second spool collecting the used paper. The advantage of the pen and paper storage is the direct access to the recorded data, the simplicity of the procedure, and low costs. Main disadvantages are high effort needed to convert the data into a digital form and possible graphical errors of these devices.

Data Logger (Local Digital Storage). A data logger stores the measured values locally in an electronic storage device. The data can then be retrieved either by a portable computer or by dialling the logger and fetching the data via a modem. The advantage of a data logger is the fast availability of digital data to the user. Main disadvantages are the loss of the primary (raw) data and the dependency on a constant energy source.

Data Transmission (Central Digital Storage). In this approach, the measured data are sent via a telephone line or radio link to a central digital storage. The advantage of this system is the instant availability of the measured data to the user in digital form. Main disadvantages are the loss of primary raw data, the dependency on a constant energy source, and the possible loss of data in the transmission line. Therefore, it is strongly recommended to couple data transmission with at least one on-site storage system as a data backup source.

3.1.4. *Examples of Raingauges*

Hellmann Raingauge. The most frequently used raingauge in Europe is the Hellmann type raingauge. It consists of a funnel of 200 cm^2, a float type measuring system and a paper chart recorder. This type has been adopted in many countries because it is robust and easy to maintain. The data collected by this device have to be processed by converting them into a digital format.

Tipping-Bucket Raingauge. The Swiss Water Pollution Control Association recommends the tipping-bucket raingauge for standard use, but distinguishes between temporary and permanent instrument at a site. For a permanent site, on-site digital data storage and remote transmission of data are recommended. Thus, data processing through digitisation is not necessary any more and data are available in almost real time. On the other hand, a redundant analog measurement is not performed any more, so that checking the electronical equipment for failure is not easy.

The Northrhine-Westphalia State Environmental Protection Agency in Germany recommends a tipping-bucket raingauge with on-site digital data storage and analog recording of data on a paper chart. This provides digital data and allows for thorough data checking during periods when the digital recordings are doubtful. However, hydrological applications requiring digital data in real time cannot be satisfied by this system.

There are other organisations (e.g. national weather services) favouring different raingauges. However, the tendency is towards digital data storage and data transmission, using a more recent measurement technology than the float type principle.

3.2. AREAL PRECIPITATION MEASUREMENT BY RADAR

A totally different measurement of rainfall is offered by radar. This device is able to measure rainfall when still in the atmosphere, providing a volumetric value for rainfall estimation. Thus, it is closer to the hydrological requirement of areal rainfall data for modelling. Unfortunately, radar does not measure rainfall directly as water volume, which is the parameter of interest to hydrologists, but as rainfall reflectivity.

3.2.1. *Radar Measurement Principle*

The detailed principle of rainfall measurement by radar has been published by many authors (e.g., Collier, 1978, 1989; WMO, 1996). In general, it uses the fact that electromagnetic waves are reflected by raindrop particles and can be captured by an antenna; the magnitude of this reflection is related to the water volume in the atmosphere. In order to obtain a precise measurement inside a well-defined cube in the air, radar beams are sent out in pulses. These pulses permit computation of the distance from the time the signal takes to return to the antenna. Thus, the area around the radar is divided into a polar grid which is regularly screened by the radar device (see Figure 9).

Major problems are caused by the relationship between reflectivity and rainfall, which depends on the (fluctuating) drop size distribution, reflections from objects other than rainfall droplets, and the conversion from polar to cartesian data needed for data processing.

More sophisticated radar measurement techniques such as Doppler, dual polarisation, and volume scanning devices are not yet widely used. Their potential is discussed in Collier (1989).

3.2.2. *Radar Types in Practical Applications*

X-band radar (9.3 - 10 GHz). The X-band radar has a wavelength of 3 cm and provides high resolution rainfall data. However, this radar type suffers from attenuation effects and has a limited regional scope. Nevertheless, its low price and a sensing radius of 40 - 50 km, sufficiently large for urban applications, has provoked discussions on the suitable radar bandwidth for different applications (e.g. Emscher region radar (Semke, 1991), Grenoble radar (Delrieu and Creutin, 1991)).

C-band radar (5.3 - 5.7 GHz). The C-band radar has a wavelength in the order of 5 cm. Offering a compromise between attenuation and measurement precision, it is a weather radar commonly used in moderate climate. Applications include both small-scale urban projects and far-reaching measurements such as in forecasts (e.g., many radars of Meteo France, all operational radars of the German Weather Service, most UK radars).

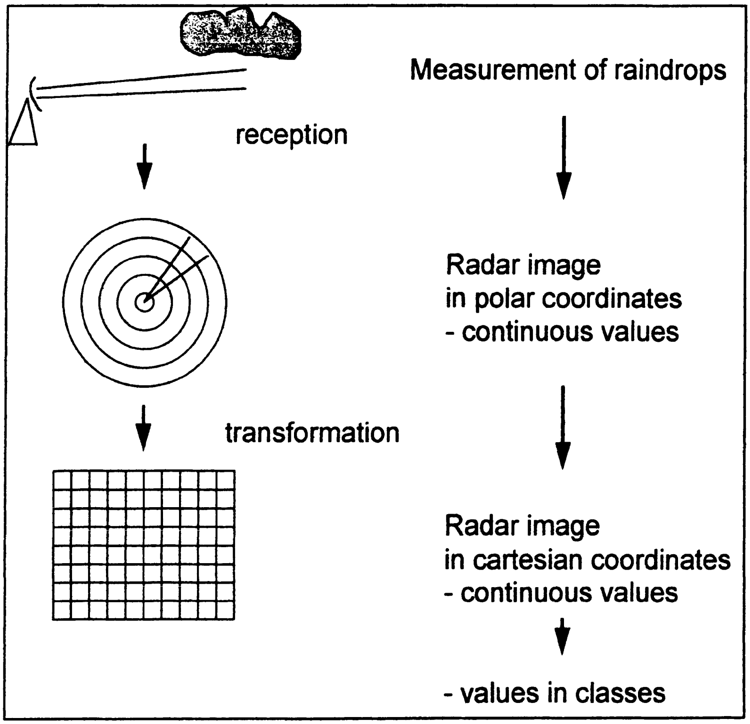

Figure 9. Radar measurement processing.

S-band radar (2.7 - 2.9 GHz). The S-band radar has a wavelength of about 10 cm. This wavelength is best suited for regions with heavy precipitation (tropical and subtropical regions) because of its limited sensitivity to attenuation (see 5.2). The spatial measurement accuracy of this radar type is limited because of the wavelength. A beamwidth of less than 2° requires a relatively large parabolic antenna of more than 5 m in diameter. This leads to high costs for the mechanical parts of the radar (e.g. CHILL radar in Chicago, Marseille radar, NEXRAD radar, early UK radars).

4. Raingauge Data

Most of the recorded rainfall data can be found on paper charts. When processing these data, extreme care is required for quality checking, particularly for the raw data. Once the raw data have been processed, such features as recording errors, temporal resolution, and temporal errors cannot be clearly identified any more. The paper chart is the primary data source; digital data after digitisation of paper charts are only secondary information hiding a number of extremely useful additional observations and remarks.

Digital data, either from direct digital recording or obtained by digitisation, have to also be carefully screened. Comparisons of neighbouring gauges and checking for outliers are useful methods. When in doubt, digital data from paper charts should be compared to the original records and eventually corrected.

4.1. ERROR SOURCES

There are a number of crucial points to consider when working with point rainfall measurement data. Some of them are related to the type of measurement device, others are generic. Among the generic potential sources of error are:

- The wind may cause a severe underestimation of measured data by up to 50%. As a function of the storm speed, a windfield may form around the raingauge and deflect raindrops from the gauge.
- The areal representativeness of point rainfall data is limited. Depending on the meteorological situation, a measurement at one location may not be valid at a point 2 km away.
- The loss by heating (in winter) is encountered in gauges heated during frost periods. Snow and rain may be underestimated by 5 - 10%.
- Evaporation on hot days causes increased initial losses in the raingauge because of the high gauge temperature.
- The loss due to initial wetting of the funnel may amount up to 10%.
- Splashing out of the gauge has been estimated at 1 - 2%.
- A change of the environment around the gauge (e.g. growing trees, buildings) may invalidate data. A minimum distance from trees and buildings has to be kept.

4.2. STANDARDS

To obtain good quality data, standards have been formulated by numerous institutions, generally under the leadership of data users, partly in collaboration with the meteorological service. These standards aim to collect comparable data at different measurement sites, and have been formulated for:

- instrumentation (VSA, 1994)
- measurement procedures (DVWK, 1985)
- digitisation (DVWK, 1985; VSA, 1994)
- data exchange format (DVWK, 1985; VSA, 1994)
- statistical data analysis (DVWK, 1985; VSA, 1994)
- procedures for data correction (DVWK, 1985; VSA, 1994)

It is beneficial for the data users if the data were collected and processed by standard procedures. The advantages are comparable data, easy identification of erroneous or useless data, and the possibility to exchange data without technical problems. Therefore, it is extremely important to follow well-defined rules in data collection.

4.3. INSTALLATION AND MAINTENANCE

Recommendations from different official bodies (WMO, 1996; VSA, 1996; DVWK, 1985) result in the following key aspects of raingauge installation:

- The site for the measurement device should be on level ground and representative of the region.
- It should be protected against wind effects but not shielded by obstacles such as trees, houses, walls, etc.
- The height of the raingauge should be between 1 m and 1.50 m above the ground.
- The collector opening of the gauge should be perfectly horizontal.
- The gauge should be protected against vandalism.
- The paper transport speed of a paper chart recorder should be at least 10 mm/hr.

A regular maintenance (at least once per week) is absolutely required to yield good data quality. The tasks of maintenance include the following:

- time check and time mark on the paper chart
- ink and pencil check
- regular change of paper
- measurement of the volume and emptying of the rain water container
- battery check
- check collector blockage by debris
- regular calibration of the measurement unit

The first three tasks are specific to paper chart recording gauges, which are still most frequently found in many countries. The other tasks apply to all gauges in general. The calibration procedure depends on the measurement technique of the gauge.

Insufficient maintenance results in doubtful or erroneous data. The next data processing steps, digitisation and data quality control, can partly detect these effects but they cannot produce the missing data.

4.4. THE DIGITISATION PROCESS

The value and reliability of results from most hydrological studies strongly depend on the length and quality of the rainfall data time series. The availability of long records is, therefore, an asset to many engineering studies. For many locations, historical paper chart records exist and have to be digitised, in order to be used in hydrologic applications. In this manner, basic data for statistics, continuous and single event simulation can be obtained.

Since historical measurements were mostly recorded on paper, the digitisation of historical rainfall records plays an important role. The quality of the hydrological simulation results depends on long time series of data and a reliable digitisation of the paper chart records.

In digitisation, substantial experience is required to distinguish between measurement errors and rare data. The first step in digitisation is visual control; the operator visually checks for missing data, suspect data, no ink, no rotation, uneven rotation speed, frost and collector blockage. Furthermore, the record is compared to the daily value which was often marked by hand on the paper by the maintenance personnel.

During the digitisation, the paper chart is screened by an operator who transfers the coordinates of the lines on the paper into a digital form. This is most often done on a digitisation table. Planimeters and scanners are used less frequently. Scanning paper charts is not a very reliable process, because of the low quality of the records and because many interpretations have to be done in order to identify errors and good data.

Possible errors were discussed by DVWK (1985), VSA (1994) and WMO (1996) in more detail. The main sources of error in digitising paper chart records are the following:

- Wrong positioning of the paper – leading to a wrong start time, non-horizontal recorded trace, the pen trace running outside the paper edge, or a combination of the aforementioned effects. Corrections are difficult to apply, often impossible.
- Timing errors – due to lack of power, the recording speed may be too fast, too slow, or non-uniform (first three hours correct, eighteen hours too fast, last three hours correct). Only a uniform timing error can be corrected; however, it is difficult to be sure that the timing error was uniformly distributed.
- Maintenance errors / omissions – such as the missing dates and times of inspection, lack of ink, no time check or any other procedures that the gauge operator has to take care of. Important tasks include checking recorded time versus real time by marking the chart, checking the mechanical parts of the system, checking that the funnel or the outflow of the rain collector is not obstructed, ink and paper, and changing the paper charts. These tasks are vital for a reliable data collection system.
- Frost – temperatures below freezing lead to the formation of ice in the raingauge. This results in a rising recorded curve which starts to descend after the ice has melted. Since during this time the measurement device was out of order, these segments of data have to be discarded.
- Wet paper – sometimes water gets into the raingauge and wets the paper, leading to poor trace of the pen or to nonuniform temporal resolution caused by paper swelling at some places. These effects are difficult to correct.
- High pen pressure – this leads to stepwise recording of the water level in the rain container. These steps may be corrected if they are not much longer than the time step required for hydrologic applications.
- Blockage of the gauge by debris – can be recognised by a uniform rise of the recorded trace. It is helpful to have additional data available from a neighbouring raingauge to prove this malfunction. No correction is possible.
- Poor digitisation – digitisation may lead to doubtful results. Good training of the personnel doing digitising is required to obtain good quality data. Furthermore, the digitisation methods are of extreme importance. A very reliable method is based on digitising those points where the recorded line bends. Between these points, a linear line can be drawn. Other methods are discussed in VSA (1994). The digitisation is a very crucial point for ensuring the data quality because the person in charge may eliminate good data, add

erroneous data, or reduce data resolution. Experienced judgement of data reliability is required for this task.

- Siphoning errors – there may be mechanical problems with the siphon leading to incomplete siphoning, premature siphoning or no siphoning at all of the rainwater container. While incomplete siphoning and premature siphoning can be recognised on the chart and corrected for, no remedy exists for water overflowing from the rainwater container.

Some of these errors may be corrected - however, there are different opinions on the circumstances under which such corrections are allowed or when it is preferable to discard the data.

4.5. QUALITY CONTROL OF DIGITAL DATA

Institutions regularly working with rain gauge measurements have well-established procedures for data checking. These procedures range from checks of digitisation to checks of digital data. Plausibility checks may include:

- Tests of high intensities: there are physical limits to the maximum rain intensity per minute. This limit depends on the type of weather and climate. In a moderate climate, the intensity of 5 mm/min is rare but possible, but 10 mm/min can be regarded as a measurement error. For southern European countries this limit will be higher.
- Tests of frequencies of certain rainfall amounts: the rainfall depths of interest depend on the hydrological application, but statistics indicate that there are certain values feasible in a given region. The Emschergenossenschaft Water Authority in Germany checks all the events exceeding the following values:
  - > 5 mm in 5 minutes,
  - > 20 mm in 1 hour, or
  - > 40 mm in 1 day.
- Comparisons of rainfall data to those from a neighbouring station: large deviations in the maximum daily value per month, the monthly and the yearly volume are good indicators of periods with doubtful data.
- Graphical comparison to a neighbouring continuous raingauge and / or a neighbouring daily measurement station.

Example: The comparison of two adjacent raingauges (within metres) showing strange differences (see Figure 10).

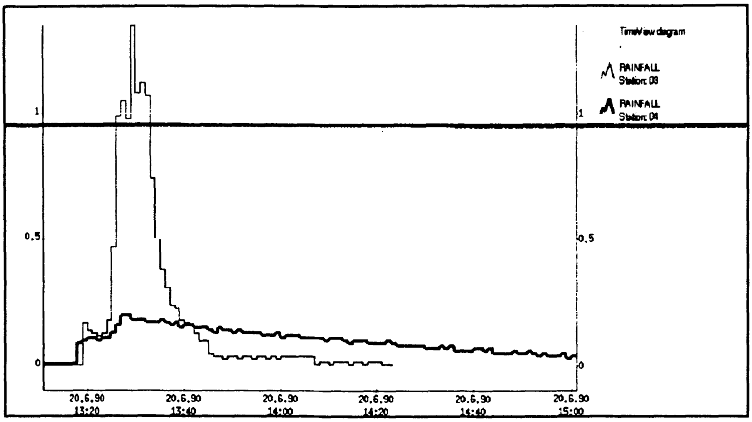

Figure 10. Hyetographs of two neighbouring raingauges .

The analysis of the data gave a strong indication that gauge no. 4 was blocked and required maintenance.

4.6. QUALITY OF HISTORICAL DATA

In many areas, the density of non-recording daily raingauges is higher than that of continuously recording gauges. The value of these daily data should not be underestimated because they are an excellent source for comparison. Such a comparison will check the data consistency, spatial variation of rainfall, rainfall depth errors over longer periods, or microclimatic differences within the area of interest.

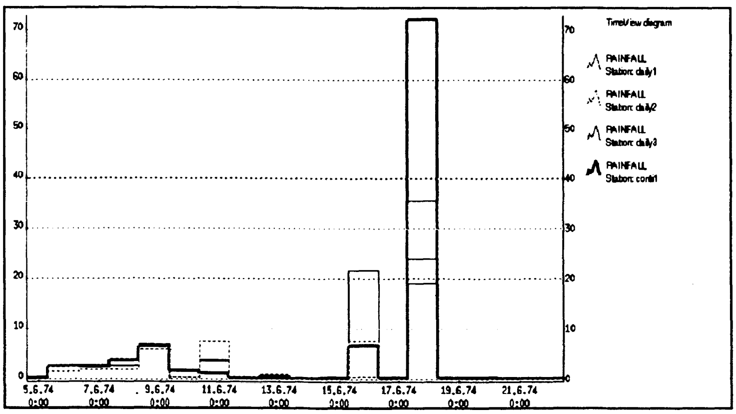

Figure 11. Analysis of representativeness of continuous rainfall measurements.

Example 1: spatial representativeness of continuous data
The following four time series belong to one catchment of a 50 km^2 size. A rainfall runoff model had to be calibrated for the catchment. Rain data were available from three

daily rain gauges and one continuous raingauge. The only raingauge inside the catchment was a daily raingauge. A graphical check (Figure 11) demonstrates that on the 17th June 1974, the high recorded rainfall depth at the continuous raingauge was not representative for the catchment and should not be used for calibration.

Example 2: measurement error in continuous data

Two time series were measured in the same catchment. After data discretisation to a comparable timestep, it was possible to judge the data quality. The graphical check, also known as the double mass curve (Figure 12), proves that the continuous data record shows much lower annual rainfall than expected in a moderate climate.

4.7. FILLING OF GAPS

Long-term simulation represents the state-of-the-art in the design of sewer systems and retention facilities, for calculations of water quality and providing a proof of an effective combined sewer overflow policy. Continuous simulation models can not deal with missing data which prevent calculations of a realistic water balance or long-term statistics (return periods of storms, floods, etc.).

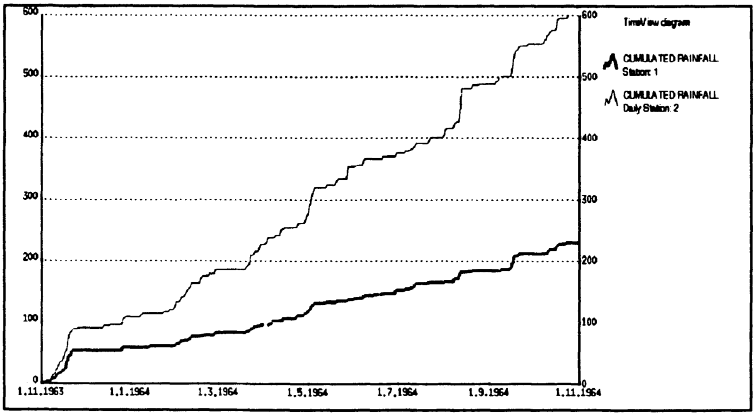

Figure 12. Comparison of annual rainfalls of two raingauges.

Thus, it is desirable to fill the periods of missing data in the time series. The objective of such a method is not to reconstruct the true event at a rainfall recording station (Einfalt and Fankhauser, 1993), but to create a time series which is plausible according to data from the neighbouring stations and which would hold in rainfall statistics analyses. This method requires a detailed analysis of the encountered meteorological situation. A routinely used method is to fill gaps with values from a neighbouring station, using a linear regression between the data from the two gauges.

The applicability of raingauge data to the gauge with missing data is very event dependent. It is high for uniformly distributed events and for events in which no raingauges are on the border of a rainfield track.

4.8. EXTREME VALUE STATISTICS OF RAIN DATA

The statistical treatment of data is always a reduction of information. The advantage of such "compacted" data is an easier way of their handling; the most prominent disadvantage is that the frequency of rainfall is not the same as the frequency of the hydrological effects (e.g., flow rate, flow volume, overflow volume, overflow pollution load, ecological impact, see Chapter 8 for details).

IDF-curves (intensity - duration - frequency) give the frequency of a maximum intensity of rainfall for different durations. While being widely used for rainfall-runoff computations, they have lost their importance for more current sophisticated hydrological applications. Their use is limited to the design of sewers in the pre-planning phase or under simple conditions. IDFs must not be used for designing important retention structures because such results are not reliable.

Hydrological design of sewers using a design storm, with time-varying intensity derived from IDF-curves, for all sewer sections is controversial. The optimal method would base the required rainfall-runoff simulation on a historical set of extreme events from (at least) ten years of rainfall data.

5. Radar Data

Weather radar provides rainfall data with a high spatial resolution (1 value per km^2), a high spatial coverage (40,000 km^2 per radar), and with forecasting capabilities. Because it is an indirect measurement of rainfall, different sources of error have to be noted and special training education is required.

5.1. RECORDING

It has already been mentioned that radar measures rainfall by measuring reflectivities. The equations for the conversion of reflectivities into rainfall intensities were extensively published and explained elsewhere (e.g., Collier, 1989). One main feature is that the assumed rainfall drop size, used in the sixth power in these equations, strongly influences the result.

Since the drop size distribution is most often unknown, a mean relationship between radar measurement Z and rainfall intensity R is used. Frequently applied is the Z-R relationship of Marshall and Palmer (1948). However, since different rainfall types imply different drop size distributions, further adjustment has to be considered. This is done by local point measurements leading to radar data adjustment by the data from raingauges.

A number of methods for radar data adjustment have been developed in the last two decades; Collier (1989) gives a good overview of these methods.

Further progress was noted in this field recently. The improved insight into rainfall generation and spatial dynamics resulted in improved methods for rainfall rate estimation, without focussing on a uniform Z-R relationship over the whole radar scope (Rosenfeld *et al.*, 1990; Braud *et al.*, 1995). New methods for rainfall rate estimation use spatially varying conversion factors (Rosenfeld *et al.*, 1995), make use of the vertical reflectivity profile information of the radar (Andrieu and Creutin, 1995), and take advantage of disdrometer measurements for the determination of the drop size distribution (Semke, 1991; Kreuels, 1991; Sauvageot, 1994).

5.2. ERROR SOURCES

There are some basic potential difficulties to be considered when working with radar data. The most important ones are:

- Ground clutter is the reflection of the radar beam by objects other than precipitation particles, typically high buildings and mountains.
- Blocking of the radar beam occurs when it is intercepted by obstacles such as high buildings or mountains, resulting in radar "blinding" (at least partially) behind the obstacle.
- Attenuation of the radar beam is caused by very high rainfall intensities. The consequence is a lack of information on the area behind the attenuation causing rainfield. Furthermore, such an effect can be observed for the whole radar image in the case of high rainfall intensities at the radar site.
- Bright band is an enhanced reflectivity belt due to snow transforming to rain, the water covered snowflakes cause disproportionately high reflectivity values.
- Overshooting of the radar beam occurs at large distances or a high elevation angle. In this case, the beam is above the rain producing cloud layer.
- Conversion of polar to cartesian coordinates leads to loss of information in the near range, where many polar grids supply the information for one grid square and to information overinterpretation at far ranges, where one polar grid may be the base for several cartesian grid squares (Atlas 1995).
- Conversion of reflectivity to intensity values is the most crucial point. Since radar measures the reflectivity of raindrops as a function of the dropsize distribution, it is not possible to deduce the rainfall amount directly from this measurement. Frequently, a telemetering raingauge network is used to adjust the radar data to ground measurements using a mean relationship (e.g., Marshall and Palmer, 1948), in spite of the comparison problems mentioned earlier.
- Practical time resolution is a limiting factor for radar systems which are serving other purposes than rainfall measurement (this is still true for most meteorological radar systems). Computer resources are not the limiting factor, since radar data processing is a matter of several seconds and data compression leads to very tightly packed data.

For each of these problems there are some at least partial solutions (see Table 2). However, the quantitative assessment of areal rainfall may still be affected, preventing a viable quantitative estimation. In particular, the first four points either depend on the event, or are regionally limited, or both. Consequently, a quantitative rainfall estimation is possible for the events and the regions which are not affected by these phenomena.

In many cases, the above mentioned solutions are not applicable because the existing radar system cannot be upgraded or only with high costs. Therefore, most radar data have to be carefully checked for the above mentioned sources of error.

TABLE 2. Radar measurement problems

Problems identified in 1985	Solutions
Bright band	two beam elevations
Ground clutter	radar filter (raw signal) / Doppler
Anomalous propagation	radar filter (raw signal) / Doppler
Adjustment to ground data	volume data and drop size distribution measurements
Intensity growing under beam	additional models required (hydrology, climate)
Evapotranspiration under beam	additional models required (hydrology, climate)
Beam overshooting	clever elevation strategies
Wavelength	depending on requirements, application
Attenuation	choice of different wavelengths

5.3. RAINFALL FORECASTING WITH RADAR

The extremely high density of points in radar measurements permits the viewing of radar data as "images" and a sequence of images as a "movie". An observer recognises the apparent movement of the rainfield as well as its development in the near future. This feature has been used for the development of a number of manual, semi-automatic or automatic radar rainfall forecasting techniques (Braud *et al.*, 1994; Bremand and Pointin, 1993).

Radar allows for more detailed spatial analyses; however, data treatment and analysis may be tedious. Furthermore, radar is not available everywhere where it would be useful.

Hydrological use has been made of automatic forecasting methods based on correlation techniques (e.g. Austin and Bellon, 1974) and pattern recognition (cell tracking) techniques (Einfalt *et al.*, 1990; Neumann, 1991). In the United Kingdom (Brown and Cheung-Lee, 1992) and the United States (Hudlow *et al.*, 1992), semiautomatic systems are in use, requiring an operator but allowing for the integration of other meteorological information sources (satellite, numerical models, etc.).

A raingauge network yields spatial information on rainfall with a very low resolution. However, since raingauges measure rainfall directly, they partly compensate for this disadvantage. Therefore raingauges are used for flood warning purposes (e.g., Delattre, 1989), even when radar data are available. A spatial-temporal analysis (e.g., Niemcynowicz, 1990; Einfalt and Fankhauser, 1993) allows under special conditions the determination of rainfield speed and direction, and furthermore a simplified forecast. However, raingauge derived forecasts have to be considered much less accurate than radar derived forecasts.

The usefulness of a forecast always depends on the hydrological application. If the user knows precisely his needs, he can be advised by the forecaster how to work with the issued forecast in an optimal manner. This improves the reliability of hydrological methods and, thus, increases confidence in their results, because the data quality is made more transparent.

For more information on limitations and use of radar rainfall forecasts see Zawadski *et al.* (1994). The most recent developments in this field can be found in Collier *et al.* (1995).

5.4. COMPARISON OF RADAR DATA TO RAINGAUGE DATA

Raingauges are point measurement devices and radar provides volumetric measurements of rainfall in the air. As a consequence, a rainfield may touch a raingauge but be neglected by the radar measurement at the same location, or a rainfield may miss a raingauge while being measured by the radar (Assem, 1989).

Furthermore, effects on the raingauge of such factors as poor siting, wind errors, miscalibration, wrong digitisation, or collector blockage may result in considerable underestimations of the actual rainfall amount.

On the other hand, radar does not measure rainfall intensities, but reflectivities, i.e., the degree of reflection of the radar beam by the raindrops. Since the size distribution of raindrops can be quite different, depending on the type of rainfall, radar data have to be adjusted by ground data. This results in a factor which has to be applied in processing the radar measurements.

Raingauges and radar measure at different times and in different temporal resolutions - the raingauge measures continuously in time, but the radar measures intermittently in steps of (usually) five minutes. Thus, the radar does not positively measure the same quantity as point raingauges which are much more sensitive to the spatial rainfall variability at the measurement site.

These two measurement techniques, radar and raingauge, complement each other. A combination of these two tools, in which the radar images are calibrated using the intensities measured by the raingauges, results in a better approximation of the areal rainfall.

6. Comparative Costs of Precipitation Data Collection

Comparative cost analyses have rarely been published. The approximate cost has to be considered as a function of

- viewpoint of the cost analyst (hypotheses, assumptions)
- standard labour costs in a country
- organisation performing the data analysis and verification
- synergistic cost effects

Therefore the two cost studies presented below differ greatly with respect to their respective costs. However, the cost of data collection cannot be considered as marginal, regardless of the assumptions in cost calculations. On the other hand, compared to the overall investment costs in urban drainage, and in particular in view of the design safety to be gained by using reliable data, a good rainfall database requires only a small part of the overall project costs. Furthermore, planning safety can most often be translated into investment savings, because project parameters can be more precisely defined and limited, rather than relying on the engineering practice to plan to be "on the safe side".

6.1. PRECIPITATION DATA COSTS FOR SWITZERLAND

The Swiss study (VSA, 1994) is based on the assumption of using the raingauge recommended by VSA; a tipping-bucket type raingauge with a collector surface of 200 cm^2, local digital storage, quartz precision timing, and data transmission. The cost for a non-permanent simple station is estimated on the following basis: the investment costs are calculated over a life time of 12 years, with an interest rate of 7%, the operation and maintenance costs are estimated on a yearly basis.

TABLE 3. Costs for the Swiss recommended tipping bucket type raingauges (in US-$)

	Simple Raingauge		**Permanent Raingauge**	
Item	unit cost	cost per year	unit cost	cost per year
measurement device	5000.00		16000.00	
installation cost	1700.00		3700.00	
investment costs		800.00		2500.00
maintenance (two visits/month)	3300.00		1200.00	
calibration (once per year)	800.00		800.00	
yearly repair costs	500.00		1200.00	
operation and maintenance costs		4600.00		3200.00
Total Yearly Costs		5400.00		5700.00

The cost of a permanent measurement station is estimated in a different way: the measurement device is more expensive because of an automatical control of the device, temperature measurement, and a modem connection for data transmission with a dedicated teletransmission line. At the same time, the cost for maintenance is reduced.

This approximate calculation of costs demonstrates clearly that the investment cost is considerably lower than the operation and maintenance costs. However, the automatic calibration and the automatic cleaning of the tipping-bucket gauge considerably reduces maintenance costs. These costs are very much a function of the distance of the measurement station from the base office of the personnel, as well as of the organisation of the maintenance work itself.

As far as possible, a single type of raingauge should be used in the measurement network, in order to obtain data with similar error characteristics and to facilitate a uniform maintenance procedure. For the same reasons, the criteria for selection of locations for measurement stations should be the same in the entire measurement network.

The costs of digitisation of historical chart data have also been estimated by VSA. Basically, the hardware investment for digitiser, plotter, and PC were taken into account as well as the development cost for suitable software. Furthermore, personnel costs were estimated for the data processing tasks of data screening (preparation), digitisation and quality control. The resulting costs ranged between 2700 and 8700 US$ per station year, with a mean of 5700 US$ per station year.

6.2. PRECIPITATION DATA COSTS FOR A LARGE RAINGAUGE NETWORK IN GERMANY

To provide a discussion basis for a future concept of rainfall data measurement for general hydrological use, an approximate cost estimation was performed (Einfalt and Weigl, 1996). The objective of the cost calculation was to prove that the traditional Hellmann gauge with paper chart recording is not the most cost effective solution for measuring precipitation over an area of 34 000 km^2.

The following table shows the costs for continuously recording raingauges (with radar in one case) providing the data of a density of at least one measurement point per 50 km^2. The costs are based on the fact that the raingauges are placed at wastewater treatment plant sites, so that there are no additional costs for personnel, travel, and maintenance can be linked to other tasks. Furthermore, digitisation is done on a routine basis without taking into account software development costs.

The result of the calculation indicated that the operation and maintenance costs (remote data transmission, operation and maintenance) were more important than the investment costs. The incorporation of new techniques such as radar does not necessarily increase the yearly cost of measurement and maintenance, while at the same time providing a higher spatial resolution of rain data. Furthermore, the more sophisticated concepts (radar plus raingauges or raingauges with data transmission) allow for more applications in the urban drainage field, such as real-time control or on-line warning.

TABLE 4. Comparison of costs of rainfall measurement for an area of 34 000 km^2 (in US-$)

	Hellman gauge		**Local storage**		**data transmission**		**radar / raingauge**	
Item	unit cost	cost per year	unit cost	cost per year	unit cost	cost per year	unit cost	cost per year
installation	3500.00	700.00	5000.00	1000.00	7000.00	1400.00	1000000.00	200000.00
operation	1000.00	1000.00	1000.00	1000.00	1000.00	1000.00	130000.00	130000.00
data processing	700.00	700.00	200.00	200.00	200.00	200.00	70000.00	70000.00
transmission	0.00		0.00		700.00	700.00	10000.00	10000.00
Total yearly costs		2400.00		2200.00		3300.00		410000.00
no. of raingauges		680		680		680		150
no. of radars								3
Overall yearly costs		1632000		1496000		2244000		1725000

6.3. CONCLUSIONS ON DATA COSTS AND DATA QUALITY

The collection of data of a reliable quality for hydrological purposes requires considerable sums of money for installation and operation of measurement devices. Traditional Hellmann type raingauges are not necessarily the most cost effective solution to creating a good quality database. More recent technology, such as tipping-bucket gauges, data teletransmission and radar measurements may considerably improve cost effectiveness and data quality.

7. Application of Rainfall Data in Urban Drainage

Rainfall measurements are subject to a large number of errors and uncertainties, whose impacts differ with respect to various hydrologic applications. Some applications require data with fine temporal detail (e.g., rapidly responding catchments), others require only the spatial detail (e.g., large catchments) or simply the data availability (e.g., real-time applications). So this chapter deals simultaneously with requirements on rainfall data quality and the choice of the appropriate kind of data for individual hydrological applications.

7.1. PROBLEM ORIENTED CHOICE OF RAINFALL DATA

The profession of urban hydrology has changed its character dramatically. The most important development is that piecemeal solutions are more and more replaced by comprehensive catchment-wide approaches that take into account the interactions between surface drainage, sewers, treatment plant, and receiving waters. Rainfall remains to be a major driving force of such processes as water and pollutant transport, hydraulic and pollution effects on treatment plants and receiving waters, etc. However, the cause-effect relations are becoming increasingly complicated. In general, the statistical significance of a rainfall event is not equivalent to the statistical significance of the resulting hydrological event.

Examination of the effects of a given rain intensity shows:

- At least for intense storms, it makes sense to assume that a flow rate in a storm sewer is proportional to the rain intensity. This is the basic assumption behind the century-old rational method for the design of storm sewers.
- It is less certain to assume that the flow volume is proportional to the rainfall intensity, because the volume also depends on the duration of rainfall.
- An overflow volume from a combined sewage storage tank is caused by rainfall, but its volume is definitively not proportional to the rain intensity. Among other reasons, the dry weather period might have been so short that the overflow storage tank is still partly filled from the previous storm. At other times, there might be no overflows since the same observed rainfall intensity might have generated flow rates that could be completely conveyed to the treatment plant.
- The overflow pollution load is not only influenced by the current and the previous rainfall, but it also depends on sewer sediment accumulation, sanitary sewage flow, etc. The current rainfall intensity is only one of many driving variables.
- Finally, a cause-effect relation between the rain intensity and ecological impact is very difficult to reveal, particularly as a proportionality of the kind "double intensity equals twice the impact".

The consequence of this complexity is that most questions in urban hydrology cannot be answered by examining (statistical) rainfall characteristics alone. In other words, the frequency distribution of causes (i.e. rain) is not equal to the frequency distribution of effects (i.e. runoff, flow, pollution, environmental impact). It is accepted now that for many hydrological investigations, unprocessed rainfall data needs to be used as input variable to simulate the hydrologic processes, and subsequently the (statistical) characteristics of the resulting effects must be analysed (Harremoes and Jensen, 1984; ATV, 1985; VSA, 1989). Figure 13 illustrates this relatively new concept.

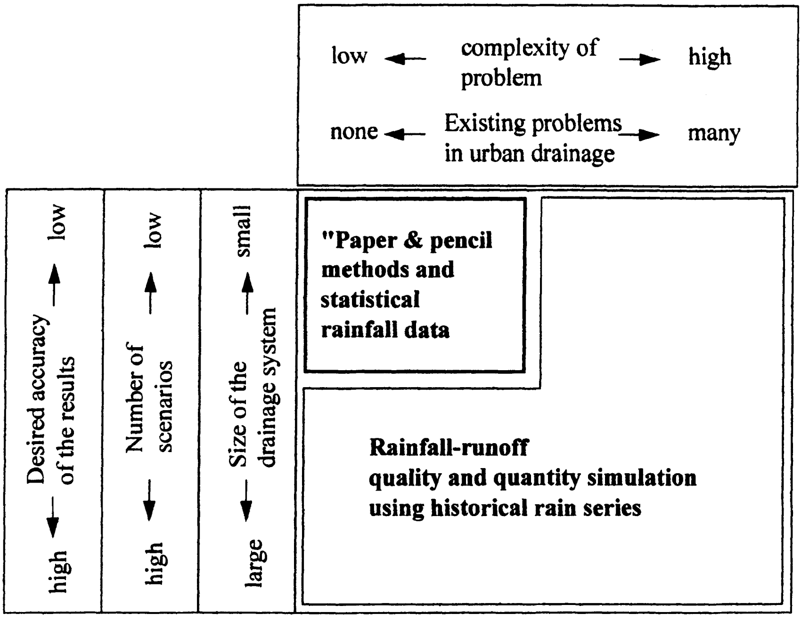

Figure 13. Modern approach to general master planning (after Schilling, 1991).

7.2. SPATIAL ACCURACY

When measuring rainfall with a point measurement device, it is implicitly assumed that the measurement on an area of 200 cm^2 is representative for a much larger area of several square kilometres. This assumption is a rough simplification of the reality which may play an important role for hydrological applications that are spatially oriented.

Table 5 postulates that design, dimensioning, planning, analysis, evaluation, and fine-planning of drainage systems do not necessarily require spatially distributed rain data. On the other hand, there is strong evidence that spatial variability of rainfall has quite an impact on the resulting runoff (Schilling, 1984; Niemczynowicz, 1984). Is that a contradiction? Schilling and Fuchs (1986) demonstrated that these effects play a major role when looking at specific events. A frequency analysis of the results, however, showed that the long-term statistics of an event series, with respect to peak runoff, were not affected by the spatial rainfall variability. What indeed are large differences in the cases of individual events makes apparently no difference in the case of long-term statistics.

In real time control (RTC), the recommendation must be different. Even in the planning phases of a project it is necessary to include spatially variable rain data in the analysis. The reason can be found in the general justification of RTC: one of its potential benefits results from the fact that the actual loading of a system is different from its design loading. Since the largest differences between the planned and actual loadings occur for spatially variable storms, it would not be meaningful to investigate the performance of an RTC system only with spatially homogeneous rainfall data. Any drainage system, by definition, is performing at 100 % for the design loading (and only for that!). The idea behind RTC is to make the system also perform better for loading and operating situations that are different from the design scenario.

7.3. TEMPORAL ACCURACY

There is an answer to the question of the required temporal resolution of rain data. If rain data time steps are chosen too long, the resulting simulated peak runoff rate is systematically too small. However, in slow responding catchments, the time steps might be chosen longer. To be on the safe side the time step, Dt, should be

$Dt \approx K / 5 .. 8$ or

$Dt \approx t_C / 3 .. 5$ or

$Dt \approx t_L / 6 .. 10$

where

Dt	the time step
K	the time constant of catchment (linear reservoir coefficient)
t_C	the time of concentration (time-to-peak, flow time)
t_L	the lag time (between centroids of the rainfall hyetograph and runoff hydrograph)

If runoff volumes are of interest, it is most important to model the transformation of volumes as water is passing through the system. A large pond that is filled within hours does not require such a fine time resolution as an overflow tank that is filled within minutes. A recommended time step might be

$Dt \approx t_F / 5 .. 10$

where

t_F = minimum filling time of a storage device

Note, that for CSO structures (tanks, weirs), the resulting time step might be rather short.

Synchronisation errors can occur if the clocks of neighbouring rain gauges and / or clocks in runoff gauges are not synchronised. These errors arise from inaccurate clockworks or their inaccurate adjustment. Whereas state-of-the art quartz clocks are very accurate, the traditional spring-driven strip chart recorders often produce timing errors in the order of 10 min/week.

TABLE 5. Required Rain Data Characteristics for Urban Hydrologic Applications (from Schilling 1991).

ENGINEERING TASK	Recording Period	Raingauge Location	Temporal Resolution	Spatial Resolution	Synchro-nisation Error	Volumetric Accuracy	Recording Gaps
1. DESIGN / DIMENSIONING							
- sewers	> 10 yr		block rain		≤ 30 min		
- retention / infiltration ponds	> 10 yr	relatively far away	≤ 30 min	homogen.	≤ 30 min	not important	not important
- CSO volumes	> 5 yr		≤ 15 min		≤ 30 min		
- real time control	some events	same region	≤ 15 min	1 gauge per sub-catchment	≤ 5 min		
2. VERIFICATION / EVALUATION							
- sewers	> 20 yr		1 min	homogeneous	≤ 10 min		not important
- retention / infiltration ponds	> 20 yr	relatively close	10 min	homogeneous	≤ 10 min	constant correction factor ?	compress data series
- CSO volumes	> 10 yr		5 min	≤ 5 km^2/gauge	≤ 5 min		
- real time control	many events	within catchment	≤ 5 min	≤ 1 km^2/gauge	≤ 1 min		not important
3. ANALYSIS / OPERATION							
- histor. event evaluation	the event						
- calibration / verification	some events	within catchment	≤ 5 min	≤ 1 km^2 per gauge	≤ 1 min	individual correction?	not allowed
- real time control	on-line						

The effects of these errors are not serious as long as only data from one rain gauge, without the corresponding runoff data, are used. Problems arise when analysing spatially variable data, or specific events. Synchronisation errors between rain gauges generate a pseudo spatial variability which might corrupt RTC investigations. Consequently, the requirements for synchronisation are quite divergent; they are low if only one rain gauge is analysed, but very stringent if more than one gauge (rain or flow) is examined.

7.4. VOLUMETRIC ACCURACY AND SYSTEMATIC ERRORS

Systematic errors are overestimations or underestimations of rainfall, which may occur for longer periods (e.g., underestimation of yearly volume), for short time steps only (e.g., insufficient time resolution), or for specific meteorological situations (e.g., snowfall, very low or high precipitation). It is usually very tedious if not impossible to correct for systematic errors in a realistic way. Therefore, data subject to systematic errors should be eliminated from detailed hydrological modelling.

The volumetric measurement error is the difference between the measured and the true rain intensity at the measurement location (i.e., a point). The problem of volumetric errors has been extensively investigated, see Sevruk (1985) for further references. For urban hydrologists, a few conclusions should be kept in mind: almost all types of rain gauges underestimate rainfall. This systematic error becomes larger with smaller drop size and higher wind speed. The annual error is in the order of 5 - 10 %.

The consequence for hydrological investigations sounds trivial; underestimated rainfall input data yield underestimated modelled runoff volumes. Because of the nonlinearity of hydrological abstractions, the runoff volume error is even larger. On the other hand, we model runoff for decades using biased rain data. If the models are calibrated, this effect is accounted for by model parameters such as the percent impervious area, etc. The question is still open whether and how the rain data should be corrected for systematic errors.

A number of studies dealt with the influence of volumetric input data uncertainties on hydrological applications. For the case of real-time control of urban drainage systems, it became obvious that real-time control strategies are sensitive to temporal and spatial (volumetric) uncertainties (Einfalt *et al.*, 1993). It can be further concluded that

- Catastrophic events are much more sensitive to a good timing of control actions than manageable events.
- The question whether an event is "catastrophic" or "manageable" is a function of storage volume of the network and thus of its control potential.
- Even in small drainage systems, the spatial rainfall distribution is crucial for a proper control strategy (see also Lei and Schilling, 1993).
- Overestimation of (future) rainfall volume is better (safer) than underestimation.

8. Availability of Rainfall Data for Urban Drainage Applications

At present, simulation tools are available to solve many of the new problems. One of the most important bottlenecks, however, is the availability of rain data that are suitable to be used as model input.

Many urban hydrologists have reflected on desirable rain data properties (e.g., Niemczynowicz, 1990). What would be the ideal situation? The urban hydrologist is "dreaming" of a database in which rain intensity data of the following properties are stored:

recording period:	20 years or more
temporal resolution:	1 min
spatial resolution:	1 km^2
time synchronisation errors:	1 min or less
volumetric accuracy:	< 3 %
recording gaps:	none

What is the real situation? Presently, only very few rain data series are available that match the requirements of urban hydrology. In Switzerland, there is no publicly available data set with a high spatial resolution. Much data have been recorded on strip charts by municipalities and sanitary districts without being analysed. Engineers apparently hardly ever use data that were recorded by the weather service. To overcome the enormous gap between reality and the "ideal" situation, we should proceed along three major tracks:

1. The weather services should be persuaded to provide rain intensity data that are currently recorded with modern equipment (remotely transmitting rain gauges, weather radar) under conditions that are acceptable to hydrologists and engineers.
2. In a cooperative approach, urban hydrologists, meteorologists, and practising engineers should review, check, correct, digitise, and disseminate historical rain data (usually available on strip charts) under acceptable conditions.
3. Researchers in hydrology and meteorology should work out some well investigated recommendations as to which kind of rain data should be used for specific urban hydrologic applications.

Unfortunately, in several countries the meteorological services are not able or willing to participate in such a cooperative effort. Therefore hydrologists have started to "help themselves" by using the available experience and expertise on rainfall measurement. These efforts should be encouraged and reported on an international level, in order to disseminate experience from one country to another and to standardise data use and the processing technology in different countries.

9. Examples

9.1. SEWER DESIGN (Zhu *et al.* 1996)

9.1.1. *Introduction*

In Europe, the common practice in the design of combined and storm sewer systems will be influenced by the new CEN standard EN 752 which is currently published as a preliminary standard ("pink print"; CEN, 1993). In this standard, it is recommended that large sewer systems should be designed using a "flood frequency criterion". For smaller systems, the traditional design storm approach might still be used. According to EN 752, allowable frequencies are a function of the drainage area and the respective hazards in case of flooding. For example, in central areas drained by gravity, the design storm frequency should be $T_r = 5$ yr, whereas the allowable flood frequency is set to $T_f = 30$ yr. In residential areas, the respective values are 2 and 20 years.

In sewer system analysis, the German terms "Einstau" and "Uberstau" are often used. These terms have the advantage that they are clearly defined with respect to hydraulics. A sewer with "Einstau" is completely full (i.e., under pressure flow), and in a sewer with "Uberstau", the water level reaches the ground level. However, the term "flooding" (in German: "Uberflutung") is often understood as a water level so high above the ground level that damage is caused. Obviously, the latter is much more difficult to assess using hydraulic models. In order to avoid this difficulty, a German task group defined that EN 752 flood frequencies are equivalent to allowable "Uberstau" frequencies that are 10 times larger (ATV, 1995). Table 6 summarises the definitions and interpretation of technical terms.

TABLE 6. Technical terms describing the hydraulic performance of sewer networks

Phenomenon	English term	German term	allowable return period for residential areas / city centres
pressure flow	surcharge	Einstau	n.a.
water level at ground level	flooding without damage	Überstau	2 yr / 3 yr (after ATV, 1995)
water level so high that damage occurs	flooding with damage	Überflutung	20 yr / 30 yr (after EN 752-4, 1993)

If urbanisation is finished, the drainage system is no longer modified and very long flood records are available, past operational experience might be sufficient to check whether the flood frequency criterion is met. In most cases, however, the application of the flood frequency criteria is only possible if the urban drainage process is simulated with realistic models; this involves a detailed description of the surface cover, the runoff losses, and the transport hydraulics including such phenomena as backwater, flows in looped sewers, internal overflows, surcharge, etc. In addition, long records of rainfall data have to be available and include a sufficient number of intense events overloading the drainage system. In principle, several occurrences per manhole must be computed, in order to create reliable statistical estimates. Even with the state-of-the-art computing power, it is very time consuming to simulate a large number of storms from a long record (e.g., 50 yr), in a system with many pipes (e.g., 1000) using a fully dynamic transport model.

It would, therefore, be very attractive to use the traditional design storm concept with only one computer run required. The validity of this design storm must be proven, though, using the performance criteria above.

9.1.2. *Methodology*

Traditional sewer system design applies a design storm that is based on local intensity-duration-frequency statistics of rainfall data. This approach suffers from the fact that the relation between the recurrence interval of the design storm and the recurrence interval of the effects (i.e., surcharge or flooding) is treated as a linear and spatially homogeneous relationship, although this is not correct in many cases.

In this paper, a different approach is chosen. Its working assumption is that a design storm can be defined such that it results in a sewer system which satisfies the required overloading frequency criteria. This design storm may not be universally valid, but is at least locally valid for the system at hand. The approach follows several stages:

1. At least one of the overloading criteria in Table 6 is chosen.
2. The hydraulic performance of a representative sub-system is simulated using a long time series of local historic rainfall input data and a calibrated fully dynamic hydraulic model.
3. For each pipe and manhole, respectively, the frequency of overloading is estimated from the long simulation output data.
4. The simulated overloading frequencies are compared to the permissible frequencies.

5. The diameters of pipes not fulfilling the requirements are increased, and the long term simulation is repeated. No more repetitions are required when all pipes satisfy the requirements ("a modified system"). Then, the modified system is just meeting the requirements.
6. A design storm with a relatively low intensity is chosen as a loading input for the modified system. If no pipe is surcharged, the design storm intensity is increased and applied again. The procedure is repeated until the first pipe is overloaded. The proper design storm intensity is found.
7. The complete sewer system is designed using the storm specified under 6.

9.1.3. *Application*

The method described above was applied to the combined sewer system of the City of Munich in Germany. The selected subcatchment is a central part of the Munich system, with some 200 pipes, a number of combined sewer overflows, internal overflow structures and outfall pipes. The total catchment area is about 910 ha, of which 325.3 ha are impervious. Three raingauges with 7 years of rain records each, SEW3, SEW6 and SEW10, are located within the selected subcatchment (Fig. 14). Nearby is another raingauge ("Riem") for which 34 years of rain data are available. All rain data are digitised with intervals of 5 minutes.

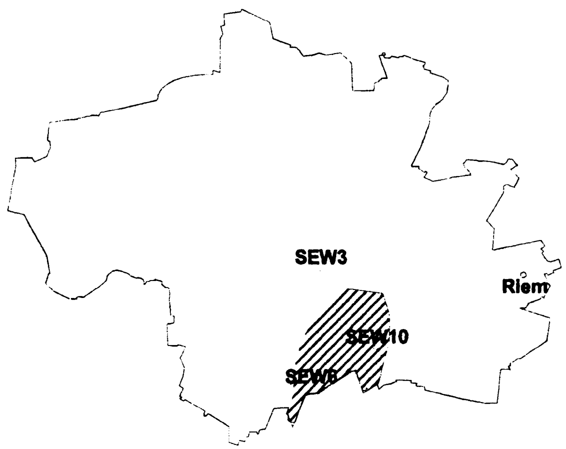

Figure 14. The study subcatchment (hatched) and rain gauges.

The HYSTEM-EXTRAN program, version 5.1 (Fuchs and Scheffer, 1993) was used to simulate the rainfall runoff process in the subcatchment. As a simplification, a manhole is considered flooded if more than 100 m^3 of sewage escape onto the ground. On a flat surface, typical for Munich, this corresponds to water ponding approximately 5 cm deep (an average pipe length 200 m, street width 10m).

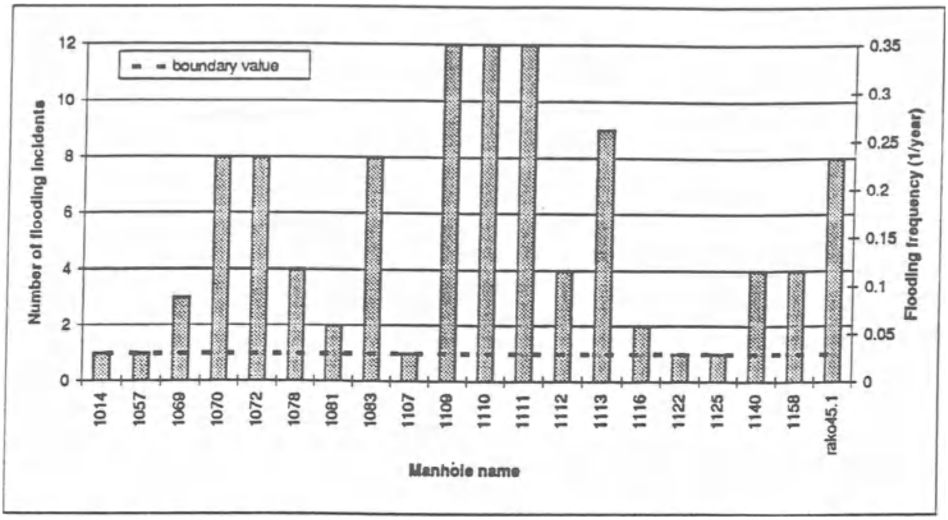

Figure 15. Flood frequencies in the subcatchment (Reim rain data, 34 years).

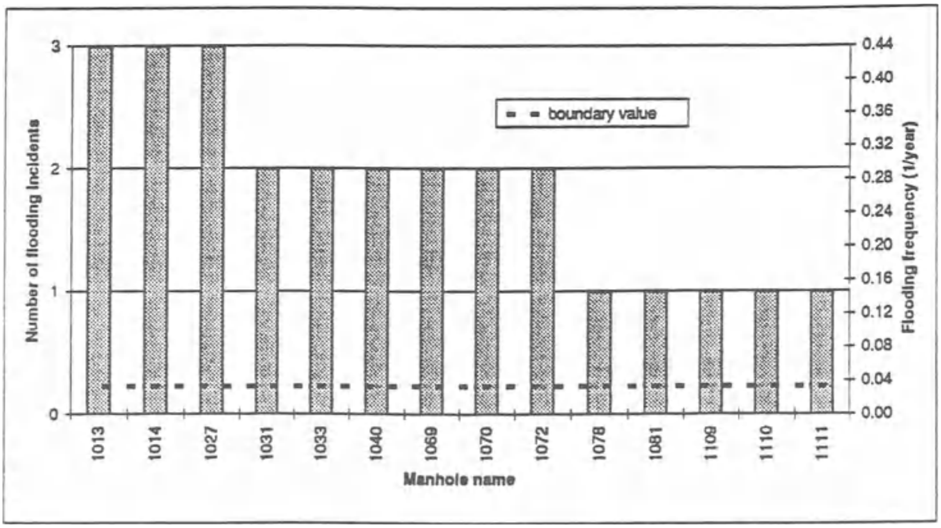

Figure 16. Flood frequencies in the subcatchment (SEW rain data, 7 years).

Using the 34 year rainfall record of the raingauge, Riem flood incidents were simulated with HYSTEM-EXTRAN. Figure 15 shows the performance of the system after the first iteration (i.e. after step 3.). At 15 manholes, flooding happens too often, i.e., flood frequencies are higher than the permissible frequency of 0.033 incidents per year. In order to confirm this finding, the 7 years of rain data from the three local raingauges within the subcatchment (SEW3, SEW6, SEW10), including their spatial variation, were applied. Here, the simulation results showed that flooding incidents were too frequent at 14 manholes (Fig. 16). Both simulation scenarios show that some pipes are under-designed with respect to flood frequency.

The system was iteratively modified until the flood frequencies simulated by the long-term rain series satisfied the required frequency at almost all manholes (Fig. 17). The modified sewer system is considered to be a properly designed system with respect to allowable flooding frequencies.

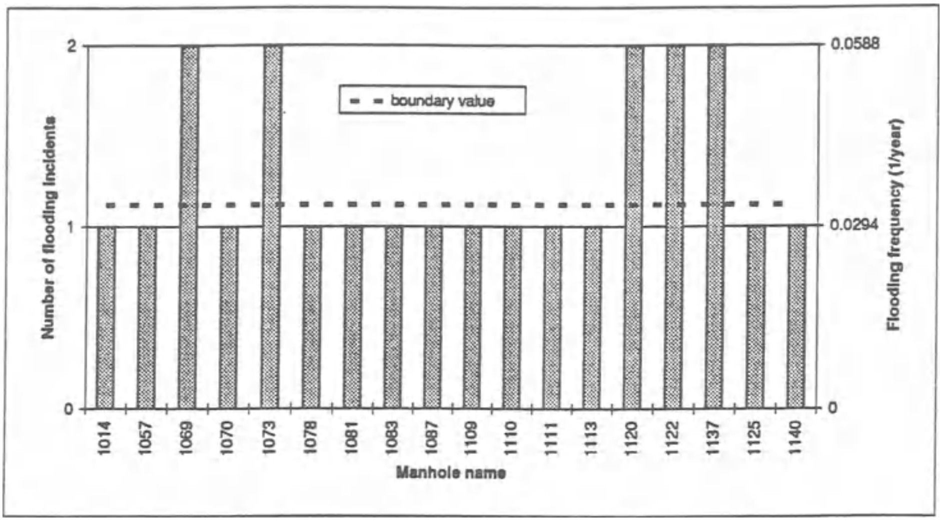

Figure 17. Flood frequencies in the modified subcatchment (Reim rain data, 34 years).

On the basis of the modified system, the design storms satisfying the flooding criterion can be found. As a first trial, traditional design storms with different durations (e.g. D = 1h , 2h and 3h) and a return period T_r = 3 years is applied. These storms are obtained from the IDF curves derived from the 34 year rain record of Riem. The 3-year design storms with different durations are applied to the modified system. As a result, no flooding is found. Hence, the design storm intensity is increased so much that some manhole water levels are just below the flooding level. Hereby, the appropriate design storm intensity is found with respect to the flooding criterion. In Table 7, these design storms are compared to the one found by the traditional IDF curve approach.

TABLE 7. Comparison of design storms obtained from two different approaches

(1) Duration (hrs)	(2) Design storm by IDF method (mm)	(3) Design storm by flooding simulation method (mm)	(4) * Relative difference (%)	(5) ** Return period of design storm by flooding simulation method (years)
1	28.2	31.83	-12.9	5
2	31.9	35.87	-12.4	4
3	34.3	40.13	-17	5
mean			-14	

* Relative difference =

** These return periods are taken from the Riem IDF curves.

9.1.4. *Results*

In the case study described above, design storms with a return period T = 3 years and different durations (D = 1h, 2h and 3h) are investigated. Table 7 shows that on the average, the design storms based on flooding simulations are 14% more intense than the design storms derived from traditional IDF curves. The return periods of the design storms in column (5) of Table 7 are taken from the IDF curves. They are greater than 3 years.

9.1.5. *Conclusions and Further Research*

The procedure described above couples the advantages of the traditional design storm approach (i.e., low computational requirements) and of multiple event simulations (i.e., statistics of the quantities of interest). By long term simulation, a locally valid design storm is derived that satisfies the sewer performance criteria in the respective system. The approach has been tested for the Munich combined sewer system, using the flood frequency criterion advocated in the new European Standard EN 752. Results in Munich show that the traditional design storms have low intensities. In Munich, a design storm with a return period of 4-5 years satisfies the flood frequency criterion. This approach depends on the assumption that a representative subcatchment behaves similarly as the rest of the system. In order to validate this assumption, the design storm should be applied to different subcatchments of the same system. If these subsystems are just about to flood too, the approach can be regarded as robust. The definition of "flooding" (i.e., an overloading causing damage) in this study is rather simplistic. Ultimately, models must be developed that allow an application of the concept of flooding which is more relevant to the local topography. These models must be fully dynamic transport models for flows, both in the sewer network and in the streets. The approach has only been applied for the "flooding with damage" frequency criterion. If other performance criteria must be met, the procedure should be tested for those criteria too. In our case, the traditional design storm is somewhat amplified. The universal validity of this finding can not be concluded from one case, and further study is needed. In the case study presented, flood damage is hypothetical and obviously its assumption affects the design storm correction coefficient. The knowledge of real flood damages or surcharge damages is lacking. Further field monitoring and studies are necessary.

9.2. GOTEBORG RADAR (Einfalt *et al.*, 1996)

9.2.1. *Introduction*

In Sweden, the first implementation of a model based real-time control (RTC) system in urban drainage is being carried out in the Goteborg region on Sweden's west coast (Lindberg *et al.*, 1993). The objective of the RTC system is to reduce combined sewer overflows (CSO) from the sewer system and at the treatment plant. One main feature of the system is flow forecasting, which up to now has been carried out using rainfall measurements by raingauges, in conjuction with a rainfall/runoff model (Gustafsson *et al.*, 1993).

Flow prediction to the treatment plant requires:

- a reliable data base with respect to rainfall, the main process influencing the flow behaviour, and
- a detailed rainfall/runoff model including the necessary topographical and network dependent information.

9.2.2. *Goteborg Catchment Objectives*

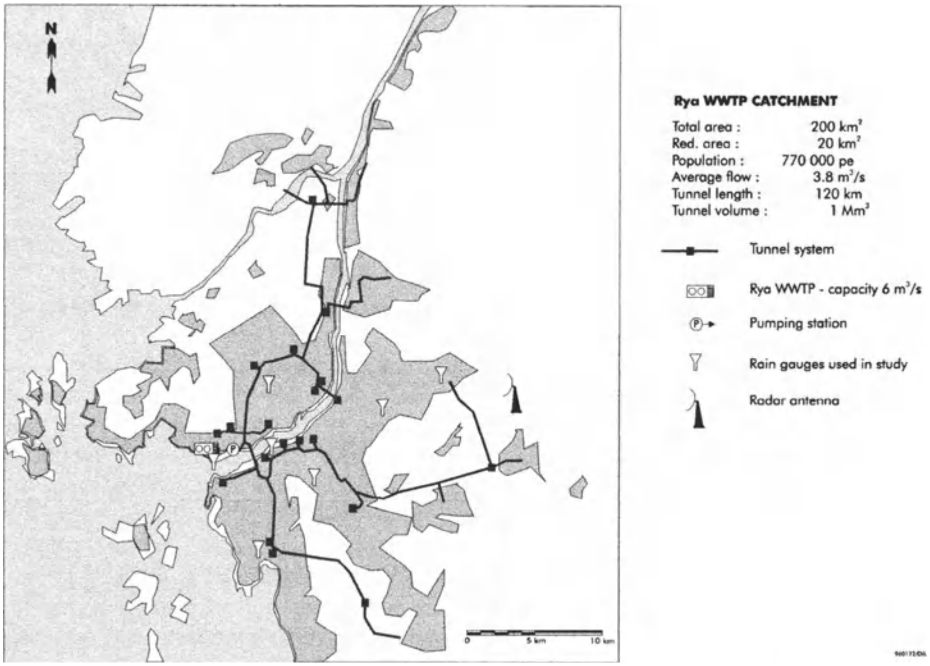

Figure 18. Goteborg catchment with locations of the raingauges and the treatment plant.

The Goteborg Regional Sewage Works (GRYAAB) serves about 770 000 people equivalent in the Goteborg region. The total catchment area of interest for the RTC system of Goteborg comprises 200 km^2 (Figure 18). An extensive combined tunnel system, of about 120 km in length, transports the wastewater to the treatment plant. The treatment plant is equipped with a sophisticated SCADA system designed for part-time unmanned operation (Lumley *et al.*, 1992).

Considerable work has been undertaken to prepare for using an on-line model based RTC system, including the use of different simulation models (MOUSE-NAM, MOUSE-PIPE, MOUSE-ONLINE), and different numbers of raingauges. The objectives of the current study are

- the comparison of raingauge based and radar derived catchment specific areal rainfall volumes (Lumley *et al.*, 1996)
- the assessment of the impact of different qualities of rainfall input data on the spatial rainfall distribution and on the accuracy of the calculated flow

9.2.3. *Tools*

For model-based applications such as RTC of drainage systems, it has been shown that spatial and temporal uncertainties may strongly affect the simulation results (Einfalt *et al.*, 1993). The tools that were used in the project had to be selected to be able to cope with spatially distributed, temporally varying input data. For the computation of the rainfall input data for the catchments, the radar rainfall processing tool SCOUT (Einfalt *et al.*, 1990) was used. The model of the urban catchment in Göteborg is based on the MOUSE simulation package (Lindberg *et al.*, 1992).

9.2.4. *Measured Rainfall Data*

Raingauge data were collected by the Göteborg Water and Sewage Works at seven locations in the Göteborg municipal area. These gauges are stand alone measurement stations, except for station no. 112, which is an on-line station located at the treatment plant.

Flow data were collected by GRYAAB at the treatment plant pumping station.

Radar data were supplied by the Swedish Meteorological and Hydrological Institute (SMHI) for a two-month period in the summer of 1995. During this time, there were three events of interest for GRYAAB, for which all the measured data were available. The radar at Göteborg is a C-band radar. SMHI provided radar images with a pixel size of 2 x 2 km, containing intensity data with a resolution of 1/10 mm/h.

9.2.5. *Results*

The comparison of the different number of raingauges used for radar data adjustment showed that for simulation of the water balance the use of seven raingauges is required, particularly for spatially variable rainfall.

TABLE 8. Event characteristics and results from adjustment using 4 or 7 raingauges

		RMS Event Error	Mean RG Intensity (mm/h)	Max. RG Intensity (mm/h)	Spatial Variation
17.05.1995	7 raingauges	0.08	0.71	3.60	1.17
(9 hours)	4 raingauges	0.07			
01.06.1995	7 raingauges	0.07	0.68	7.60	1.20
(16 hours)	4 raingauges	0.12			
12.06.1995	7 raingauges	0.69	2.13	33.20	3.64
(8 hours)	4 raingauges	1.11			

9.2.6. *Main Conclusion*

The project has demonstrated significant differences between radar-derived and raingauge-derived areal rainfall data. Due to a higher information density in space, radar measurements adjusted according to raingauge data, were not corrupted by non-representative local extreme values and yielded significantly better and more consistent simulation results than the raingauge measurements only.

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TOOLS FOR DATA ARCHIVING, VISUALISATION AND ANALYSIS, APPLIED IN MASTER PLANNING

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1. Hydroinformatics – Rapidly Developing Technology

Rapidly developing software (SW) and hardware (HW) components make it possible to design and develop completely new tools for master planning and global studies of highly urbanised areas [1].

Experience gained through pilot studies applied to sewer systems has clearly shown that mathematical modelling tools can solve very little unless data are available for the specified study. The inadequacy of input data sets has been a real limitation preventing wide-scale application of mathematical models in the past. The reason was very simple, data collection campaigns need to be extensive, are rather expensive and delay urgently required results. Because of the cost of data collection, very sophisticated tools and methods for data archiving, error checking, and data presentation and visualisation must be used.

There have also been problems with transferring existing data because of differences in operation systems, and incompatibility of SW and HW. Modelling tools have introduced a completely different viewpoint on structure and contents of the required data (such as long time series of rainfall data, continuous records of discharges, stages, and water quality parameters in sewers) [5]. Monitoring systems have been set up in order to provide answers to common questions by continuous evaluation of recorded parameters, and to analyse them after some necessary data processing. When selecting the type of facilities and equipment and their density for a monitoring network, the responsible team usually adopts a pragmatically sound approach being limited by financial sources. It is a hard job to set up monitoring systems in urbanised areas without some basic knowledge about the behaviour of such systems.

A very good example of a need for large-scale data collection campaign is a master plan of a sewer system. In this case, mutual interconnections among hydroinformatic elements (basic data, results of models, GIS tools, data from the monitoring network) provide a platform for a completely different approach to master planning the sewer system with its existing infrastructure [4].

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2. Data Collection During Master Plan Preparation

Many cities, towns and villages are going to reconstruct or at least improve their water infrastructure systems, because the pressure on polluters has been rising in recent years and losses of potable water in networks are high.

Municipal politicians often face a very different situation, and there are many mistakes that they can make:

1. they ask a consultant to apply classical design methods for sewer systems or water supply lines (as in the past), which will not provide them with a master plan in the modern sense,
2. they decide to start some action (such as data collection, GIS, mathematical model application etc.) without setting up priorities and objectives in the area of interest,
3. they set up a range of general objectives (sometimes conflicting with each other) which cannot be met because of financial limits, and
4. they neglect the possibility of applying most modern methods in urban drainage (such as real time control, new structures, etc.).

Sewer networks and wastewater treatment plants have to be considered as parts of a single complex system. The clear specification of objectives related to Master Plans is often severely underestimated. The basic aim of engineers in designing urban sewer systems is not only wastewater collection and flood protection, but nowadays also the improvement of water quality in receiving waters. During preparation of a master plan and setting up its objectives, it is recommended to focus on the following items:

- to improve the sanitary conditions in the city,
- to decrease the discharge of pollutants into receiving waters,
- to evaluate and/or decrease infiltration into sewer systems
- to reduce fast runoff components
- to understand the sewerage system and how it can be further developed,
- to prepare a platform for cost-effective reconstruction, operation and maintenance of sewer systems,
- to ensure the best possible joint operation of sewer networks and waste water treatment plants.

From this list of objectives, it is obvious that such a sewer system Master Plan cannot be a one-phase project. It is recommended to work out a Master Plan in several logically interconnected phases and divide each phase into sub-phases. Each sub-phase will be divided into stages through which the given objectives will be achieved partly or completely. Parallel processes should be followed in the Master Plan, although most of the results of previous phases will influence the structure and contents of subsequent phases. The phases of the Master Plan should include some of the following activities:

- methodology based on accepted objectives,
- data collection, pre and post processing,
- mathematical modelling of processes in the systems (sewer systems, waste water treatment plants, receiving waters, water distribution lines, reservoirs),
- short-term and long-term monitoring,

- presentation of selected information at various user levels (application of GIS),
- presentation of results, recommendations, and predictions.

The Master Plan, after completing the phases suggested in the result oriented mode, should consist of recommended operational and technical measures, including cost-benefit analysis of the application of such measures, and possible consequences with respect to the environment. An important component of the Master Plan is the transfer of knowledge from the consultant to the users. Implementation of new technologies in daily operation of aquatic systems may lead to the need for changes in the organisational structure of the Water Board. Also, the data collected and validated during this process must be used in daily operation of the system [7].

Some of those data must also be prepared for transfer into a more general information system (City Information System). In such a system, the data from the sewer system creates one layer, and if it already exists, then the other layers (streets, water supply network, owner...) can be used during data preparation and presentation.

It is clear that a proper set of hydroinformatic tools can greatly assist the team in preparing the Master Plan and support the city's interests [7].

3. Collection and Processing of Input Data

For a given strategy, specified by the objectives of the Master Plan, input data for further activities have to be collected. Data collection has to start exactly according to the confirmed phases of the Master Plan. It is only in exceptional cases that all data are available in regularly updated information systems for any given city [3]. The opposite case is more frequent – data have not yet been processed. For the purpose of collecting and processing the data required in most phases of investigation of an integrated aquatic system in urbanised areas, it is possible to identify four main groups of data:

1. basic data describing the sewer system,
2. hydrological data on the catchment receiving waters, reservoirs, and rainfall data,
3. data describing the quality of all water components included in the process (rain, waste water, receiving waters, drinking water).
4. data obtained from monitoring and results of simulations.

The amount of data and/or information which is collected or processed has to be limited by the detailed planning of the Master Plan. Detailed planning with a time schedule gives the consultant a chance to collect only those data which are important for a given phase or sub-phase. Relevant basic data are often available in a written form, and they have sometimes been sorted or validated using advanced hydroinformatic tools. Databases and/or Geographic Information Systems provide the opportunity for users to process and control the input data with high efficiency and accuracy. The need for a database system increases with the size of the investigated area. If the GIS package is not fully implemented with all the necessary data for simulations, it is highly recommended to use a specialised database, which will enable the user to collect the required data without special knowledge and special SW and HW requirements. The VABAS package designed in Sweden by Commune Data or VaKBase by Hydroinform

Ltd., are examples of such databases available on the market. Fig. 1 shows an example of such an approach applied to a sewer system in the Town of Vyskov (Czech Republic).

Data obtained from monitoring and results of simulations in the form of time series can also be stored in database. Tools for data transfer are a logical requirement of hydroinformatics.

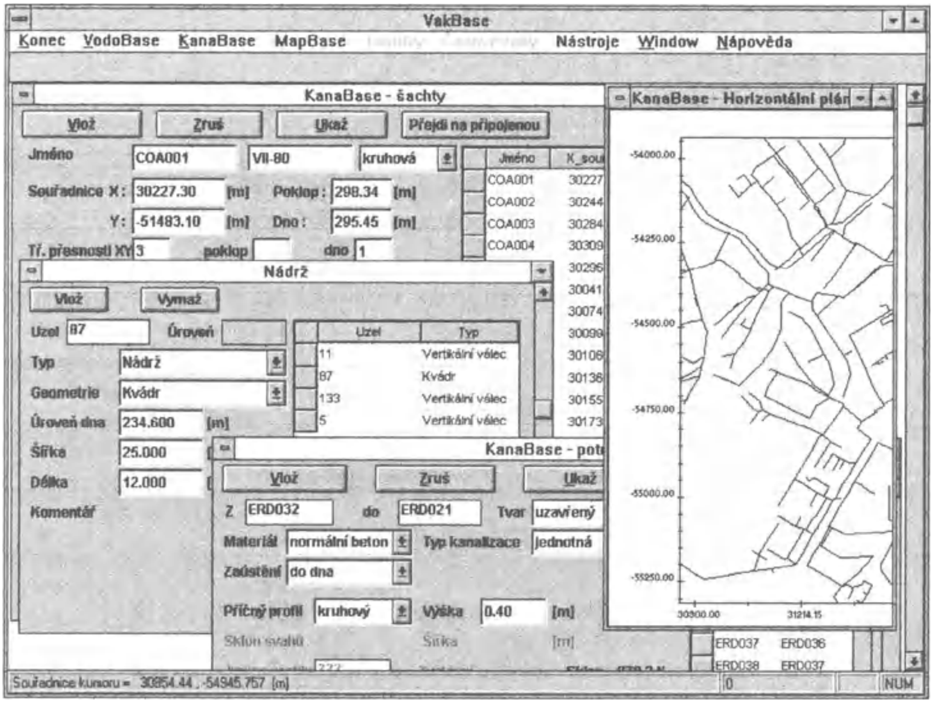

Figure 1. Sewer data in Vyskov processed in VaKBase (several windows are opened: table of manholes, table of pipes, table of retention basins, and horizontal plan of the sewer system in Vyskov)

4. Tools for Data Handling

Collected data must be processed by a tool, which can offer at least the following features:

- Data protection against damage – data must be protected against HW errors (power supply failure, physical damage of part of a hard disk...). Also protection against unauthorised changes must be guaranteed.
- Data access by authorised users – user friendly data access through a computer network must be guaranteed for personnel authorised to work with the specific data.

- Data protection against unauthorised use – a system of the names used, passwords and groups, together with rights to access and modify specific data, must be built very carefully. This system must on one hand protect the data against misuse or damage, but on the another hand, it cannot be unfriendly, or inoperational for daily routine work. The rights of different groups of users typically change during the project (different rights for collected data, validated data, data used for computation, and data used in daily operation of the system).
- Querying the data (preferably by both QBE [Query By Example] and SQL [Structural Query Language]).
- Basic analytical tools for global error checking (some predefined queries can be provided).
- Data exchange in different formats for communications with measurement devices (standard ASCII formats, or the formats used by mathematical models must be provided).

All mentioned services can typically be provided by any database system with a simple application written in it. For more advanced services, like graphical visualisation or even graphical editing, special database application, or application developed in GIS, a GIS system must be used. Such applications can run much more complex data analysis and error checking based on topological (network consistency) or even spatial attributes. Many times, different database applications are used for spatial (sewer network) and non-spatial (time series) data. Tools for handling time series are typically equipped with some statistical functions for data analysis.

5. Data Structure Design

If database application is created or modified for a project, the data structure must be defined very carefully. The following requirements must be fulfilled:

- Data must cover all demands of mathematical models, which will be used for project (items used in model must be directly stored in the data structure, or must be directly derived from the data stored in data structure; e.g., slope can be computed from the length and z co-ordinates, hydraulic resistance from material...)
- Basic spatial and topological data must be stored even if mathematical model is not used it (x, y co-ordinates) Those items open possibilities for transferring data into GIS.

A very important task is also to set-up criteria for acceptable accuracy of the data. In some cases we must accept a high price for very accurate data. While a 0.5 m error may be acceptable for x and y co-ordinates of a manhole, it is questionable for the z co-ordinate of the top of the manhole. It is unacceptable for the z co-ordinate of the bottom of the manhole, because it would completely change the computed slope of the connected pipes.

6. Coupling GIS with Processed Data and with the Results of Modelling Tools

Numerical models for hydrodynamic analysis are based on a certain topological schematisation. The solution domain consists of branches connected by means of nodes and various hydraulic structures (such as weirs, storage structures, pumps, regulation elements etc.), including manholes. The models usually have their internal database, in which the data are stored together with results of hydraulic calculations. Most models support the import and/or export of ASCII data. Hence the data can be presented in specialised graphical packages or can be imported into geographic information systems. A GIS model can be efficiently used for data preparation and verification, and for data post-processing and visualisation. The input to the model may involve creating a digital model of the terrain, map plans, digitising, scanning and vectoring data, creating cross-references for the numerical model, etc. The output part may use GIS as a graphical data processor for displaying results from a numerical simulator in the form of graphs, maps, thematic maps, 2D, pseudo 3D and 3D images, browsing results, using grid-analyst tools to work on the computed data and for geo-coding the data [1, 3].

GIS commands for selecting objects with the help of querying, selecting and SQL Select enable us to prepare input data for the model and to analyse the results. Using SQL Select, it is possible to create query tables containing information which was only implicit in the data base tables. SQL utilisation in combination with other basic units greatly improves the efficiency of the application of simulation packages.

There are two different types of input spatial data describing a model topology: we can distinguish between CAD oriented data models and data from numerical models. CAD models usually work with entities such as points, lines, polygons and surfaces. Each object has its own spatial co-ordinates and the objects are stored and visualised independently. Descriptive data attached to these objects may contain attributes such as the colour and shape of an object or data base attributes, such as the element number, values from catalogues etc. On the other hand, data from numerical models do not necessarily have spatial co-ordinates (x, y co-ordinates are usually used for pipe network visualisation, but not really for the simulation itself), but ground elevation data may be included. There are several types of cross-reference data describing the pipe network topology (start and end node attached to a branch). Data exchange between these two models requires the creation of additional operations such as geo-coding, creating cross-references and data pre-processing. As a result the data can either be used in CAD systems and visualised with respect to their spatial co-ordinates, or the same data can be used as an input into a numerical model.

Raster data are mainly used for creating an intelligent background for map layers [2]. The map layers contain additional information that is neither stored in a database nor available in a vector form. Raster data can represent descriptive data in more detail, e.g., images of a certain place in the pipe network, schematic pictures or drawings.

The choice of GIS modules should respect the specific requirements of the end user, the type of data that will be maintained, and the size of the project. It is possible to distinguished between “small” and “large” GIS, because the hardware and software requirements (the choice of GIS modules, database, hardware platform) may limit the project. The coupling of GIS with numerical models can be one of the following three types:

- GIS does not exist; if there is no GIS at the beginning of the project it is possible to develop a new application in the selected GIS and use it for data preparation, verification, analysis, and also for presenting hydraulic modelling results.
- GIS exists, but does not include layers with the data needed for hydraulic modelling. In this case the existing GIS is a source of general information. We need to develop additional layers and data attributes, and these layers are later entered into the GIS.
- GIS exists and contains a layer with the data we need. Then GIS is a source of data, there is a structure already created to contain results from the hydraulic model, and the structure may be only slightly modified, if and when necessary.

The following picture shows a map consisting of different layers (Fig.2); pipe networks (drawn by lines between two points with known x, y co-ordinates), manholes (a symbol at a x, y position), several layers with map information such as blocks of flats, green areas, and a river (drawn by means of lines, polylines and areas). Despite their common cross-references, the individual objects on the map are drawn independently. Since the descriptive data are attached to graphical objects, the database information can be displayed interactively on the screen or stored in a browse table. Each graphical object can be shaded according to the value or function of its descriptive or graphical attribute, e. g. the object length is shown in the picture. Descriptive data without any graphical representation can be geo-coded, i.e., using a common attribute they can be assigned to the existing geographical data and can be visualised in that data. It is also possible to create joint tables or to aggregate data.

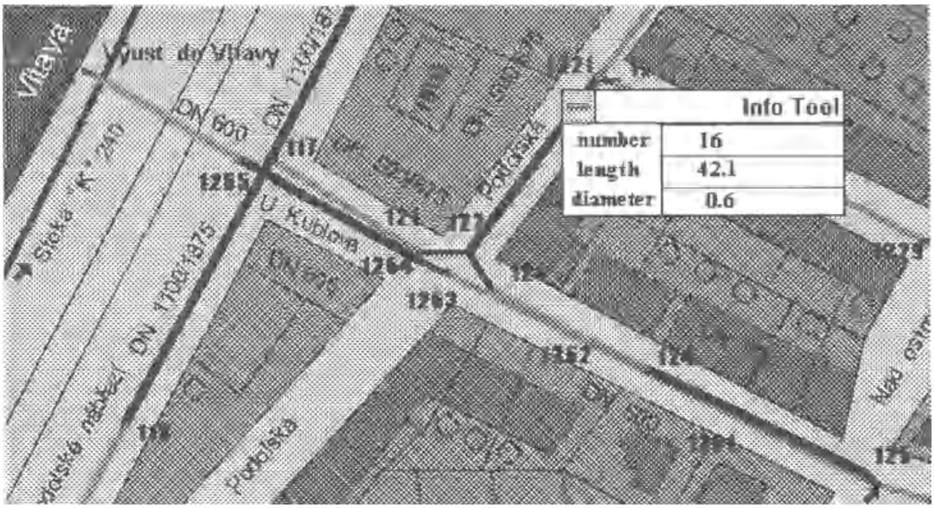

Figure 2. Map with different layers.

A powerful SQL language makes it possible to display objects satisfying certain conditions. We are able to display pipes within a given length range or with a diameter bigger than a given value, etc. Moreover, geographical operators "within", "intersects", "contains", "contain entire" can query data according to their geographical location. Hence it is possible to display intersecting pipes and manholes that are located within a certain street, etc. The results of SQL queries are stored in separate tables and they can be later used to perform desired operations, or to create a new mapper. Graphs and simple statistical functions are also usually a part of GIS modules. Data are mostly stored in an internal or external database and can be imported or exported in several data types. ASCII and xBASE files describing both geographical and descriptive data are almost always supported; most systems usually recognise DXF files for graphical data.

A Digital Elevation Model (DEM) can create terrain contour lines. Such DEM can help in manual or automatic generation of the catchment. Coupling DMT with a numerical result of simulator can bring a different view of the computed results. Various GIS modules, including statistical tools, grid-analysts, databases and data visualisation packages, represent very powerful general tools, which can extend the use of numerical models and support the analysis of their results.

GIS, as a part of hydroinformatics systems, has the potential to perform complex data analyses. Data visualisation techniques and analytical tools can improve the everyday work of engineers, scientists and researchers. The link between these models and numerical simulators supports integration of modelling systems, knowledge-based tools and information systems.

7. Conclusion

Utilisation of hydroinformatic tools in sewer network projects and master planning is unavoidable, especially for the investigation of complex integrated systems of sewers – combined sewer overflows – wastewater treatment plants – receiving water systems in large cities. The current generation of simulation tools and other related hydroinformatic components make it possible to study most of problems in their full complexity in both natural and man-made aquatic systems in urban areas. Proper selection and use of the tools for data archiving, processing, validation, visualisation and analysis can provide good quality data for simulation and decision making process.

This contribution showed the role of data handling in all processes of the applications of hydroinformatic tools to solving basic problems of sewer systems. The Master Plan of a sewer system was used as an example. A procedure for relevant data preparation was presented and proper steps in data surveys were recommended.

Once the pertinent data for sewer networks, together with time series of hydrological or hydrodynamic data become available, it is possible to prepare a Real Time Control strategy for aquatic systems in urbanised areas [6]. RTC investigation and consequent application, primarily in sewer systems, leads to cost savings. Unfortunately in most cities, such data are not yet available in the required quality and quantity to allow an adequate analysis of the system.

In order to build a well-established platform for investigating and describing the behaviour of complex systems, and to integrate all elements of the system by application of simulation tools (models), new approaches to master planning have recently been publicised.

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APPLICATION OF GIS IN URBAN DRAINAGE

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1. Introduction

The first part of this contribution deals with the application of GIS technologies in acquisition and manipulation of data for physical assets. In principle, there are two major groups of GIS applications; the above mentioned being one of the two. The second group of applications is matching the physical asset data with simulation models, both for rainfall/runoff and water quality simulations, and it is dealt with later.

The practice of water management projects is changing. This is characterised by a shift from an almost independent application of simulation models towards a more integrated approach, characterised by interaction between high levels of information processing and simulation packages. This section deals with the number of improvements in simulation packages that become inevitable after the introduction of a new information product (GIS in this case). Designing a storm drainage system by means of an existing simulation package with a relatively new information processing tool, if done properly, results in improved reliability of the final product (the project design).

The emphasis here has been placed on the cases in which a number of changes have been introduced in simulation packages, and a number of interfaces (gluing routines) have to be developed for this purpose. The aim is to make GIS a real problem solving tool rather than just a new method of presenting graphical inputs and results of simulation. It is believed that this is a way of proving the mutual and everlasting interaction among knowledge, information, domain information and again information and knowledge (Abbot 1994). An example of application of the object oriented GIS under the framework of an object oriented programme language is given by Ruland and Rouve (1994). The work presented in this paper is based on the application of geometry centred GIS packages and of the related database.

2. Data Suitable for GIS Operation and Selection of GIS for Storm Drainage Projects

The following two major GIS concepts are currently in use (as shown in Fig. 1):

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- geometry-oriented, in which the system of points, lines and polygons is applied for creating its contents, and
- object-oriented, in which the real world objects (parks, houses, streets, etc.) are dealt with directly by forming object classes, prototype and instances in a hierarchical manner.

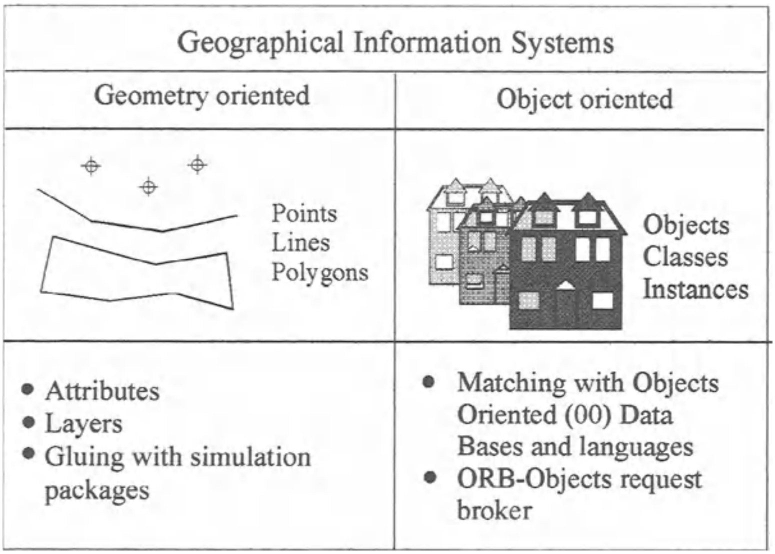

Figure 1. Concept of GIS.

The following groups of data needed for urban drainage modelling are suitable for GIS operations:

- rainfall (single event, or series of events) obtainable either by manual or automatic digitising of the existing historical records, or by automatic transfer from digital recording equipment (the preferable time resolution is of the order of 1 minute);
- catchment boundary – separating runoff contributing areas from the noncontributing ones;
- digital terrain model with the attributes necessary for further analysis (catchment delineation, advanced analysis of the flow pattern, “islands”, etc.);
- general purpose land use maps that serve for the assessment of land use in subcatchments and for the assessment of the contribution of parts of subcatchment to runoff, as well as for creation of the appropriate input files;
- drainage network information (proposed or existing);
- surface runoff obstacles and their attributes, and
- catchment delineation, or discretisation of a catchment into subcatchments, assigned to the computational nodes (inlets) of the drainage network (this

can be either already available or just an intermediate product in data handling).

Most of the above information can be obtained from a combination of information sources (paper maps, photogrammetry images, scanned images, automatic field surveying) with the commercially available GIS packages. In order to match the sources of information with GIS packages and the output of GIS with rainfall-runoff models, several matching (or gluing) routines were developed, and will be presented in the following.

Instead of being limited to one particular data base, the present approach is conceived so that it enables an easy incorporation of any of the following selected GIS or graphical packages in current use for handling a storm drainage project: *ArcInfo, Intergraph, IDRISI, AutoCAD, MapSoft* (Mihajlovic and Cvijetinovic 1992). This approach, of course, does require the appropriate interface routines to be developed for matching the various sources of data (photogrammetry, satellite image, GPS, field surveying or digitised paper maps) with the particular GIS package. The concept of a particular user interface depends not only on the particular GIS package, but also on the particular field of application (flood-plain management, water supply, irrigation, storm drainage, etc.). As pointed out in many papers, the introduction of GIS does provide an enormous potential for improvements in versatility and reliability of simulation and management packages, but many of the developers and users of interfaces fail to use this opportunity for making a real breakthrough by benefiting from the full potential that this tool offers. The selection of a GIS package for a particular application should be accompanied by a careful examination of each of the elements in the string of products applied. It seems appropriate to mention that some of the software packages are versatile, reliable and flexible enough not only for matching with GIS, but also for creating a modern product of an integrated hydroinformatics chain of activities. On the contrary, many of the existing simulation packages used in hydraulic engineering are based on assumptions and methods that were used before the introduction of computers (or even pocket calculators), and such methods and products (databases, computer graphics, etc.) are not improved by modelling.

The author and his colleagues have developed a series of interfaces linking sources of data with GIS packages and simulation models for urban storm drainage projects. They have witnessed a number of problems that the users of particular simulation packages faced after purchasing a GIS package without being aware of the range of problems and of the amount of additional development needed before their simulation package interfaces with the purchased GIS. Selection of GIS has to be preceded by thorough analysis of what operations are already included and what has to be developed, since most GIS dealers overestimate the potential of their packages in particular fields of hydraulic engineering. In addition to this, the feedback from the application of a GIS supported simulation package has to be used for upgrade of both the simulation package and users' interface (or of the GIS itself in the long run). As a conclusion, one can state that the selection of GIS for a particular (group of) application has to be made after the assessment of the amount of additional developmental work that has to be done.

3. Development of Interfaces

Gluing routines at stage 1 in Fig. 2 (which shows a general scheme of phases of application of GIS and user interfaces) are closely related to the sources of input data. Since the conversion from scanned and digitised paper maps is well supported by the majority of commercial GIS packages, and can be performed by the general purpose routines, and since automated field surveying is closely related to hardware used (generally equipped by its own software), the greatest effort was placed on the development of automatic pattern recognition and texture analysis for use with photogrammetry images. For example, in order to generate a land use map, high resolution satellite and/or airborne remote sensing images could be used for automatic land cover features detection. General principles of data pre-processing and post-processing are shown in Fig. 3.

Once the data are made fully compatible with GIS packages, they can be treated in a standard way (GIS operations). A number of quality packages serving this need are available on the market and new ones, even more advanced, are emerging. For the application of storm drainage simulation models, several operations have to be performed, such as (stage 2 in Fig. 2):

- discrimination of various types of pervious and impervious areas;
- analysis of DEM (Digital Elevation Models) for possible recognition of flow patterns;
- matching underground infrastructure (sewer network) with surface terrain features;
- delineation of the catchment in subcatchments and assignment of land features;
- creation of the proper input files etc.

One particular issue is the division of a catchment into subcatchments. Having all the input data in the form of images or files, this problem can be defined as finding all cells that supply the chosen cell, representing an inlet of drained water. It may appear straightforward to solve the subcatchment problem by simple calculations of slopes for all cells and assuming that water from each cell will flow in the direction of the steepest slope. A simple method is to take a seed cell and map all cells from which water can flow into the seed cell. All such cells are then made seed cells and the process is repeated until no new cells are added to the catchment.

However, most of the approaches can be used only for large-scale problems, where local irregularities are masked by the averaging procedure (in making any kind of grid, regular or irregular, the space averaging is carried out; all local data inside one element are substituted with an average cell value). For small-scale problems, such as the subcatchment delineation in urban areas, the man-made irregularities are important and they can not be neglected because they determine the flow pattern. The program must take into account the land cover data, and some set of rules must be introduced for different situations. Ideally, the program would use computer vision techniques together with land cover data to make a numerical evaluation of different situations, and, by accessing the knowledge base built around this procedure, an expert system would produce results that would be comparable to those of manual work.

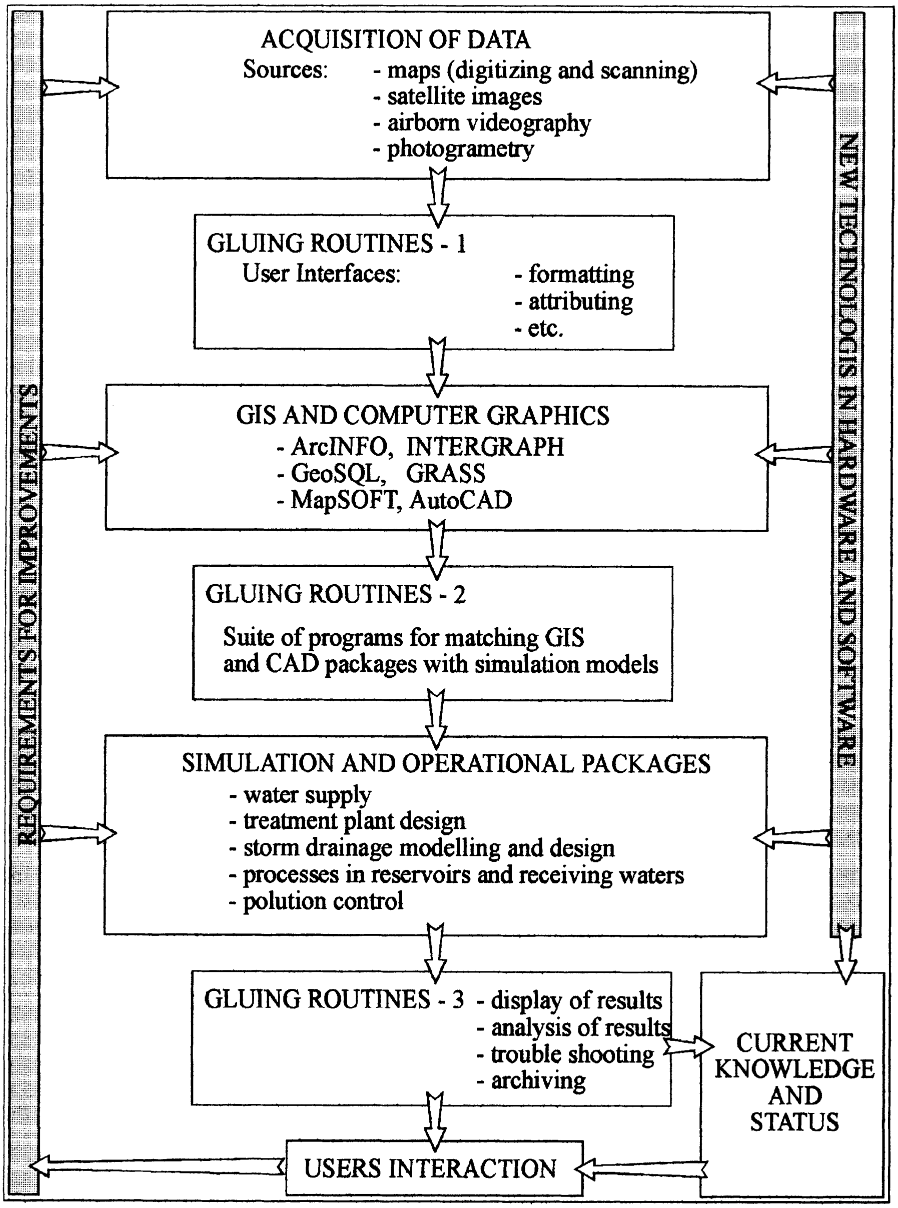

Figure 2. Phases in application of GIS and remote sensing and users interfaces (gluing routines).

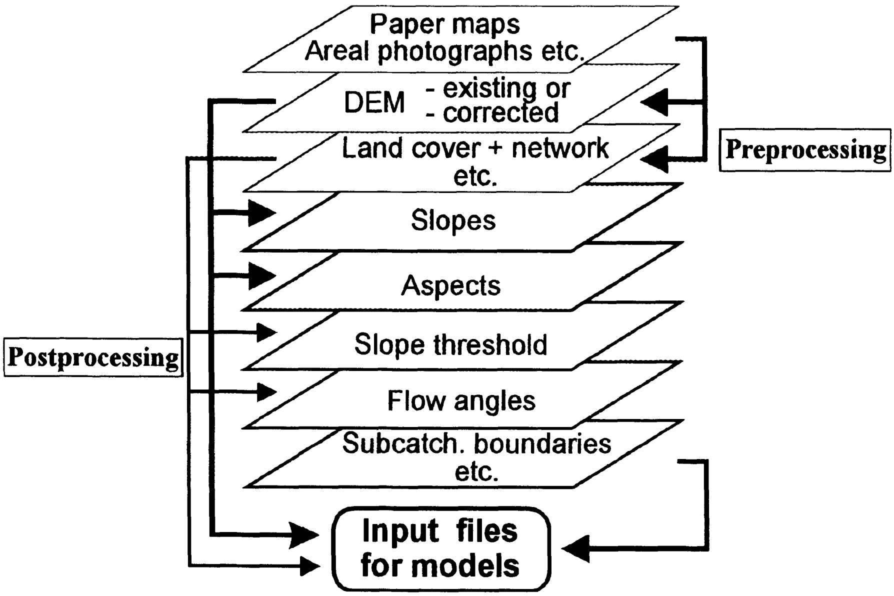

Figure 3. Pre- and post-processing of land use and Digital Terrain Model data for rainfall runoff modelling in an urban catchment.

There are several approaches to this problem. The general idea used in the present approach to subcatchment delineation (with regular spaced grids) is to use a temporary image that can be created in one pass, and that will hold data of all possible directions of inflow to each cell. Using this technique, the computational time can be significantly reduced. Also, during the creation of that temporary file, any exception caused by the cover file can be easily introduced. The addition of new images that hold some calculation rules may affect only this temporary image.

Keeping in mind the fact that each cell can have only eight neighbours, a very compact presentation of the flow direction map can be made by using bitwise operations. Assigning one byte to each cell, all neighbours of one cell will have their accompanying bit, that indicate whether that cell can supply the referenced one with water or not. The value of the flow direction map for each cell carries sufficient information to build the subcatchment image in the next stage. Because of bitwise operations, the algorithm is fast and efficient.

After making the flow direction map, the procedure for finding the boundaries of subcatchments is quite simple:

1. Find the highest inlet, mark it as a current cell,
2. Search through the fired bits of current cell, go to the first set bit, and use that cell as a current one,
3. Store the path used for the later return procedure,
4. If all bits are zero, then the cell is at the boundary of the current subcatchment, mark it as a member of the current subcatchment,
5. Step one cell back, and make it as a current one,

6. If this is the last cell in return path, end the search for the current subcatchment,
7. If there are more cells in the return path, continue with the search procedure from step 2.

4. Examples

4.1. EXAMPLE 1

Miljakovac is an experimental catchment in Belgrade (area = 25 ha) that has been used for rainfall-runoff and runoff water quality measurements for more than a decade. Figure 4 shows some intermediate results of GIS operations applied in study of runoff processes in the catchment, in which input files for different urban drainage models were prepared (IRTCUD 1992).

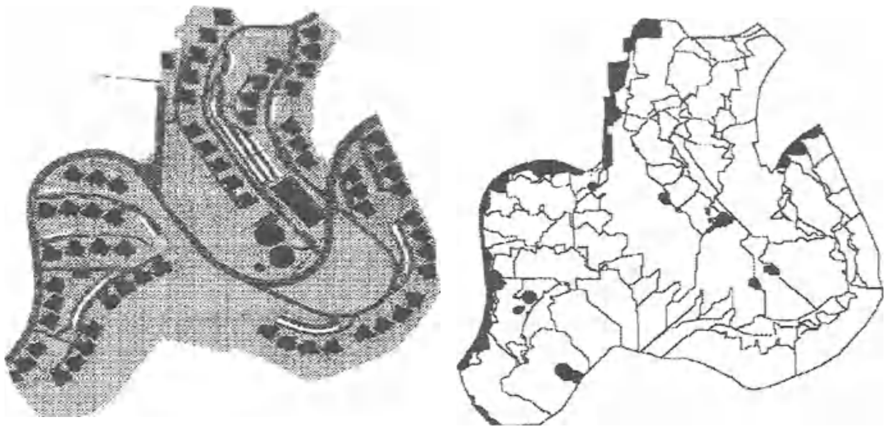

Figure 4. Miljakovac experimental catchment – land use pattern (left) and calculated subcatchment boundaries (right).

4.2. EXAMPLE 2

Developed tools were used for processing the raw information (maps, satellite images, digital information on ground levels) for the portion of the City of Dresden having the area of about 5.9 km^2 and about 800 sewer pipes (IRTCUD-ITWH 1993). Detailed analysis enabled preparation of reliable input data for HYSTEM-EXTRAN (German urban drainage simulation package). Some results are shown in Fig. 5. Estimation of needed resources was also done as a part of this study.

4.3. EXAMPLE 3

In a recent design of the storm sewer system of Kladovo (a small city by the Serbian-Romanian border), input files for the BEMUS model were created using a custom developed GIS-based software (Djordjevic and Prodanovic 1995). The catchment is about 2.7 km^2 in area and is serviced with about 400 sewer pipes. Some results are shown in Fig. 6.

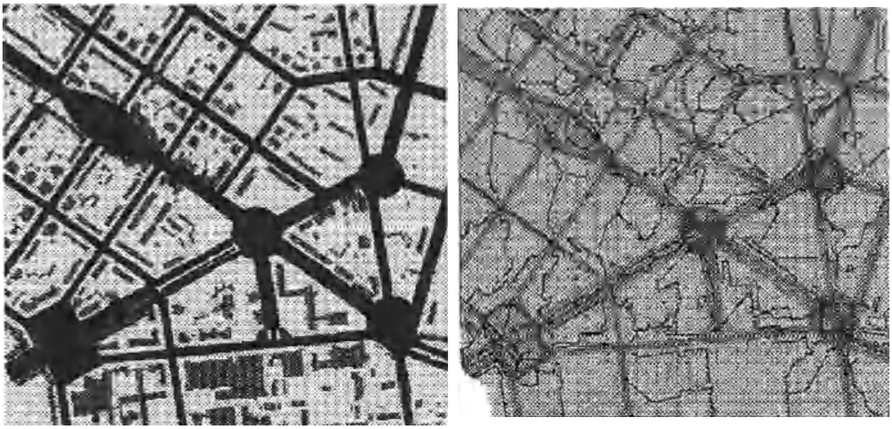

Figure 5. Central part of the City of Dresden – land use pattern (left) and calculated subcatchment boundaries (right).

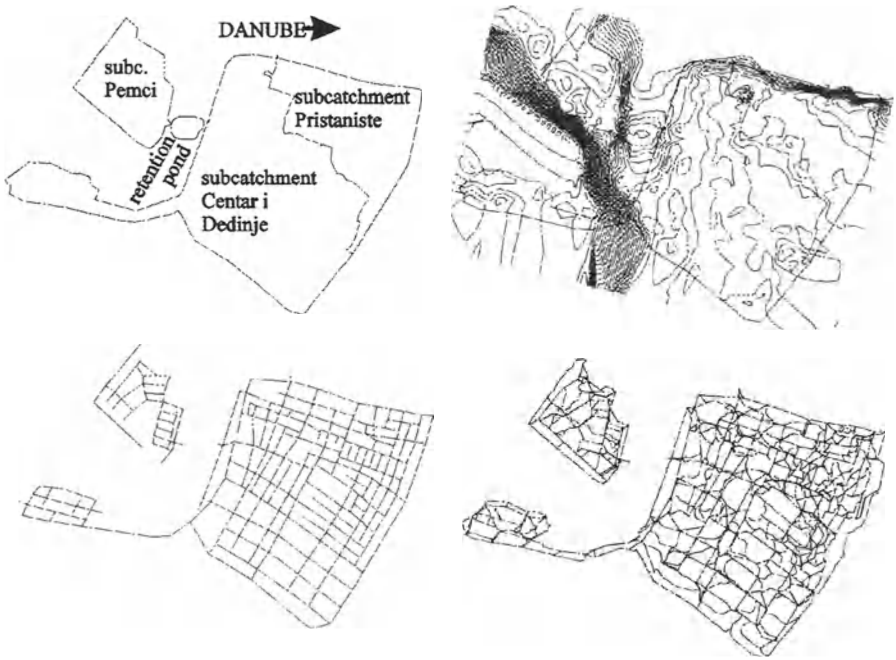

Figure 6. City of Kladovo – three main subcatchments (upper left), digital terrain model (upper right), sewer system (lower left) and subcatchments.

4.4. EXAMPLE 4

A GIS approach to data preparation and interpretation has been applied for reconstruction of the network of open channels for a part of the City of Novi Sad. A step by step presentation of the procedure will be done during the ASI, in the form of computer animation.

5. Conclusions

The emphasis in this section is placed on two approaches that inevitably should be used in the study of runoff in urban areas, namely, the physically based modelling of the runoff process including the simulation of surface flow by solving mass and momentum conservation equations, and using GIS-based tools for preparation of good quality input data. Some results of originally developed tools based on GIS operations have been presented here. The advantage of the physically-based modelling over standard hydrological models is wider applicability of physically based parameters than those of the conceptual models. Obvious benefits of the GIS tools over the manual preparation of data are the time savings, high resolution and quality of information.

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HYDROLOGIC MODELLING OF URBAN CATCHMENTS

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1. Introduction

The transformation of rainfall over a catchment area into a flow hydrograph that may be used as an input to a pipe flow model involves two steps. First, the rainfall losses due to interception, wetting, depression storage, infiltration and evaporation are abstracted from rainfall. Only the remaining part, called net rainfall, will produce storm runoff. The net rainfall is then transformed into a flow hydrograph for the area. This transformation, called surface routing, accounts for temporal distribution of the net rainfall at the outlet of the area, caused by overland flow processes.

In traditional methods, relatively little importance was given to the description of the surface runoff process. For computation of stormwater sewer systems, the runoff process is however at least as important as the pipe flow process, and of equal importance as the selection of the design rainfall. At present, with an increased knowledge of these phenomena, computational methodologies tend to incorporate more detailed models for simulation of the rainfall-runoff process. Figure 1 shows schematically the rainfall-runoff process in urban areas.

Urban runoff can be regarded as a sequence of processes in which water flows are governed by the laws of hydrodynamics; conservation of mass, momentum and energy. However, because of extreme complexity of flow boundaries, in most cases, it is impossible to develop models that are fully based on these laws, without considerable simplifications. Consequently, several basic modelling concepts (co-)exist.

Physically based models remain as close to the basic laws of hydrodynamics as possible. Their advantage consists of a higher universality of model components and in the physical interpretation that can be given to model parameters. As stated before, their use is hampered by the complexity of real world catchments and also by constraints with respect to the computation time.

Empirical models are developed from observations, with minimal considerations given to the physics of underlying processes. An easily forgotten characteristic of empirical models is that their applicability is often limited to narrowly specified environmental conditions, e.g., a limited range of the degree of urbanisation, catchment slopes, climatic conditions, etc.

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In between those two extremes are the conceptual models that rely on the mass conservation law and on empirical or semi-empirical relations. The conceptual models recognise explicitly that different processes may cause rainfall losses and integrate these processes for calculation of the net rainfall. The losses that may be accounted for are caused by interception, wetting, depression storage, evaporation and infiltration.

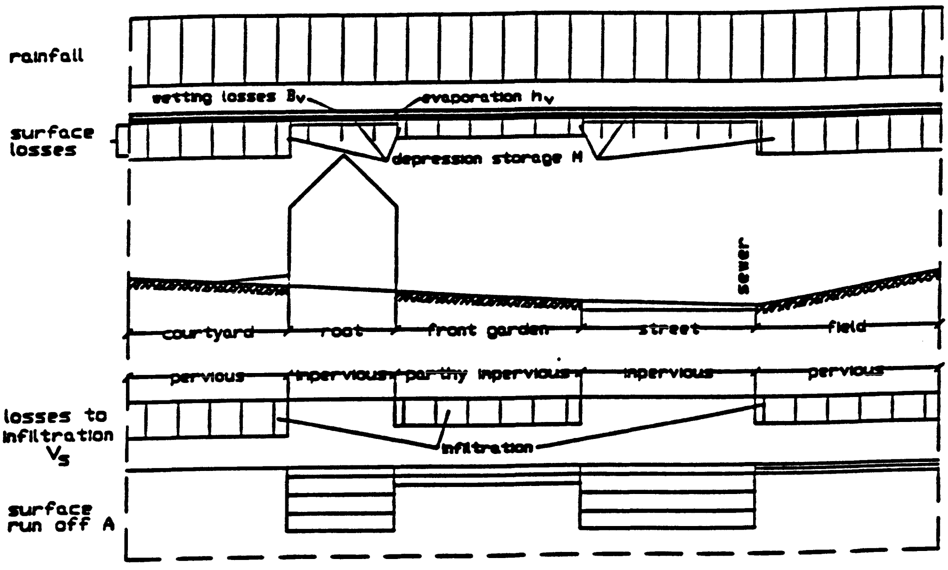

Figure 1. Rainfall losses in urban areas.

The runoff process may be simulated according to different spatial discretisation, depending on the actual problem and the model available. For approximate calculations, a lumped approach may be sufficient; calculation of the flow hydrograph will be done for the whole area, without spatial discretisation. For a detailed sewer system simulation, the catchment will be subdivided into subcatchments and calculation of the flow hydrograph will be done for each subcatchment separately. This approach is classified as the distributed approach.

Globally distributed models may be locally lumped or locally distributed. In locally distributed models, each subcatchment is further divided, according to the type of surface: pervious, impervious, roads, flat roofs, sloping roofs, etc. Again, calculation of the net rainfall may be done for each type of surface separately, using specific model components and parameters for each surface type.

The conceptual and physically based models recognise explicitly that different processes may cause rainfall losses and integrate these processes for the calculation of the net rainfall. Thus, they are in general globally and locally distributed. On the other hand, empirical models are often based on a lumped approach.

An overview of such urban hydrologic models as BEMUS (IRTCUD 1992), HYSTEM (Fuchs and Scheffer 1991), MOUSE (DHI 1990) and HydroWorks (Wallingford Software 1992) will show that different approaches co-exist in urban hydrology. According to the previously mentioned classification, BEMUS can be considered as following the physically based approach; HYSTEM follows a conceptual approach; and HydroWorks and MOUSE offer several options for the representation of rainfall-runoff processes, ranging from empirical to conceptual.

2. Initial Abstractions

Often, several types of losses occurring during the initial stage of a rainfall storm are combined and designated as initial abstractions. These abstractions include the losses due to interception, initial wetting and depression storage. For intense rain storms in urban areas, the initial losses are less significant. However, for less severe storms or for basins with low imperviousness, they should not be neglected.

The following figure indicates the initial abstractions for impervious and pervious areas.

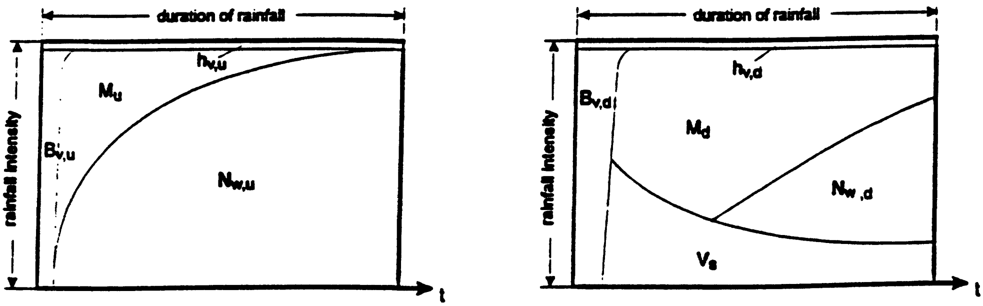

Figure 2. Initial abstractions and continuous losses for impervious and pervious areas.

2.1. THE INTERCEPTION AND WETTING LOSS

The interception loss is caused by the interception of rainfall by vegetation. After a sufficient amount of rainfall has fallen, additional rainfall will fall through or flow to the soil along plant stems, and the interception rate rapidly approaches zero.

In urban hydrology, the interception loss on pervious areas is usually not modelled in detail, as its importance is minimal. If considered at all, the interception loss will be modelled in a way similar to that for the wetting loss. The latter may be considered as a generalised form of the interception loss, which is also applicable to impervious areas. It is modelled as a certain amount of rainfall that has to be subtracted from the rainfall at the beginning of a storm. Typical values for these losses are about 0.5 to 1 mm for impervious areas, 1 mm for corn and 2 mm for meadow grass.

2.2. THE DEPRESSION STORAGE

The depression storage accounts for rainwater trapped in small depressions, and not contributing to runoff. This water will be removed from the depressions by evaporation or infiltration only. Typical values of the depression storage capacities are 0.5 to 2 mm for impervious areas, 2.5 to 7.5 mm for flat roofs, up to 5 mm for bare soil, up to 10 mm for lawn grass, and up to 15 mm for wooded areas. Other factors that affect the depression storage are the slope and the connection of the area to the sewer system.

Different conceptual approaches may be distinguished with regard to the depression storage losses. HydroWorks combines the wetting losses with the depression storage losses, and their modelling is then similar to the modelling of the wetting loss. In the MOUSE model, the initial and depression losses are combined for impervious surfaces. For pervious areas, it is assumed that no net rainfall is generated during the filling of depressions. However, infiltration takes place after enough rainwater has been supplied to satisfy the wetting loss. BEMUS and HYSTEM assume that some net rainfall is generated on impervious areas from the beginning of the filling of depression storage. These models also assume an exponential decay of the depression loss rate (Figure 3). Conceptually, this decay represents the distribution of depressions of different sizes in the catchment.

In BEMUS, the empirical equation proposed by Linsley *et al.* (1949) is used, and the depression storage loss rate, i_d, is calculated as:

$$i_d(t) = i_e(t)\, e^{-\frac{I_e(t)}{L_d}} \tag{2.1}$$

The cumulative effective rainfall volume, I_e, is defined here as the rainfall minus infiltration, interception, wetting and evaporation losses; L_d represents the maximum depression capacity and i_e is the effective rainfall rate.

An alternative empirical equation for calculation of the depression storage was proposed by Fuchs and Verworn (1990):

$$i_d(t) = i_e(t)\left(1 - \frac{I_e(t)}{L_d} \exp(1 - \frac{I_e(t)}{L_d})\right) \tag{2.2}$$

One may notice that in eq. (2.1), the loss rate becomes zero when the cumulative effective rainfall equals the maximum depression capacity. Linsley, on the other hand, accounted for the fact that, due to evaporation and infiltration, more water may be stored in the depression than what its actual volume would allow. Such differences in the conceptual approach contribute to the wide range of depression storage values found in the literature.

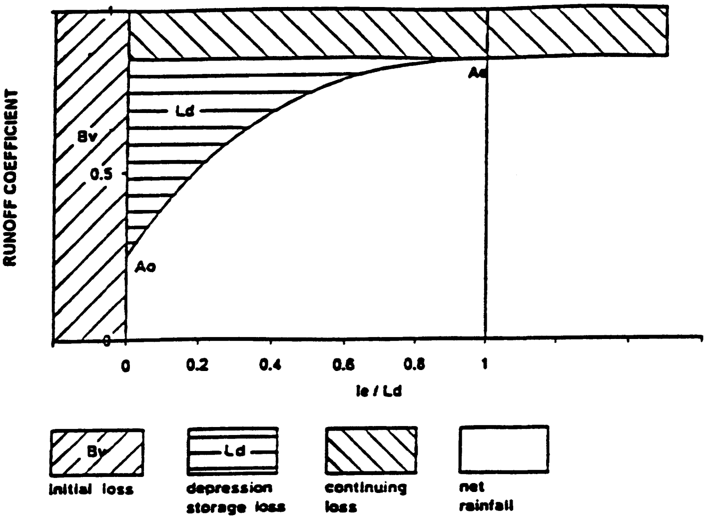

Figure 3. The rainfall losses.

3. Infiltration

The infiltration capacity of the soil defines the rate at which the water may infiltrate into the soil, under the condition of a sufficient water supply. The ability of soil to infiltrate water depends on a number of parameters, such as the soil type, structure and compaction, the initial moisture content, the surface cover, the viscosity of water and the depth of water on the soil surface. A typical temporal evolution of the infiltration capacity is shown in Figure 4. High rates observed at the beginning of the infiltration process tend to decrease exponentially to a final quasi-steady infiltration rate when the upper soil zone becomes saturated.

3.1. THE RICHARDS EQUATION

The basic equations for describing the infiltration process are the equations expressing the conservation of mass and momentum of water in unsaturated porous media. By approximating the latter equation by Darcy's law and assuming the flow to be one-directional, the Richards (1931) equation is obtained:

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z}\left(D\frac{\partial \theta}{\partial z} + K \right) \tag{3.1}$$

with

$$D = K\frac{\partial\phi}{\partial\theta} \tag{3.2}$$

where θ is the soil moisture content, ϕ = the suction head, K = the hydraulic conductivity, D = the soil water diffusivity, t = the time, and z is the depth.

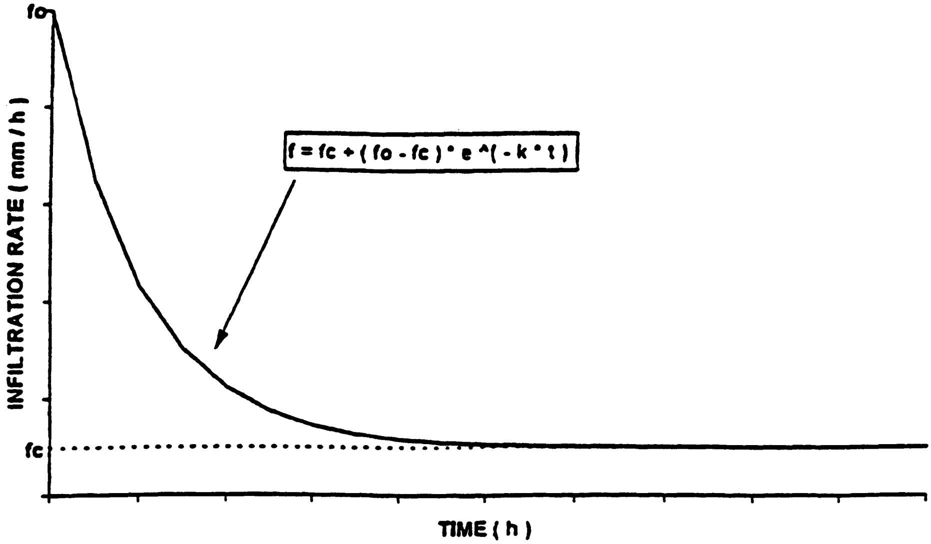

Figure 4. The evolution of the infiltration rate with time.

When combined with the appropriate initial and boundary conditions, the Richards equation may be solved numerically. The practical application of the Richard equation is limited by the difficulty of assessing the unsaturated hydraulic conductivity and the diffusivity or hydrostatic pressure as a function of the moisture content. Several environmental factors are indeed responsible for large variations in the latter functions, e.g., the heterogeneity of the soil and surface cover, the compaction and/or swelling of the soil, entrapped air, surface crusts, filling of the soil pores by entrained sediment, etc.

Therefore, approximate methods are often used for practical purposes. In general, those methods may be formulated in such a way that they describe the overall infiltration process relatively well, with the exception of the initial stage of infiltration.

3.2. THE GREEN AND AMPT EQUATION

Green and Ampt (1911) proposed an approximate method for calculation of the vertical infiltration, which is well suited for coarse soils that are initially dry. It is assumed that the soil is homogeneous and that the initial soil moisture content, θ_i, is constant over the depth (Figure 5). It is further assumed that the infiltrating water forms a sharp infiltration front, and above this front, the water content is uniformly distributed (θ_s, near saturation).

The infiltration rate may be calculated, using Darcy's law :

$$f(t)=K\frac{H_0+\phi+L_f(t)}{L_f(t)} \tag{3.3}$$

where H_0 is the ponding depth and L_f is the depth of the infiltration front.

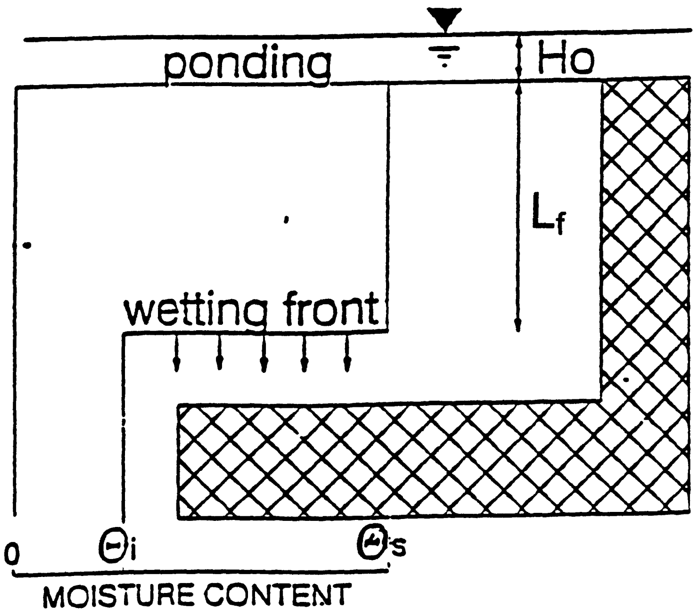

Figure 5. The Green and Ampt method.

Defining $M = \theta_s - \theta_i$ and $F = M\,L_f$ (= the cumulative infiltration), and neglecting the depth of ponding, the equation may be rearranged:

$$f(t)=K\left(1+\frac{M\phi}{F(t)}\right) \tag{3.4}$$

For F(0) = 0, integration of eq. (3.4) over time yields:

$$F(t)=Kt-\phi\,M\ln\left(1+\frac{F(t)}{M\phi}\right) \tag{3.5}$$

The value of θ_i strongly depends on the antecedent rainfall conditions. The moisture content above the wetting front is limited to the effective porosity of the soil but full saturation is seldom observed in the field, due to air entrapment. For the same reason, the value of the hydraulic conductivity above the wetting front, K, is often set to 0.66 K_s, where K_s represents the conductivity at saturation. The suction head is the variable most difficult to estimate as it depends on soil characteristics and the moisture content. Several equations for the assessment of ϕ may be found in the literature, e.g., Brooks and Corey (1964) and Carman (1939).

3.3. THE MEIN AND LARSON MODEL

The Green and Ampt equation allows for calculation of the infiltration rate, provided that ponding occurs, i.e., under conditions that the rainfall intensity is at least as high as the infiltration rate. The method was extended by Mein and Larson (1973) to account for ponding conditions by distinguishing the periods with and without ponding. During the periods without ponding, all rainfall infiltrates, while during the periods with ponding, the Green and Ampt equation is used. Ponding occurs when the potential infiltration rate is less than or equal to the rainfall intensity. The Mein and Larson model is incorporated into the BEMUS model.

3.4. THE NEUMANN EQUATION

In HYSTEM, the infiltration is calculated using the Neumann (1976) equation. The actual storage of moisture in the unsaturated soil zone, W, is used as the independent variable for calculation of the infiltration rate:

$$f(t) = cW(t) + b(W_s - W(t)) \tag{3.6}$$

where W_s is the storage capacity of the unsaturated soil. Constants b and c depend on the soil type. Storage in the unsaturated zone at the beginning of simulation has to be provided as an additional parameter. The initial storage value depends on the antecedent conditions in the catchment, especially on the rainfall depth and the time lapsed from the previous rainfall event.

As in the Mein and Larson method, the Neumann equation makes a distinction between the potential infiltration and the actual infiltration. The variation of storage during a storm is described by an equation of the type

$$W(t) = W(t)e^{-d\,dt} + \frac{f(t)}{c}(1 - e^{-d\,dt}) \tag{3.7}$$

where d is set equal to b (ponding) or c (no ponding).

3.5. THE HORTON EQUATION

The empirical relation developed by Horton (1933, 1939) assumes that the potential infiltration rate decreases exponentially with time (Figure 4):

$$f(t) = f_c + (f_0 - f_c)e^{-kt} \tag{3.8}$$

where f_c is the final infiltration rate, f_o is the initial rate, and k is a decay constant. All these parameters depend primarily on the soil type and the initial moisture content of the soil.

It can be shown that the Horton equation can be derived from the Richards equation, assuming that the hydraulic conductivity and diffusivity are independent of the water content of the soil. However, with time being the only independent variable for calculation of f, eq. (3.8) can hardly be modified to an actual infiltration model. The Horton equation may be optionally used in the MOUSE model.

4. Evaporation and Evapotranspiration

Evaporation occurs at the potential rate, with respect to the intercepted and adsorbed water, water stored in depressions and surface waters. For pervious areas, the actual evapotranspiration should be accounted for. In urban hydrology, complex processes of evapotranspiration are often neglected or drastically simplified. The effect of evapotranspiration during rainfall events is indeed negligible; the average daily value of the potential evaporation in Europe, during summer, amounts to about 3 mm. Values of evapotranspiration during the growth season in Europe range from 0.5 to 1 mm/day for forests to 2 to 7 mm/day for wheat and grasslands. Among the models considered here, only the MOUSE model allows for inclusion of a constant evaporation loss.

5. Lumped Rainfall Abstractions

5.1. THE PROPORTIONAL LOSS MODEL

In the proportional loss model, the net rainfall rate, i_n, is considered to be a constant fraction of the rainfall intensity, so that

$$i_n(t) = C(T)\,i(t) \tag{5.1}$$

The runoff coefficient, C, depends mainly on land use, the soil and vegetation type and the slope. Rainfall characteristics (intensity, duration) also affect this coefficient. As the runoff coefficient for a given event also depends on the antecedent rainfall in the basin, it may be assigned some probability, as reflected by the return period, T, in the equation.

The values of C range from 0.7 to 0.95 for pavements and roofs, and from 0.05 to 0.35 for pervious areas (Geiger *et al.* 1987).

The proportional loss rate model may be used in all models for lumped modelling of impervious areas, where it is usually combined with an initial loss model. It is not advisable to apply this model to highly pervious catchments.

5.2. THE WALLINGFORD MODEL

In the Wallingford model (Anonymous 1983), the runoff coefficient is calculated using a regression equation that was established for typical urban catchments in the UK. This runoff coefficient depends on the density of development, the soil type and the antecedent wetness of the basin.

In the HydroWorks model, a global runoff coefficient for the total sub-basin is calculated using the Wallingford model. Runoff is then divided among different surfaces, using weighting coefficients.

5.3. THE SCS METHOD

The U.S. Soil Conservation Service developed a method for computing abstractions from a storm rainfall for agricultural and urban land uses (Anonymous 1972). This method is based on the hypothesis that no runoff occurs before the initial abstraction volume, L_i, has been satisfied, and that the ratio of the net rainfall volume, I_n, to the potential runoff volume, $(I - L_I)$, is equal to the ratio of retention after runoff initiation, L_c, to the maximum retention capacity, S:

$$\frac{L_c}{S} = \frac{I_n}{I - L_i} \tag{5.2}$$

The initial loss volume can be related to the retention capacity by an empirical equation:

$$L_i = 0.2\,S \tag{5.3}$$

Combining the previous equations yields:

$$I_n = \frac{(I - 0.2\,S)^2}{I + 0.8\,S} \tag{5.4}$$

The time distribution of the abstractions is found by solving eqs. (5.2) and (5.4) for L_c and differentiating:

$$i_c(t) = \frac{dL_c}{dt} = \frac{S^2\, i(t)}{\{I(t) - L_i + S\}^2} \tag{5.5}$$

where i_c is the loss rate, i is the rainfall intensity, and I(t) is the cumulative rainfall volume at time t.

For normal antecedent moisture conditions, empirical values of S may be derived, using the equation:

$$S = 25.4\left(\frac{1000}{CN} - 10\right) \tag{5.6}$$

where CN is the runoff curve number. CN values are given in the literature (Anonymous 1972) for various surface covers, soil types and antecedent moisture conditions. Alternative equations were proposed for calculation of the retention capacity for dry and wet conditions.

The SPIDA model offers the SCS method as an option, though its use is advised for agricultural and semi-urbanised areas only (Wallingford Software 1992).

6. Physically Based Overland Flow Models

6.1. SAINT VENANT EQUATIONS

If the overland flow is considered as a horizontal one-dimensional unsteady gravity flow, it may be described by Saint Venant equations. Morris and Vierra (1981) defined the regions of applicability of the full set and the diffusive and kinematic wave approximations of Saint Venant equations for overland flow simulation. Several authors have shown that the kinematic submodel is sufficient in most practical cases (e.g., Maksimovic 1990).

6.2. KINEMATIC WAVE MODEL

In a kinematic wave model, the acceleration and pressure terms in the momentum equation are negligible and the gravity and friction forces are balanced. This results in a flow that does not change rapidly. The motion of a kinematic wave is described primarily by the continuity equation, and the momentum equation is reduced to:

$$S_0 = S_f \tag{6.1}$$

where S_o and S_f are the surface and friction slope, respectively. The latter equation may also be substituted by:

$$q = \alpha\, y^{\beta} \tag{6.2}$$

where q is discharge per unit width, y is the flow depth, and parameters α and β depend on the friction formula used. If the Manning formula is used to describe roughness,

$$\alpha = \frac{\sqrt{S_0}}{n} \qquad \beta = \frac{5}{3} \tag{6.3}$$

and the roughness coefficient, n, depends on surface characteristics of the catchment. Corrections to the traditional values may be needed, as the Manning equation is valid for fully turbulent flow only. For overland flow, this condition is rarely met and the roughness will depend on the Reynolds number. Moreover, an increased resistance will be encountered by the flow, due to the impacts of rain drops. Finally, the actual flow is often two-dimensional. The simplification made by using the one-dimensional kinematic wave model will be reflected in the value of the roughness.

Radojkovic and Maksimovic (1987) extended the general equation for bottom shear stress to account for the additional shear stress due to the impact of rainfall drops. They found

$$\alpha = \sqrt{\frac{2gS_0}{C}} \qquad \beta = \frac{3}{2} \tag{6.4}$$

Based on extensive analysis of experimental data, they recommend the following formulae for the computation of the effective friction coefficient:

$$C = \frac{c_1 + d_1 i_e^{d_2}}{\mathrm{Re}} \qquad \text{for Re} < 2500 \tag{6.5}$$

$$C = \frac{c_2\,(1 + d_3)}{\mathrm{Re}^{c_3}} \qquad \text{for Re} > 2500 \tag{6.6}$$

where Re is the Reynolds number, i_e is the effective rainfall, and c_i and d_i are parameters which depend on the surface type.

The kinematic wave model is implemented for surface routing in the BEMUS and MOUSE models.

6.3. THE UNIT HYDROGRAPH

6.3.1. *The Principles*

The unit hydrograph is a model for the transformation of a net rainfall hyetograph into a hydrograph, under the assumption that the surface runoff process behaves like a linear system. Using the terminology of systems theory, the unit hydrograph is the unit pulse response function of a linear, time invariant system; it represents the outflow of a catchment if a unit amount of net rainfall were applied over a duration Δt. The assumption of linearity facilitates the use of the principle of superposition; time invariance indicates that the system processes input into output independently of time.

The response of such a linear system is completely defined by its impulse response function – the instantaneous unit hydrograph (IUH) – which describes the response of the system if a unit amount is applied instantaneously as an input to the system. Knowing the impulse response function, u, the response of a complex input time function, i, can be found as a convolution integral of the response for its constituent impulses:

$$q(t)=\int_0^t u(\tau)i(t-\tau)d\tau \tag{6.7}$$

where q(t) is the surface runoff at time t, u(τ) = the ordinate of IUH at time τ, i(t-τ) = the net rainfall intensity at (t-τ), and τ represents time measured into the past.

The pulse response function, h, describes the response for an input of unit amount and duration Δt. It is produced by applying the principle of superimposition:

$$h(t)=\frac{1}{\Delta t}\int_{t-\Delta t}^{t} u(l)dl \tag{6.8}$$

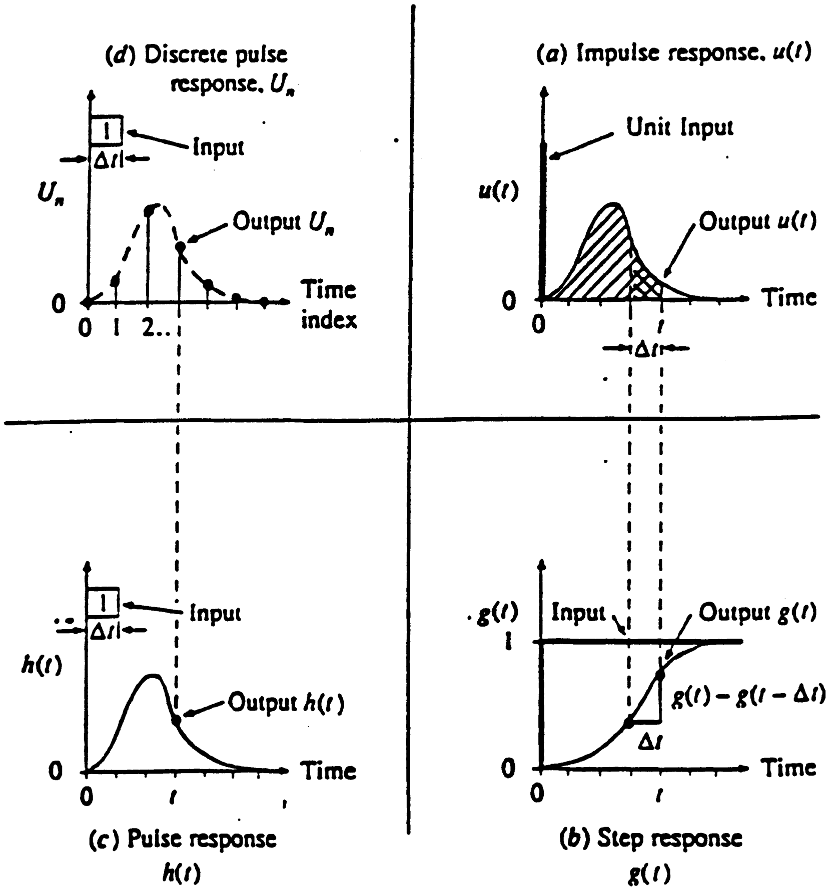

Figure 6. Response functions of a linear system (Chow *et al.*, 1988).

The previous response functions were defined for the continuous time domain. To apply the response functions in discrete time, the time domain is divided into discrete intervals of duration Δt, and the input time function is represented as a succession of pulses, and sampled data are used for the flow output series.

If I_m is the rainfall amount between the times $(m-1)\Delta t$ and $m\Delta t$, and q_n is the instantaneous flow rate at the end of the n-th interval, then the flow at the end of the n-th interval may be calculated, using the pulse response function and the principle of superposition. First, function U may be defined as:

$$U_n = h(n\Delta t) \tag{6.9}$$

The response of input pulse P_1 at the end of the n-th interval is given by $I_1.U_n$, the response of P_2 by $I_2.U_{n-1}$,..., so that

$$q_n = I_1 U_n + I_2 U_{n-1} + \ldots + I_m U_{n-m+1} + \ldots + I_m U_{n-M+1} \tag{6.10}$$

or

$$q_n = \sum I_m U_{n-m+1} \tag{6.11}$$

The summation is performed for m = 1 to M, if n > M, or to n, if n < M, where M is the number of pulses in the rainfall series. This principle is illustrated in Figure 7.

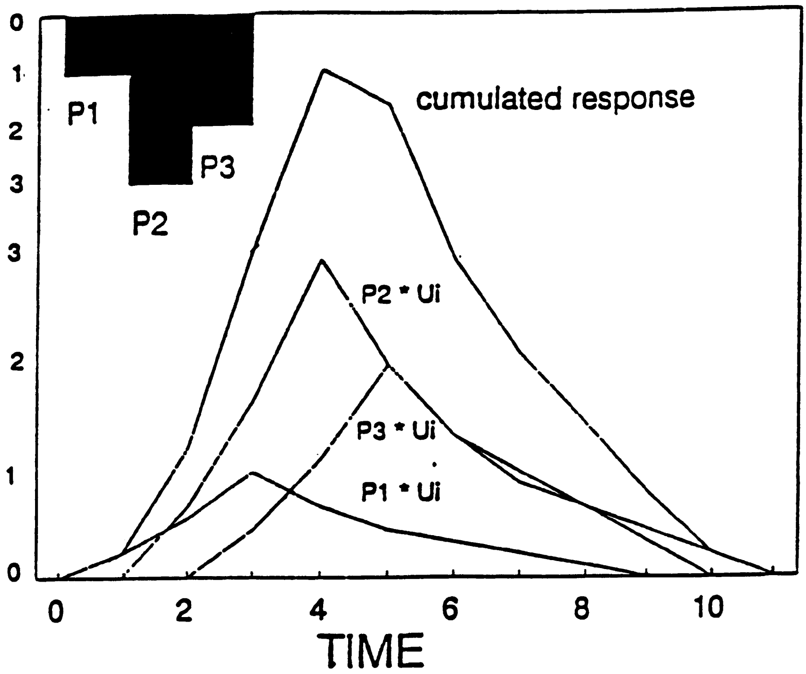

Figure 7. Application of the discrete convolution.

In urban hydrology, no data are available to define the unit hydrograph for each individual sub-basin. Therefore, synthetic unit hydrographs are used. To develop them, several methods can be used, e.g., the standard unit hydrograph, the time-area method or the linear reservoir models.

6.3.2. *The Standard Unit Hydrograph*

Harms and Verworn (1984) defined a standard unit hydrograph with the following characteristics (Figure 8):

- a linear increase up to the peak Q_p at the time t_p;
- an exponential recession, similar to the linear reservoir model; and
- the end of the recession at $0.01 * Q_p$

Based on the analysis of 20 dimensionless unit hydrographs for different catchments, they found:

$$Q_p = \frac{0.96A}{t_L} \tag{6.12}$$

$$t_p = 0.49\, t_L \tag{6.13}$$

$$k = \frac{\frac{A}{Q_p} - \frac{t_p}{2}}{0.99} \tag{6.14}$$

where A is the area and t_L is the lag time.

The lag time t_L is calculated by empirical formulae. For impervious areas, it depends on the area and the length of flow path; for pervious areas it is a function of the length of the flow path, the slope, surface roughness and rainfall intensity. This standard unit hydrograph is used in the HYSTEM model.

6.3.3. *The Time-Area Method*

For the time-area method, isochrones that connect all points at equal travel time to the outlet are defined (Figure 9). The maximum travel time represents the time of concentration of the basin. The response function of the basin is defined by the time-area diagram that is constructed by integrating the areas between the isochrones.

This method can be optionally used in the MOUSE model, where the user has to select a standard profile for the time-area curve and the time of concentration of the subbasin. Many empirical equations for estimation of the concentration time may be found in the literature (McCuen 1984; Chow 1988).

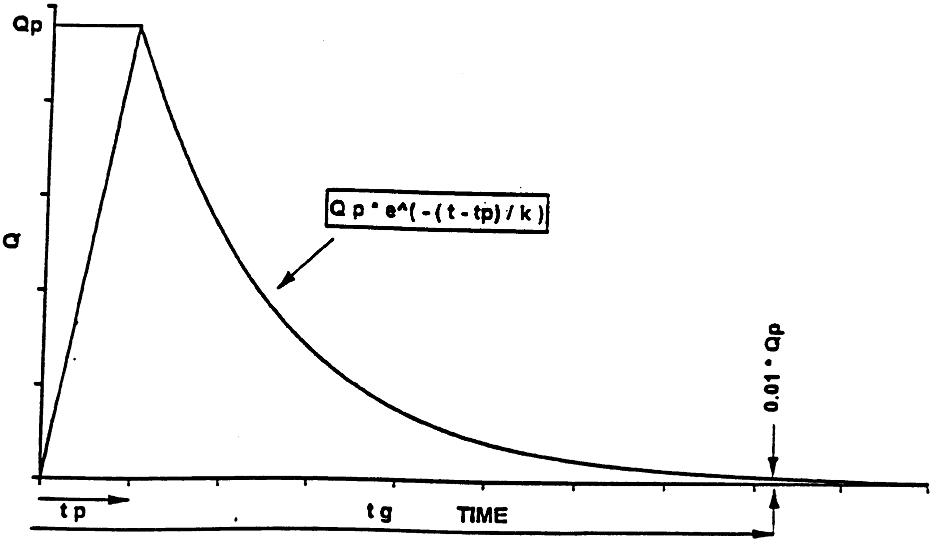

Figure 8. The standard unit hydrograph.

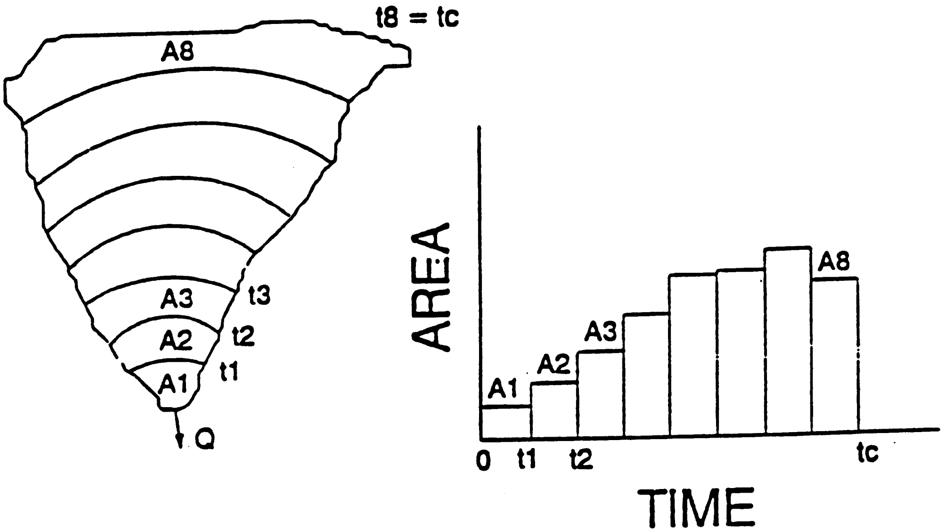

Figure 9. The time-area method.

6.3.4. *The Linear Reservoir Models*

In the reservoir models, the catchment is considered to act as a reservoir or as a series of reservoirs in series or in parallel.

The linear reservoir models are based on the continuity equation

$$\frac{dS(t)}{dt} = i(t) - q(t) \tag{6.15}$$

and on a storage equation

$$S(t) = Kq(t) \tag{6.16}$$

where i is the inflow rate, q is the outflow rate, S is the storage and K is the reservoir time constant.

Nash (1957) proposed to conceptualise a catchment by a cascade of n identical reservoirs (Figure 10). The instantaneous unit hydrograph of this model is expressed as:

$$u(t) = \frac{1}{K(n-1)!} \left(\frac{t}{K}\right)^{n-1} e^{-\frac{t}{K}} \tag{6.17}$$

Empirical relations for the estimation of n and k for urban catchments have been proposed e.g., by Desbordes (1978) and Viessman (1968).

In SPIDA (Wallingford Software 1992), a Nash cascade of two reservoirs is used to represent the unit hydrograph. The reservoir time constant is defined by a regression equation including sub-basin characteristics (slope, area, imperviousness) and the rainfall intensity.

7. Conclusions

As opposed to the current pipe flow simulation models, which all use the same basic equations, a whole gamut of hydrologic modelling approaches is used for calculation of inflow hydrographs to the hydraulic models. Table 1 illustrates this for the BEMUS, HYSTEM, MOUSE and HydroWorks models, which were discussed in detail in this paper.

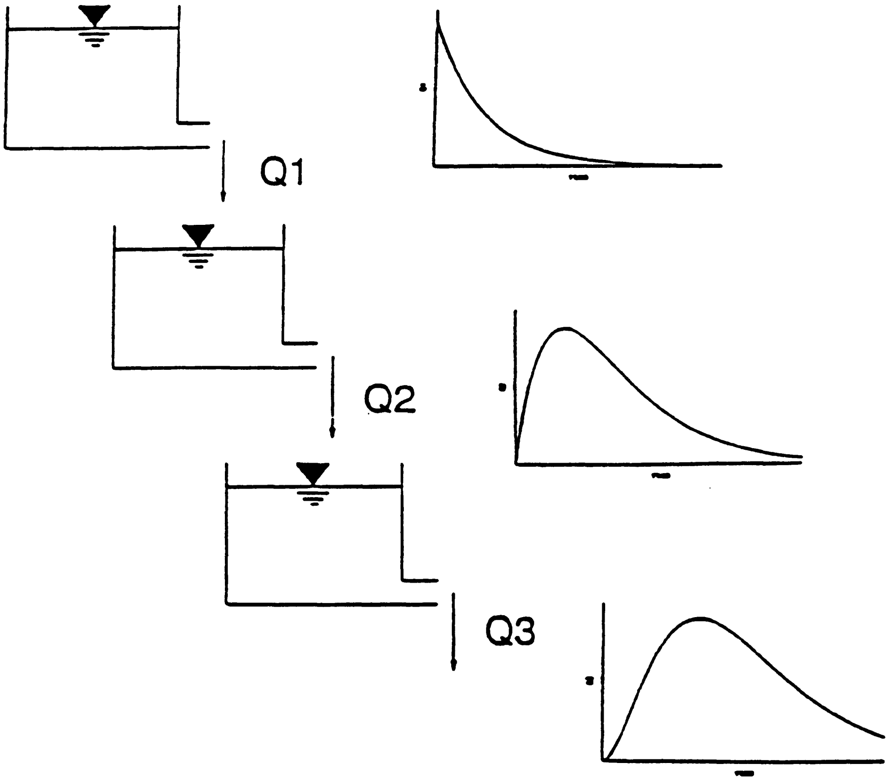

Figure 10. The Nash cascade.

TABLE 1.

model	HYSTEM			BEMUS	MOUSE		HydroWorks		
submodel	a	b	c		a	b	a	b	c
impervious area - initial losses	yes	yes	yes	yes	yes	yes	yes	yes	yes
impervious area - depression loss	yes	yes	(yes)	yes	yes	(yes)	no	no	no
impervious area - continuing loss	runoff coefficient	runoff coefficient	runoff coefficient	runoff coefficient	runoff coefficient	(Horton)	Wallingford	runoff coefficient	SCS
pervious area - initial loss	no	no	no	yes	yes	yes	yes	yes	yes
pervious area - depression loss	yes	yes	yes	Linsley	no	(yes)	no	no	no
pervious area - continuing loss	Neumann, runoff coefficient	Horton runoff coefficient	Horton runoff coefficient	Green Ampt	runoff coefficient	(Horton)	Wallingford	runoff coefficient	SCS
evaporation	no	no	no	no	no	yes	no	no	no
surface runoff	unit hydrograph	multiple reservoir	linear reservoir	kinematic wave	time-area	kinematic wave	linear reservoir	linear reservoir	linear reservoir

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MODELLING SEWER HYDRAULICS

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1. Mathematical Representation of Flow in Sewers

Conservation laws of mass, momentum and energy are used to describe open channel flow in sewers. The mass conservation principle yields the continuity equation, whereas Newton's second law yields the momentum equation. Two flow variables in one-dimensional flow, such as the flow depth y and velocity V, or the flow depth y and the rate of discharge Q, are sufficient to define the flow conditions at a cross section. Therefore, two governing equations may be used to describe one-dimensional unsteady open channel flow. Except for the velocity distribution coefficient, α, and the momentum distribution coefficient, β, the momentum and energy equations are equivalent as shown in [4], provided that the flow depth and velocity are continuous. This condition is met if there are no flow discontinuities, such as a hydraulic jump or a bore. However, the momentum equation should be used for flows with discontinuities, since, unlike in the energy equation, it is not necessary to know the magnitude of losses at the discontinuities in the application of the momentum equation.

1.1. SAINT-VENANT GOVERNING EQUATIONS

The following assumptions were made by Barre de Saint-Venant in 1871 in his derivation of flow equations [19]:

1. The flow is one-dimensional. The depth and velocity vary only in the longitudinal direction. Thus, the velocity at any cross section is constant and the water surface is horizontal across any section perpendicular to the longitudinal axis.
2. The fluid is incompressible and of constant density.
3. The flow varies gradually along the channel so that the pressure distribution is hydrostatic and the vertical acceleration can be neglected.
4. The longitudinal axis of the channel can be approximated by a straight line. The channel is prismatic - i.e., the channel cross section and the channel bottom slope do not change with distance. The variations in the

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cross section or bottom slope may be taken into consideration by dividing the channel into several prismatic reaches.

5. The bottom slope of the channel is small and the channel bed is fixed; that is, the effects of scour and deposition are negligible.
6. The resistance effects in unsteady flow may be described by using the steady-state hydraulic resistance laws, such as the Manning or Chezy equation.

The governing de Saint-Venant equations consisting of the continuity equation (1.1) and the dynamic equation (1.2) may be written as cited in [18]

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{1.1}$$

$$\underbrace{\frac{\partial Q}{\partial t}}_{1} + \underbrace{\frac{\partial}{\partial x}\left[\beta \frac{Q^2}{A}\right]}_{2} + \underbrace{g A \frac{\partial y}{\partial x}}_{3} + \underbrace{g A S_f}_{4} = \underbrace{g A S_0}_{5} \tag{1.2}$$

KWM

DIFWM

DYNWM

where t is time; x is the distance along the longitudinal direction of channel; Q is the discharge; A is the flow cross-sectional area perpendicular to x; y is the flow depth measured from channel bottom and normal to x as shown in Figure 1; S_f is the friction slope; S_o is the channel slope; β is a momentum distribution coefficient; and g is the gravitational acceleration. In the above governing equations, there are two independent variables, x and t, and two dependent variables, namely y and Q.

The dynamic equation consists of the local acceleration term 1, which describes the change in momentum due to the change in velocity over time, the convective acceleration term 2, which describes the change in momentum due to the change in velocity along the channel, the pressure force term 3, proportional to the change in the water depth along the channel, the friction force term 4, proportional to the friction slope S_f, and the gravity force term 5, proportional to the bed slope S_o. The local and convective acceleration terms represent the effect of inertial forces on the flow.

Assuming that the bottom slope is small, then S_o can be expressed as a function of the water depth and water surface gradient

$$S_o \approx \frac{\partial y}{\partial x} - \frac{\partial h}{\partial x} \tag{1.3}$$

It is thus possible to use the water height, h above a certain reference level (datum), as the dependent variable instead of the water depth, y (see Figure 1). The dynamic equation can hence be written as:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left[\beta\frac{Q^2}{A}\right] + gA\frac{\partial h}{\partial x} + gAS_f = 0 \tag{1.4}$$

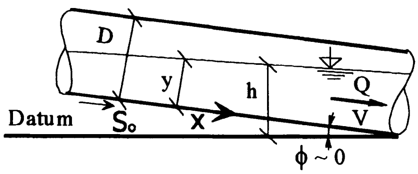


Figure 1. Height h above the datum.

Alternative formulations can be produced by using the full continuity equation while eliminating some terms of the dynamic equation. The simplest model is the kinematic wave model KWM, which neglects the local acceleration, convective acceleration, and pressure terms. It assumes $S_o = S_f$, i.e., the friction and gravity forces balance each other. The diffusion wave model DIFWM neglects the local acceleration and convective acceleration, but incorporates the pressure term. The dynamic wave model DYNWM considers all the acceleration and pressure terms.

Empirical resistance formula for steady uniform flow Q_o may be expressed as

$$Q_o = CSR^m\sqrt{S_o} \tag{1.5}$$

where C depends on the formula used to describe the friction slope S_f

$$C = \frac{1}{n}R^{2/3} \tag{1.6}$$

for the Manning's formula, and

$$C = \left(\frac{8gR}{f}\right)^{1/2} \tag{1.7}$$

for the Darcy-Weisbach formula in which R is the hydraulic radius. Similar to Equation (1.5), a resistance formula for unsteady, non-uniform flow may be written as

$$Q = CSR^m\sqrt{S_f} \tag{1.8}$$

Rewriting Equation (1.2) and considering Equations (1.5) and (1.8), a different form of the approximations can be expressed:

$$Q = Q_o \sqrt{1 - \frac{1}{S_o}\frac{\partial y}{\partial x} - \frac{V}{S_o g}\frac{\partial V}{\partial x} - \frac{1}{S_o g}\frac{\partial V}{\partial t}} \tag{1.9}$$

KWM

DIFWM

DYNWM

Figure 2 shows that Q is less than Q_o (dotted line) for the same depth for a falling stage. Thus the rating curve has hysteresis. The difference between the discharge during rising and falling stages is caused by the unsteadiness and nonuniformity of the flow depth.

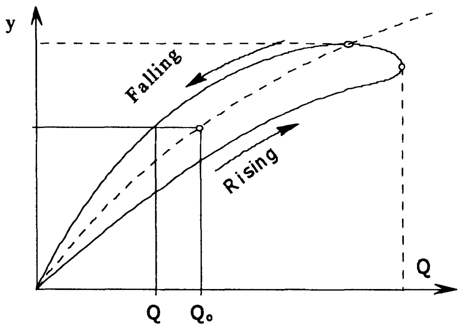


Figure 2. Rating curve.

By dropping both the local and convective acceleration terms, the diffusive wave model (approximation) requires the flow to vary gradually in both time and space. Since sewer channels are prismatic, gradually varying flow requires no rapid change in depth or hydraulic radius. Retaining the pressure term $\partial h / \partial x$ preserves the effect of backwater from downstream, which is important for sewers. Although not as accurate as the dynamic wave equation, when the local and convective acceleration terms are significant, the diffusion wave approximation is much simpler and quicker to solve numerically. The diffusion wave approximation requires two boundary conditions in addition to the initial condition, in order to obtain a unique solution. In a subcritical flow, one of the boundary conditions reflects the downstream backwater effect.

The kinematic wave model (approximation) ignores all the dynamic effects of the flow except the friction slope, i.e.,

$$S_o - S_f = 0 \tag{1.10}$$

In other words, the free surface is assumed to be parallel to the channel bed, i.e., the flow is uniform. This equation is solved together with the continuity equation. By eliminating the inertia terms as well as the pressure term, the kinematic wave approximation requires only one boundary condition, in addition to the initial condition, for unique solution. The boundary condition is specified at the upstream end of the channel, usually as an inflow hydrograph, and hence it permits solution irrespective of the downstream condition. It can therefore only be applied in supercritical flow. The kinematic wave approximation has no mechanism to attenuate the flood peak, any attenuation observed in computation is purely numerical. And finally, the kinematic wave approximation is not suitable for computations of pressurised flow problems. Table 1 shows some major features of a theoretical comparison among the approximations.

TABLE 1. Theoretical comparison of approximations of the Saint-Venant equation after [17]

Approximation ⇒	Kinematic wave	Diffusion wave	Dynamic wave
Boundary conditions required	1	2	2
Accounts for downstream backwater effect and flow reversal	No	Yes	Yes
Damping of flood peak	No	Yes	Yes
Accounts for flow acceleration	No	No	Yes

1.2. CALCULATION OF FRICTION SLOPE

For steady flow in uniform flow sections, any of three common formulae are used for the resistance coefficient. The Darcy-Weisbach friction factor, f, is described by equation

$$S_f = f\frac{1}{4R}\frac{V^2}{2g} = \frac{fQ^2}{8gRA^2} \tag{1.11}$$

Using the Colebrook-White equation

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{2.51}{\mathrm{Re}\sqrt{f}} + \frac{\varepsilon}{4R\,3.71}\right] \tag{1.12}$$

where Re = 4VR/ν, ε is the roughness height, and ν is the kinematic viscosity, the mean velocity is given by:

$$V = -\sqrt{32gRS_o}\log\left\{\frac{1.255\nu}{R\sqrt{32gRS_o}} + \frac{\varepsilon}{14.84R}\right\} \tag{1.13}$$

The expression given by Chezy for the mean velocity is

$$V = C\sqrt{RS_f} \tag{1.14}$$

so from Equation (1.11)

$$C = (8g/f)^{1/2} \tag{1.15}$$

Manning's relationship yields

$$V = \frac{1}{n} R^{2/3} S_f^{1/2} = \frac{1}{n} R^{1/6} \sqrt{R S_f} \tag{1.16}$$

where n is the Manning's roughness.

2. Solution of Saint-Venant Equations

The Saint-Venant governing equations (1.1) and (1.2) are a set of first order nonlinear hyperbolic partial differential equations and hence no analytical solutions are known. Therefore, these equations are solved for sewer flow numerically, with appropriate initial and boundary conditions.

2.1. DISCRETISATION OF THE SPACE - TIME DOMAIN

The differential terms in the partial differential equations are approximated by finite differences of selected grid points in a space and time domain shown in Figure 3. This process is described as discretisation. Substitution of the finite differences into a partial differential equation transforms it into an algebraic equation. Thus, the original set of differential equations can be transformed into a set of finite difference algebraic equations for numerical solution. Theoretically, the computational grid of space and time need not be rectangular. Neither need the space and time differences Δx and Δt be kept constant. However, it is usually easier for computer coding to keep Δx and Δt constant throughout the computation. It is normally advisable to subdivide the sewer length into two or three computational reaches of Δx, unless the sewer is unusually long or short. A single computational reach tends to introduce significant inaccuracy, because of the sewer entrance and exit conditions, and does not reflect well the flow inside the sewer [17]. Some numerical schemes do not allow only one computational reach. For example, the Abbott-Ionescu scheme [2] requires a minimum of three computational gridpoints in each pipe. Conversely, too many computational reaches would increase the computational complexity and costs without significant improvement in accuracy.

2.2. NUMERICAL SCHEMES

There are many numerical schemes for solving partial differential equations. For solving sewer flows, explicit schemes, implicit schemes and the method of characteristics can be used. Some examples of these schemes, as cited in [1], [2], [5], [10], [11], [17] and [18], are presented below.

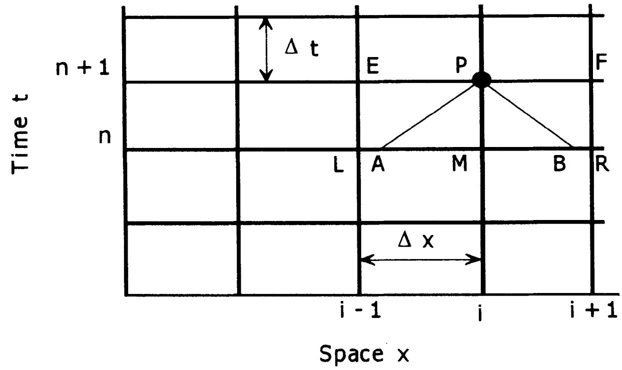


Figure 3. Computational grid for difference schemes. Reprinted from [17] with permission.

2.2.1. *Explicit scheme*

As an example, consider the continuity equation for a rectangular sewer for which the width B is constant and for simplicity let us assume that V is also constant. The continuity equation is

$$\frac{\partial y}{\partial t} + V\frac{\partial y}{\partial x} = 0 \tag{2.1}$$

Referring to the computational grid shown in Figure 3, the flow variables y and V are initially known at grid points L, M, R, etc., at a time $n\Delta t$. The unknowns to be found are the depths at $(n + 1)\Delta t$. Suppose the depth at grid point P, y_P, is sought. By taking the time and space backwater difference to approximate the derivatives of the depth, and using three grid points P, M, and L, one can write the following equations:

$$\left.\frac{\partial y}{\partial t}\right|_P \approx \left.\frac{\Delta y}{\Delta t}\right|_P \approx \frac{y_P - y_M}{\Delta t} \tag{2.2}$$

$$\left.\frac{\partial y}{\partial x}\right|_P \approx \left.\frac{\Delta y}{\Delta x}\right|_P \approx \left.\frac{\Delta y}{\Delta x}\right|_M \approx \frac{y_M - y_L}{\Delta x} \tag{2.3}$$

Substituting Equations (2.2) and (2.3) into Equation (2.1) and solving for y_P yields

$$y_P = y_M - \frac{\Delta t}{\Delta x}V(y_M - y_L) \tag{2.4}$$

Thus, the unknown y_P is solved explicitly in terms of known quantities on the right-hand side of the algebraic equation. Explicit finite difference schemes do not lead to a system of algebraic equations.

Depending on the number and position of the grid points used in expressing the finite difference to approximate the derivatives, there are many different explicit schemes. Even if the scheme is numerical stable, the solution varies with the value of

$\Delta t / \Delta x$ used, and often the accuracy is in doubt. For example, using $\Delta t / \Delta x = 0.5$, Equation (2.4) with V = 1 yields

$$y_P = 1/2\,(y_M + y_L) \tag{2.5}$$

Using $\Delta t / \Delta x = 1$

$$y_P = y_L \tag{2.6}$$

and with $\Delta t / \Delta x = 2$

$$y_P = -y_M + 2y_L \tag{2.7}$$

One can easily observe that for a given input stage hydrograph and initial depth profile throughout the sewer, the solution for the depths at subsequent time steps is different, depending on the values used for $\Delta t / \Delta x$. Usually the true solution is unknown. For this simple example Equation (2.6) is the true solution. The case $\Delta t / \Delta x = 0.5$ introduces numerical damping of the flood peak, whereas the case of $\Delta t / \Delta x = 2$ is numerical unstable. Hence the explicit scheme is conditionally stable and the Courant number $Cr \leq 1$ is the necessary and sufficient condition for stability (see section 2.3).

2.2.2. *Implicit schemes*

A popular implicit formulation utilises the "box", "four point" or Preissmann schemes. Consider the points between i and (i + 1) in space and n and (n + 1) in time in the computational grid shown in Figure 4. The time and space derivatives of a variable y can be approximated as

$$\frac{\partial y}{\partial t} \approx \frac{1}{\Delta t}\left\{\left[w_x\, y_{i+1}^{n+1} + (1 - w_x) y_i^{n+1}\right] - \left[w_x\, y_{i+1}^{n} + (1 - w_x) y_i^{n}\right]\right\} \tag{2.8}$$

$$\frac{\partial y}{\partial x} \approx \frac{1}{\Delta x}\left\{w_t\left(y_{i+1}^{n+1} - y_i^{n+1}\right) + (1 - w_t)\left(y_{i+1}^{n} - y_i^{n}\right)\right\} \tag{2.9}$$

where w_x and w_t are weighing factors with their respective values between zero and one. The first subscript of y denotes the space step, the second subscript represents the time step.

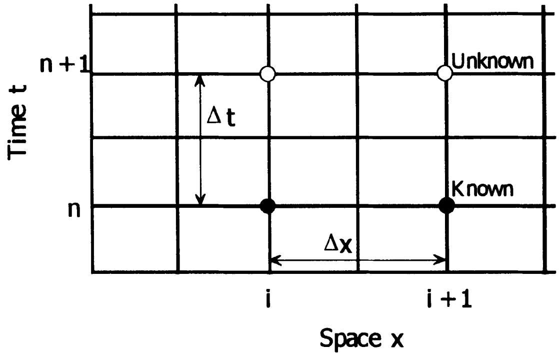


Figure 4. The Preissmann scheme.

Preissmann scheme. A special case of Equations (2.8) and (2.9), for $w_x = 0.5$, is the scheme suggested by Preissmann (1961). Accordingly, Equation (2.8) becomes

$$\frac{\partial y}{\partial t} \approx \frac{1}{2\,\Delta t}\left\{\left[y_{i+1}^{n+1} + y_i^{n+1}\right] - \left[y_{i+1}^{n} + y_i^{n}\right]\right\} \tag{2.10}$$

assuming that

$$y(x,\, t) = \frac{w_t}{2}\left(y_{i+1}^{n+1} + y_i^{n+1}\right) + \frac{1 - w_t}{2}\left(y_{i+1}^{n} + y_i^{n}\right) \tag{2.11}$$

where the value of the weighting factor w_t lies between 0.5 and 1.0. For $w_t = 1$ both y and its space derivative are expressed in terms of unknowns at the time step (n + 1), and this is called a full implicit scheme. Taking $w_x = w_t = 0.5$, we get a doubly-centred case. Considering $w_t > 0.5$ introduces truncation errors [2]. Substitution of Equations (2.9) - (2.11) for variables Q and y into the Saint-Venant equations yields two algebraic equations

$$C_i\left(Q_{i+1}^{n+1},\, Q_i^{n+1},\, y_{i+1}^{n+1},\, y_i^{n+1}\right) = 0 \tag{2.12}$$

$$M_i\left(Q_{i+1}^{n+1},\, Q_i^{n+1},\, y_{i+1}^{n+1},\, y_i^{n+1}\right) = 0 \tag{2.13}$$

where C denotes the finite difference equation from the continuity equation and M the finite difference equation from the momentum equation. Similar algebraic equations are written for other grid points of the sewer for the same time step (n + 1) Δt. If a sewer is divided into m space steps, there are (2m + 2) algebraic equations to be solved simultaneously for the unknown depth y and discharge Q at the grid points. This set of equations is solved using known conditions at time step n (initial conditions) and specified boundary conditions. Finite difference schemes which must be solved for all grid points at time level (n + 1) Δt simultaneously are called implicit schemes. Since application of equations (2.12) and (2.13) at every spatial grid point leads to a system of nonlinear algebraic equations, suitable methods must be used for solutions. Among others, the double sweep method is widely accepted [2]. This method is commonly used to solve matrices of a tri-diagonal or pentadiagonal form.

Abbott-Ionescu scheme, Figure 5. The continuity equation as cited in [18] reads

$$\frac{\partial Q}{\partial x} + b_s \frac{\partial h}{\partial t} = 0 \tag{2.14}$$

where b_s is the storage width, i.e., the width of the flow cross-section measured at the surface and h (water level) is height above the datum. The continuity equation only contains the derivative of Q with regard to x. The corresponding finite difference equation is therefore centred at points labelled as h-points in Figure 5a (a generalised scheme after [18]). The individual derivative terms in Equation (2.14) are centred at time (n + ½) and written in finite difference form as:

$$\frac{\partial Q}{\partial x} \approx \frac{1}{2\Delta x_j}\left\{\left[\frac{1}{2}\left(Q_{j+1}^{n+1} + Q_{j+1}^{n}\right) - \frac{1}{2}\left(Q_{j-1}^{n+1} + Q_{j-1}^{n}\right)\right]\right\} \tag{2.15}$$

$$\frac{\partial h}{\partial t} \approx \frac{1}{\Delta t}\left(h_j^{n+1} - h_j^{n}\right) \tag{2.16}$$

$$b_s = \frac{A_{o,j} + A_{o,j+1}}{2\Delta x_j} \tag{2.17}$$

where $A_{o,j}$ is the surface area between grid points (j – 1) and j; $A_{o,j+1}$ is the surface area between grid points j and (j + 1); and $2\Delta x_j$ is the distance between points (j – 1) and (j + 1). Substituting equations (2.15), (2.16) and (2.17) into the continuity equation yields a system of algebraic equations, whose general formulation is expressed by Equation (2.21).

The momentum equation is centred at points labelled in Figure 5b as Q-points at time level (n + ½). The derivatives in Equation (1.4) are expressed as follows:

$$\frac{\partial Q}{\partial t} \approx \frac{1}{\Delta t}\left(Q_j^{n+1} - Q_j^{n}\right) \tag{2.18}$$

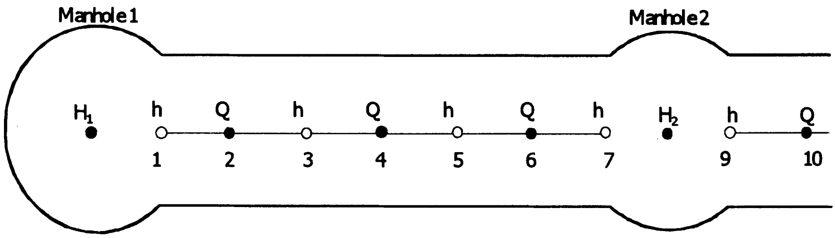
$$\frac{\partial}{\partial x}\left(\beta\frac{Q^2}{A}\right) \approx \frac{1}{2\Delta x_j}\left[\left(\beta\frac{Q^2}{A}\right)_{j+1}^{n+1/2} - \left(\beta\frac{Q^2}{A}\right)_{j-1}^{n+1/2}\right] \tag{2.19}$$

$$\frac{\partial h}{\partial x} \approx \frac{1}{2\Delta x_j}\left\{\left[\frac{1}{2}\left(h_{j+1}^{n+1} + h_{j+1}^{n}\right) - \frac{1}{2}\left(h_{j-1}^{n+1} + h_{j-1}^{n}\right)\right]\right\} \tag{2.20}$$

The remaining parameters in Equation (1.4) are expressed at time (n + ½). A formulation for the quadratic term Q^2 is used after [18] as $Q_j^{n+1}\, Q_j^n$. Inserting all the finite difference approximations in Equation (1.4) yields the general formulation

$$\alpha_j Z_{j-1}^{n+1} + Z_j^{n+1} + Z_{j+1}^{n+1} = \delta_j \tag{2.21}$$

The general variable Z equals h in grid points with odd numbers, and Q in grid points with even numbers. Coefficients α, β, γ and δ are specified in [18]. There is no time (n + ½) and the corresponding discharge values must be obtained by interpolation. For N computational points, there are N unknowns (Q^{n+1}, h^{n+1}) and (N – 2) equations. Two additional boundary conditions are needed to close the system. However, the boundary conditions must be formulated properly, it is not possible to prescribe Q(t) condition at a point labelled as "h-point", Figure 5.



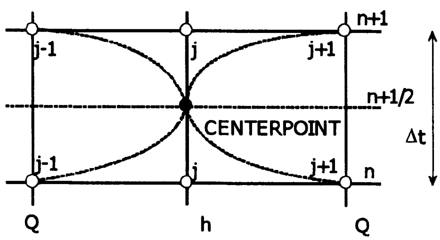


Figure 5a. Centering the Abbott-Ionescu scheme for the continuity equation.

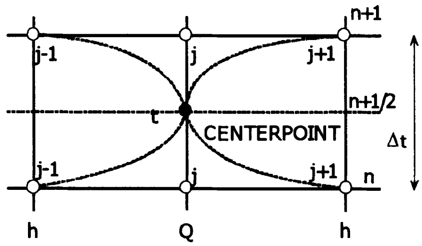


Figure 5b. Centering the Abbott-Ionescu scheme for the momentum equation.

2.2.3. *The method of characteristics.*

The method of characteristics solves two sets of "characteristic" equations, each set consisting of a pair of ordinary differential equations. These equations are transformed mathematically from the Saint-Venant equations or similar hyperbolic type equations. For example, governing Equations (1.1) and (1.2) expressed in a form containing the mean velocity are combined linearly and reduced to two total differential equations called characteristic equations [5].

$$\frac{dV}{dt} \pm \frac{g}{\sqrt{g.A/B}}\frac{dy}{dt} + g\left(S_f - S_o\right) = 0 \tag{2.22}$$

which need to be integrated along characteristic curves whose directions are

$$\left(\frac{dx}{dt}\right)_{\pm} = V \pm \sqrt{gA/B} = V \pm c \qquad (2.23)$$

The characteristic directions can be interpreted as the path of propagation of an infinitesimal disturbance. The characteristic direction with the + sign is called the forward characteristic (labelled as C^+) and the one with - sign is the backward characteristic, labelled as C^-. The integration of Equations (2.22) yields the values of y and V at the points where C^+ and the C^- characteristics intersect. The method of characteristics provides a solution that can serve as a benchmark for the evaluation of the solution by explicit or implicit schemes. However, because the characteristic curves do not form a rectangular grid in the x - t plane, interpolations are necessary as discussed in the method of specified intervals [15].

For a general channel shape, the wave propagation velocity (celerity) reads

$$c = \pm\sqrt{gA/B} \qquad (2.24)$$

and reduces for a rectangular channel to

$$c = \pm\sqrt{gy} \qquad (2.25)$$

Thus equations (2.22) and (2.23) provide a basis for a finite difference solution. Referring to Figure 6, if the variables V and y are known at R and S, then four equations can be written in terms of the unknowns at P,

$$V_P - V_R + g\int_{y_R}^{y_P} \frac{1}{c}dy + \int_{t_R}^{t_P} g\left(S_f - S_o\right)dt = 0 \qquad (2.26)$$

$$x_P - x_R = \int_{t_R}^{t_P} \left(V + c\right)dt \qquad (2.27)$$

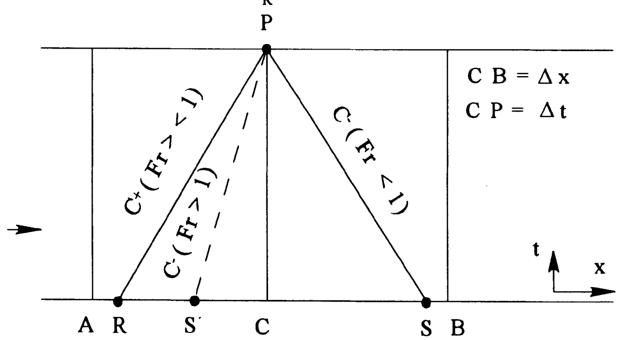


Figure 6. Method of characteristics solution.

$$V_P - V_S - g\int_{y_S}^{y_P} \frac{1}{c} dy + \int_{t_S}^{t_P} g\left(S_f - S_o\right) dt = 0 \tag{2.28}$$

$$x_P - x_S = \int_{t_S}^{t_P} \left(V - c\right) dt \tag{2.29}$$

These equations, forming two pairs of relationships, are the C^+ and C^- characteristic equations, and characteristic directions for unsteady flow prediction in open channels. As such the expressions can be related to a fixed grid in Δx and Δt as shown in Figure 6. In subcritical flow the local celerity c exceeds the local mean flow velocity, $c > V$; thus characteristics intersecting at P originate in both adjacent grid sections. However, in supercritical flow $V > c$; thus downstream information cannot propagate upstream and the slopes of both characteristics are positive, i.e., both C^+ and C^- lines originate in the upstream grid section.

The relative magnitudes of V and c are such that V cannot be ignored relative to c, and the characteristic base points R and S shown in Figure 6 do not lie at the nodes upstream and downstream of P. For a rectangular, regular grid, the length Δx may be fixed. However, the value of Δt must be such that R and S are located as shown in Figure 6. Using the Courant criterion to assure stability (eq. 2.40), R and S cannot fall at the nodes on either side of P. Hence interpolation for the base values is necessary. Referring to Figure 6, for the *subcritical case*, Equation (2.30) reads:

$$\frac{c_C - c_R}{c_C - c_A} = \frac{V_C - V_R}{V_C - V_A} = \frac{x_C - x_R}{x_C - x_A} = \left(V_R + c_R\right)\frac{\Delta t}{\Delta x} = \frac{y_C - y_R}{y_C - y_A} \tag{2.30}$$

As $x_P = x_C$, $x_P - x_R = (V_R + c_R)\,\Delta t$, $\gamma = \Delta t / \Delta x$

$$V_R = \frac{V_C + \gamma\left(-V_C c_A + c_C V_A\right)}{1 + \gamma\left(V_C - V_A + c_C - c_A\right)} \qquad c_R = \frac{c_C\left(1 - V_R \gamma\right) + c_A V_R \gamma}{1 + c_C \gamma - c_A \gamma} \tag{2.31}$$

$$y_R = y_C - \left(y_C - y_A\right)\left[\gamma\left(V_R + c_R\right)\right] \tag{2.32}$$

Similarly for the node S,

$$V_S = \frac{V_C - \gamma\left(V_C c_B - c_C V_B\right)}{1 + \gamma\left(V_C - V_B - c_C + c_B\right)} \qquad c_S = \frac{c_C + V_S \gamma\left(c_C - c_B\right)}{1 + c_C \gamma - c_B \gamma} \tag{2.33}$$

$$y_S = y_C + \gamma\left(y_C - y_B\right)\left(V_S - c_S\right) \tag{2.34}$$

For the supercritical flow regime the equations relating to R remain unchanged, and those for S′ read

$$V_{S'} = \frac{V_C\left(1 + \gamma c_A\right) - V_A c_C \gamma}{1 + \gamma\left(V_C - V_A + c_A - c_C\right)} \qquad c_{S'} = \frac{c_C + V_{S'} \gamma\left(c_A - c_C\right)}{1 + c_A \gamma - c_C \gamma} \tag{2.35}$$

$$y_{S'} = y_C - (y_C - y_A)\left[\gamma\left(V_{S'} - c_{S'}\right)\right] \tag{2.36}$$

The finite difference characteristics, Equations (2.26) and (2.28) are reduced to a simpler form of two linear simultaneous algebraic equations to be solved for the two unknowns y_P and V_P,

$$V_P = V_R - g\frac{y_P - y_R}{c_R} - g\Delta t\left(S_{fR} - S_o\right) \tag{2.37}$$

$$V_P = V_S + g\frac{y_P - y_S}{c_S} - g\Delta t\left(S_{fS} - S_o\right) \tag{2.38}$$

Simultaneous solution yields the depth at node P,

$$y_P = \frac{1}{c_R + c_S}\left[y_S c_R + y_R c_S + c_R c_S\left(\frac{V_R - V_S}{g} - \Delta t\,(S_{fR} - S_{fS}\right)\right] \tag{2.39}$$

2.3. NUMERICAL STABILITY AND CONVERGENCE

The selection of the time step Δt is often a compromise of three criteria [17]. The first criterion is the physically significant time required for the flow to pass through the computational reach. In sewers, this can be approximately 0.25 - 2 minutes. This criterion is not important for a slowly varying unsteady flow when larger computational time steps Δt may be used, but it must be considered for a rapidly varying unsteady flow. The second criterion is a sufficiently small Δt to ensure numerical stability (see the Courant criterion below). This criterion sometimes requires a time step Δt of 1 or 2 seconds. The third criterion is the time interval of the available input data. If we require Δt to be smaller than the data time interval (for example for the inflow hydrograph), the corresponding data can be only interpolated. All three criteria should be considered, but the computational point of view, the second, numerical stability, is the most important.

Since we compute discrete solutions of the governing flow equations, we have to guarantee the numerical stability and convergence. As far as the term convergence is concerned, discrete solutions of the finite-difference scheme should approach the exact solution of the governing equations when Δt, $\Delta x \rightarrow 0$. However, the true solution of the full Saint-Venant equations is not known and we must adopt some indirect means for verification of the solutions, e.g., comparison with the method of characteristics, proving the ability to conserve volume, proving the ability to compute steady state flow conditions, etc. Numerical stability requires that truncation and round-off errors stay small during computation. The main restriction on the use of explicit schemes for the solution of unsteady, one-dimensional, real fluid flows is that the Courant number must be less than or equal to unity. The Courant criterion is the ratio of analytical to numerical celerity, and for nearly-horizontal, free-surface flow can be written as

$$Cr = \frac{dx/dt}{\Delta x/\Delta t} = \frac{V + \sqrt{gA/B}}{\Delta x/\Delta t} \leq 1 \quad \text{resp.} \quad \Delta t \leq \frac{\Delta x}{|V + c_{max}|} \tag{2.40}$$

The criterion $Cr \leq 1$ is a necessary and sufficient condition for stability of the system of equations for nearly-horizontal flows [2, 4]. The time step cannot be chosen arbitrarily but it must satisfy the Courant criterion. Therefore, all explicit finite difference schemes, when applied to the hyperbolic flow equations, are conditionally stable.

Implicit schemes have no time step limit for stability and can be run successfully at Courant numbers greater than unity. Generally speaking, they can be made unconditionally stable. However, for $Cr \gg 1$, phase-errors can be excessive so that a control has to be kept on Cr in practical computations. A linear stability analysis of the Priessmann scheme can be found in [2] together with the accuracy displayed as amplitude and phase portraits. A definitive study, taking into account also bed slope, boundary shear and the flow regime, has not been yet performed. The analysis of instability is understood only for quasi-linearised equation systems.

3. Flow Instabilities

Hydraulic instabilities may be classified as follows [17]:

- A near-dry bed flow instability.
- The transition between supercritical and subcritical flow (moving hydraulic jumps).
- The transition between open-channel flow and pressurised flow.
- Surface waves.
- Hydraulic transients.

A near-dry bed flow instability. Dry-bed instability occurs when the channel is nearly dry and there is not a sufficient amount of water to flow. These hydraulic conditions can occur at the beginning and end of the runoff, and are of little practical importance. In order to avoid numerical problems when simulating small discharges, it is possible to assume some minimum discharge corresponding to the relative depth y/D of about 2%.

The transition between supercritical and subcritical flow. When there is a transition from supercritical to subcritical flow, water surface rises abruptly and this transition is called a hydraulic jump. As long as the flow is unsteady, the jump is moving either upstream or downstream. This transition is sometimes called a positive jump because a stationary observer sees an increase in depth as the wave front passes. The transition from subcritical to supercritical flow creates negative jumps which appears to a stationary observer as a lowering of the water surface. Figure 7 illustrates four types of moving hydraulic jumps. These transitions can be modelled numerically using the diffusion wave approximation. The jump loses its abrupt nature during computation and becomes smeared. Another method based on tracking the jump is rather tedious [12].

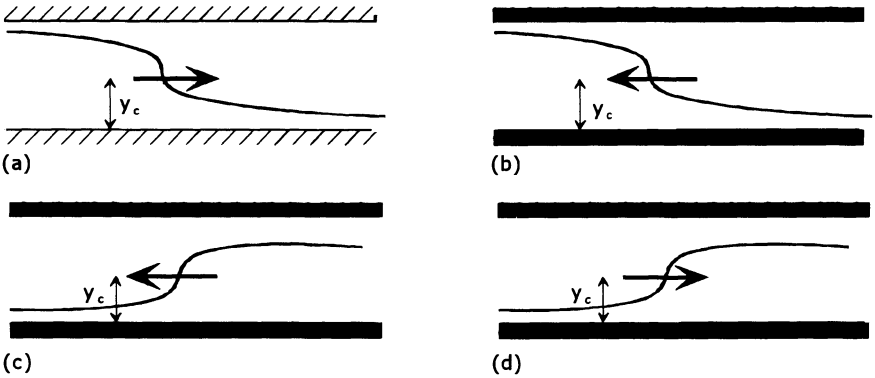


Figure 7. Moving hydraulic jumps; a) Positive jump moving downstream; b) Negative jump moving upstream; c) Positive jump moving upstream; d) Negative jump moving downstream.

The transition between open-channel flow and pressurised flow. In routing a flood flow through a sewer, the open-channel flow is described by a looped rating curve (see Fig. 2). Even for a steady uniform flow, the discharge-depth relation is not unique as seen in Figure 8. If Q_f denotes the discharge when the pipe is full, D the pipe diameter, Q the discharge corresponding to a given depth y, then the maximum discharge occurs approximately for y/D = 0.95, at point G. The decrease of the discharge when the pipe is nearly full, i.e., between points G and F, is caused by the rapid increase of the wetted perimeter. Furthermore, for a given discharge it is not possible to obtain a unique flow depth.

When the sewer pipe is completely full, the flow equations must change to those of pressurised flow, but this transition is not simple nor smooth. Additional assumptions must be made to specify the switch between open-channel flow and surcharged flow, such as along line MN, or line GE of the maximum steady uniform part-full pipe flow, or a line JF. A more complicated but computationally less stable assumption is switching to surcharged flow along GE or MN lines and switching back to open-channel flow along FJ line. From the numerical point of view it is suitable to define a transition along line GH.

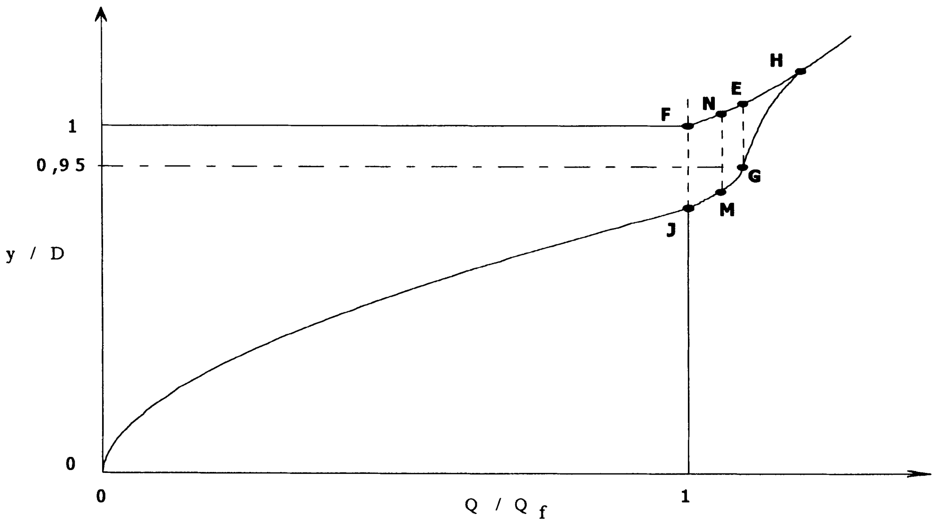


Figure 8. Discharge and depth assumptions for transition between open-channel flow and surcharge flow. Reprinted from [17] with permission.

Surface waves. When the depth to diameter ratio y/D is greater than 0.9, the interfacial shear between the air and water generates waves with a small amplitude but high frequency. This may occur both for subcritical and supercritical open-channel flows.

When a flow in the sewer system is supercritical with Froude number greater than 2, for circular conduits, roll waves are formed (see Figure 9). This instability is caused by high friction. The water is moving considerably faster near the free surface than near the sewer bottom.

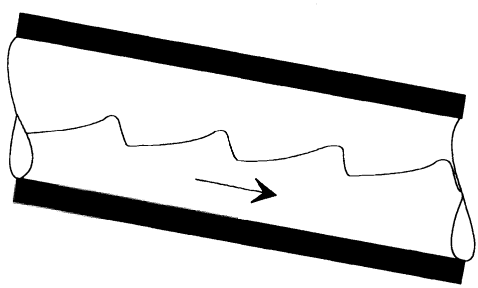


Figure 9. Roll waves in a sewer. Reprinted from [17] with permission.

4. Mathematical Simulation of Surcharged Flow

4.1. GOVERNING EQUATIONS

The governing equations of closed conduit transient flow, see e.g., [3], [14], [15], consist of the continuity equation

$$\frac{\partial H}{\partial t} + V\frac{\partial H}{\partial x} + \frac{a^2}{g}\frac{\partial V}{\partial x} = 0 \tag{4.1}$$

and equation of motion

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial H}{\partial x} + \frac{f\,V|V|}{2.D} = 0 \tag{4.2}$$

in which H(x, t) is piezometric head (hydraulic grade line) above the datum; V(x, t) mean velocity; D is pipe diameter; and a is the wave-speed (celerity) of pressure surge. This set of hyperbolic quasi-linear partial differential equations can be transformed into four ordinary differential equations by the method of characteristics and solved numerically using the finite difference method [3], [15]. Another solution is based on a simplified approach cited in [17].

4.2. SIMPLIFIED APPROACH

In the surcharged flow (see Figure 10), the cross-sectional area is constant, being equal to the full pipe area A_f. Hence $\partial A / \partial x = 0$ and the continuity equation can be written as

$$Q = A_f V \tag{4.3}$$

The dynamic equation for an incompressible fluid, applicable to the entire pipe length is [17],

$$\frac{L}{gA_f}\frac{\partial Q}{\partial t} = H_u - H_d - K_u\frac{Q^2}{2gA_f^2} - K_d\frac{Q^2}{2gA_f^2} - S_f L \tag{4.4}$$

in which H_u and H_d are the total head at the upstream and downstream manholes, respectively; K_u and K_d are the entrance and exit loss coefficients; L is the length of the sewer; S_f is the friction slope; t is time and g is the gravitational acceleration.

Assuming that in a time step Δt, from n to (n + 1),

$$\frac{\partial V}{\partial t} \approx \frac{V^{n+1} - V^n}{\Delta t} \tag{4.5}$$

a simple difference form of Equation (4.4) with the variables expressed as their respective values form Equation (4.5), at the time step (n + 1) is

$$H_u^{n+1} - H_d^{n+1} - \left[K_u^{n+1} + K_d^{n+1}\right]\frac{(V^{n+1})^2}{2g} - LS_f^{n+1} - \frac{L}{g\Delta t}\left(V^{n+1} - V^n\right) = 0 \quad (4.6)$$

This equation is solved with a specified upstream boundary condition at the sewer entrance and a downstream boundary condition at the sewer exit together with a properly specified initial condition. The equation can be rearranged to solve for Q_{n+1}, explicitly.

$$Q^{n+1} = Q^n + \frac{g\Delta t}{L}A_f\left[H_u - H_d - \left(K_u + K_d\right)\frac{(Q^n)^2}{2gA_f^2} - LS_f^n\right] \quad (4.7)$$

It has been shown [17] that for surcharged sewers eq. (4.7) is essentially as good as the transient model which considers the elastic effect.

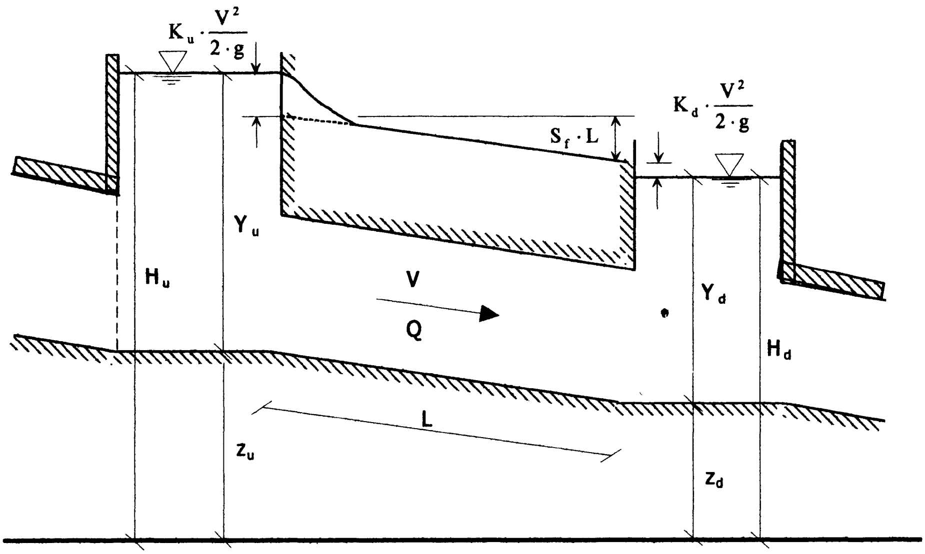


Figure 10. Surcharged flow in a sewer. Reprinted from [17] with permission.

4.3. PREISSMANN SLOT CONCEPT

To reduce the problems of accounting for which reaches are surcharged and switching the equations, Preissmann suggested an open-slot concept [1], [17], [18]. The idea is to transform the pressurised conduit flow situation into a conceptual open-channel flow situation by a hypothetical slot, shown in Figure 11. Hypothetical, continuous, narrow slot attached to the sewer crown and running over the entire length of the sewer must not introduce an appreciable error in the volume of water. Conversely, the slot cannot be too narrow in order to avoid the numerical problem associated with a rapidly moving pressure surge.

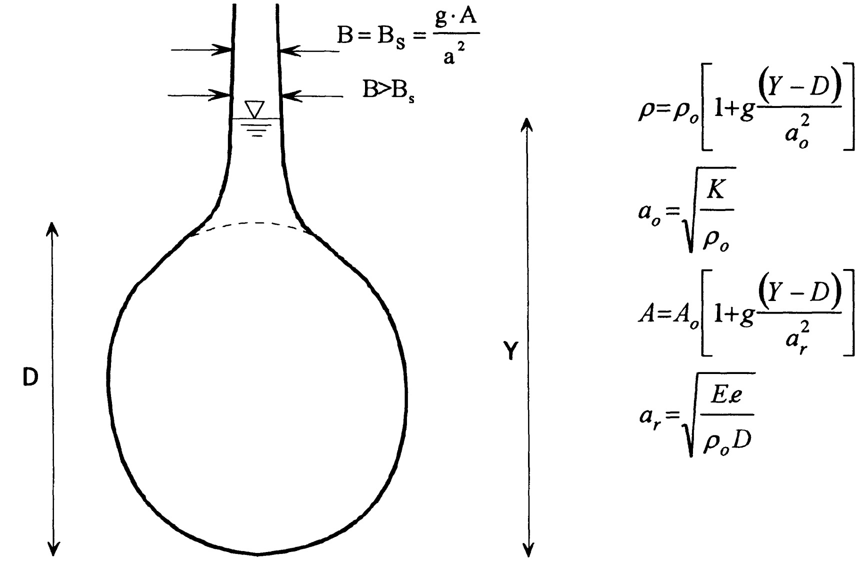


Figure 11. Hypothetical Preissmann open slot.

A theoretical basis for the determination of the width of the slot is to consider the surge celerity in the sewer accounting for the compressibility of water and elasticity of the pipe, making the width of the slot such that the wave celerity in the slot is the same as that of the actual elastic pipe. The celerity c_l of the slot pipe is given as

$$c_1 = \sqrt{\frac{g A}{B}} \tag{4.8}$$

in which B is the slot width and A is the flow cross-sectional area. The general expression for the wave speed under pressurised flow is

$$c = \sqrt{\frac{K}{\rho\left[1 + (K/E)\psi\right]}} \tag{4.9}$$

In Equation (4.9), E is the Young's modulus of elasticity of the conduits walls; ψ is a nondimensional parameter that depends upon the elastic properties of the conduit; and K and ρ are the bulk modulus of elasticity and density of the fluid, respectively. Neglecting the area contribution of the slot and hence $A = A_f = \pi D^2 / 4$ for a circular pipe, and equating c_l to the pressure wave speed c in the elastic pipe described by Equation (4.9) gives the theoretical slot width

$$B = \pi g D^2 / 4c^2 \tag{4.10}$$

For small pipes Equation (4.10) may give too small a slot width, which would cause numerical problems [17]. The change of the cross-sectional area and the water density, once the flow depth is above the pipe crown, has been recommended in [18] by means of equations shown in Figure 11. The subscript "o" denotes the density and the flow area for open-channel flow conditions in a sewer.

Use of the hypothetical Preissmann slot has the following advantages:

- It uses only the Saint-Venant equations and avoids switching between the surcharge equation and open-channel equations. The treatment of the boundary conditions is simpler than it would be for both separate flow regimes.
- There is no need to define surcharge criteria.
- It is not necessary to identify the pipes that are surcharged at different times.
- It permits the flow transition to progress computationally reach by reach in a sewer regardless of the fact whether only part of the pipe length is full.
- It does not require so many assumptions as the standard approach to achieve numerical stability.
- It is not difficult for programming.

However, there are some disadvantages. It introduces a potential accuracy problem in the mass and momentum balance of the flow, if the slot is too wide, and stability problems, if it is too narrow. It is hypothetical rather than real, and it has not been verified in a physical model.

5. Initial and Boundary Conditions

In order to obtain solution of unsteady flow in open channels, we need to know the flow conditions (dependent variables) at some starting time. These flow conditions are referred to as initial conditions. Since our physical system is limited, we have to specify the conditions for dependent variables at the limits or boundaries. It is appropriate to distinguish between external boundary conditions and internal boundary conditions. While the external boundary conditions describe the interaction of the modelled sewer system with its surroundings, the internal boundary conditions express hydraulic relations for the sewer network components which are located inside the modelled area.

5.1. SIMULATION OF INITIAL CONDITIONS

The unsteady flow process in a sewer depends on the initial depth and discharge (velocity) inside the sewer when the flood begins, and also on the flow conditions at the sewer entrance and exit. The initial condition is the flow condition in the sewer pipe when computation starts at time $t = 0$, described by such dependent variables as discharge $Q(x, 0)$, or the velocity $V(x, 0)$ and the depth $y(x, 0)$.

For a storm sewer, the magnitude of the base flow depends on the characteristics of the inflow hydrograph and the sewer pipe. For combined sewers the initial condition is usually the dry-weather flow. From the hydraulic point of view, initial open-channel flow in sewers is usually computed as steady uniform flow, or more correctly, steady non-uniform flow [6].

5.2. BOUNDARY CONDITIONS IN A SEWER

When the Saint-Venant equations are used to solve unsteady-flow problems, in addition to the initial conditions, two boundary conditions must be specified in order to obtain a unique solution. The boundary conditions of an interior reach need not be explicitly specified, because they are implicitly accounted for in the flow equations of the adjacent reaches. For the exterior reaches containing either the sewer entrance or exit (see Figure 12), the upstream boundary condition is the sewer entrance flow condition over the period of runoff [17]. Usually this is the inflow discharge or depth hydrograph from the upstream junction, $Q(0, t)$ or $y(0, t)$. In the reach containing the sewer exit, the downstream boundary condition can be a discharge-time function, stage-time function, stage-discharge relationship $Q(y)$ (rating curve), or the stage-velocity relationship $V(y)$.

Classification of unsteady non-uniform flows in a sewer is schematically depicted in Figure 13. If the flow is subcritical, one boundary condition is at sewer entrance and the other is at the sewer exit. If the flow is supercritical, both boundary conditions are at the upstream end, the entrance, one of them often is a critical depth criterion. If at one instant a hydraulic jump occurs in an interior reach inside the sewer, two upstream boundary conditions at the sewer entrance and one downstream boundary condition at the sewer exit should be specified. If a transition from subcritical to supercritical flow occurs inside the sewer, two boundary conditions (one at the sewer entrance and one at the exit) are needed and a critical depth relation is used as an interior boundary condition. Handling the moving surface discontinuity is a complex matter [3], [12].

5.3. BOUNDARY CONDITIONS IN A SEWER NETWORK

The boundary conditions can be classified as external and internal. The external boundary conditions represent an interaction of the external hydraulic parameters with the modelled system, while internal boundary conditions are described by hydraulic equations of the overflow weir structures, pump stations, gate valves, etc.

Upstream and downstream boundary conditions of sewers are illustrated in Figure 12. There are four main cases of entrance hydraulic conditions and four cases for the downstream exit conditions [17].

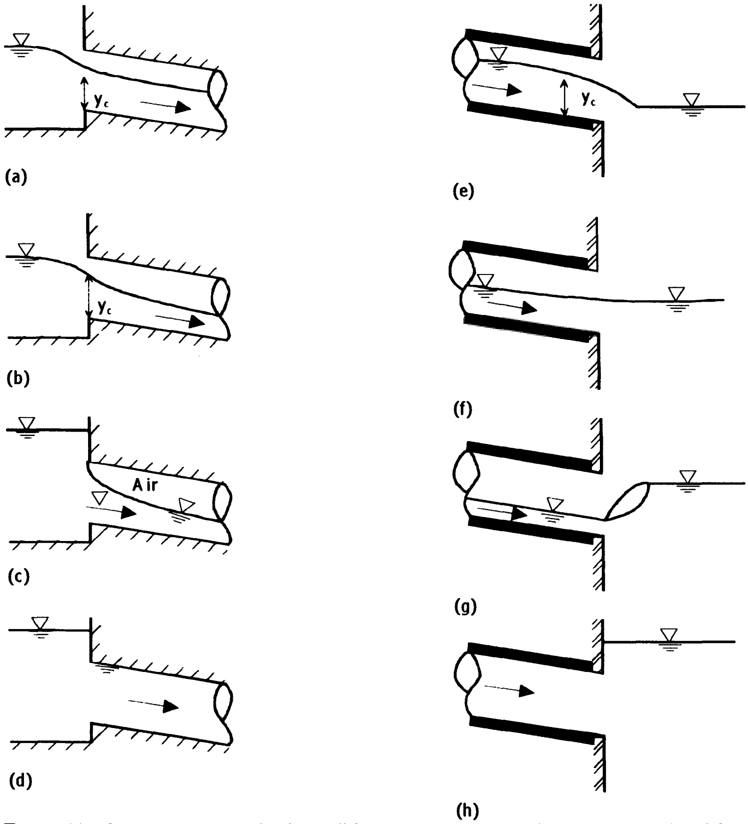


Figure 12. Sewer entrance and exit conditions; a) Nonsubmerged entrance, subcritical flow; b) Nonsubmerged entrance, supercritical flow ; c) Submerged entrance, air pocket; d) Submerged entrance, water pocket; e) Nonsubmerged, free fall; f) Nonsubmerged, continuous; g) Nonsubmerged, hydraulic jump; h) Submerged. Reprinted from [17] with permission.

In Figure 13, 10 different cases of the unsteady non-uniform flow in sewers are shown and demonstrate that for a given time the flow depends on the flow regime, whether it is subcritical, supercritical or surcharged [17].

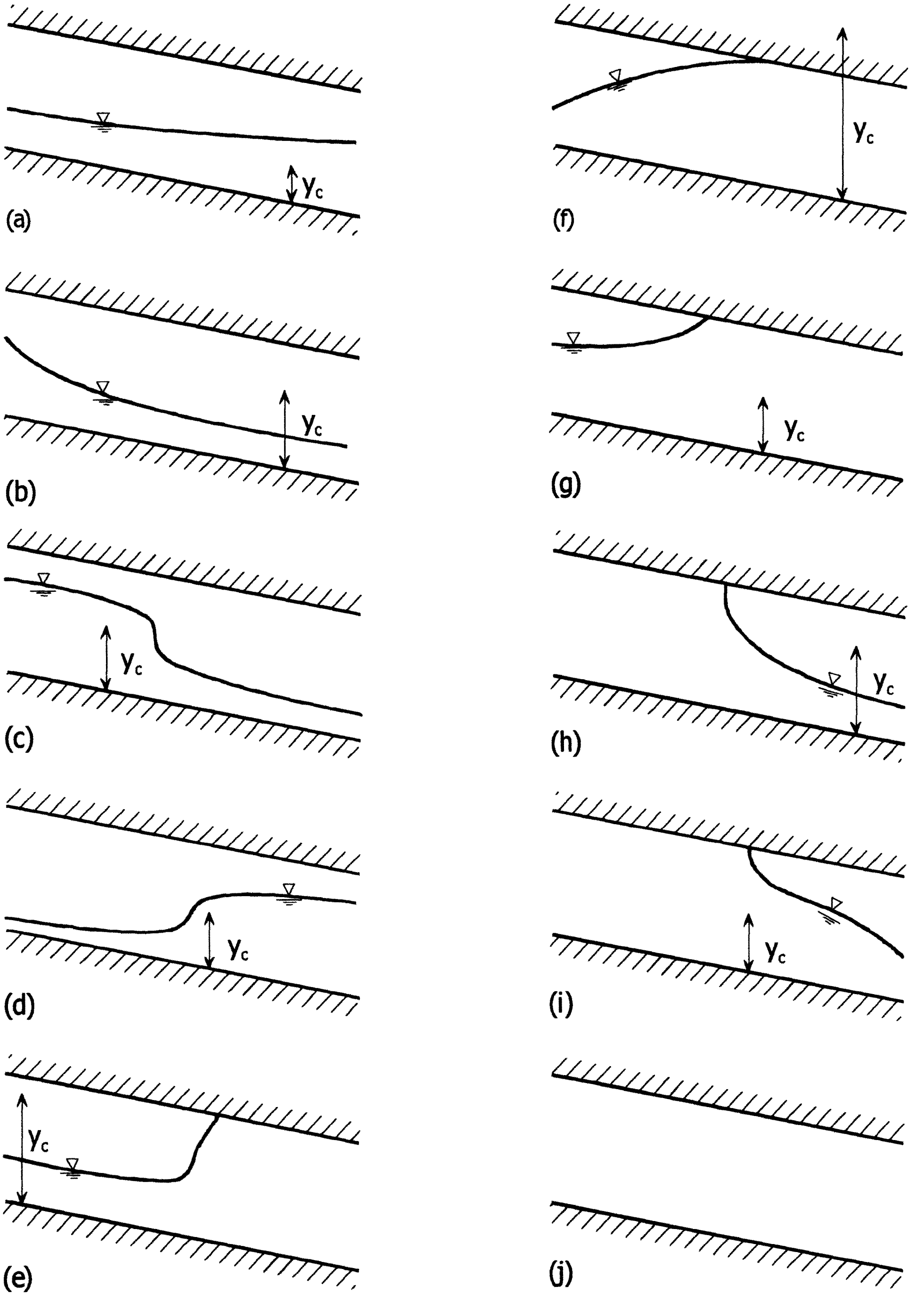


Figure 13. Classification of unsteady non-uniform flow in a sewer: case a - subcritical, case b - supercritical, case c - subcritical to supercritical, case d - supercritical to subcritical, case e - supercritical jump to surcharge, case f - supercritical to surcharge, case g - subcritical to surcharge, case h - surcharge to supercritical, case i - surcharge to subcritical, case j - surcharge. Reprinted from [17] with permission.

A summary of the boundary conditions is illustrated in Figure 14. In addition to the boundary conditions at inflow nodes (manholes) and outflow nodes (outlets), the corresponding hydraulic relationships must be used for internal boundary conditions, e.g., for sewer overflows and pumping stations.

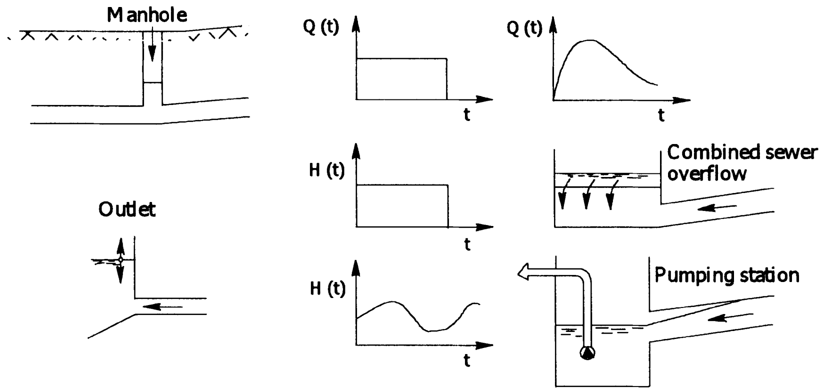


Figure 14. Boundary conditions.

6. Combined Sewer Overflows

The impact of combined sewer overflows (CSOs) on receiving waters should be determined as thoroughly as possible in order to minimise the pollution load and to prevent water quality problems in receiving streams. Various pollution prevention and control strategies have been used for CSOs. The main control criteria being considered are the annual volume and frequency of overflows, percent wet weather volumetric control and percent pollution control.

The CSO abatement strategy requires hydraulic analysis of the sewer performance. Detailed knowledge of actual flow conditions leads to reductions of dry weather and wet weather overflows. Every simulation model has to perform hydraulic calculation of CSOs, however, this calculation is dependent on the purpose of the model and its capability to solve governing equations. The level of hydraulic calculation is different for master planning and for detailed modelling of flow in sewer systems.

There are more possibilities for classification of CSO structures, e.g., from the structural or hydraulic point of view. According to the latter one, CSO structures can be classified as follows:

1. Weirs without regulation, e.g., front weir, side weir etc.
2. Weirs with regulation, e.g., with a downstream throttling pipe, downstream control gate, movable weir crest, vortex flow controller etc.

3. Special overflow structures, e.g., an extended stilling pond, a single and a double high-side weir, air-regulated saddle siphon, hydrodynamic separator etc.

Many CSO structures are designed in the form of a side weir. Lateral overflow over side weirs is spatially varied flow. There are in principal two approaches for describing the flow over side weirs, namely the energy equation, e.g., [13], or the momentum equation [9]. Only the energy principle, which is easier for practical use, will be discussed.

6.1. GOVERNING EQUATIONS

The governing equation obtained from the energy principle is

$$\frac{dy}{dx} = \frac{S_o - S_f - \dfrac{\alpha Q}{gA^2}\dfrac{dQ}{dx} + \dfrac{\alpha Q^2}{gA^3}y\dfrac{dB}{dx}}{1 - \dfrac{\alpha Q^2 B}{gA^3}} \tag{6.1}$$

in which x is the distance along the channel, measured in a downstream direction; y is the flow depth; Q is the overflow discharge; A is the flow cross-sectional area; B is the surface width; S_o is the bottom slope; S_f is the frictional slope; α is the kinetic energy correction factor; β is the momentum correction factor and g is the gravitational acceleration. Although the flow over side weirs forms a considerable angle with the direction normal to the weir, a conventional weir equation for discharge per unit length is assumed in the form

$$-\frac{dQ}{dx} = q = m_s\sqrt{2g}\left(y - s\right)^{3/2} \tag{6.2}$$

$$m_s = cm \tag{6.3}$$

where q is the discharge per unit length over the side weir; m_s is the discharge coefficient for a side weir; m is the discharge coefficient for a perpendicular inflow condition; c is the shape coefficient (see below); y is the depth of flow and s is the height of weir crest above the channel invert. The shape coefficient for vertical broad-crested weirs, shown in Figs. 15 and 16, is

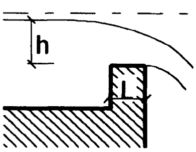


Figure 15.

$$c = 1 - \frac{2}{9\left(1 + \zeta_b^4\right)} \tag{6.4}$$

$$\text{and} \quad \zeta_b = h_o / L \tag{6.5}$$

where $h_o = h + \alpha V^2/2g$ is the energy head.

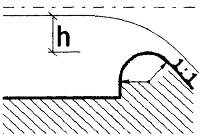


Figure 16.

$$c = \frac{\sqrt{3}}{2}\left[1 + \frac{22/81\zeta_r^2}{1 + 1/2\zeta_r^2}\right] \tag{6.6}$$

where $\zeta_r = h_o/r$ (6.7)

In sub-critical flow, the discharge coefficient for flow over side weirs in circular channels [16] is expressed as

$$m_s = \left[0.21 + 0.094\sqrt{1.75\frac{L_s}{D} - 1}\right] + \left[0.22 - 0.08\sqrt{1.68\frac{L_s}{D} - 1}\right]\sqrt{1 - Fr_1} \tag{6.8}$$

and for supercritical flow as

$$m_s = -\left[0.046 + 0.0054\sqrt{1.67\frac{L_s}{D} - 1}\right]Fr_1 + \left[0.024 + 0.021\sqrt{1 + 35.3\frac{L_s}{D}}\right] \tag{6.9}$$

where D is the pipe diameter; L_s is the length of the side weir and Fr_1 is the Froude number for the inlet flow depth. In [18], a built-in overflow formula is used, assuming the water depth to be constant and equal to the critical depth, as shown in Figure 17.

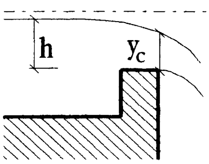


Figure 17.

$$Q_b = L.\sqrt{g}\left[\frac{2}{3 + K_c}H\right]^{3/2} \tag{6.10}$$

In eq. (6.10), L is the crest length and K_c is the energy loss coefficient, whose value depends on the configuration of the overflow structure.

Calculation of the water surface profile described by y = f(x) can be performed using an iterative procedure

$$y_{i+1} = y_i + \frac{dy}{dx}\Delta x \tag{6.11}$$

in which Δx is the distance between the sections. The depth for small Δx at position (i + 1) is

$$y_{i+1} = y_i + \frac{1}{2}\text{SIGN}\left[\left(\frac{dy}{dx}\right)_{i+1} + \left(\frac{dy}{dx}\right)_i\right]\Delta x \tag{6.12}$$

The value of SIGN is -1 for computation in the upstream direction and +1 for computation in the downstream direction. The term dQ/dx in Equation (6.1) is evaluated from Equation (6.2). Discharge Q_{i+1} is then calculated from

$$Q_{i+1} = Q_i + \frac{1}{2}\text{SIGN}\left[\left(\frac{dQ}{dx}\right)_{i+1} + \left(\frac{dQ}{dx}\right)_i\right]\Delta x \tag{6.13}$$

The water surface profile calculation can be made as accurate as required by using a sufficiently small value of the increment Δx.

Calculation of the discharge over side weirs can be verified by comparisons with another theoretical solution, physical models or field measurements [6, 7, 8]. Two case studies will illustrate that the energy solution simulates correctly the CSO structures with side weirs.

Case Study 1. Figure 18 shows a side weir structure in a circular sewer of 2.88 m in diameter. The bottom slope is $S_o = 0.0009$, the Manning's roughness $n = 0.013$, side weir height $s = 2.22$ m, and the crest length $L = 16$ m. Measurements in a hydraulic scale model yielded the following scaled-up values: $Q_{in} = 15.504\ m^3 \cdot s^{-1}$, flow depth behind the overflow structure $y_d = 2.51$ m, and the overflow discharge $Q_m = 2.841\ m^3 \cdot s^{-1}$. For illustration, the solution from Equations (6.1, 6.2 and 6.3) using the explicit scheme gave for the overflow coefficient $m_s = 0.47$ the overflow discharge $Q_s = 2.898\ m^3 \cdot s^{-1}$, which is only 2% higher than the measured value. The MOUSE model calculation using eq. (6.10) yielded $Q_M = 3.391\ m^3 \cdot s^{-1}$, which is 19.3% higher than the measured value. Overestimation is caused by the assumption of the constant critical overflow depth over the crest length that is rather long. Sensitivity analysis with respect to the number of length steps is shown in Table 2.

TABLE 2. Sensitivity analysis of the iterative procedure, Equation (6.11)

Overflow coefficient $m_s = 0.44$		Overflow coefficient $m_s = 0.47$	
Number of equal Δx steps	CSO discharge Q_s	Number of equal Δx steps	CSO discharge Q_s
5	2.850	5	2.910
10	2.841	10	2.898
20	2.838	20	2.951

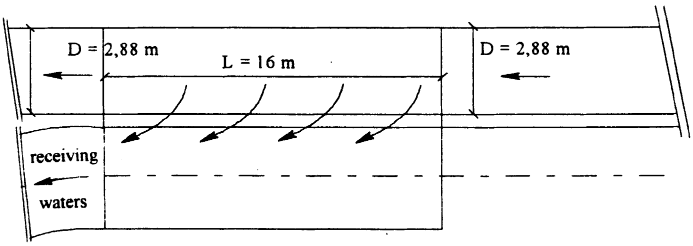


Figure 18. Schematic of the side weir structure.

Case study 2. The Waste Water Board of Prague (WWB) has started to use simulation tools in order to achieve more effective operation of their complex sewer system. One of the main reasons was also to evaluate hydraulic behaviour of various structures, including combined sewer overflows and large inverted siphons. This study shows hydraulic behaviour of an existing double inverted siphon, with insufficient capacity, under the Vltava River. The aims of the study were: a) model the existing siphon in order to determine its flow capacity; b) assess the interaction between a CSO structure located close to the inlet chamber and the siphon; and c) suggest possible improvements which could increase the flow capacity of the siphon.

The inverted siphon in Figure 19 consists of two steel parallel pipelines of 974 mm and 674 mm in diameters. The length of the structure is 190 m, the inflow sewer is a vertically oriented ellipse and the outfall sewer has a pear-shape cross-section. The measuring points for the inlet discharge Q_{in} , water level in the inlet chamber H_{257} and the outlet chamber H_{254} are also shown in Figure 19. In June and July 1995, a large set of data was collected and used for calibration and verification of a mathematical model. The time interval for discharge and water level measurements at H_{257} was two minutes, while water levels at H_{254} were measured every hour. Unfortunately, the discharge over the side weir was not measured and had to be calculated. The crest length was L = 4.75 m and the side weir height s = 1.27 m. Upstream of the inlet chamber and in the downstream pipe, sludge deposits were found. It was assumed that the bottom sediments were also present inside the inverted siphon, but no direct measurement was carried out.

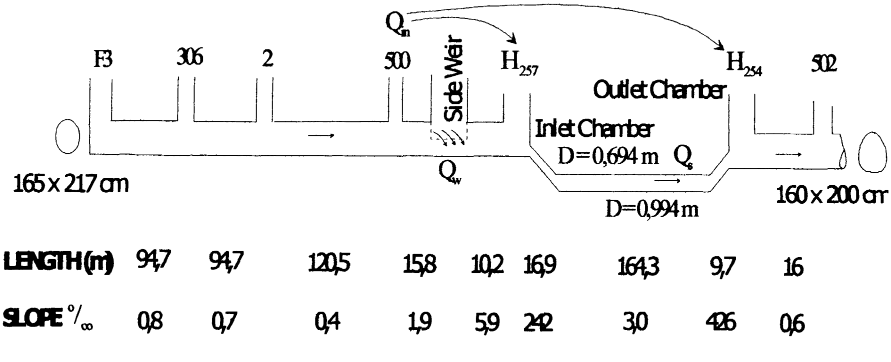


Figure 19. Schematic of the inverted siphon.

The MOUSE system [18], and the Steady Non-uniform Flow in Sewers (SNFS) model described in [6] were used to perform calibration for quasi-steady flows. It was observed that for a dry weather discharge of about 0.5 m^3s^{-1}, the side weir started to overflow at (0.7 - 0.75) m^3s^{-1}. Independent sets of measured data were used for validation. In order to obtain an overflow time history, the MOUSE dynamic pipe model was used. Hydraulic analysis showed that as the discharge increased, the self-cleansing process became important and the Manning's roughness decreased. After the flood event, the sediments tended to settle again. Using the measured data, the rating curves for the side weir were calculated using both models. Figure 20 shows the differences between both models due to the theoretical assumptions for the flow over side weirs.

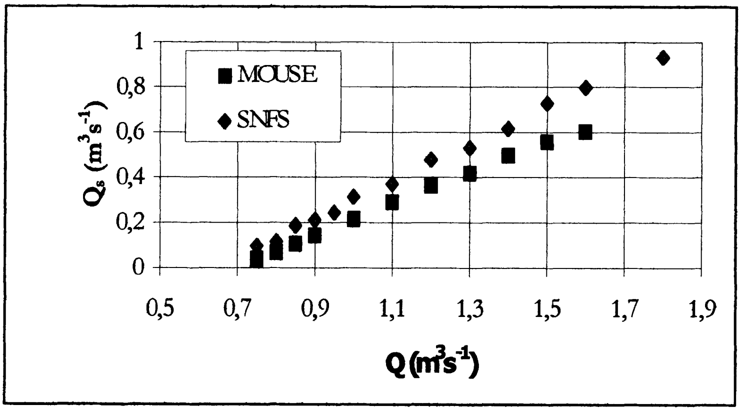


Figure 20. Calculated CSO discharge Q_s for the inlet discharge Q.

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MODELLING QUALITY OF URBAN RUNOFF

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1. Introduction

In this contribution, the characteristics of urban runoff quality are described first. Next the quality procedures used in the **RUNOFF** block of **SWMM** are presented, including the generation of pollutants (quality constituents). The quality of stormwater is generated by two mechanisms: pollutant **buildup** and **washoff.** It is assumed that, on an impervious area, the constituents build up during the dry period preceding a rainfall event, and are washed off during the storm. Both the buildup and washoff are treated empirically and their mathematical formulations require local data for calibration and verification. Then alternative formulations used in **MOUSE** are examined, followed by a review of the atmospheric dustfall model, **ATMDST,** the new buildup model, **NEWBLD,** and the **Particle Transport Model (PTM)**. In PTM the water quality parameters are based on erosion and deposition processes and use of a potency factor. Different methods of calibration of SWMM, including the use of expert systems, are discussed next. The paper concludes with probabilistic and statistical methods of runoff quality prediction.

2. Urban Runoff Quality

2.1. CHARACTERISTICS OF URBAN RUNOFF WATER QUALITY

The term quality relates to the constituents that are present in the water. The constituent may be any substance dissolved in water, such as dissolved oxygen, or any substance carried by water, such as suspended sediments, or a pollutant, such as lead.

The concentration of a constituent, C, is its mass, M, per unit volume of water, V, namely

$$C = \frac{M}{V} \tag{2.1}$$

The concentration is generally given in mg/L, which is numerically equivalent to parts per million (ppm). The *load* is either the mass M in a given volume V (that is

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$M = C\,V$), or the mass per unit of time ($L = C\,Q$, where Q is the discharge, m^3/s). The plot of the concentration C as a function of time is the *pollutograph,* while the plot of the mass per unit of time or load, L, is the *loadograph.*

The event mean concentration, *EMC,* is the ratio of the total mass of pollutant, M, to the total runoff volume, V,:

$$EMC = \overline{C} = \frac{M}{V} = \frac{\int C(t)\,Q(t)\,dt}{\int Q(t)\,dt} \tag{2.2}$$

where $C(t)$ is the concentration and $Q(t)$ is the flow rate, both of which vary in time. The event mean concentrations vary from storm to storm (at a site) and from site to site. The *site median EMC* is the median or the 50th percentile of the EMCs at a given site. The variability across the sites is expressed by the *coefficient of variation, CV,* (standard deviation /mean) when the site EMCs of several locations are pooled.

The principal sources of pollutants accumulated in urban areas are wet and dry atmospheric deposition, erosion, littering and traffic emissions. Urban rainfall is generally acid with pH values below 5. This acidity damages pavements, sewers and buildings. Rain contains nitrates, ammonia, sulfates and sulfites, phosphorus, lead, mercury etc. Dry atmospheric deposition includes fine particles carried by wind over some distance and particles from local sources such as traffic, construction, and industrial activities. Dust originating from urban litter and from soils is also present in urban air. Traffic generates dust as road surfaces are abraded by vehicles, traffic resuspends dust and shifts it to the curb or the median, and contributes pollutants such as oil, grease, lead, zinc and copper. Most of the street refuse deposits within one metre of the curb. For this reason it is often expressed as mass per unit of curb length. The fraction of street refuse passing a 3.175 mm (1/8 inch) sieve is referred to as "dust and dirt". Many of the particles are picked up and transported by the runoff on impervious surfaces. Additional information on pollutant deposition and washoff can be found in Novotny and Chesters (1981).

The largest and most comprehensive investigation of urban storm runoff quality was the Nationwide Urban Runoff Program (NURP) undertaken by the US Environmental Protection Agency (EPA) (1983) and the US Geological Survey (USGS, Mustard *et al.* 1987). It included 2300 storm events at 81 sites in 22 different cities geographically distributed across the United States. The principal conclusions quoted from the executive summary are:

1. Heavy metals (especially copper, lead and zinc) are by far the most prevalent priority pollutant constituents found in urban runoff.
2. The organic priority pollutants are detected less frequently and at lower concentrations than heavy metals.
3. Coliform bacteria are present at high levels in urban runoff and can be expected to exceed EPA water quality criteria during and immediately after storm events in many surface waters, even those providing a high degree of dilution.
4. Nutrients are generally present in urban runoff.

5. Oxygen demanding substances are present in urban runoff at concentrations approximating those in secondary treatment plant discharges.
6. Total suspended solids concentrations in urban runoff are fairly high in comparison to treatment plant discharges.

For the Nationwide Urban Runoff Program EPA adopted the following constituents as standard pollutants characterising urban runoff:

TSS	Total suspended solids
BOD	Biochemical oxygen demand
COD	Chemical oxygen demand
TP	Total phosphorus (as P)
SP	Soluble phosphorus (as P)
TKN	Total Kjeldahl nitrogen (as N)
NO_{2+3}	Nitrite and nitrate (as N)
Cu	Total copper
Pb	Total lead
Zn	Total zinc

These standard pollutants can be considered as representative of others. These pollutants are examined both in point and nonpoint source studies and represent solids, oxygen consuming constituents, nutrients and heavy metals. Most of the data in the EPA report consist of flow weighted average concentrations, that is, the event mean concentration of each pollutant for each runoff event.

The median and the coefficient of variation of the event mean concentrations (EMC) of several constituents found in urban runoff are given in Table 1. These data can be used for planning purposes, for example for assessing water quality impacts in rivers and streams. They were obtained from the pooled site data for all urban sites, as the geographic location, the land use category and other factors did not contribute significantly to the explanation of the site-to-site variability or to the prediction of the characteristics of unmonitored sites.

Driscoll (1986) showed that for most sites and most pollutants, the EMC's are *lognormally* distributed and are weakly correlated with runoff volumes. For lognormally distributed data the following relationship holds

$$Mean = Median\ (1 + CV^2)^{1/2} \qquad (2.3)$$

where CV is the coefficient of variation (standard deviation / mean). Hall *et al.* (1990) have shown that the distribution of EMC's of pollutants in stormwater runoff is equally well described by a mixture of two normal distributions.

TABLE.1. Water Quality Characteristics of Urban Runoff

Constituent	Event to Event Variability in EMCs (Coef. Var.)	Site Median EMC For Median Urban Site	For 90th Percentile Urban Site
TSS (mg/l)	1-2	100	300
BOD (mg/l)	0.5-1.0	9	15
COD (mg/l)	0.5-1.0	65	140
Tot. P (mg/l)	0.5-1.0	0.33	0.70
Sol. P (mg/l)	0.5-1.0	0.12	0.21
TKN (mg/l)	0.5-1.0	1.50	3.30
NO_{2+3}-N (mg/l)	0.5-1.0	0.68	1.75
Tot. Cu (μg/l)	0.5-1.0	34	93
Tot. Pb (μg/l)	0.5-1.0	144	350
Tot. Zn (μg/l)	0.5-1.0	160	500

Source: US EPA, 1983, Vol. 1, Table 6.17, p. 6-43.

The EMC mean concentrations listed in Table 2 were obtained from the medians and coefficients of variation listed in Table 1, using eq. (2.3). The range of the site mean concentrations and of the 90th percentile urban sites reflects the use of the higher or the lower values of the coefficient of variation listed in Table 1. The range of values used in load comparison reflects the median and the 90th percentile site mean concentrations, using the average of the range caused by coefficient of variation effects. The mean concentrations are more appropriate than median values when analysing cumulative effects such as water quality impacts on lakes or when making comparisons of annual or seasonal loads.

The mean annual load can be estimated for urban runoff constituents by choosing the appropriate rainfall and runoff coefficient values and selecting the EMC value from Table 2. Estimates of annual pollutant loads in kg/ha/yr for different types of urban developments are listed in Table 3, for a 100 cm (40 inch) annual rainfall and the runoff coefficients, Rv, shown in Table 3.

TABLE 2. EMC Mean Values Used in Load Comparison

Constituent	Median Urban Site	Site Median EMC 90th Percentile Urban Site	Site Median EMC Values Used in Load Comparison
TSS (mg/l)	141 – 224	424 - 671	180 – 548
BOD (mg/l)	10 – 13	17 - 21	12 – 19
COD (mg/l)	73 – 92	157 - 198	82 – 178
Tot. P (mg/l)	0.37 - 0.47	0.78 - 0.99	0.42 - 0.88
Sol. P (mg/l)	0.13 - 0.17	0.23 - 0.30	0.15 - 0.28
TKN (mg/l)	1.68 - 2.12	3.69 - 4.67	1.90 - 4.18
NO_{2+3}-N (mg/l)	0.76 - 0.96	1.96 - 2.47	0.86 - 2.21
Tot. Cu (μg/l)	38 – 48	104 - 132	43 - 118
Tot. Pb (μg/l)	161 – 204	391 - 495	182 - 443
Tot. Zn (μg/l)	179 – 226	559 - 707	202 - 633

Source: US EPA 1983, Vol. 1, Table 6.24, p. 6-60.

In addition to other pollutants, high fecal bacteria counts are found in urban runoff. In residential and light commercial areas, such the bacteria originate predominantly from non-human sources (Queresshi and Dutka, 1979). The NURP project performed coliform bacteria counts in 156 separate storm events at 17 sites. A summary of these results is shown in Table 4. The seasonal effect is seen to be very important. Coliform counts in urban runoff were about 20 times greater during the warmer periods that during the colder ones. According to the EPA report "the substantial seasonal differences do not correspond to comparable variations in urban activities. This suggests that seasonal temperature effects and sources of coliform unrelated to those traditionally associated with human health risk may be significant". The data were too limited to identify any land use distinctions.

Makepeace *et al.* (1995) presented an extensive review of the specific chemical, physical and biological parameters describing the quality of urban runoff that are necessary to assess the impact of storm water on aquatic life in receiving water bodies and on downstream water users.

TABLE 3. Annual Urban Runoff Loads kg/ha/yr

Constituent	Site Mean Con. mg/l	Residential	Commercial	All Urban
Assumed Rv		0.3	0.8	0.35
TSS	180	550	1460	640
BOD	12	36	98	43
COD	82	250	666	292
Total P	0.42	1.3	3.4	1.5
sol. P	0.15	0.5	1.2	0.5
TKN	1.90	5.8	15.4	6.6
NO_{2+3}-N	0.86	2.6	7.0	3.6
Tot. Cu	0.043	0.13	0.35	0.15
Tot. Pb	0.182	0.55	1.48	0.65
Tot. Zn	0.202	0.62	1.64	0.72

Note: Assumes 100 cm/yr rainfall as a long-term average.
Source: US EPA, 1983, Vol. 1, Table 6-25, p. 6-64.

TABLE 4. Fecal Coliform Concentrations in Urban Runoff

Warm Weather 11 sites			Cold Weather 9 sites		
No. of Events	Median EMC 1000/100ml	C.V.	No. of Events	Median EMC 1000/100ml	C.V.
76	21	0.8	52	1	0.7

Source: US EPA, 1983, Vol. 1, Table 6-18, p.6-45.

2.2. MECHANISMS OF CONSTITUENT TRANSPORT

The two principal modes of constituent transport are advection and diffusion or dispersion. *Advection* is the transport by the motion of the fluid. The center of gravity of the constituent moves with the mean velocity of the flow. Advection does not cause any spreading of the constituent. The flux due to advection is

$$F = U C \tag{2.4}$$

where F is the *flux* (g/m^{2}s), U is the mean flow velocity and C is the concentration. *Diffusion* causes the constituent to spread in the direction of decreasing concentration. It is expressed by Fick's law, which in one dimension is

$$F = -E\frac{\partial C}{\partial x} \tag{2.5}$$

where E is the diffusion coefficient (cm^2/s). For liquids at rest or in laminar flow, E is the *molecular diffusion coefficient.* For turbulent flows, E is the *turbulent diffusivity.* In surface runoff, the flow is generally turbulent, but laminar flow can exist, for example, in thin sheet flow after the cessation of rainfall. In addition there is the *longitudinal dispersion* (also called *shear flow dispersion)* caused by the variation of velocity across a section. The longitudinal dispersion obeys Equation (2.5) in which E is replaced by E_L, the *coefficient of longitudinal dispersion.* This term also includes all other processes that increase mixing, such as secondary currents. For shear flow dispersion, U is the mean velocity and C is the average concentration in the flow section. The molecular diffusion coefficient is of the order of 1×10^{-5} to 2×10^{-5} cm^2/s, turbulent diffusivity is of the order of 1 to 10^4 cm^2/s and the coefficient of longitudinal dispersion is 10^2 to 10^3 times greater. Values of the molecular diffusion coefficient can de found for several constituents in the Handbook of Chemistry and Physics (Lide, 1996) and information concerning the dispersion coefficients can be found in Fisher *et al.* (1979) and in Schnoor *et al.* (1987).

The conservation of mass of a constituent is obtained by equating the change of mass in an elementary volume to the net sum of the advection and diffusion through the sides of the volume plus sources and sinks. Assuming a first order decay, the one-dimensional advection-dispersion equation is

$$\frac{\partial C}{\partial t} + \frac{\partial (UC)}{\partial x} = \frac{\partial}{A \partial x}\left(A E_L \frac{\partial C}{\partial x} \right) - KC \tag{2.6}$$

where U is the average longitudinal velocity, A is the flow cross section and E_L is the longitudinal dispersion coefficient. For *complete mixing* there is no concentration gradient and equation (2.6), for a continuously stirred reactor, becomes

$$\frac{dC}{dt} = -KC \tag{2.7}$$

The solution of this equation is

$$C(t) = C_0 e^{-Kt} \tag{2.8}$$

where C_0 is the well mixed concentration at $t = 0$.

In addition to the physical transport by advection and diffusion/dispersion there may be *kinetic processes* that involve production or decay of substances. Reaeration is an example of production or gain of dissolved oxygen, while oxidation of carbonaceous substances and bacterial decay are examples of loss of constituent. A *first order decay* is represented by the equation

$$\frac{dC}{dt} = -KC \tag{2.9}$$

where K is the first order rate coefficient (s^{-1}). If dC / dt is proportional to C^2 The process is second order. When the concentration rate has a limit, the *Monod* or *Michaelis-Menton* formulation can be used:

$$\frac{dC}{dt} = -\frac{k_s C}{k_{1/2} + C} \tag{2.10}$$

where k_s is the limiting reaction rate and $k_{1/2}$ is the half saturation constant ($dC / dt = \frac{1}{2} k_s$ when $C = k_{1/2}$). This equation is often applied to the kinetics of nitrogen and phosphorus in eutrophication studies. It is also applied to the buildup of dust and dirt and of pollutants on urban watersheds (see section 3.2 and Fig. 4).

One of the important sinks of *dissolved oxygen* is the *biochemical oxygen demand (BOD)* which measures the oxygen required by bacteria in oxidation of organic matter. The oxidation of BOD is generally represented by a first order process

$$\frac{dL}{dt} = -K_1 L \tag{2.11}$$

where L is the BOD concentration (mg/l) and K_1 is the deoxygenation coefficient (d^{-1}). The amount of oxygen consumed by organic matter in 5 days is reported as BOD_5. The BOD_5 and the ultimate carbonaceous BOD, L_0, are related by

$$L_0 = \frac{BOD_5}{1 - e^{-5K_1}} \tag{2.12}$$

For a continuous point source discharge in a stream with complete mixing, the BOD concentration, L_D, at the point of discharge is given by the *Streeter-Phelps* equation

$$L_D = \frac{Q_u L_u + Q_s L_s}{Q_u + Q_s} \tag{2.13}$$

where Q_u and Q_s are the upstream and the source discharges, respectively, L_u is the upstream ultimate BOD concentration, and L_s is the source strength.

Like any other constituent, BOD is subject to advection and diffusion/dispersion. Usually dispersion can be neglected compared to advection and the steady state BOD distribution downstream is

$$L(x) = L_0 e^{-K_1 t} \tag{2.14}$$

where L_0 is the BOD concentration at $x = 0$ and $t = x/U$ is the travel time.

For further details on contaminant transport in surface waters see, for example, Huber (1993) and references therein.

3. Storm Water Management Model (SWMM)

The Stormwater Management Model (SWMM) was developed in the U.S.A. by the Environmental Protection Agency (Metcalf and Eddy *et al.*, 1971) and has been continuously improved. The current version (4.31) was issued in 1995. A brief description of SWMM can be found in Huber (1995) and a more detailed one in Nix (1994). The most complete account is in the user's manual (Huber and Dickinson, 1988). James (1993) has given an introduction to the SWMM environment that includes a classified list of 2222 papers that appeared in the SWMM user group conference proceedings from 1976 to 1992. A guide to the 485 papers presented at the Kentucky Symposia between 1975 and 1985 was also published (James, 1994).

3.1. OVERVIEW

The Storm Water Management Model (SWMM) simulates the mechanisms of runoff quantity and quality, the transport of the runoff and of constituents through a separate or combined sewer system, and the storage/treatment followed by the discharge or overflow into the receiving waters. A simplified schematic of an urban watershed for SWMM analysis is shown in **Figure 1**. The computer program consists of four computational blocks and five service blocks connected by an executive block as shown in **Figure 2.**

The RUNOFF block simulates the processes involved in the generation of the surface runoff and pollutant loads. The watershed is subdivided in a number of homogeneous subwatersheds. One of these subcatchments is shown in **Figure 1.** Each subwatershed is simulated as a nonlinear reservoir with a capacity equal to the maximum depression storage provided by interception, surface wetting and ponding **(Figure 3)**. The rainfall is the inflow and the surface runoff, infiltration and evaporation are the outflows. The infiltration can be simulated using the Horton equation or the Green-Ampt equation. The flow rate, Q, is estimated by Manning's equation

$$Q = W\,(\kappa/n)\,(d - d_p)^{5/3}\,S^{1/2} \tag{3.1}$$

where W is the subwatershed width, n is the roughness coefficient, d - d_p is the depth in excess of the depression storage and κ is unity for metric units or 1.486 for U.S. customary units.

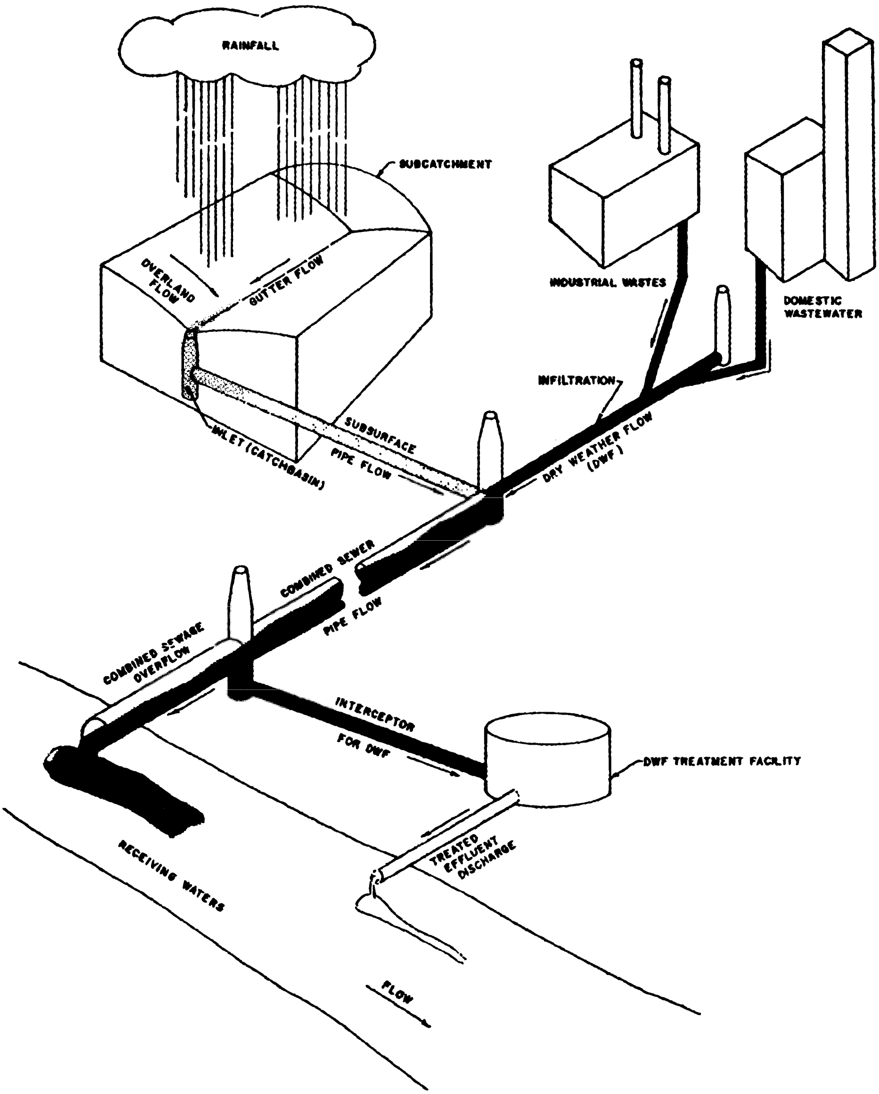

Figure 1. Simplified Schematic of Urban Watershed for SWMM Analysis.
Source: Metcalf & Eddy, 1971, EPA 11024DOC07/71 P.25.

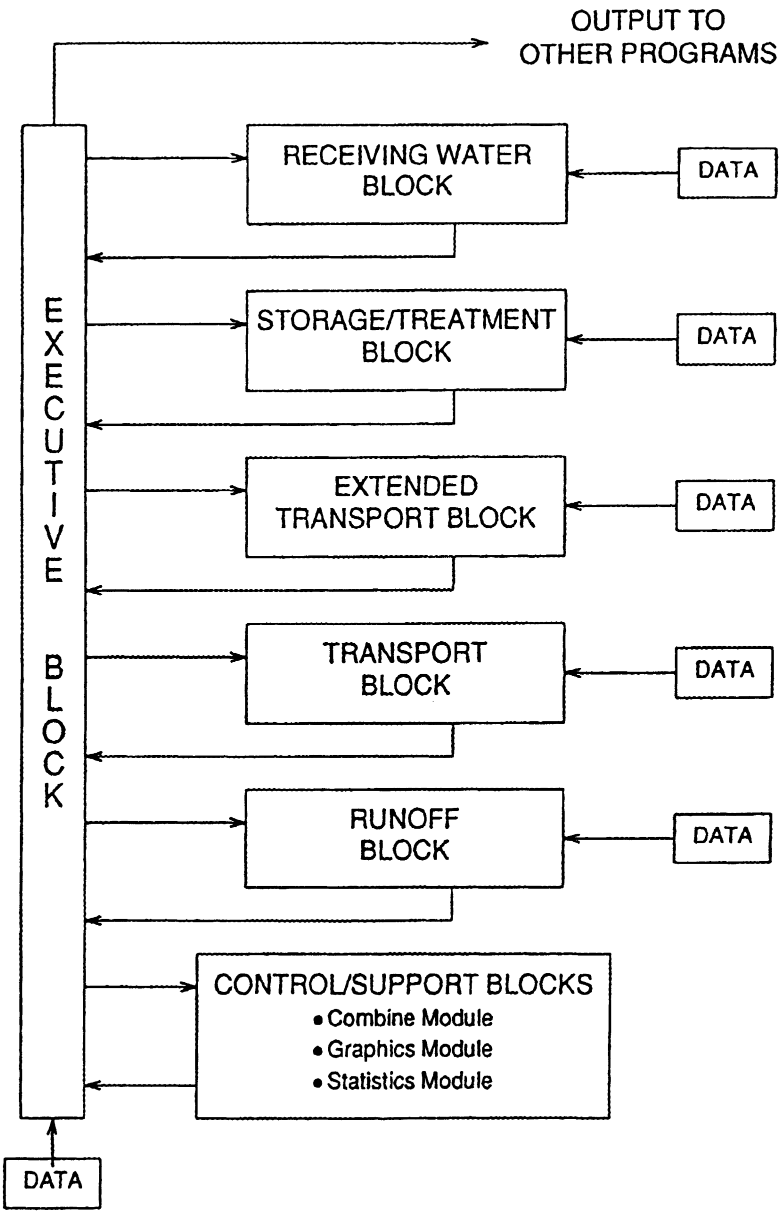

Figure 2. SWMM Structure.
Source: Baffaut *et al.* 1987, Fig. 3.1 p. III-2.

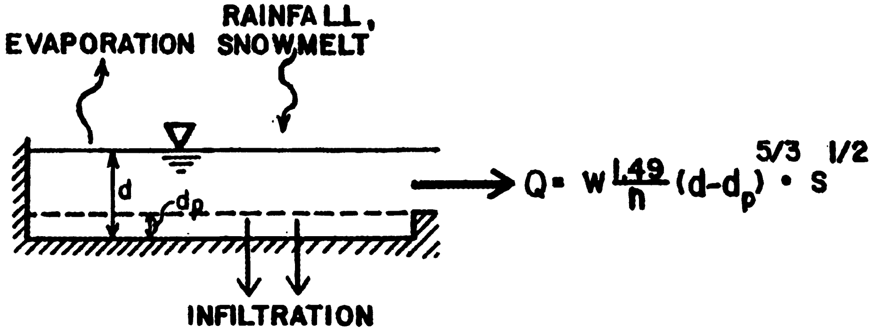

Figure 3. Nonlinear Reservoir Representation of Runoff Block Subcatchment.
Source: Huber and Dickinson, 1988, Fig. 4-12 p. 97.

The continuity equation for a subarea is

$$\frac{dV}{dt} = A\frac{dd}{dt} = A\,i*-Q \tag{3.2}$$

where $V=A\ d$ is the volume of water on the subarea, d is the depth of water, A is the surface area of the subarea, $i*$ is the rainfall excess (rainfall/snowmelt intensity - evaporation/infiltration rate) and Q is the outflow. Combining the continuity equation (3.2) and Manning's equation (3.1) yields the nonlinear reservoir equation (3.3)

$$\frac{dd}{dt} = i* - \frac{\kappa W}{An}(d-d_p)^{5/3}S^{1/2} \tag{3.3}$$

A finite difference approximation of the reservoir equation is

$$\frac{d_2-d_1}{\Delta t} = \bar{i}* - \frac{\kappa W}{An}(\bar{d}-d_p)^{5/3}S^{1/2} \tag{3.4}$$

where $\bar{d}$ is the average depth $(d_2+d_1)/2$ during the time step Δt, and $\bar{i}*$, likewise, is the average rainfall excess. This equation is solved numerically for d_2 at each time step using a Newton-Raphson technique. It should be noted that the width W of the overland flow affects the runoff velocity. As W increases, the length of the reservoir decreases for a constant volume of water. As a result a large W results in quick rise of the runoff hydrograph, a short W yields a slower rise of the hydrograph. The parameter W can be used as a calibration measure to adjust the shape of the hydrograph. For further details see Huber and Dickinson (1988).

3.2. POLLUTANT BUILDUP

Buildup occurs over the number of dry days prior to the beginning of simulation (DRYDAY) and also during continuous simulation when runoff is very small (< 0.013 mm/hr or 0.0005 in/hr). The buildup formulation is based on a study conducted by the American Public Works Association in Chicago in 1969 and subsequent studies. Two sets of formulation are used: one for dust and dirt, and the other for pollutants. *Dust and Dirt* was defined as any materials passing through a 6.35 mm mesh screen (0.25 inch sieve). Dust and dirt can be expressed in kg or lbs., or in terms of mass per gutter length (kg/km of curb or lb/100ft of curb) or in terms of mass per subcatchment area (kg/ha or lb/ac).

SWMM includes three empirical build-up formulations:
For *Dust and Dirt,* with *DD* = Dust and Dirt in kg or lb., *t* = time in days:

Power -- linear

$$DD = DDFACT.t^{DDPOW} \qquad (3.5)$$
$$DD \leq DDLIM$$

Exponential

$$DD = DDLIM.(1 - e^{-t.DDPOW}) \qquad (3.6)$$

Michaelis- Menton

$$DD = \frac{DDLIM.t}{DDFACT + t} \qquad (3.7)$$

The parameters *DDLIM, DDPOW, DDFACT* are single subscripted by land use.

For the *constituents* the build-up equations similarly are, with P_{SHED} = amount of pollutant on the sub-watershed at time *t* (load/ha or load/ac) and *QFACT(i,K)* = ith build-up parameter for *K* th pollutant:

Power-linear

$$P_{SHED} = QFACT(3).t^{QFACT(2)} \qquad (3.8)$$
$$P_{SHED} \leq QFACT(1)$$

Exponential

$$P_{SHED} = QFACT(1).(1 - e^{-t.QFACT(2)}) \qquad (3.9)$$

Michaelis-Menton

$$P_{SHED} = \frac{QFACT(1).t}{QFACT(3) + t} \qquad (3.10)$$

The equation that best fits the data at the site should be selected. However, the differences between runoff quality simulated by the several equations are usually less than the uncertainty in the model and in the runoff quality measurements.

Figure 4 shows an example of several time distributions of the accumulations of dust and dirt. For any constituent, the build-up can be expressed as a fraction of dust and dirt or individually for that specific constituent. The parameter *QFACT* is double subscripted (second subscript is constituent number). The *exponential* and the *Michaelis-Menton* functions have definite asymptotes and an upper limit can be imposed for the linear and power functions. In the case of linear build-up (*DDPOW* = *1),* making $DDFACT \geq DDLIM\ /\ DELT$ results in a fixed amount, *DDLIM,* of dust and dirt at the beginning of a storm event. For the *exponential* build-up, the exponent (*DDPOW* or *QFACT(2,K)*) can be obtained from the slope of the line of build-up vs. time on a semilog plot. For the *Michaelis-Menton* build-up, the parameter *DDFACT* or *QFACT(3,K)* is the time at which the build-up is half the asymptotic value.

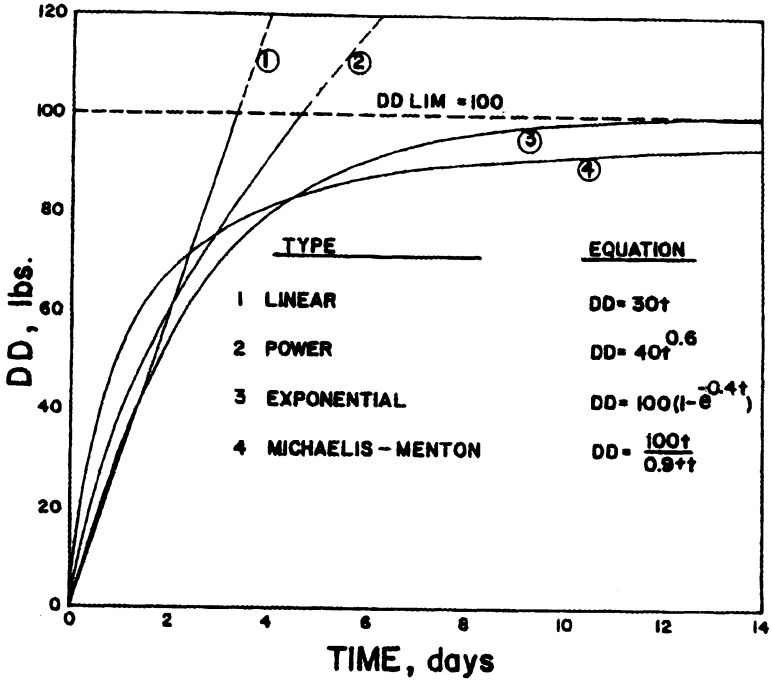

Figure 4. Comparison of Buildup Equations for Dust and Dirt.
Source: Huber and Dickinson, 1988, Fig. 4-29, p. 150.

3.3. POLLUTANT WASHOFF

Washoff is defined as the process of erosion or dissolution of constituents from a subcatchment during a runoff event. Two formulations are used. The first assumes a first order relationship for the washoff rate, namely, the rate at which a pollutant is washed off is proportional to the amount of pollutant remaining on the subwatershed P_{SHED}:

$$P_{OFF} = -\frac{dP_{SHED}}{dt} = k.P_{SHED} \tag{3.11}$$

where P_{OFF} = rate of pollutant washoff (mg/s, MPN/s for coliform bacteria, JTU/s for turbidity) and k is a coefficient (s^{-1}) which is assumed to be proportional to the runoff rate:

$$k = R_{COEFF}.r \tag{3.12}$$

where R_{COEFF} = washoff coefficient (mm^{-1} or in^{-1}) and $r = Q/A$ = runoff rate at time t (mm/s or in/s). Thus,

$$P_{OFF} = -\frac{dP_{SHED}}{dt} = R_{COEFF}.r.P_{SHED} \tag{3.13}$$

The solution of (3.11) is

$$P_{SHED}(t) = P_{SHED}(0).e^{-kt} \tag{3.14}$$

where $P_{SHED}(0)$ is the initial amount of pollutant. Since $P_{SHED}(t) = P_{SHED}(0) - P_{OFF}$, this solution is equivalent to

$$P_{OFF}(t) = P_{SHED}(0).[1 - e^{-kt}] \tag{3.15}$$

This exponential relationship appeared to fit the experimental data of Sartor and Boyd (1972). However, equation (3.11) has the inconvenience of producing runoff pollutant concentrations that decrease with time as

$$C = \frac{P_{OFF}}{Q} = const.\frac{R_{COEF}.r.P_{SHED}}{A.r} = const\frac{R_{COEF}.P_{SHED}}{A} \tag{3.16}$$

where C = concentration, Q = flow rate and A = subcatchment area and the constant includes conversion factors. From (3.16) it is seen that the concentration keeps decreasing as P_{SHED} decreases as a function of time. Although a decreasing concentration is possible, it also could be that the concentrations near the middle of the storm are higher than those preceding, due to a higher rate of runoff. To remedy this problem, the washoff rate can be made proportional to a power *WASHPO* of the runoff rate:

$$P_{OFF} = -\frac{dP_{SHED}}{dt} = R_{COEFX}.r^{WASHPO}.P_{SHED} \tag{3.17}$$

where R_{COEFX} = $R_{COEF}/360$ = washoff coefficient. The concentration now is proportional to $r^{WASHPO-1}$ as

$$C(t) = const \frac{R_{COEFX} . r(t)^{WASPO-1} . P_{SHED}(t)}{A} \tag{3.18}$$

A finite difference approximation of the solution of (3.17) is

$$P_{SHED}(t + \Delta t) = P_{SHED}(t) . \exp\{-R_{COEFX} \frac{1}{2}[r(t)^{WASPO} + r(t + \Delta t)^{WASPO}] . \Delta t\} \tag{3.19}$$

There are thus two coefficients to be determined: R_{COEFX} and *WASHPO*. Equation (3.19) is used to calculate the remaining load on the subcatchment at the end of each time step. Then equation (3.17) is used to calculate the washoff load.

An alternative formulation consists in simulating the washoff as a function of the runoff rate as

$$P_{OFF} = R_{COEF} \cdot Q^{WASHPO} \tag{3.20}$$

in which the coefficients R_{COEF} and *WASHPO* are specified for each pollutant.

3.4. SEDIMENT LOAD and RATING CURVES

The following sediment rating curve is often used to describe the sediment load rate:

$$G = a.Q^b \tag{3.21}$$

where G is the sediment load rate (mg/s), Q is the flow rate (cms or cfs)and a and b are coefficients. A rating curve can also be used instead of the washoff calculation.

Instead of employing the preceding washoff equations, the following rating curve can be used:

$$P_{OFF} = R_{COEF} \cdot W_{FLOW}{}^{WASHPO} \tag{3.22}$$

where W_{FLOW} is the subcatchment runoff (cms or cfs). The coefficient R_{COEF} and the exponent *WASHPO* can be determined on a storm event basis by plotting P_{OFF} versus W_{FLOW}. Typically the exponent *WASHPO* takes values between 1.5 and 3.0 for particulate and less than 1.0 for dissolved constituents.

Constituents transported in the solid state can be estimated by the erosion equation using a *potency factor* which is the ratio of the concentration (or mass load rate) of the constituent to that of the solids.

3.5. STREET CLEANING and CATCH BASINS

After street cleaning, the fraction of the load (constituent or dust and dirt) remaining on the surface, *REMAIN*, is given by

$$REMAIN = 1.0 - AVSWP(J) . REFF(K) \tag{3.23}$$

where $AVSWP(J)$ is the availability factor for land use J, which accounts for the fraction of the catchment area that is swept, and $REFF(K)$ is the removal efficiency for constituent K. The user's manual gives a table of removal efficiencies.

The constituent load in a catch basin, $P_{BASIN,}$ (mg) at the beginning of a storm is calculated as

$$P_{BASIN} = CB_{VOL} \cdot \quad BASINS. \quad CB_{FACT} \cdot \quad FACT3 \tag{3.24}$$

where CB_{VOL} is the volume of the sump not occupied by sediments (m^3 or ft^3), $BASINS$ is the number of basins in a subcatchment, CB_{FACT} is the constituent concentration at the beginning of the storm (mg/l) and $FACT3$ is a conversion factor (28.3 l/ft^3 if dimensions in ft^3). Assuming a completely mixed tank, the rate of flushing is

$$\frac{d\,P_{BASIN}}{dt} = -\frac{W_{FLOW}}{k.BASIN} P_{BASINS} \tag{3.25}$$

where W_{FLOW} is the flow through the catchbasin, e.g., the runoff from the subwatershed, $BASIN$ is the total volume of the catch basins for the catchment and $k = 1$ for complete mixing, otherwise k is a constant to be determined experimentally. A value of $k = 1.3$ is normally used. An approximate integration yields

$$P_{BASIN}(t + \Delta T) \approx P_{BASIN}(t) + \frac{dP_{BASIN}}{dt} \Delta t \tag{3.26}$$

(Note that dP_{BASIN}/dt is negative.)

For a constant flow rate, the constituent load remaining in the catch basin at time t after the runoff began is

$$P_{BASIN} = P_{BASIN0} \exp\{-\frac{W_{FLOW}}{kBASIN} t\} \tag{3.27}$$

where $P_{BASIN\,0}$ is the initial catch basin load. Catch basins contribute a first flush, but, in terms of the total storm load, their effect is negligible.

3.6. PRECIPITATION

Rainfall may contain organics, solids, nutrients, metals and pesticides. The various nitrogen forms probably are important contributors to urban runoff pollution. All runoff is assumed to have at least the concentrations found in the rainfall or snowmelt. The precipitation load is calculated by multiplying the rainfall concentration by the runoff rate. This is then added to the loads generated by other mechanisms.

3.7. EROSION

SWMM uses the Universal Soil Loss Equation (USLE) which estimates the average soil loss per unit area, L, (in metric tonnes per ha per year or U.S. tons per acre per year.) as

$$L = R \cdot K \cdot LS \cdot C \cdot P \tag{3.28}$$

where $R =$ the rainfall energy factor; $K =$ the soil erodibility factor; $LS =$ the slope-length factor; $C =$ the vegetative cover factor; $P =$ the erosion control practice factor

The above factors can be estimated by empirical formulas and nomograms that can be found in the SWMM user's manual (Huber and Dickinson, 1988). The USLE was originally developed for the estimation of average annual erosion from agricultural watersheds. A revised version (RUSLE) has been developed by the Agricultural Research Service. In this revised version, the empirical equations and nomograms are replaced by algorithms that can be executed on a personal computer. However, it has not yet been incorporated in the current version of SWMM.

3.8. DATA INPUT

Referring to the arrangement of input data, after the titles contained in the A Line, the control data are listed in lines B1 through B3, the C lines hold the data concerning the snow input, the E lines contain the rainfall data, and the F lines are concerned with the evaporation data. The channel and control structure data are listed in G lines, H lines provide the subcatchment data for the quantity simulation, including groundwater flow. The subcatchment snow data are listed on lines I.

The quality data are input in four lines labelled J1 to J4. Line J1 identifies the number of constituents to be modelled and the land use types, line J2 sets up the dust and dirt characteristics, line J3 sets up the constituent buildup equations and line J4 allows a fraction of one constituent to be added to another. The K line provides the erosion data and the L lines provide subcatchment surface quality data. The remaining M lines are concerned with print controls.

Details on the input parameters can be found in Huber and Dickinson (1988) and in Nix (1994).

4. MOUSE

The MOUSE package was developed by the Danish Hydraulic Institute. (Lindberg *et al.*, 1986, 1989). MOUSE includes the runoff simulation from pervious and impervious areas as well as a pipe flow model that solves the full St. Venant equations for looped sewer networks. A companion module is MOUSETRAP that simulates the transport of sediments, pollutants attached to the sediment and dissolved pollutants. MOUSETRAP itself is subdivided into four modules: Surface Runoff Quality (SRQ), Sediment Transport (ST), Advection - Dispersion (AD) and Water Quality module (WQ). The transport processes on the catchment are described in SRQ. Further discussion of the other three modules can be found in the paper on "Water Quality Modelling in Sewer Systems" later in this chapter.

The sediment transport module consists of two submodels: one for the accumulation of particles and another for the washoff by the overland flow (Mark *et al.* 1993). The accumulation of particles on the catchment can be formulated as a linear or exponential function of time. The exponential buildup is expressed as

$$\frac{dM}{dt} = A_c - D_{rem}\, M \tag{4.1}$$

where M is the accumulated mass of particles at time t (kg), t is the time in days, A_c is the daily accumulation rate (kg/d), and D_{rem} is the removal coefficient (d^{-1}). This removal includes the effects of wind, traffic, street sweeping, and biological and chemical degradation. The accumulated mass increases until a specified limit is reached.

The washoff of sediment is produced by two processes: first the detachment of particles caused by the rain drops and second the erosion by the overland flow. The detachment by rainfall, V_{sr}, is expressed as

$$V_{sr} = D_r \left(\frac{i_r}{i_d} \right)^2 LW(1-\varepsilon)\, A_s \tag{4.2}$$

where D_r is the detachment coefficient for rainfall, i_r is the rainfall intensity (mm/hr), i_d is the rainfall intensity constant (mm/h), L is the catchment length (m), W is the catchment width (m), ε is the perviousness of the catchment and A_s is the fraction of the area covered by sediment.

The total washoff transport is limited by the transport capacity of the surface runoff. This capacity is calculated as the sum of the suspended load and bed load capacities. The suspended load capacity is calculated by the Einstein (1950) equation while the bed load capacity is estimated by the Meyer-Peter formula, (See, for example, Shen and Julien, 1992 or Meyer, 1971).

5. Alternate Accumulation Models

The Particle Transport Model (PTM) uses an exponential accumulation rate. A number of improved models for atmospheric dustfall and for buildup of dust and dirt have been introduced. The atmospheric dustfall process ATMDST (Shivalingaiah and James, 1986) and buildup model NEWBLD (James and Shivalingaiah, 1985) have been developed in the context of the SWMM framework.

5.1. PARTICLE TRANSPORT MODEL (PTM)

The PTM (Alley, Ellis and Sutherland, 1980) formulates the particle accumulation as

$$\frac{dL}{dt} = K - K_2 L \tag{5.1}$$

where L is the particle load (lb./acre), K is a constant rate of deposition (lb./acre -- day), K_2 is a constant rate of particle removal (day^{-1}) and t is the time in days. This equation integrates as

$$L = K_1[1 - \exp(-K_2 T)] \tag{5.2}$$

where $K_1 = K/K_2$ is the maximum load (lb./acre) and T is the accumulation time (days). To account for the load remaining after the previous washoff or sweeping, set $T = t + t_0$ where t_0 is the time required to accumulate a load equal to the load L_0 since the previous washoff or sweeping. The time t_0 is calculated as

$$t_0 = \frac{-1}{K_2}\ln(1 - \frac{L_0}{K_1}) \tag{5.3}$$

5.2. ATMOSPHERIC DEPOSITION MODEL (ATMDST)

The model ATMDST uses regression equations to predict mean monthly dustfall on individual subcatchments using prevailing meteorological conditions. The equation yielding the best correlation for the Hamilton, Ontario data is:

$$Y_{im} = a_i + b_i V_{md} + c_i P_m \tag{5.4}$$

where Y, V, and P are the predicted dustfall, the average wind velocity and the total precipitation, respectively, and the subscripts i, m and d correspond to the dust collection station, the month and the wind direction.

5.3. DUST AND DIRT BUILDUP (NEWBLD)

The model NEWBLD estimates the dust and dirt build-up based on the atmospheric dustfall, wind effects, vehicles, intentional removals (such as street sweeping), special activities (such as construction and demolition), biological decomposition and population related actives (such as lawn cutting). The effective hourly atmospheric dustfall, L_a, is calculated from (4). The *vehicle input*, L_v,(mg/d) is estimated as

$$L_v = N L_{dv} M \tag{5.5}$$

where N is the number of vehicle-axles per day, L_{dv} is the mass of dust and dirt produced per vehicle per km travelled (800-1200 mg/axle-km) and M is the total length of road in the subcatchment (km). The *population input*, L_p, is calculated as

$$L_p = P_s L_l \tag{5.6}$$

where P_s is the total population in the catchment and L_l is the mass of pollutant per capita per day 100 - 1000 (mg/cap - d). The *vegetation input*, L_c, due to leaves and grass clipping is estimated as

$$L_c = V A \tag{5.7}$$

where V is the vegetation load rate in DD (300 000 - 6 000 000 mg/ha-d) and A is the subcatchment area (ha). The special activities, L_s, include construction and demolition, and are given by

$$L_s = S_L A \tag{5.8}$$

where S_L is the daily load of DD produced by special activities, mg/ha-d, and A is the area of special activities. The *biological removal*, R_b , is formulated as

$$R_b = P_{DD} F(1 - e^{-Kt}) \tag{5.9}$$

where P_{DD} is the mass of DD available at the surface, mg/d, F is the fraction of decomposable DD (0.05 - 0.15), K is a decay rate coefficient (0.05 - 0.1 d^{-1}) and t is the number of days. The *vehicle removal*, R_v is

$$R_v = N F P_{DD} \tag{5.10}$$

where N is the number of vehicles in a subcatchment, P_{DD} is the pollutant available for eddy transport, mg/d-vehicle and F is calculated as

$$F = \theta_1 (1 - e^{-\theta_2 s}) \tag{5.11}$$

where θ_l is the maximum fraction of DD that could be removed (0.00001 - 0.0001), θ_2 is a decay coefficient (0.0015 - 0.002 h/km) and s is the vehicle speed, km/h. The fraction of *pollutant lost due to wind*, R_w, is calculated from

$$R_w = 0.0116(e^{-0.088H})W \tag{5.12}$$

where H is the curb height, cm, and W is the wind speed, km/h. The *intentional removal*, R_i, is given by

$$R_i = \frac{L_a - R_{ma}}{L_a} \tag{5.13}$$

where R_{ma} is the DD remaining on the surface, (mg), L_a is the DD present before sweeping, (mg), R_i is the fraction removed by sweeping. The *daily flux* of DD accumulated on an impervious area, (mg/d), is thus

$$P_a = L_a + L_v + L_p + L_c + L_s - R_b - R_v - R_w - R_i \tag{5.14}$$

6. Alternate Washoff and Particle Transport Models

6.1. LIMITATIONS OF THE FIRST ORDER DECAY EQUATION

In the early versions of SWMM, the washoff was described by eq. (3.13) instead of eq. (3.17) which is equivalent to using *WASHPO* = *1*. In that case, with the notation X_T = pollution load washed off at the end of runoff duration *T*, P_0 = initial mass of pollutant, $R_{COEF} = b$, and V_T = volume of runoff for the event, the expression is (Ellis and Sutherland, 1979)

$$X_T = P_o(1 - e^{-bV_T}) \tag{6.1}$$

The fraction, Y_n, of the total washoff removed through the *n*th interval is

$$Y_n = \frac{X_n}{X_T} = \frac{(1 - e^{-bV_n})}{(1 - e^{bV_T})} \tag{6.2}$$

where X_n is the volume of pollutant washed off at the end of the *n*th time interval. With Z_n defined as the fraction of total runoff $Z_n = V_n / V_T$ and assuming $V_t = 1$ unit of depth, so that $Z_n = V_n$ then equation (6.2) becomes

$$Y_n = \frac{(1 - e^{-bZ_n})}{(1 - e^{-b})} \tag{6.3}$$

A plot of the fraction of the total pollutant load, Y_n, vs. the fraction of the total runoff volume, Z_n, shows that for $b>1$ an early first flush occurs, for $b = 1$ a straight line or uniform flush occurs, and for $b<1$ a late flush occurs, as shown in **Figure 5.** SWMM does not allow negative values of $b = R_{COEF}$. However Ellis and Sutherland (1979) show observations of early, late and uniform flush and flush rates changing through the storm event. As stated above, this has been corrected in the newer versions of SWMM by introducing the exponent *WASHPO.*

6.2. THE PARTICLE TRANSPORT MODEL (PTM)

The PTM considers first the sediment movement by size and then the pollutant transport using a potency by particle size. The sediment transport model was developed by Li (1974) and is summarised in Simons *et al.* (1977). For overland flow the total potential sediment transport rate, q_t, is

$$q_t = \sum_{i=1}^{N} (q_{si} + q_{bi}) i_s \tag{6.4}$$

where q_{si} is the suspended load calculated by Einstein's method (1950), q_{bi} is the bed load calculated by the Meyer-Peter, Muller formulas (USBR,1960), i_s the percentage of each sediment size of bed material and N is the number of sizes considered. The sediment yield is controlled by the supply or by the capacity. If the supply is greater than the capacity, then the capacity controls, otherwise the supply controls. The individual *capacity* for a particle size, V_{ti}, is

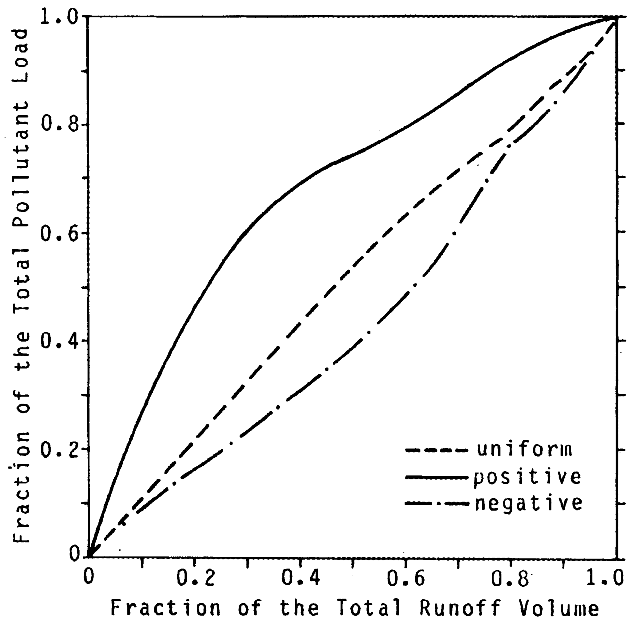

Figure 5. Some observed suspended solids loading characteristic curves.
Source: Ellis and Sutherland, 1979, reproduced with permission.

$$V_{ti} = \frac{q_{ti}\, i_s (D_t - t_p) W}{\gamma\, S_s} \tag{6.5}$$

where q_{ti} is the potential transport capacity rate of size i, D_t is the storm duration, t_p is the time to ponding since the beginning of effective rainfall, W is the width of the overland flow, γ is the specific weight of the water and S_S is the specific gravity of the sediments. The available *supply* for the *i*th particle size V_{ai} is

$$V_{ai} = i_s (V_r + V_f) \tag{6.6}$$

where V_r is the detachment due to rainfall splash and V_f is the detachment due to overland flow. The rainfall splash varies with the square of the rainfall intensity and is determined by an empirical equation (Meyer, 1971). The detachment due to overland flow is given by

$$V_f = D_f (V_t - V_r) \tag{6.7}$$

where D_f is a calibration parameter. The transport capacity V_{ti} and the available supply V_{ai} are compared; if $V_{ti} > V_{ai}$ then supply controls, if $V_{ai} > V_{ti}$ then the demand controls. A schematic of the model is shown in **Figure 6.**

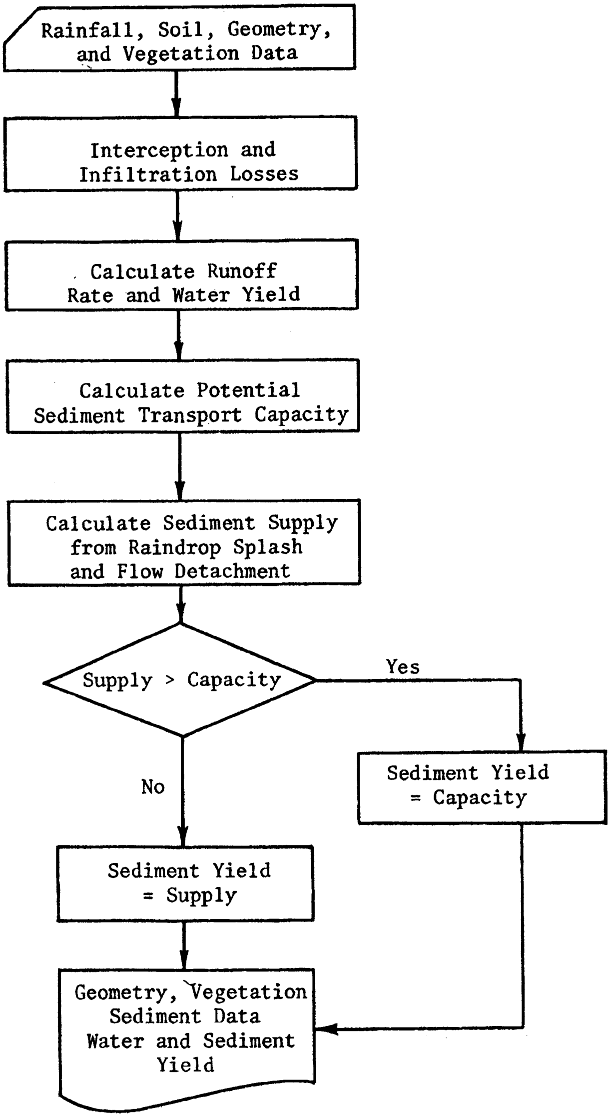

Figure 6. Flow Chart of Sediment Transport Model.
Source: Simons *et al.* 1977, reproduced with permission.

The PTM uses 10 particle size categories. The effect of armouring is introduced as it can control the supply of entrained particles. For a given particle size range, the fraction of sediment available for transport increases with the amount of sediment eroded. For the first and for the following time steps in which the required entrainment criterion has been reached, the particles supply available for transport is, respectively, (Alley *et al.* 1980):

$$S_a(J) = \frac{S(J)^2}{2S_T} \qquad S_a(J) = \left[1 - \frac{S(J)}{S(J)+T(J)}\right]S(J) \tag{6.8}$$

where $S_a(J)$ is the weight of particles in size J available for transport, $S(J)$ is the weight of particles of size J that is part of the armored bed, S_T is the total weight of sediment currently in the bed and $T(J)$ is the total weight of sediment of size J that has been transported from the bed in the previous time steps. A schematic of the model is shown in **Figure 6.**

The PTM has been tested using data from Ross Ade upper watershed, an urban watershed adjacent to the Purdue University campus in West Lafayette, Indiana. These data were collected by McElroy *et al.* (1976) as part of a project under the coordination of the writer. **Figure** 7 shows a sample result.

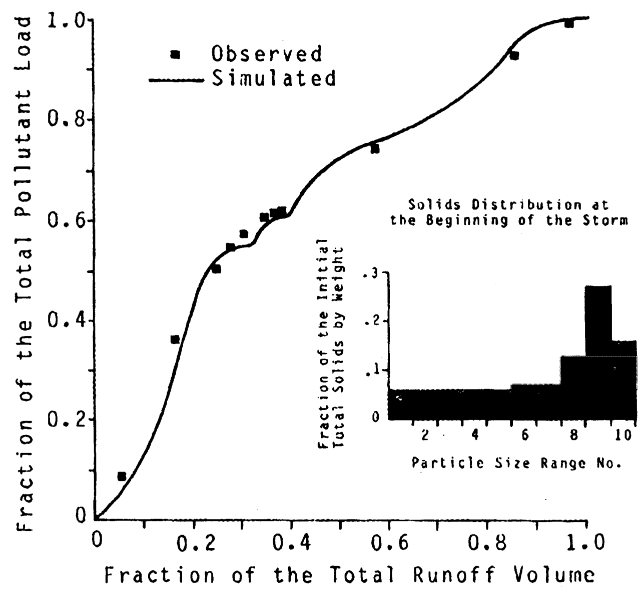

Figure 7. Particle Transport Model (PTM).
Calibration Results for Ross Ade Storm of November 3, 1972
Source: Ellis and Sutherland, 1979, reproduced with permission.

6.3. THE EQUIVALENT SOLID RESERVOIR (ESR) MODEL

Good calibration of models like SWMM requires expensive water quality data. PTM requires data that are not generally available and does not simulate flows in a sewer system. Therefore, Schroeter and Watt, (1989) developed the Equivalent Solid Reservoir (ESR) model to provide an accurate simulation of sediment transport that is not unnecessarily complex. The ESR model is part of the Queen's University Urban Runoff Model (Q'URM).

An initial sediment load has to be specified as no build-up of sediment is included in the model. The sediment transport includes two mechanisms: translation and scour/deposition. The mass flow routing is achieved by the time-offset method applied to gutters and pipes only. The routing technique is the same as used in Q-ILLUDAS (Noel and Terstriep, 1982).

The scour/deposition is simulated by means of ESR. One such conceptual reservoir is located at the downstream end of each surface, gutter and pipe element. The basic equations used are the conservation of the sediment mass M and a mass balance in the ESR. These are, respectively:

$$\frac{dM}{dt} = D - E \tag{6.9}$$

$$O = I - D + E \tag{6.10}$$

where D and E are the deposition and erosion rates, respectively, O and I are the sediment outflow and inflow, respectively, as shown in **Figure 8.** The deposition rate is based on an equation used for the design of sedimentation tanks (Camp, 1946):

$$D = I\exp[-k_d(Q/Q_r)^\alpha \tag{6.11}$$

where Q_r is a reference flow, k_d and α are the deposition coefficient and exponent, respectively. The erosion rate is based on the rating curve equation

$$E = k_e(Q/Q_r)^\beta M \tag{6.12}$$

where k_e and β are the scour coefficient and exponent, respectively. The reference flow has a velocity sufficient to suspend a representative particle. This occurs when the fall velocity is equal to the shear velocity that, in turn, is about 1/9 the mean velocity v_r. Thus $v_r \geq 9\ w$, where w is the fall velocity. The reference discharge for surfaces is calculated by Manning's formula as

$$Q_r = \kappa W v_r^{5/3}(n_s/\sqrt{S})^{3/2} \tag{6.13}$$

where κ is unity for SI units and 0.552 for Imperial units, W is the width, S is the slope and n_s is the roughness coefficient. A sensitivity analysis showed that the particle size, the relative density, the amount and location of sediment loads and the rate of deposition were significant variables. Examples of good simulation results were obtained as shown in **Figure 9.**

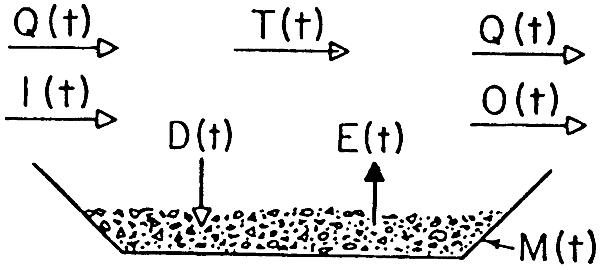

Figure 8. ESR Model Symbol Definition.
Source: Schroeter and Watt, 1989, reproduced with permission.

7. Calibration

The theory and techniques of model calibration have been reviewed by Sorooshian and Gupta (1995). They discuss the effect of uncertainties on model calibration and methods of obtaining unique and realistic parameter values in the presence of uncertainty. The reliability of hydrologic models has been reviewed by Melchin (1995). He is concerned with the sources of uncertainty, the measures of output reliability and methods of reliability analysis. For water quality models in particular, Beck (1987) listed four areas of concern: 1) the uncertainty about the model structure, 2) the uncertainty in the model parameters, 3) the aggregation of uncertainty in model structure, model parameters, etc., and 4) the reduction of uncertainties by means of thoughtfully designed experiments and monitoring programs. This section focuses on the calibration of urban runoff models that include a quality component and the emphasis is on the calibration of SWMM.

7.1. RUNOFF QUANTITY

Proper calibration of the runoff quantity model is necessary before attempting to calibrate the quality portion. Sangal and Bonema (1994) propose the following methodology for the calibration of the runoff process. They use the following sequence of steps:

1. Calibrate the *runoff volume from impervious* areas by adjusting the percentage of impervious area and the initial abstraction (depression storage). This is done for a storm occurring after a prolonged dry spell so as to minimise runoff from pervious areas. An event with fairly continuous and constant rainfall and with an intensity less than the soil infiltration capacity is preferred. Note that the runoff volume and the depression storage are related. For example, a decrease in depression storage to obtain a less rapid response of the runoff

hydrograph will decrease the runoff volume. The percentage imperviousness thus needs to be increased proportionally.

2. Calibrate the *shape of the hydrograph*: the peak and the time to peak are calibrated by adjusting the subcatchment width, Manning's roughness coefficient, the ground and channel slopes with the same storm used in the previous step. Decreasing the width forces the peak to occur later as the travel time is increased. Increasing the roughness coefficient also will increase the travel time and hence the time to peak.

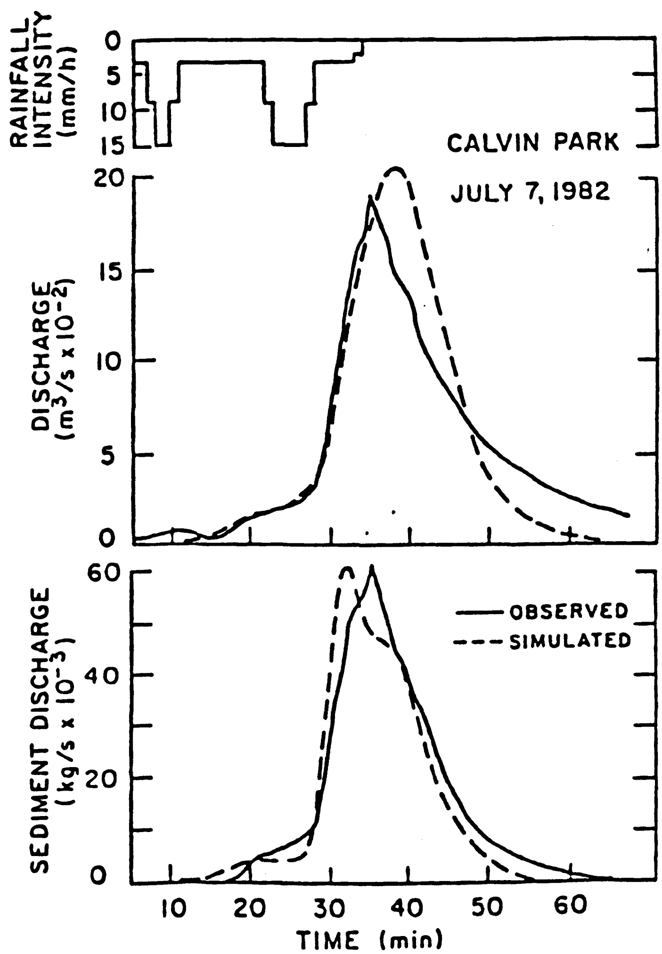

Figure 9. Results of ESR Model: Hydrograph and Pollutograph in Kingston, Ontario. Source: Schroeter and Watt (1989), reproduced with permission.

3. Calibrate the *runoff volume from pervious areas* by adjusting the soil infiltration capacity parameters using storm events with rainfall intensities larger than the soil infiltration capacity.
4. *Fine tune* using measured data from other storms.

Sangal and Bonema insist on the importance of having good data for the calibration process. They use a spreadsheet for the purpose of archiving and plotting the rainfall data, the calibration data and the simulation results. The use of charts linked to the spread-sheet facilitates the plotting of the hydrographs generated in the calibration process.

Calibration is an iterative process requiring several storm events or several continuous rainfall sequences. As a perfect fit cannot be obtained for all events or rainfall sequences, goodness-of-fit criteria may be used. Some of these criteria are:

1. Minimise the sum of squares of the differences between predicted and observed data

$$\text{Minimise } F = \sum_{i=1}^{n} (q_i - q_i^{'})^2 \tag{7.1}$$

where q_i' is the simulated value and q_i is the observed value.

2. Minimise the sum of the absolute values

$$\text{Minimise } F = \sum |q_i - q_i^{'}| \tag{7.2}$$

3. Minimise the sum of the squares of the differences between the logarithms

$$\text{Minimise} F = \sum_{I=1}^{n} [\ln(q_i - \ln(q_a^{'})]^2 \tag{7.3}$$

Other calibration criteria have been used such as those found in Baffaut and Delleur (1989):

4. Minimise the volume difference

$$F = \frac{1}{N} \sum \frac{V_m - V_p}{V_m} \tag{7.4}$$

5. Minimise the peak difference

$$F = \frac{1}{N} \sum \frac{P_m - P_p}{P_m} \tag{7.5}$$

6. Minimise the time difference

$$F = \frac{1}{N}\sum(T_m - T_p) \tag{7.6}$$

7. Minimise the weighed error

$$F = \frac{1}{N}\left[a\frac{\sqrt{\sum_{j=1}^{N_i}(m_j - p_j)^2}}{m_i} + b\left|\frac{V_m - V_p}{V_m}\right| \right] \tag{7.7}$$

where V is the volume of runoff, P is the peak flow, T is the time to peak, the subscripts m and p refer to the measured and predicted quantities, N is the number of storm events used for calibration, N_i is the number of data for event I, and p_j ($j=1,\ldots,N_i$) are the measured and predicted flows of the ith hydrograph and m_i is the average flow of event i.

8. Yang and Parent (1996) minimise the root-mean-square error

$$F = \sqrt{\frac{1}{n}\sum_{i=1}^{n}[Q_m(i) - Q_c(i)]^2} \tag{7.8}$$

where the subscripts m and c refer to measured and calculated flows, respectively. Many users prefer to rely on their judgement rather than on rigorous criteria.

7.2. QUALITY CALIBRATION

There are few guidelines for choosing the initial value of the washoff coefficient. Haster and James (1994) obtained a linear regression for the washoff coefficient for sediment removal from impervious areas of the watersheds in Austin and Houston (TX):

$$w = 0.007N + 0.304 \tag{7.9}$$

where w is the washoff coefficient (m^2/L or mm^{-1}) designated by R_{COEFF} in section 3.3 and N is the antecedent time since the last rainfall occurred (days). The coefficient of determination (R^2) for this regression is 0.90. On impervious areas the total rainfall has little effect on the amount of sediment transported.

Jewell *et al.* (1978) presented a flow chart for the calibration of the quantity and quality parts of RUNOFF (**Figure 10**). They used the standard error of estimate, *SEE*, to measure the accuracy of fit between measured and predicted data:

$$SEE = \left(\frac{\sum_{i=1}^{n} (p_i - m_i)^2}{n-2} \right)^{1/2} \tag{7.10}$$

where n is the number of measured and predicted data points, p_i and m_i are the predicted and measured values on the ith data point. Because of the variability of the predicted flow volumes and pollutant mass emissions from storm to storm, they recommend calibrating for average conditions across several storms in an effort to reduce the predictive error and to increase confidence in the result.

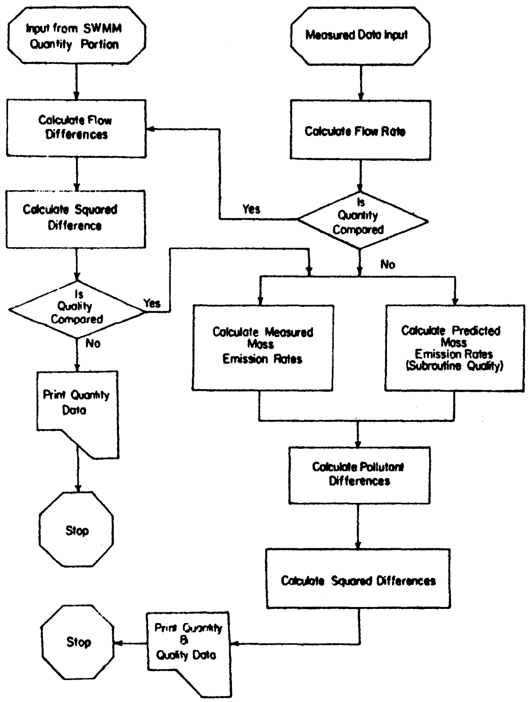

Figure 10. SWMM Calibration Program.
Source: Jewell *et al.*, 1978, with permission

Baffaut and Delleur (1990) used the relative load difference as an efficient means of quantifying how well the washoff load of an event is simulated:

$$F = \frac{1}{N} \sum_{1}^{N} \frac{L_{m_i} - L_{p_i}}{L_{m_i}} \tag{7.11}$$

where L_{mi} and L_{pi} are the measured and predicted loads, respectively, of event i and N is the total number of events used for calibration. To avoid misinterpretation of the relative load difference when overpredicted events balance with underpredicted events, the average of the absolute value of the load differences is also used:

$$F = \frac{1}{N}\sum_{1}^{N}\left|\frac{L_{m_i} - L_{p_i}}{L_{m_i}}\right| \tag{7.12}$$

7.3. KNOWLEDGE-BASED CALIBRATION

An introduction to the application of expert systems in hydrology has been given by Delleur (1991). Liong *et al.* (1991) presented a knowledge-based calibration of the RUNOFF block. The procedure includes: a sensitivity analysis of the calibration parameters and a strategy for the parameter selection and the assignment of a value that attempts to match the simulated and observed hydrographs. Two objective functions are used:

$$\textit{Relative runoff volume ratio} = \frac{V_m - V_s}{V_m} \tag{7.13}$$

$$\textit{Relative peak flow ratio} = \frac{(Q_p)_m - (Q_p)_s}{(Q_p)_m} \tag{7.14}$$

where V is the runoff volume, Q is the flow rate and the subscripts p, m, s indicate peak, measured and simulated, respectively. A third order polynomial is used to represent the value of each objective function in terms of the ratio of the current parameter value to its default value for five different storm events. The more sensitive parameter exhibits a steeper objective function. The strategy to reduce the error of the runoff prediction consists of the following steps:

1. Determine which of the two objective functions is larger.
2. If the runoff volume ratio is larger than the peak ratio, select the most appropriate parameter to reduce the error in the runoff volume prediction. The most appropriate parameter is that which has the steepest relative slope.
3. Compare the objective functions with the corrected parameter.
4. Repeat up to four times.
5. Run SWMM with the new parameter value.

The calibration strategy is implemented using the expert system "Nexpert" (KES PS, 1987).

7.4. EXPERT SYSTEM CALIBRATION

Baffaut and Delleur (1989, 1990) developed two expert systems for the calibration of the quantity and quality aspects of SWMM runoff block, respectively. Each expert system performs three tasks: 1) the selection of computational options and the choice of initial values of the inputs, 2) the determination of the parameters that need to be adjusted and 3) the estimation of the parameter adjustments. The emphasis is on the last two tasks that comprise the calibration phase. The runoff quantity calibration is summarised because of the importance of a correct quantity model in order to make quality predictions.

7.4.1. *Runoff Quantity Calibration.*

For the quantity of runoff, the assessment of the goodness of fit between measured and predicted values uses four criteria: the volume difference, the peak difference, the time difference and the weighed error defined in eqs. (7.5), (7.6), (7.7) and (7.8). In the latter equation, weights of 0.7 for the standard error and 0.3 for the volume difference are used initially. The goals are to limit the maximum volume and peak errors to 10% of the measured values and the delayed or advanced peak times to 10 minutes while minimising the weighed error. When these goals cannot be attained, the focus is on reaching approximately the same number of event hydrographs with underpredicted and overpredicted runoff volumes and flow peaks. Not all parameters are used for calibration. The adjustable parameters, from the most to the least sensitive are: the percentage of impervious area, the characteristic width, Manning's coefficient, the slope, the depression storage and the infiltration parameters. Bounds of the range of values of the depression storage and Manning's coefficient can be set from values found in the literature. For the other parameters, the user can give a level of certainty. A certainty of 1.0 indicates that the statement is completely true, a certainty of -1.0 indicates it is completely false, and a certainty of 0.0 characterises unknown values. Ranges of possible values are then

$$\textit{maximum value} = \textit{initial value} \,.\, (2 - \textit{certainty}) \tag{7.15}$$

$$\textit{minimum value} = \textit{initial value} \,.\, \textit{certainty} \tag{7.16}$$

The expert system uses rules to assess the goodness of fit and to adjust parameters during the calibration. An example of rules to assess the goodness of fit is:

> *If the peak difference criterion is greater than 10%, then the peaks are too low with a certainty of 1.00.*

Other rules decide whether a correct calibration has been attained. For example:

> *If the number of storms with overpredicted peaks is equal to the number of storms with underpredicted peaks and if the number of storms with underpredicted volumes is equal to the number of storms with overpredicted volumes then we believe that the calibration is correct to a degree of 0.7.*

Some rules assess possible actions. For example:

> *If the predicted volumes of runoff are too low and if the percentage of impervious area is not at its maximum, then it has to be increased (with a certainty factor of 0.9).*

Certainty factors are given in order to rank the conclusions. A complete list of the rules may be found in Baffaut (1988).

Once the parameters that need to be changed have been identified, the magnitude of the change needs to be determined. Difference criteria are very sensitive to the impervious area percentage, the characteristic width and the slope. Changes in these parameters have an impact even if the volumes and peaks are poorly simulated. The depression storage, Manning's coefficient and the infiltration parameters have an impact only if the simulation is almost correct.

7.4.2. *Runoff Quality Calibration*

Focusing on the total event washoff load, the most sensitive parameters are the buildup limit, *DDLIM* or *QFACT(1)*, the buildup exponent, *DDPOW* or *QFACT(2)*, (assuming an exponential buildup model) and the washoff coefficient, R_{COEFF}, defined in sections 3.2 and 3.3. The assessment of the goodness of fit is based on the relative and absolute load differences which are defined by eqs. (7.11) and (7.12). The calibration is considered good when the absolute load difference is less than 0.2. If this criterion cannot be met, the calibration is considered satisfactory, when the difference between the number of events with overpredicted and underpredicted loads is not more than one.

For an individual event the washoff load is considered overpredicted, if the relative load difference $(L_m - L_p) / L_m < -0.1$ and underpredicted if $(L_m - L_p) / L_m > 0.1$. For multiple events, the relative load should be within the interval (-0.15, +0.15). An example of a calibration rule expressing the direction of the improvement is as follows:

> *If, for a pollutant P, the absolute load difference is more than 0.20 and if the number of events with underpredicted loads is larger than the number of events with overpredicted loads, then the simulated storm loads for this pollutant are too small.*

Another rule is:

> *If, for pollutant P, the absolute load difference is larger than 0.20 and the relative load difference is larger than 0.15 and the difference between the number of events with overpredicted and underpredicted loads is not more than one, then the simulated storm loads are too small.*

If needed, the values of the buildup exponent and of the washoff coefficient are modified by a small amount. The washoff loads are proportional to the buildup coefficient for the exponential model and to the buildup limit for the Michaelis model. Thus, neglecting secondary sources of pollutant, the variation of washoff loads can be used to calculate the variation of the buildup parameters:

$$a = \frac{\Delta\ parameter}{parameter} = \frac{\Delta\ washoff\ load_i}{washoff\ load_i} \tag{7.17}$$

The adjustment is calculated so that the new relative load difference (defined in eq. 7.11) is zero:

$$\frac{1}{N}\sum_{1}^{N}\frac{L_{m_i}-L_{p_i}-aL_{p_i}}{L_{m_i}}=0 \tag{7.18}$$

The value of a is therefore

$$a=\frac{\dfrac{1}{N}\sum_{1}^{N}\dfrac{L_{m_i}-L_{p_i}}{L_{m_i}}}{\dfrac{1}{N}\sum_{1}^{N}\dfrac{L_{p_i}}{L_{m_i}}}=\frac{\textit{average load difference}}{1-\textit{average load difference}} \tag{7.19}$$

from which the new parameter value is obtained. The change is made in the input files and the model is run again. The process is repeated until the calibration is obtained for each pollutant.

7.5. TRAINING WITH EXPERT SYSTEM

Griffin *et al.* (1994) developed a tool kit based on expert system to transfer knowledge and to provide training in the use of several models including MOUSE, HYSTEM,-EXTRAN WALLRUS/SPIDA and BEMUS.

8. Statistical and Probabilistic Methods

Statistical methods can be divided in three categories: statistical methods proper such as regression analysis, probabilistic methods, such as derived distributions, and stochastic methods such as autoregressive-moving average models.

8.1. REGRESSIONS

Simple linear regressions are often used to estimate the pollutant load during a storm event:

$$Y_i = a + bX_1c + cX_2 \tag{8.1}$$

where Y_i is the concentration or the loading of the *i*th pollutant and X_j's are related variables such as rainfall depth or duration or volume, peak runoff, dry weather duration etc. The curvilinear regression is

$$Y_i = a\,X_1^b\,X_2^c \tag{8.2}$$

Equations of these types are often used to predict impacts of runoff pollution on receiving waters, because total event loads are sufficient and the detailed pollutographs are not needed. For example, Jewell *et al.* (1982) give the following equation for the storm event load of total suspended solids (kg/ha), MTSS, in terms of the antecedent dry period (days), DTS and the average runoff duration (h), DP and the discharge (m^3/h), Q :

$$MTSS = 213\ DTS^{0.481}\ DP^{0.313}\ Q^{1.20} \tag{8.3}$$

which was obtained from a sample of 118 cases and has coefficient of determination of $R^2 = 0.63$.

Driver and Tasker (1990) presented multiple regressions to estimate runoff constituent loads, runoff volumes, constituents mean concentrations and mean seasonal or mean annual constituent loads. They used data from the NURP project described in section 2.1. The United States were divided in three areas depending upon the mean annual rainfall: less than 20 inches (50.8 cm), 20 to 40 inches (50.8 to 101.8 cm) and larger than 40 inches (101.8 cm). The explanatory variables included: The total contributing area, the impervious area as a percent of total contributing area, the percent of industrial area, the percent of commercial land use, the percent of residential land use, the percent of non-urban land use, the population density, the storm rainfall depth, the maximum 24-hour precipitation intensity that has a 2-year recurrence interval, the mean annual rainfall, the mean annual nitrogen load in precipitation, and the mean minimum January temperature. The equations are of the form

$$\hat{Y} = \beta_{0'}\ X_1^{\beta_1} X_2^{\beta_2} \cdots X_n^{\beta_n}\ BCF \tag{8.4}$$

where *BCF* is a bias correction factor to obtain an unbiased estimate of the mean due to the logarithmic transformation that is used in calculating the regression coefficients. For example the equation for the total nitrogen storm load for region I is

$$\begin{aligned} TN_I &= 1132\ TRN^{0.798}\ DA^{0.960}\ (LIU+1)^{0.462}\ (LUC+1)^{0.260} \\ &\cdot (LUN+2)^{-0.194}\ MNL^{-0.951}\ 1.139 \end{aligned} \tag{8.5}$$

where TN_I is the total nitrogen storm load (lb), *TRN* is the total storm rainfall (in.), *DA* is the total contributing area (Mi_2), *LUI* is the industrial land use (percent), *LUC* is the commercial land use (percent), *MNL* is the mean annual nitrogen load in rainfall (lb. per acre) and the bias correction factor is 1.139. The coefficient of determination is $R^2 = 0.95$.

8.2. ERROR ANALYSIS

Water quantity/quality models are subject to a number of errors. There are random input and output measurement errors, and modelling errors due to the model inability to formulate exactly the dynamic nonlinear characteristics of the process. Water quality models that use rainfall or runoff data containing random errors generally lead to inaccurate predictions. Patry and Kennedy (1989) examined the effects of noise corrupted runoff data on the estimates of pollutant loading using the first order pollutant washoff model (see section 3.3, eq. 3.14).

$$P_n = P_{n-1}\, e^{-kr_n\Delta t_n} \tag{8.6}$$

$$\Delta P_n = P_{n-1}[1 - e^{kr_n\Delta t_n}] \tag{8.7}$$

where P_n is the mass of pollutant remaining on the surface at the end of the nth time step (kg) , r_n is the runoff intensity (mm/h), k is the washoff coefficient (mm^{-1}), Δt is time increment (h) and ΔP_n is the mass of pollutant washed off during the nth time interval. Assuming that the true rainfall $r_n{}^* = \mu_r(n)$ is corrupted by a normal random variable $\varepsilon_r\,(n)$ of zero mean and variance $\sigma_R{}^2$,

$$r_n = r_n{}^* + \varepsilon_R\,(n) = \mu_R\,(n) + \varepsilon_R\,(n) \tag{8.8}$$

and using derived probability concepts, they obtained the statistics of P_n and ΔP_n as:

$$E[P_n] = P_n^*\ \eta^n \qquad\qquad Var[P_n] = P_n^{*2}\ \eta^{2n}(\eta^{2n} - 1) \tag{8.9}$$

$$E[\Delta P_n] = \eta^{\,n-1}(P_{n-1}^* - P_n^*\ \eta) \tag{8.10}$$

$$\begin{aligned} Var[\Delta P_n] = {} & P_{n-1}^{*\ 2}\eta^{\,2(n-1)}(\eta^{\,2(n-1)} - 1) \\ & + P_n^{*2}\eta^{\,2n}(\eta^{\,2n} - 1) - 2P_{n-1}^* P_n^* \eta^{\,2n-1}(\eta^{\,2n-2} - 1) \end{aligned} \tag{8.11}$$

where $\eta = \exp[(k\sigma_R\Delta t)^2 / 2]$ and the superscript * indicates the true value. The bias in the pollutant load is seen to increase as a function of t but the magnitude of the pollutant load, P_n, decreases exponentially. Patry and Kennedy (1989) suggest that the functional dependence of the bias on t, Δt and σ_R should be considered particularly for continuous simulations.

8.3. STOCHASTIC MODELS

An example of a model based on the Markov process quoted by Hemain (1986) is:

$$\ln M(t) = a + b\ln M(t-1) + ct + d\,Q(t) \tag{8.12}$$

where $M(t)$ is the pollutant mass and $Q(t)$ is the runoff discharge. Results of application to data collected in West Lafayette in a project coordinated by the author are shown in **Figure 11.**

8.4 DERIVED PROBABILITY DISTRIBUTIONS

Benjamin and Cornell (1970) present methods for deriving distributions of random variables whose general distributions are known. In particular if the dependent variable

Z is the sum of two continuous independent random variables X and Y, then the density distribution function $f_Z(Z)$ can be found from the convolution integral

$$f_Z(z) = \int_{-\infty}^{\infty} f_X(x) f_Y(z-x) dx \tag{8.13}$$

where $f_X(x)$ is the probability density of the random variable X evaluated at $X = x$, and $f_Y(z\text{-}x)$ is the probability density of the random variable Y evaluated at $Y = z - x$.

Loganathan and Delleur (1984) used exponential density functions for the volumes of runoff, inter-event times and durations of runoff events, and a beta distribution for the relative pollutant concentrations and a gamma density for the river flow volumes to derive new distributions for sewer overflow volumes and for the pollutant concentrations in the receiving river after mixing with untreated overflows. These probabilities of overflow volumes and stream pollutant concentrations can be estimated with a simple calculator and do not require computer simulations. Expressions of this type are, therefore, useful in preliminary studies.

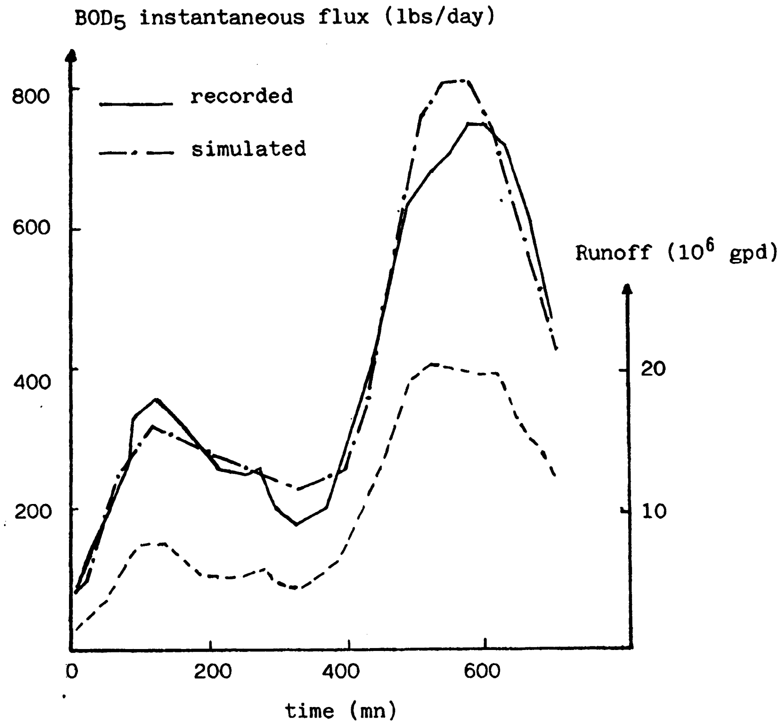

Figure 11. Results of Autoregressive Model fitted to data. Collected at the Ross Ade Watershed, West Lafayette, IN. Source: Hémain (1986), reproduced with permission.

Figure 12 shows a schematic representation of a typical urban runoff system with treatment, storage and overflow to a receiving stream. The volume of runoff X_1 (in or mm) of duration X_2 (hours) is treated at a rate of a (in or mm per h.) If the runoff volume is larger than the volume that can be treated during the runoff duration time X_2, then the excess runoff is stored so that it can be treated at a later time. The volume of the storage facility is b (in or mm). The next storm occurs with an interevent time X_3 (hr). If the volume in excess of the treated volume is larger than the available storage, then an overflow Y (in or mm) occurs. The overflow containing a pollutant concentration C_e (mg L^{-1}) reaches the receiving stream and mixes with the river water that also contains a concentration C_R. The probability of exceedance of the mixed concentration over a permissible level C_o, that is the probability of a regulation violation, is calculated.

The random variables X_1, X_2, X_3 are assumed to be exponentially distributed with parameters α, β, and γ. So that

$$f(x_1) = \alpha \exp(-\alpha x_1) \qquad x_1 > 0 \qquad f(x_2) = \beta \exp(-\beta x_2) \qquad x_2 > 0$$

$$f(x_1) = 0 \qquad x_1 \leq 0 \qquad f(x_2) = 0 \qquad x_2 \leq 0$$

$$\begin{aligned} f(x_3) &= \gamma \exp(-\gamma x_3) \qquad x_3 > 0 \\ f(x_3) &= o \qquad\qquad\qquad\quad x_3 \leq 0 \end{aligned} \tag{8.14}$$

Assuming that X_1, X_2, X_3 are independent and the worst possible scenario that the previous storm completely fills the storage, the probability density of the overflow volumes, Y (in or mm), is (Loganathan and Delleur, 1984)

$$f(y) = K \alpha \exp(-\alpha y) \qquad y > 0 \tag{8.15a}$$

$$f(y) = 1 - K \qquad y = 0 \tag{8.15b}$$

$$f(y) = 0 \qquad y < 0 \tag{8.15c}$$

where K is a function of the parameters α, β, γ, a, and b:

$$\begin{aligned} K = &\{[\beta\gamma / ((\gamma + \alpha a)(\beta + \alpha a))][1 - \exp(-b(\alpha + \gamma/a))]\} \\ &+ [\beta / (\beta + \alpha a)]\{\exp - b(\alpha + \gamma/a)]\} \end{aligned} \tag{8.16}$$

Equation (8.15a) provides a simple way of estimating the probability of a given volume of overflow.

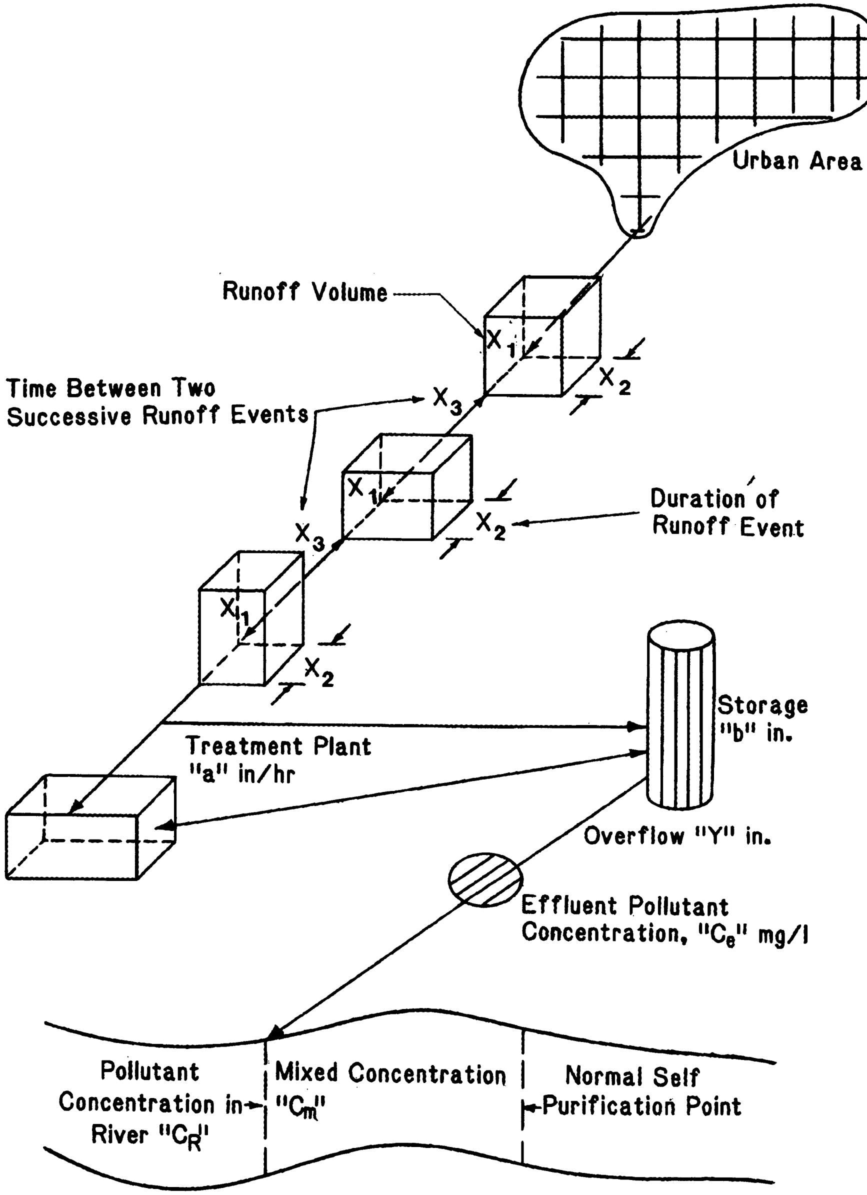

Figure 12. Schematic of the urban storm runoff system.
Source: Loganathan and Delleur (1982), Fig. 2.1 p. 16.

The total pollutant load, L_p, discharged in the receiving stream is $L_p = C_e\,Y$, where C_e is the effluent concentration. In BOD analysis the Streeter-Phelps equation provides a critical distance x_c at which the minimum dissolved oxygen occurs. The flow travel time from the discharge point to point x_c is the critical time. The volume of flow in the river during the critical time is designated by V_R. A common assumption for river flows is that V_R is gamma distributed with parameters ρ and θ:

$$f(v_R) = \frac{\rho^{\theta}}{\Gamma(\theta)} v_R^{\theta-1} \exp(-\rho\, v_R) \quad ; \qquad \rho > 0 \qquad \theta > 0 \qquad v_R > 0 \tag{8.17}$$

$$f(v_R) = 0 \quad ; \qquad v_R \le 0$$

The pollutant concentration, C_m, in the receiving stream at the overflow point after mixing is

$$C_m = \frac{C_e Y + C_R V_R}{Y + V_R} \approx C_e \frac{Y}{V_R} + C_R \tag{8.18}$$

where C_R is the pollutant concentration in the receiving stream just upstream of the overflow point. The approximation on the right hand side of (8.18) is based on the assumption that, in general, the receiving flow volume V_R is much larger than the effluent volume Y. Thus, the probability that C_m exceeds a threshold concentration C_o is

$$P(C_m \ge C_{ov}) \approx P\left[\frac{C_e}{C_o}\frac{Y}{V_R} + \frac{C_R}{C_o} \ge 1\right] \tag{8.19}$$

As the relative river concentration C_R/C_o varies between 0 and 1, a beta distribution with parameters p and q can be fitted:

$$f(s) = K_p s^{p-1}(1-s)^{q-1} \qquad ; \quad 0 < s < 1 \tag{8.20a}$$

$$f(s) = 0 \qquad ; \quad s \le 0 \tag{8.20b}$$

where

$$K_p = \frac{\Gamma(p+q)}{\Gamma(p)\Gamma(q)} \tag{8.21}$$

Loganathan and Delleur (1984) showed that the critical exceedance probability (8.19), i.e., the probability of an environmental violation is

$$P[\text{violation}] = K\left(\frac{\rho\, k_1}{k_2}\right)^{\theta} F(\theta, p, r, k_3) \tag{8.22}$$

where $k_1 = C_e/C_o$ $\quad k_2 = k_1+\alpha \quad k_3 = \alpha/k_2 \quad r = p + q$.
and

$$F(\theta, p, r, k_3) = K_p \int_0^1 s^{p-1}(1-s)^{r-p-1}(1-sk_3)^{-\theta} ds \tag{8.23}$$

is a Gaussian hypergeometric function for which Abramowitz and Stegun (1972) present an infinite series expansion. The probability of the river concentration exceeding the permissible maximum concentration, C_o, can be calculated by means of the closed form relationship (8.22) that can be evaluated by means of a simple calculator and does not require a computer-based simulation.

Loganathan and Delleur(1984), using data for West Lafayette, Indiana, verified the assumptions of independence of X_1, X_2, X_3 and with $\alpha = 1/\overline{X}_1 = 16.7$, (in^{-1}), $\beta = 1/\overline{X}_2 = 0.4761$ (h^{-1}), and $\gamma = 1/\overline{X}_3 = 0.014$ (h^{-1}), a = 0.006 (in/h) and b = 0 found K = 0.8261 and $P(0 < Y \leq y) = 0.8261[1-\exp(-16.7y)]$. **Figure 13** shows a comparison of results obtained with the above equations and simulations made using the model STORM which in many respects is similar to SWMM.

The derived distributions give an excellent approximation for small storage values, b, as is the case in **Figure 13,** because the assumption that the previous storm fills the storage is nearly fulfilled. This assumption was later removed by Loganathan *et al.* (1985) and the methodology was extended to the design of detention facilities (Segarra and Loganathan 1993, 1994 and Segarra 1995).

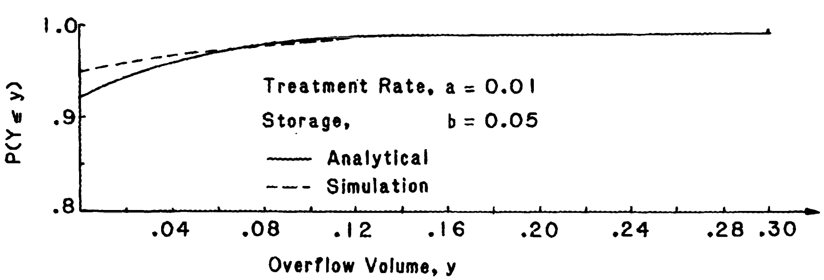

Figure 13. Comparison of analytical and simulation results.
Source: Loganathan and Delleur (1982), Fig. 5.7 p. 79

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10. Appendix

This appendix lists some of the sources of the principal models mentioned in this chapter, namely SWMM and MOUSE. This does not constitutes an endorsement of the models nor of the sources by the authors nor by the NATO ASI organisers. Approximate prices are given when known. Contact vendor for exact price.

MODEL AVAILABILITY

	Version	Description and Price	Source	Address & Telephone
SWMM	4.31	EPA original Manuals and programs Prog. \$50, Man. \$85 +postage	Dr. Wayne C. Huber	Dept. of Civil Engineering Oregon State University Corvallis, OR 97331-2302 USA 503 737 4934
		EPA original Manuals and programs	Dr. William James	CHI 36 Stuart Street Guelph, Ontario N1E 4S5 Canada 519 767 0197
		EPA original Manuals only	National Technical Information Service	5285 Port Royal Road Springfield, VA 22161 USA 703 487 4650
	PCSWMM4	Windows Menu interface Sensitivity Analysis \$200	Dr. William James	CHI 36 Stuart Street Guelph, Ontario N1E 4S5 Canada 519 767 0197
	Windows SWMM	Windows input menus and some post-processing Free	Gerald D. LaVeck or D. King Boynton	EPA Office of Science and Technology 401 M Street, NW Washington, DC USA 202 260 7771
	XP-SWMM	graphical interface and post-processor \$3000 or less.	XP Software	5553 West Waters Ave. #302 Tampa, FL 33634 USA 813 886 7724
	MTVE`	post-processor for Extran Dynamic hydr. grade line	10 Brooks Software	3744 West Huron River Dr. Suite 200 Ann Arbor, MI 48103 USA 313 761 1511
MOUSE	3.3 Classic	MOUSE TRAP Includes sediment transport MOUSE RTC real-time-control	Danish Hydraulic Institute	DHI Software Agern Alle5 DK-2970 Horsholm Denmark 45 45 76 95 55
	4.0 for Windows 95	For use with Windows 95 and Windows NT	Danish Hydraulic Institute	DHI Software Agern Alle5 DK-2970 Horsholm Denmark 45 45 76 95 55

WATER QUALITY MODELLING IN SEWER NETWORKS

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1. Introduction

This contribution first describes the routing of flows and pollutant loads through a sewer network, following the simplified techniques used in the TRANSPORT block of SWMM. The inflows and loads are generated by the RUNOFF block or by some other program. Then the advanced flow routing procedures are described. These are used in the EXTRAN block of SWMM, in the form of the complete one-dimensional unsteady flow equations. This block can also be adopted for constituent transport. A short presentation of MOUSE ST follows a review of recent investigations concerning the transport of sediments in sewers. The chapter concludes with a discussion of the prediction of suspended solids using stochastic transfer functions.

2. Flow Routing

2.1. BASIC EQUATIONS

The basic flow routing equations for unsteady free surface flows are the momentum and continuity equations known collectively as the Saint Venant Equations. Their derivations can be found in standard textbooks on open channel flow such as Chow (1959), French (1985), Chaudhry (1993) and standard textbooks on hydrology such as Chow *et al.* (1988).

The *momentum* equation is:

$$\frac{\partial h}{\partial x}+\frac{v}{g}\frac{\partial v}{\partial x}+\frac{1}{g}\frac{\partial v}{\partial t}=S_0-S_f \qquad (2.1)$$

where h is the flow depth (m) and v is the average flow velocity (m/s), which are the dependent variables; x is the distance (m) and t is the time (s), which are the independent variables; g is the acceleration of gravity (m/s^2), S_0 is the slope of the bottom of the conduit and S_f is the slope of the energy grade line, that is the energy loss

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per unit length. This equation is simply a statement of $F = m\, a$ or $a = F / m$. On the left hand side the first term is the rate of change of the depth along the conduit (which represents the change in hydrostatic pressure force per unit mass). The second term is the ratio of the convective acceleration (due to a change in flow velocity between cross sections) to the acceleration of gravity. This term represents the net flux of momentum in the stream reach. The third term is the ratio of the local acceleration (due to a change in velocity at a given cross section) and the acceleration of gravity. This term represents the change of momentum inside the stream reach. On the right hand side, the first term is the component of the gravity force per unit mass in the flow direction and the last term is the friction force per unit mass.

The *continuity* equation is

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (2.2)$$

where Q is the flow rate (m^3/s) and A is the flow cross section area (m^2). This equation is a simple statement of the conservation of volumes. The first term represents the balance between the inflows and outflows in the stream reach and the second term represents the change of volume in the stream reach.

The *kinematic wave* approximation assumes that the left hand side terms of the momentum equation are negligible. Disturbances are allowed to propagate only in the downstream direction. This means that *backwater effects* cannot be simulated and *downstream boundary conditions* (such as tide gates or diversion structures) are assumed not to affect upstream computations. Surcharge conditions (pressure flow) cannot be simulated. However considerable simplification is gained. The *momentum* equation (2.1) thus reduces to

$$S_f = S_0 \qquad (2.3)$$

which states that the slopes of the energy grade line and of the bottom of the conduit are the same, and the flow is essentially uniform. Using Manning's equation

$$Q = \frac{\kappa}{n} A R^{2/3} S_0 \qquad (2.4)$$

where n is Manning's roughness coefficient, R is the hydraulic radius (m) and $\kappa = 1.0$ for metric units or $\kappa = 1.486 \cong 1.49$ for US units. The *friction slope* S_f is calculated as

$$S_f = \frac{Q^2}{\left(\frac{\kappa}{n}\right)^2 A^2 R^{4/3}} \qquad (2.5)$$

3. Pollutant Routing

3.1. BASIC EQUATIONS

For the routing of the pollutants, each conduit is treated as a completely mixed reactor with first order decay (Nix, 1994):

$$\frac{dVC}{dt} = \frac{V\,dC}{dt} + \frac{C\,dV}{dt} = Q_i\,C_i - QC - KCV \pm L \qquad (3.1)$$

where V is the volume of water in the conduit (m^3), C is the pollutant concentration in the conduit and in the discharge, C_i is the concentration in the inflow, Q_i is the rate of inflow in the conduit (m^3/s), Q is the rate of outflow, K is the decay coefficient (s^{-1}) and L is the source or sink of pollutant in the conduit (quantity/s).

Equation (3.1) is a mass balance equation. The left hand side represents the change of the constituent mass per unit time in a conduit element. On the right hand side, the first term represents the mass inflow rate of constituent, the second term is the mass outflow rate of constituent, the third term is the mass decay in the conduit and the last term is the mass source or sink.

An analytical solution of equation (3.1) is possible if Q, Q_i, C_i, V, and L are assumed constant over the time interval. Medina *et al.* (1981) obtained

$$C(t+\Delta t) = \frac{Q_i C_i + L}{V.DENOM}\left(1 - e^{-DENOM.\Delta t}\right) + C(t)e^{-DENOM.\Delta t} \qquad (3.2)$$

where $DENOM = \frac{Q}{V} + K + \frac{1}{V}\frac{dV}{dt}$

4. Scour and deposition

Verbanck *et al.* (1993) state that "urban drainage design and analysis must take intrinsically into account various problems linked to sewer sediments:

- washout of pollutants by sediment re-entrainment during rain events (impact of stormwater discharges on the environment)
- premature functioning of combined sewer overflows
- reduction of hydraulic capacity of urban conduits
- large operational costs incurred for the cleaning out of sewers and tanks".

4.1. BASIC EQUATION

The solids are generally assumed to behave like ideal non-cohesive sediments. (For cohesive sediments see section 8.2.2 in this paper). The Shields criterion is used to determine the critical particle diameter, which indicates the onset of particle movement scouring. Particles smaller than the critical size are eroded and the particles in suspension are routed downstream. The Shields' diagram gives a dimensionless critical shear stress when the particle motion starts, that is when the hydrodynamic drag and lift forces acting on a particle balance the submerged weight of the particle:

$$\frac{\tau_c}{(\gamma_s - \gamma)d} = f\left(\frac{u^* d}{\nu}\right) \tag{4.1}$$

where τ_c is the critical shear stress to induce particle motion, γ_s is specific weight of the sediment, γ is the specific weight of the water, d is the particle diameter, $u^* = (\tau_c/\rho)^{1/2} = (gRS)^{1/2}$ is the shear velocity, ν is the kinematic viscosity, ρ is the water density, g is the acceleration of gravity, R is the hydraulic radius and S is the slope of the conduit which is assumed to be equal to the slope of the energy grade line.

5. Flow Routing in SWMM

A general description of the Storm Water Management Model (SWMM) has been given earlier in this chapter ("Modelling Quality of Urban Runoff"). The flow routing characteristics of the Runoff, Transport and Extran blocks are summarised in Table 1. The full St. Venant equations are implemented in the EXTRAN block (see section 7).

6. TRANSPORT Block

6.1. FLOW ROUTING IN THE TRANSPORT BLOCK

In the TRANSPORT block the *kinematic wave* approximation is used. A finite difference approximation of the continuity equation (2) is utilised:

$$\frac{(1-w_x)(Q_{j+1,n} - Q_{j,n}) + w_x(Q_{j+1,n+1} - Q_{j,n+1})}{\Delta x} + \frac{(1-w_t)(A_{j,n+1} - A_{j,n}) + w_t(A_{j+1,n+1} - A_{j+1,n})}{\Delta t} = 0 \tag{6.1}$$

where Δt is the time interval $(t_{n+1} - t_n)$ (s), Δx is the distance increment $(x_{j+1} - x_j)$ (m), the j, $j+1$ subscripts indicate the conditions at the upstream and downstream ends of the conduit element, respectively, and the n, $n+1$ subscripts indicate the nth and $(n+1)$st time steps, respectively. The weights w_x and w_t are chosen for best numerical stability; a value of 0.55 is normally used. **Figure 1** shows the definition of the notation used in describing the sewer network.

6.1.1. *Sewer Elements*

The *link* elements can be of 16 predefined shapes of conduits or channels, a HEC-2 format for natural channels or a user supplied shape. The *nodes* include: lift station, flow divider, storage unit, flow divider - weir, and backwater element.

When the flow exceeds the free surface capacity of a conduit then surcharge occurs. TRANSPORT stores the excess capacity at the upstream end node of the conduit. This excess is released when the flow is less than the conduit capacity. The hydrograph at

TABLE 1. Flow Routing Characteristics of Runoff, Transport and Extran Blocks

		Runoff Block	Transport Block	Extran Block
1	Flow routing method	Nonlinear reservoir. cascade of conduits	Kinematic Wave, cascade of conduits	Complete equations, interactive conduit network
2	Relative computational expense for identical network schematisations	Low	Moderate	High
3	Attenuation of hydrograph peaks	Yes	Yes	Yes
4	Time displacement of hydrograph peaks	Weak	Yes	Yes
5	In-conduit storage	Yes	Yes	Yes
6	Backwater and downstream control effects	No	No[a]	Yes
7	Flow reversal	No	No	Yes
8	Surcharge	Weak	Weak	Yes
9	Pressure flow	No	No	Yes
10	Branching tree network	Yes	Yes	Yes
11	Network with looped connections	No	No	Yes
12	Number of preprogrammed conduit shapes	5	16	8
13	Alternative hydraulic elements e.g., Pumps, Weirs, Orifices)	WO	PWO	PWO
14	Dry weather flow and infiltration generation (base flow)	Some	Yes	Yes
15	Pollutograph routing	Yes	Yes	No
16	Solid scour/deposition	No	Yes	No
17	Input of hydrograph / pollutograph from external source	No	Yes	Yes

a. Backwater may be simulated as a horizontal water surface behind a storage element.

Source: Huber (1995), reproduced with permission

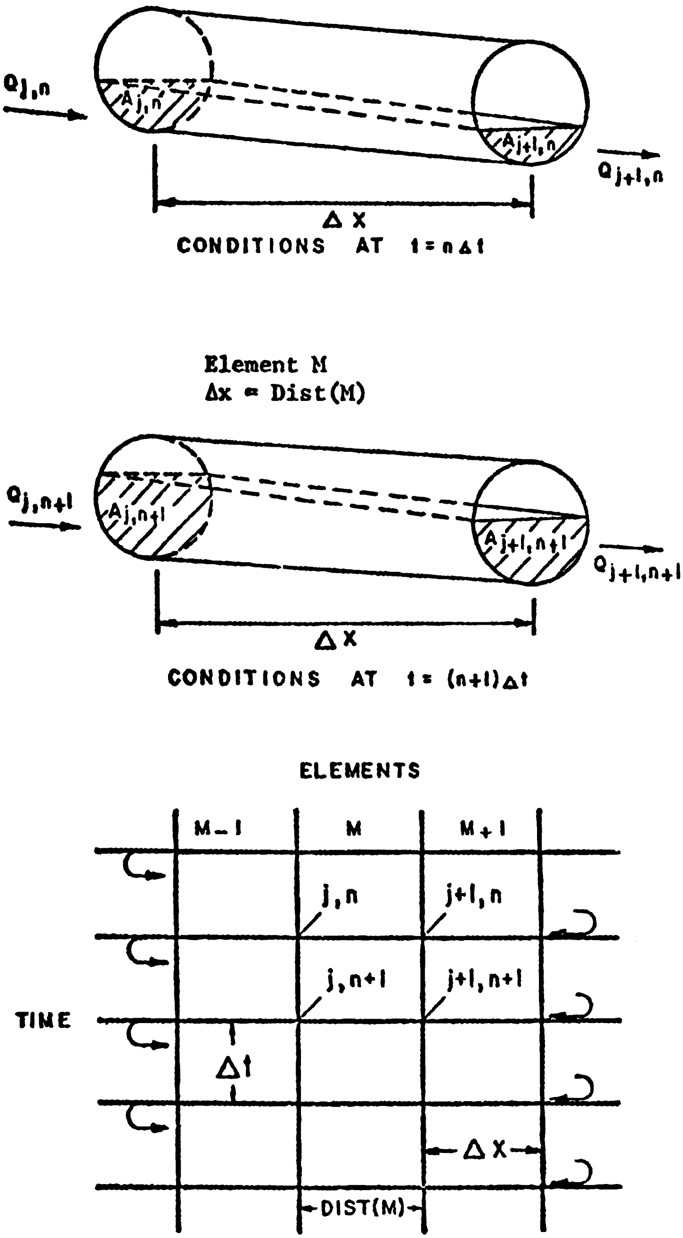


Figure 1. Finite Difference Definition Sketch.
Source: Metcalf AND EDDY (1971), EPA 11024DOC 07/71.

the downstream element is therefore truncated and the downstream conditions are deformed.

The flow area is obtained from Manning's equation and normalised by dividing by that corresponding to full pipe flow

$$\frac{Q}{Q_f} = \frac{A R^{2/3}}{A_f R_f^{2/3}} \tag{6.2}$$

6.2. POLLUTANT ROUTING IN TRANSPORT BLOCK

In the TRANSPORT block, a maximum of four contaminants can be routed. The constituents can be introduced into the sewer network in four ways:

1. Pollutographs generated by the Runoff, Storage/Treatment or transport blocks.
2. Other pollutographs entered at designated nodes
3. Resuspended bottom sediment
4. Dry weather flow pollutographs in combined sewers, entered at designated nodes.

Travel time in sewers is generally short so that the decay during the routing is unimportant. However, for BOD a deoxygenation coefficient is generally desired. The quality routing is not amenable to direct calibration. Routing becomes closer to advection as the number of elements is increased.

6.3. SCOUR AND DEPOSITION IN TRANSPORT BLOCK

The same particle size distribution and average specific gravity are used for each pollutant, i.e., no difference is made between dry weather flow and storm water pollutants. Scour-deposition is simulated only in the conduits but not in the other elements, such as storage elements. The effect of the deposited sediments on the geometry of the bed is not considered. Thus the hydraulic radius of the conduit is used. For programming purposes, the Shields' curve is represented by linear and parabolic segments (**Figure 2**) as follows:

$R^* \leq 1.47$ $\qquad Y = 0.1166\, R^{*-0.977842}$ (6.3a)

$1.47 \leq R^* \leq 10$ $\qquad y = -0.9078950 - 1.2326090\, X + 0.7298640\, X^2 - 0.0772426\, X^3$ (6.3b)

$10 \leq R^* \leq 400$ $\qquad Y = 0.0227\, R^{*0.1568}$ (6.3c)

$R^* \geq 400$ $\qquad Y = 0.06$ (6.3d)

where $Y = \tau_c / (\gamma_s - \gamma)\, d$ $\qquad y = \log_{10} [\tau_c / (\gamma_s - \gamma)\, d]$

and $X = \log_{10} R^*$ $\qquad R^* = u^* d / \nu$.

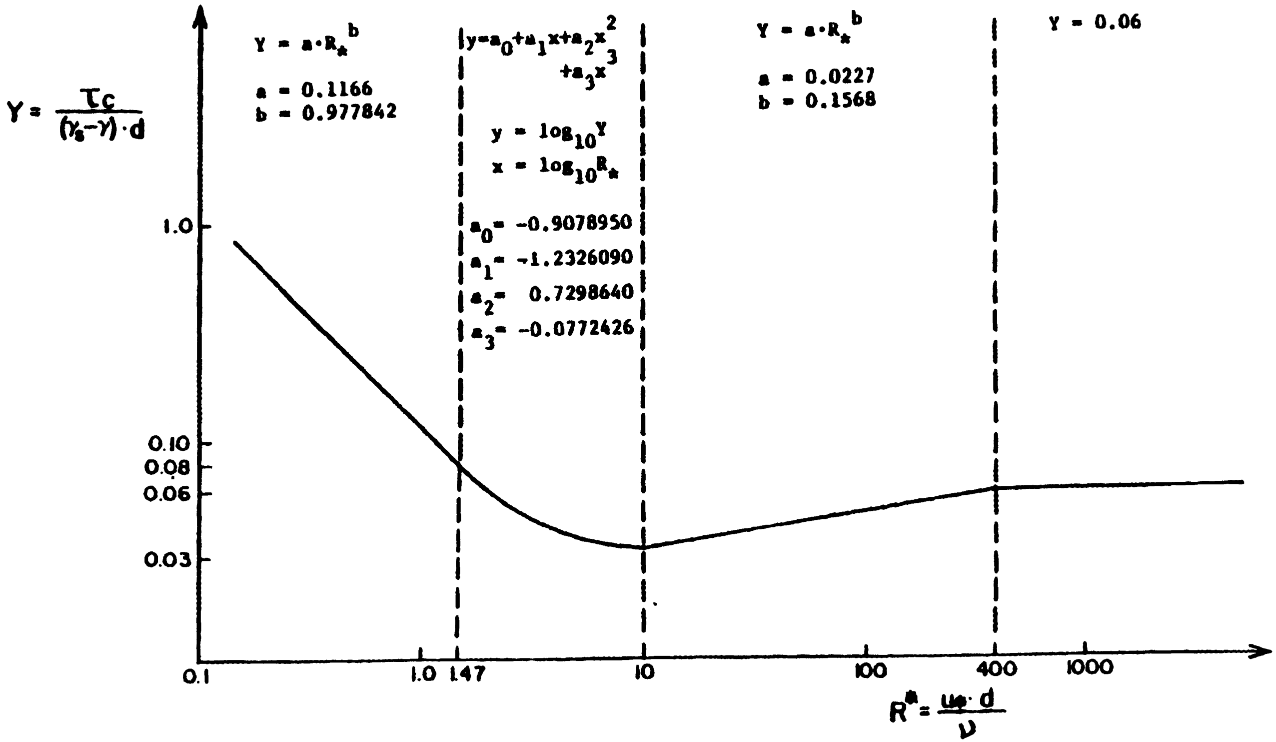


Figure 2. Linear and Parabolic Approximation of Shields Diagram.
Source: Huber and Dickinson (1988) p. 500.

Equation (6.3a) corresponds to the laminar case and there is essentially no particle motion. At R^* = 1.47 the critical diameter is $d = 1.47\ \nu/[RSg]^{1/2}$.

For the region of equation (6.3b) the critical diameter is obtained by a Newton-Raphson iteration. For the straight line segments the equations are solved directly for the critical particle diameter.

The particle size distribution for each pollutant is approximated using up to four straight line segments as shown in (**Figure 3a**). Suppose that at the beginning of the time step the minimum particle size in the bed is $DB_1 = 0.6$ mm. Also suppose that a new critical diameter for the time step is calculated as $CRITD = 1.5$ mm. Then the fraction of mass that is scoured is (72 - 35) / 72 = 0.51 or 51% (see **Fig. 3a**). If the suspended material is deposited, the *new* minimum particle size in the bed DB_2 is

$$DB_2 = \frac{DB_1 M_b + CRITD\, M_d}{M_b + M_d} \tag{6.4}$$

where M_b is the original mass of bed material and M_d is the mass deposited (mg). Similarly for deposition, if the initial distribution is as in Fig. 3c and the maximum diameter in suspension at the beginning of the time interval is $DS_1 = 1.6$ mm, and a new critical diameter for the time step is calculated as $CRITD = 0.9$ mm, then the fraction of suspended material that is deposited is (56 - 34) / (100 - 34) = 0.33 or 33%. (See Fig 3c). The new value of the maximum particle size in suspension DS_2 is

$$DS_2 = \frac{DS_1 . M_s + CRITD. M_e}{M_s + M_e} \tag{6.5}$$

where M_s is the original mass in suspension and M_e is the mass eroded from the bed.

At nodes the new value of the maximum particle size in suspension of the mixture DS_m is

$$DS_m = \frac{\sum_{i=1}^{3} DS_{ui} Q_{ui} C_{ui} + PSDWF\, Q_{DWF} C_{DWF}}{\sum_{i=1}^{3} Q_{ui} C_{ui} + Q_{DWF} + Q_{INF}} \tag{6.6}$$

where Ds_{ui} is the maximum particle size in suspension in the ith upstream conduit, Q_{ui} and C_{ui} are the outflow from upstream conduit i, and the concentration in upstream conduit i, respectively. The subscripts DWF and INF refer to the dry weather flow and infiltration, respectively.

6.4. INFILTRATION INTO SEWER SYSTEMS

The TRANSPORT block has the capability of estimating the average daily infiltration inflows into the sewer system prior to the routing. The components of infiltration are the following **(Figure 4)**:

1. *DINFIL* = dry weather infiltration
2. *RINFIL* = wet weather infiltration
3. *SINFIL* = snow infiltration
4. *GINFIL* = groundwater infiltration (when the water table is above the sewer invert)

The total infiltration, *QINFIL,* is thus

$$QINFIL = \begin{cases} DINFIL + RINFIL + SINFIL \\ GINFIL \text{ (for high water table)} \end{cases} \tag{6.7}$$

The dry weather infiltration is obtained from local observations or calculated by multiplying the unit infiltration rate (flow rate per unit diameter per unit length) by the diameter and the pipe length. The snow infiltration during the snowmelt period is calculated as

$$SINFIL = RSMAX . \sin [180\ (NDYUD - MLTBE) / (MLTEN - MLTBE)] \tag{6.8}$$

where *RSMAX* is the residual peak contribution obtained from local observations, *NYUD* is the day on which the estimate is desired, *MLTBE* is the beginning day and *MLTEN* is the ending day of the melt period. The antecedent precipitation infiltration is calculated by a regression equation of the type:

$$RINFIL = a + a_0 R_0 + a_1 R_1 + a_2 R_2 + a_3 R_3 + \ldots + a_9 R_9 \tag{6.9}$$

where R_n is the precipitation n days prior to the estimate. Likewise the groundwater infiltration is calculated from a regression equation of the type:

$$GINFIL = b + b_1\ GWHD + b_2\ GHHD^2 + b_3\ GWHD^{0.5} \tag{6.10}$$

where *GWHD* is the water table elevation above the sewer invert.

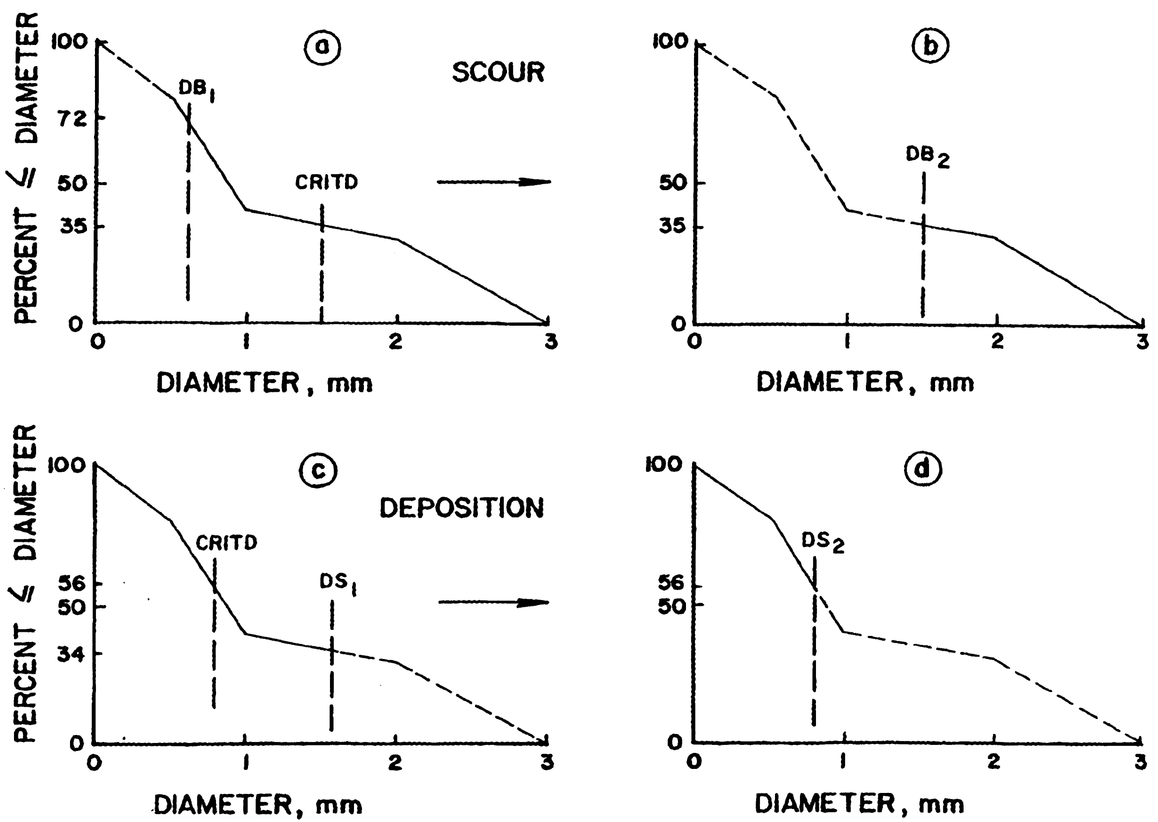
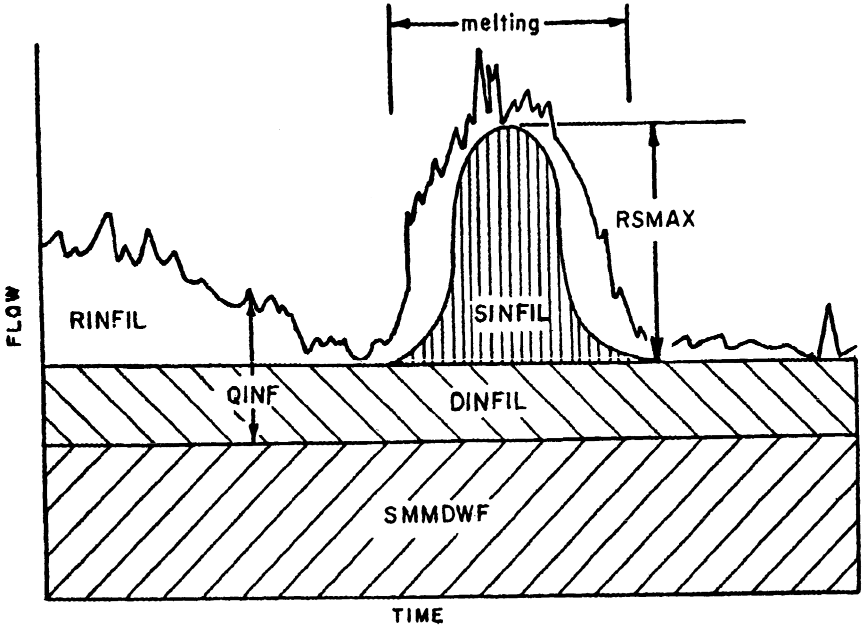


Figure 3. Truncation of Particle Size distribution During Scour and Deposition
Source: Huber and Dickinson (1988) p. 504.



QINF	= Total infiltration
DINFIL	= Dry weather infiltration
RINFIL	= Wet weather infiltration
SINFIL	= Melting residual ice or snow infiltration
RSMAX	= Residual moisture peak contribution
SMMDWF	= Accounted for sewage flow

Figure 4. Components of infiltration.
Source: Metcalf and Eddy, 1971 EPA 11024DOC07/71, p. 102

6.5. DATA INPUT

The input data are arranged in lines B-Q, as described below. The lines B1, B2 and B3 set the run control parameters such as the number of time steps, the number of non-conduit elements, where input hydrographs and pollutographs are entered or printed, or where routed pollutographs are printed, the number of constituents, the starting date, the time step size, the number of dry days prior to start of simulation, the starting time of simulation, the kinematic viscosity of the water, and the parameters needed for estimates of infiltration, sanitary flows and constituents loads.

The lines C1, D1 and D2 are used to enter the hydraulic characteristics of user supplied conduit shapes. Line E1 is used to enter the hydraulic characteristics of the sewer elements and must be repeated for each element. The lines E2 to E4 are used to enter the data for natural channels. Line F1 is used to enter the water quality data. Lines G1 through G5 are concerned with internal storage elements and are omitted, if there is no internal storage. Line I2 specifies the conduits for which the flow depths are to be printed. Lines J1 and J2 specify the non-conduits where the inputs and routed flows and water quality constituents are to be printed, respectively. Lines K1 and K2 are concerned with infiltration flows and constituents. Lines L1 to L3, M1 to M4, O1 to O2, P1 and Q1 are concerned with dry weather flow from residential, commercial and industrial areas.

7. EXTRAN

The purpose of EXTRAN is to route inlet hydrographs through a network of pipes, junctions and flow diversion structures (see **Figure 5**). EXTRAN must be used (instead of TRANSPORT) whenever there are backwater conditions, unsteady flow conditions resulting in transition from free surface to pressurised flow, and special flow devices such as weirs, orifices, pumps, storage basins and tide gates.

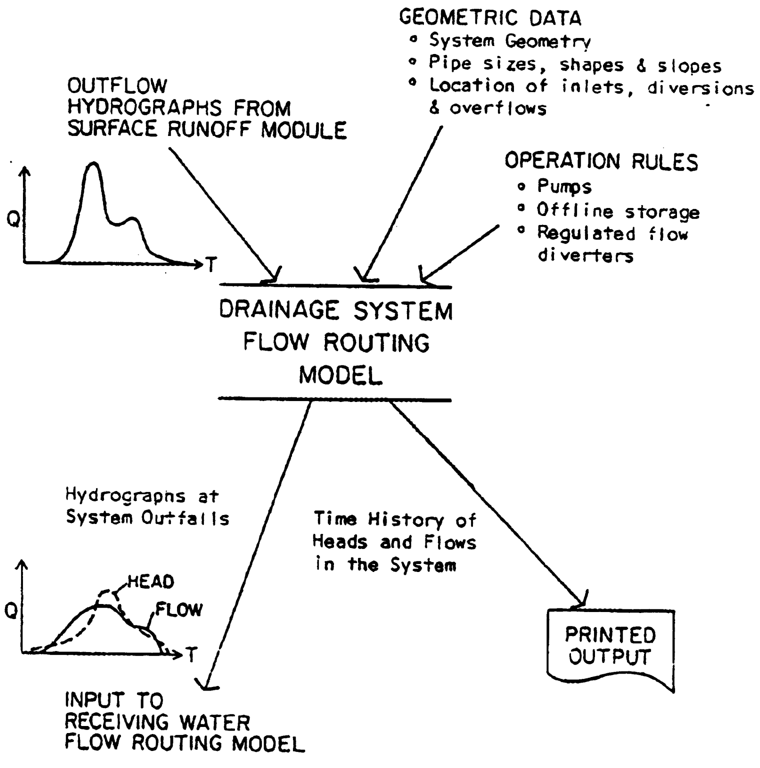


Figure 5. Schematic Illustration of EXTRAN.
Source: Roesner and Aldrich (1989), Fig 5-1, p.136

The sewer system is represented as a series of links or pipes and nodes or junctions (**Figure 6**). The links convey the flow between the nodes. The average discharge is the primary dependent variable in each link. It is assumed to be constant during a computational time step, but the velocity and the depth are allowed to vary within a link. The nodes are storage elements. The primary dependent variable in a node is the head (i.e. the elevation of the pipe invert plus the depth of water). Inflows and outflows take place at the nodes. The volume associated with a node includes that of the half lengths of all pipes connected to the node.

EXTRAN does not include a pollutant routing component. However, a post-processor has been developed for this purpose and is presented in section 7.2.

7.1. FLOW ROUTING

The extended transport block or EXTRAN uses the full Saint Venant equations expressed in a slightly different form, which uses the flow Q and the hydraulic head H as dependent variables instead of the velocity and the depth as in eq. (2.1). The momentum and the continuity equations are, respectively:

$$gA\frac{\partial H}{\partial x}+\frac{\partial (Q^2/A)}{\partial x}+\frac{\partial Q}{\partial t}+gAS_f=0 \tag{7.1}$$

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=0 \tag{7.2}$$

where $H = z + h$ is the hydraulic head, z is the elevation of the conduit invert, and the remaining notation is the same as for eqs. (2.1) to (2.5). The friction slope S_f is obtained from Manning's equation as

$$S_f=\frac{Q^2}{(\kappa/n)^2A^2R^{4/3}}=\frac{Q|v|}{(\kappa/n)^2AR^{4/3}} \tag{7.3}$$

Note that on the right hand side of (7.3) the absolute value of the velocity is taken in order to guarantee that the friction force is in the direction opposite to the flow should flow reversal occur (e.g., in the case of closure of a conduit). Since $Q^2/A = v^2A$, the momentum equation is rewritten as

$$gA\frac{\partial H}{\partial x}+2Av\frac{\partial v}{\partial x}+v^2\frac{\partial A}{\partial x}+\frac{\partial Q}{\partial t}+gAS_f=0 \tag{7.4}$$

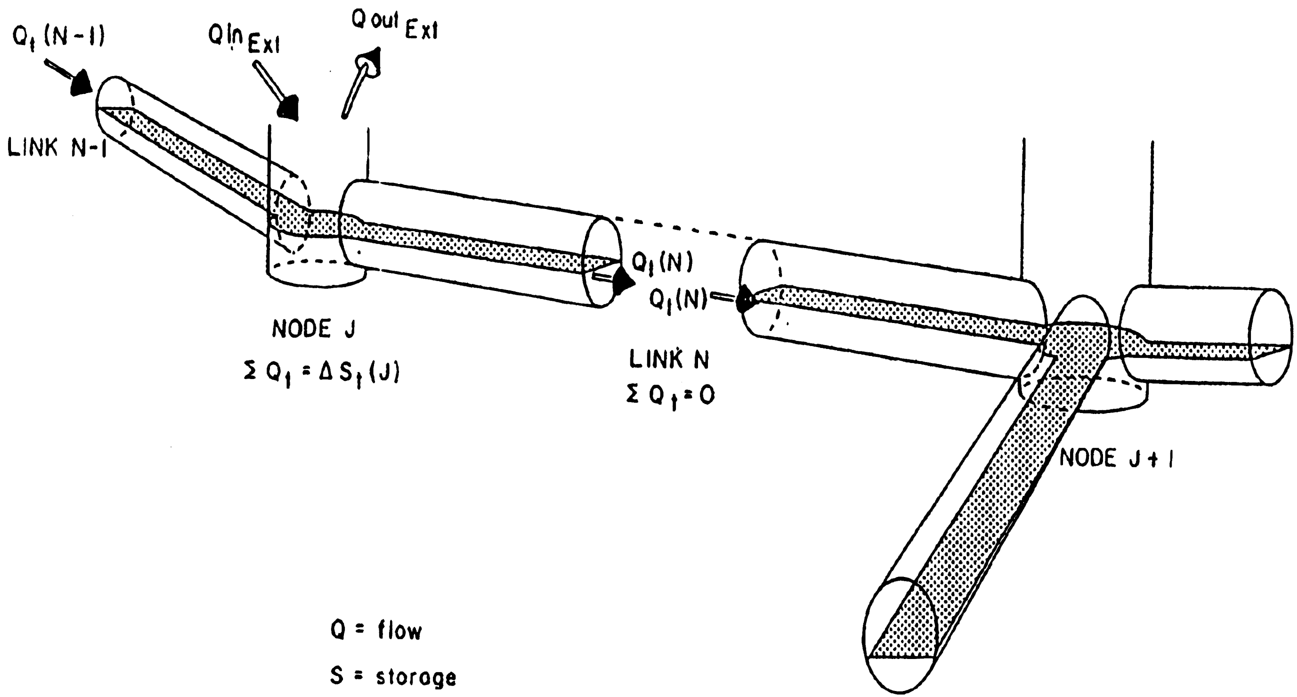


Figure 6. Representation of EXTRAN.
Source: Roesner and Aldrich, (1989). Fig. 5-2, p. 137.

EXTRAN offers three methods of solution: the *explicit*, the *enhanced explicit* and the *iterative* methods.

For the default *explicit* method, the continuity equation (7.2) is modified making use of the fact that $Q = A\,v$ and after multiplying all terms by v, it is rewritten as

$$Av\frac{\partial v}{\partial x} = -v\frac{\partial A}{\partial t} - v^2\frac{\partial A}{\partial x} \tag{7.5}$$

Substituting the right hand side of (7.5) into the second term of the momentum equation (7.4) yields

$$gA\frac{\partial H}{\partial x} - 2v\frac{\partial A}{\partial t} - v^2\frac{\partial A}{\partial x} + \frac{\partial Q}{\partial t} + gAS_f = 0 \tag{7.6}$$

Equation (7.6) is expressed in finite difference form, using (7.3) for S_f, as

$$Q_{t+\Delta t} = Q_t - \frac{\kappa\,\Delta t}{R^{4/3}}|v_t|Q_{t+\Delta t} + 2v(\frac{\Delta A}{\Delta t})_t\Delta t + v^2\frac{A_2 - A_1}{L}\Delta t - gA\frac{H_2 - H_1}{L}\Delta t \tag{7.7}$$

where L is the length of the conduit and Δt is the time step. The discharge at the end of the time interval is:

$$Q_{t+\Delta t} = \frac{1}{1 + \frac{\kappa\,\Delta t}{\overline{R}^{4/3}}|v|}\{Q_t + 2\overline{v}\left(\frac{\Delta A}{\Delta t}\right)_t\Delta t + \overline{v}^2\frac{A_2 - A_1}{L}\Delta t - g\overline{A}\frac{H_2 - H_1}{L}\Delta t\} \tag{7.8}$$

where $\overline{v}$, $\overline{R}$, $\overline{A}$ are weighed averages of the conduit end values at time t, and $(\Delta A/\Delta t)_t$ is the time derivative from the previous time step. The unknowns are $Q_{t+\Delta t}$, H_2, H_1. This is the relationship used along with the continuity equation at the nodes, and relates Q and H by the following expression:

$$\frac{\partial H}{\partial t} = \sum\frac{Q_t}{A_{St}} \tag{7.9}$$

where A_{St} is the surface area of the node at time t. In finite difference form this is rewritten as

$$H_{t+\Delta t} = H_t + \sum\frac{Q_t\Delta t}{A_{St}} \tag{7.10}$$

Equations (7.8) and (7.10) are used to calculate sequentially the discharge in each link and the head at each node, using a modified Euler method.

The *enhanced explicit* method (ISOL=1) and the *iterative* method (ISOL=2) are based on the following form of the dynamic equation:

$$gA\frac{\partial H}{\partial x} - 2v\frac{\partial A}{\partial t} + Q^2\frac{\partial \frac{1}{A}}{\partial x} + \frac{\partial Q}{\partial t} + gAS_f = 0 \tag{7.11}$$

For the *enhanced explicit* method equation (7.3) is substituted into (7.11), and the result is expressed in finite difference form as

$$\begin{aligned} Q_{t+\Delta t} &= Q_t - \frac{\kappa\,\Delta t}{R^{4/3}}|v_t|Q_{t+\Delta t} + 2v(\Delta A/\Delta t)_t\,\Delta t \\ &- Q_{t+\Delta t}Q_t\frac{1/A_2 - 1/A_1}{L}\Delta t - gA\frac{H_2 - H_1}{L}\Delta t \end{aligned} \tag{7.12}$$

where L is the length of the conduit. The solution for $Q_{t+\Delta t}$ is

$$Q_{t+\Delta t} = \frac{Q_t + 2\bar{v}(\Delta A/\Delta t)_t\,\Delta t - g\bar{A}\Delta t(H_2 - H_1)/L}{1 + \frac{\kappa\,\Delta t}{\bar{R}^{4/3}}|v| + Q_t\Delta t(1/A_2 - 1/A_1)_t\,/L} \tag{7.13}$$

where $\bar{v}$, $\bar{R}$, $\bar{A}$ are weighed averages of the upstream, middle and downstream values of the flow velocity, hydraulic radius and cross section area, respectively. The unknowns are $Q_{t+\Delta t}$, H_2, H_1. This equation is used along with the finite difference form of the continuity equation (7.10) to calculate sequentially the discharge in each link and the head at each node. A modified Euler method is used to perform the calculation.

The *iterative method* is also based on equation (7.12), but a time weighting coefficient w is introduced so that the terms evaluated at the current iteration time $t + \Delta t$ are multiplied by w and the terms evaluated at the previous iteration time t are multiplied by $1-w$. The value of $w = 0.55$ is usually taken. The expression for the discharge becomes

$$\begin{aligned} Q_{t+\Delta t} &= \{Q_t + (1-w)[-g\bar{A}\,\Delta t\frac{H_2 - H_1}{L} + \frac{\kappa\Delta t}{\bar{R}^{4/3}}|v| + \Delta t\frac{Q/A_2 - Q/A_1}{L}]_t \\ &+ w(-g\bar{A}[\Delta t\frac{H_2 - H_1}{L}]_{t+\Delta t} + v\frac{\Delta A}{\Delta t}\}\,/ \\ &w\{\frac{\kappa\Delta t}{\bar{R}^{4/3}}|v| + \Delta t\frac{Q/A_2 - Q/A_1}{L}\}_{t+\Delta t} \end{aligned} \tag{7.14}$$

The continuity equation at the nodes is

$$\left(\frac{\partial H}{\partial t}\right)_t = \sum \frac{Q_t + Q_{t+\Delta t}}{A_{s_t} + A_{s_{t+\Delta t}}} \tag{7.15}$$

where A_{s_t} and $A_{s_{t+\Delta t}}$ are the surface areas at the node at times t and $t+\Delta t$, respectively. Equations (7.14) and (7.15) are solved iteratively for the discharge in each link and the head at each node at the end of the time step. The iterative method uses an under-relaxation factor U_f so that

$$Q_{t+\Delta t} = (1 - U_f)Q_j + U_f Q_{j+1} \tag{7.16}$$

$$H_{t+\Delta t} = (1 - U_f)H_j + U_f H_{j+1} \tag{7.17}$$

where the subscripts j and $j+1$ correspond to the jth and $j+1^{st}$ iterations, respectively. U_f is taken as *0.75* or *0.50* for the first and subsequent iterations, respectively. The iterations stop when the following convergence criteria are satisfied:

$$\frac{|Q_{j+1} - Q_j|}{Q_{full}} \leq TOL \quad , \quad \frac{|H_{j+1} - H_j|}{H_{crown}} \leq TOL \tag{7.18}$$

where Q_{full} is the conduit design flow and H_{full} is the height between the invert and the crown of the junction.

For stability of the explicit method, the time increment Δt should not exceed the *Courant number (C#),*

$$C\# = \frac{L}{v + \sqrt{gD}} \quad \text{or} \quad C\# = \frac{L}{v + \sqrt{gA/T}} \tag{7.19}$$

where D is the current pipe hydraulic depth, A is the conduit cross sectional area and T is the free surface width. The *iterative* method uses the maximum allowable time step at each iteration. For further details see Roesner *et al.* (1989) and Dickinson *et al.* (1990).

7.2. QUALITY ROUTING

EXTRAN does not perform any routing of constituents. However, O'Connor *et al.* (1994) developed a pollutant post-processor which is used after running RUNOFF and EXTRAN to model complex hydraulic and pollutant situations. EXTRAN provides the unsteady flow hydraulic modelling capability, while RUNOFF and the post-processor provide the pollutant modelling capability. O'Connor *et al.* (1994) applied the methodology to a CSO problem. **Figure 7** shows a flow chart of the post processor. The program searches the output of RUNOFF for the concentration of total suspended

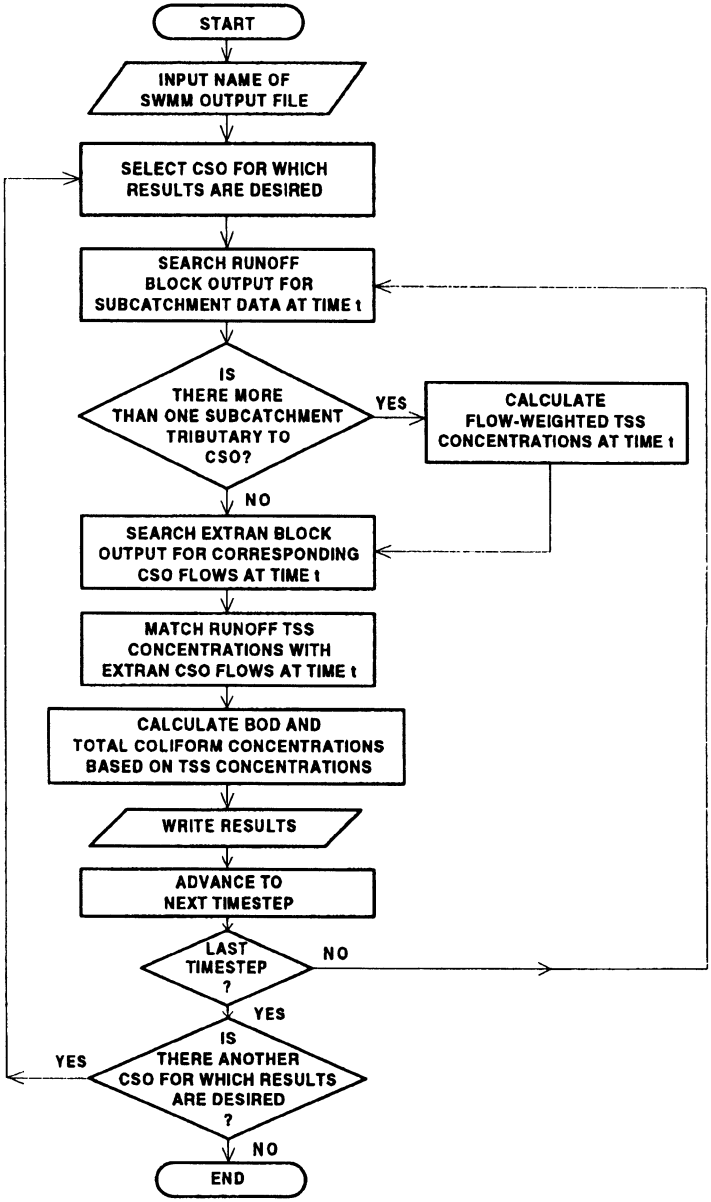


Figure 7. Pollutant post-processor for EXTRAN.
Source : O'Connor *et al.* (1994), reproduced with permission.

solids (TSS) at a particular time step. If there is more than one subcatchment, it flow-weighs the concentrations at that time step. Flow-weighed average concentrations are assigned to matching CSO flows calculated by EXTRAN. BOD and total coliform are calculated based on the TSS concentrations. The empirical equations developed by O'Connor *et al.* are as follows:

$$TSS = \frac{(100mgl/l)DWF + TSS_{SW}\,SWF}{DWF + SWF} \tag{7.20}$$

$$BOD_5 = \frac{(65mg/l)DWF + (5mg/l)SWF}{DWF + SWF} + C_p TSS \tag{7.21}$$

$$Total\ Coliform = \frac{1x10^6\,DWF + 1x10^4\,SWF}{DWF + SWF} + TSS^{3.3} \tag{7.22}$$

where DWF is the dry weather flow (cfs), SWF is the stormwater flow (cfs), BOD_5 is the five-day biochemical oxygen demand (mg/l), TSS_{SW} is the stormwater TSS concentration calculated in RUNOFF (mg/l) and C_p is a coefficient (0.25 to 0.35). It is important to note that the method assumes an instantaneous transport of the pollutants in the sewer system. This is a reasonable assumption if the travel time in the sewer is small compared to the time resolution used to model the water quality in the receiving stream and the time resolution of the field data.

7.3. DATA INPUT

Lines B1, B2 and B3 set some of the run controls such as NTCYC, the number of time steps desired, DELT, the length of the time step (which should be selected sufficiently small to satisfy the Courant condition, 5 to 15 s is typical), TZERO, simulation starting time, NSTART, time step at which output printing starts, and INTER, the number of time steps between print cycles, usually 10. Lines B4 through B8 set the locations where time histories of head and flow are printed. The C1 line sets the conduit data, using one line per conduit. Lines C2 through C4 are concerned with the hydraulic data for natural channels. Line D1 sets the junction data and is repeated for each junction. Line E1 sets the storage junction data and is repeated for each storage junction. The line E2 follows line E1, whenever the storage has a variable area-depth relationship. The F1 line sets the orifice parameters at junction with an orifice. The G1 line is concerned with the weir data and is repeated for each junction containing a weir. Each weir junction must also appear in the D1 line. The H1 line sets up the pump data. Each pump junction must also be entered in the D1 line. The I1 line sets the outfall data. It is repeated for each outfall. Each junction containing an outfall should also be listed on the D1 line. The outfalls with tide gate are listed in the I2 line. Lines J1 through J4 set the boundary conditions, for example, to determine if there is an elevated discharge with no water surface at the outfalls, or to enter the elevation of the water surface or to set the tide values or the stage history of surface elevations. The lines K1 to K# are used for user input hydrographs. More complete data input information can be found in Nix (1994) and full details in Roesner and Aldrich (1989).

8. Sediment Transport

8.1. FLUVIAL SEDIMENT TRANSPORT

Much more is known about sediment transport in rivers than in sewers. Sand bed streams can display different bed forms depending on the flow and sediment characteristics. With increasing discharge these are: ripples, dunes, plane beds, and antidunes. Vanoni (1974) has presented charts that allow the prediction of the type of bed form knowing the Froude Number [$V/(gD)^{1/2}$, where V is the flow velocity and D is the depth] and the water depth to sediment size ratio [D / d_{50}, where d_{50} is the sediment size for which 50% is finer] for fine, medium and coarse sand. Different bed forms result in different flow resistances and different stage-discharge relationships. Shen *et al.* (1990) have shown that it is possible to determine the total resistance by adding the grain resistance and the form resistance.

The forces acting on a single cohesionless sediment particle are: the drag force $F_D = \tau_o \alpha_1 d_e^2$, the lift force $F_L = \tau_o \alpha_2 d_e^2$, and the submerged weight of the particle $W = \alpha_3 g (\rho_s - \rho) d_e^3$, where τ_o is the bed shear stress, d_e is the equivalent grain size, $\alpha_1 d_e^2$ is the bed area per particle, $\alpha_2 = \alpha_1 C_L / C_D$ where C_L and C_D are the lift and drag coefficients, respectively, ρ and ρ_s are the fluid and sediment densities, respectively, and $\alpha_3 d_e^3$ is the volume of the particle. The particle motion is incipient when $F_D / (W - F_L) = \tan\phi$, where ϕ is the angle of repose of the material.

The hydraulic criterion for the beginning of motion of nearly *uniform cohesionless* sediment is thus given by

$$\tau^* = \frac{\tau_o}{g(\rho_s - \rho)d_e} = \frac{\rho U_*^2}{g(\rho_s - \rho)d_e} = \frac{\alpha_3 \tan\phi}{\alpha_1 + \alpha_2 \tan\phi} \tag{8.1}$$

where τ^* is a dimensionless shear stress and $U^* = [\tau_o/\rho]^{1/2}$ is the shear velocity. The second term of equation (8.1) is known as the Shields parameter. It was previously mentioned in sections **4.1** and **6.3**. The Shields' parameter is a function of the particle Reynolds number $R_e = U^* d_e / \nu$ shown graphically in **Figure 2.** A first-order approximation of the Shields' curve is a straight line for which the critical dimensionless shear stress $\tau^* = 0.03$ serves as a conservative criterion of no motion.

For *ripples and dunes*, the critical dimensionless critical shear stress is $\tau^* = 0.06$ (Chabert and Chauvin, 1963). For *nonuniform* cohesionless sediment sand bed streams, Shen and Lu (1983) have shown that the critical dimensionless shear stress is

$$\tau^* = \frac{\tau_0}{g(\rho_s - \rho_f)d_{30}} < 0.028 \tag{8.2}$$

where d_{30} is the sediment size for which 30% is finer. The movement of *cohesive* sediments is much more complex and there is no general criterion to predict incipient motion. The behaviour of cohesive coastal sediments has been described by Mehta (1986).

The sediment load, that is the mass per unit of time, is given by

$$Q_S = \xi\, QC \tag{8.3}$$

where Q is the discharge and C is the concentration of sediments and

$\xi = 2.697 \times 10^{-3}$	for Q in ft^3/s	and C in mg/l	and Q_S in US tons/day
$= 86.4$	m^3/s	kg/m^3	tonnes/day

The *total sediment load* is the sum of the bed load and the suspended load. The *bed load* is the rate of transport of sediment particles that move along the bed by rolling, sliding and saltation. The *suspended load* is the rate of transport of sediment particles that are maintained in suspension by the turbulence of flow. The suspended load is sometimes divided into suspended bed material and wash load. The *wash load* is made of the finer particles that cannot be discerned individually on the bed. The most common equations for the prediction of bed load are those of Einstein, Meyer-Peter and Muller and Bagnold. Ackers and White, and Toffaleti gave relationships to predict the total sediment load. These equations are summarised in Shen and Julien (1993).

8.2. SEDIMENT TRANSPORT IN SEWERS

The previously referred to equations are concerned with noncohesive sediments. This is not the case in sewers where sediment deposits clearly are *cohesive*. However, water quality models usually neglect this fact due to lack of knowledge. Recently, experiments have been performed on the initiation of motion of noncohesive and cohesive sediments in sewers at a number of laboratories.

In the UK sewer sediments have been classified in five categories (Crabtree, 1988):

Type A - Inorganic, coarse granular material
Type B - As type A, but concreted with fats and tars
Type C - Organic, mobile fine grained material
Type D - Organic wall slimes
Type E - Deposits found in tanks.

Types A and C have been identified of primary importance with respect to hydraulic transport and pollution.

8.2.1. *Noncohesive Sediments*

Results at the University of Newcastle upon Tyne (Kleijwegt *et al.* 1990) reveal that Shields criterion for incipience of motion of *noncohesive* sediments is not valid for circular cross sections with flat sediment beds. Particles tend to move at a lower shear stress. The critical shear stress was found to be about 70% of the Shields' value. It was also found that the circular shape causes various distributions of the shear stress over the width of the bed and that one, two or even three local maxima may exist. Experiments made at Delft by Kleijwegt (1993) with noncohesive sediments resulted in the following bed forms:

1. Continuous flat bed
2. Continuous bed with ripples
3. Continuous bed with dunes
4. Isolated bed forms

Dunes and ripples migrate downstream. Antidunes, which are bed forms travelling upstream, were not observed. The flow resistance differs for each type of bed form because of the form drag that has to be added for ripples and dunes. As a result the water level, the energy grade line slope and the sediment transport do not increase monotonically with the discharge but depend on the shape of the bed form. **Figure 8** shows some of the experimental results.

Extensive experimental investigations of noncohesive bed load transport in sewers at the University of Newcastle upon Tyne (Nalluri *et al.* 1994) yielded prediction equations for the limit deposit and for the friction factor. The limiting sediment concentrations ($C_v = Q_s / Q$) for no deposit were established. At the limit deposit condition, with a deposited flat bed, the mean shear stress ($\tau_0 = \rho g R S_0$) was found to be given by

$$\frac{\tau_0}{\rho g \Delta d_{50}} = 0.55 C_v^{0.33} \left(\frac{b}{y_0}\right)^{-0.76} \left(\frac{d_{50}}{D}\right)^{-1.13} (\lambda_s)^{1.22} \tag{8.4}$$

$$\lambda_s = 0.88 C_v^{0.01} \left(\frac{b}{y_0}\right)^{0.03} (\lambda_c)^{0.94} \tag{8.5}$$

where $\Delta = (\rho_s / \rho) - 1$ is the relative density of sediment, d_{50} is the median particle size, C_v is the volumetric concentration, b is the bed width, y_0 is the uniform flow depth, D is the pipe diameter, λ_s is the overall friction factor during transport, and λ_c is the clear water friction factor (**Figure 9**). The limiting velocity, V_s, is then given by

$$\frac{V_s}{\sqrt{g \Delta d_{50}}} = 1.94 C_v^{0.165} \left(\frac{b}{y_0}\right)^{-0.4} \left(\frac{d_{50}}{D}\right)^{-0.57} \lambda_{sb}^{0.10} \tag{8.6}$$

$$\lambda_{sb} = 6.6 \lambda_s^{1.45} \tag{8.7}$$

For application to clean pipe data, substitute $b = 0.5D$ in equation (8.6).

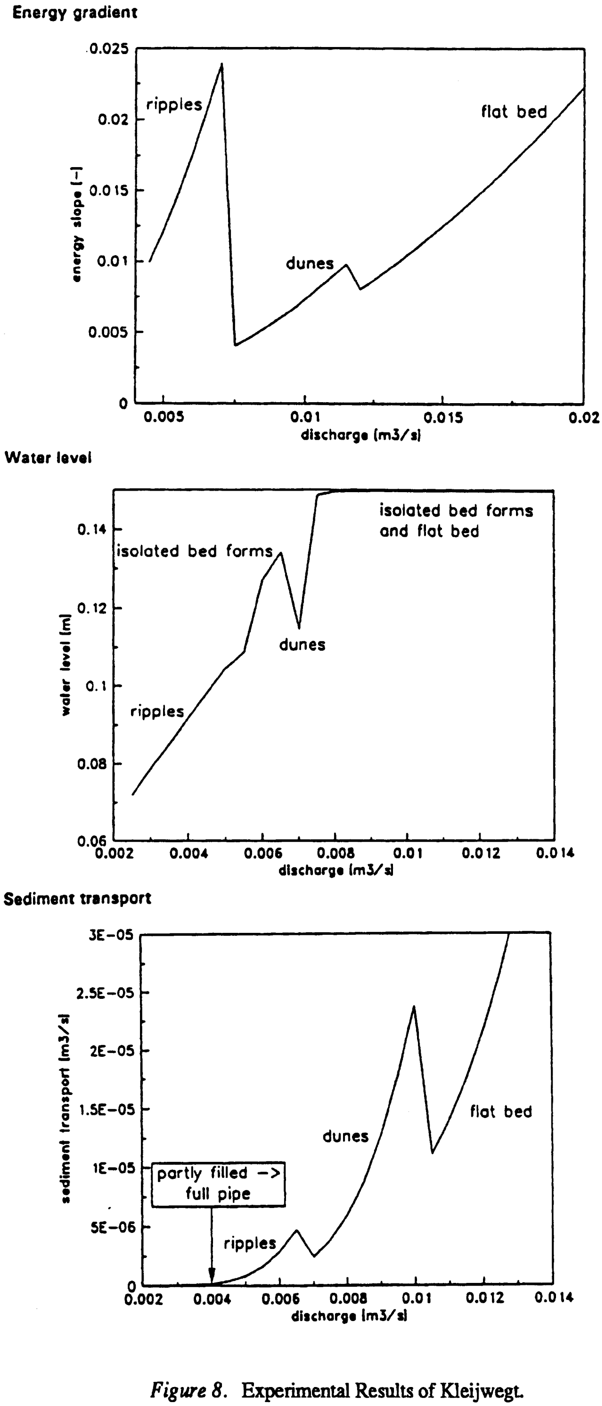


Figure 8. Experimental Results of Kleijwegt.
Source: Kleijwegt (1993), reproduced with permission

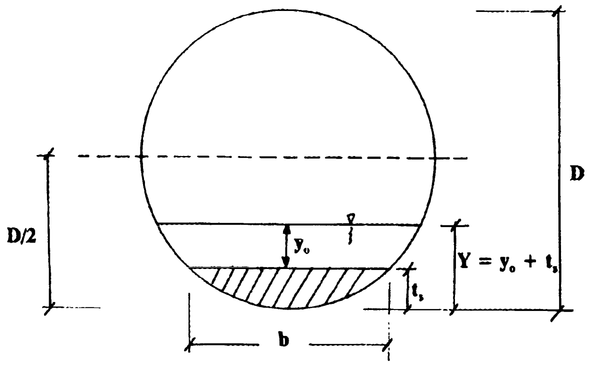


Figure 9. Pipe Cross Section With Deposited Bed.
Source: Nalluri *et al.* (1994), reproduced with permission.

8.2.2. *Cohesive Sediments*

The theory of *cohesive* sediments has been reviewed by Mehta (1991). Using the rate theory, he states that surface erosion is dependent on the excess shear bed stress and the absolute temperature. The dynamics of erosion of cohesive sediment is influenced by the deformation of the bed matrix and by the occurrence of a "stirred" layer which controls the near-bed dynamics. There are three modes of erosion which are not fully independent. In surface erosion, flocs of particles break up from the bed and are suspended in the fluid. The mass erosion is the rapid removal of large pieces of soil at a deeply embedded plane. The re-entrainment of fluid mud is due to the destabilisation of the fluid mud-water interface.

Torfs (1995) uses a critical shear stress criterion for cohesive sediments proposed by Mehta and Dyer (1994), which is an extension of Shields critical shear stress criterion for cohesionless sediments:

$$\frac{\tau_0}{g(\rho_s - \rho)d_e} = \frac{\alpha_3 \tan\phi}{(\alpha_1 + \alpha_2 \tan\phi)} + \frac{F_c \tan\phi / (\alpha_1 + \alpha_2 \tan\phi)}{g(\rho_s - \rho)d_e^3} \tag{8.8}$$

In equation (8.8), the meaning of the symbols is as in (8.1) and F_c represents the net cohesive force. This equation can be rewritten as

$$\frac{\tau_0}{g(\rho_s - \rho)d_e} = \Theta + \frac{\tau_s}{g(\rho_s - \rho)d_e} \tag{8.9}$$

where Θ is the Shields parameter (obtained from Shields' diagram) and τ_s is the shear strength of the material (which can be obtained experimentally).

In experiments with cohesive sediments with various concentrations of clay gel, Nalluri and Alvarez (1992), tested consolidated sediment beds simulating type B sediment consisting of inorganic and coarse granular material concreted with fats and tars (Crabtree, 1988). They obtained the following Shields parameter:

$$\frac{\tau_b}{(\rho_s - \rho) g d_{50}} = 0.964 C_v^{0.457} \left(\frac{d_{50}}{R_b} \right)^{-0.765} \lambda_{sb}^{0.41} \tag{8.10}$$

where $\tau_b = \rho g R_b S_0$ is bed shear stress, R_b is the bed hydraulic radius and λ_{sb} is the bed friction factor with transport. The bed shear stress can be calculated using the side wall elimination technique described in French (1985, pp 178-181).

Experiments with *partly cohesive* sediment mixtures have been performed at several Belgian universities (Torfs *et al.*, 1994, Torfs, 1995). For low contents of fines (0-7% pottery clay, mostly montmorillonite), the sediment transport in a 40 cm diameter circular channel was represented by

$$\Phi = 1.46(\Theta_b' - \Theta_{cr})^{0.96} \tag{8.11}$$

with

$$\Phi = \frac{Q_S}{\sqrt{g(s-1)d_{50}^3}} \qquad \Theta_b' = \frac{\tau_b'}{\rho g (s-1) d_{50}}$$

Transport parameter — Grain mobility parameter

where τ_b' is the grain shear stress, i.e., the difference between the bed shear stress and the bed-form shear stress, Θ_{cr} is the critical mobility number for initiation of sediment transport, Q_S is the sediment discharge per unit width (m^2/s) and $s = \rho_s / \rho$ is the ratio of the sediment and water densities. The bed behaved as a noncohesive material exhibiting ripples and dunes as the discharge was increased. The critical shear stress is shown in **Figure 10.**

For high content of fines (>13% pottery clay), no bed forms were observed and the material was transported primarily as bed load. After the bulk shear strength was reached (Mehta, 1991), the erosion rate increased linearly with the dimensionless excess shear stress (in a 40 cm wide x 40 cm deep rectangular flume):

$$E = 0.0104 \frac{\tau_b - \tau_{cr}}{\tau_{cr}} \tag{8.12}$$

where E is the erosion rate (kg/m^2s) and τ_{cr} is the critical shear stress for erosion. As the erosion proceeded, a groove appeared in the center of the channel similar to that previously observed by Parteniades (1965). Further increase in the velocity resulted in massive erosion and eventual destruction of the bed. For intermediate content of fines (between 7 and 13%), neither of the two equations applies. Some irregular bed forms are exhibited but they are not the usual ripples or dunes. The erosion begins at irregularities in the bed.

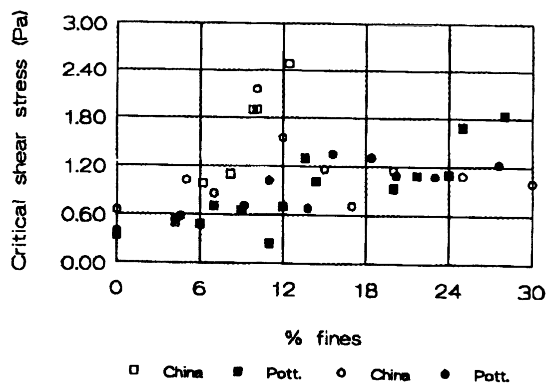


Figure 10. Critical Shear Stress for Incipient Motion. Squares correspond to rectangular section 0.40 m wide by 0.40 m deep, circles correspond to 0.40 m diameter circular cross section. Source: Torfs *et al.* (1994), reproduced with permission.

Mixtures of sand and much finer china clay (Kaolinite) are more cohesive. For less than 3% of Kaolinite in the circular cross section, the sediment rate is well approximated by equation (8.11). For mixtures with more than 3% Kaolinite, the erosion rate was approximated by

$$E = 0.00097(\tau_b - \tau_{cr})^{0.95} \tag{8.13}$$

where the erosion rate is in kg/m^{2}s, the shear stresses are in Pa and the critical shear stress values are shown in **Figure 10.** With Kaolinite, the bed exhibited ripples and dunes only with <3 % Kaolinite. For larger amounts, the bed exhibited a wavy surface. These experiments clearly show that the composition strongly influences the behaviour of the sediments.

Torfs (1995) generalised equation (8.12) as

$$E = E_m\left(\frac{\tau_b - \tau_{cr}}{\tau_{cr}}\right)^{\alpha} \tag{8.14}$$

with the values of E_m and α given in Table 2. Torfs *et al.* (1994) also found that the shape of the cross section affects the erosion rate. Mixtures that could be considered as cohesive in rectangular channels behaved as noncohesive in circular sections. The erosion rates were higher in the circular sections. They conclude that formulas derived for rectangular channels should not be applied to circular cross sections.

TABLE 2. Coefficients for equation (8.14)

Mixture type	E_m *(kg/cm²)*	α	r
Montmorillonite(upper limit)	12.7 x 10^3	1.0	-
Montmorillonite (lower limit)	0.12 x 10^{-3}	1.0	-
Montmorillonite (low density, 6.5%-15.9%)	3.73 x 10-3	1.839	0.86
Montmorillonite (low density, 20%)	0.16 x 10^{-3}	1.181	0.85
Mud1(10.3%), Scheldt River, Belgium, intertidal zone[1]	1.3 x 10^{-3}	1.0	0.99
Mud2 Scheldt R., near Antwerp, Belgium, subtidal zone[2]	2.0 x 10^{-3}	1.0	0.75
Kaolinite	-	-	-
Kaolinite (low density)	1.8 x 10^{-3}	1.0	0.75

1. 33% sand, 79% organic. 2) 15% sand, 10% organic. Clay fraction: 30% smectite, 55% illite, 15% kaolinite.
2. Adapted from Torfs (1995).

8.2.3. *Cohesive sediment erosion prediction model*

Torfs (1995) developed a model to predict erosion rates. It consists of the following steps:

1. Determination of the mixture composition: % fines (< 63 μm) and the amount of clay (<2 μm), grain size and porosity of sand and density profile. Sediments with more than 10 to 20% fines by weight are cohesive.
2. Calculation of the bed shear stress τ_b taking into account the channel geometry. For cohesionless sediments take into account the bed form to calculate the grain shear stress.
3. Calculation of the erosion resistance τ_{cr} from equation (8.8).
4. Comparison of τ_b and τ_{cr}. If $\tau_b > \tau_{cr}$ then erosion occurs.

5a. When the sediment mixture is noncohesive then calculate the erosion rate using the empirical sediment transport equations.

5b. When the mixture is cohesive, determine if surface erosion or massive erosion occurs. Torfs (1995) uses equation (8.14) in both cases with coefficients from Table 2.

6. Calculation of the incremental erosion depth Δ_{ze} for each time step Δt from

$$\Delta z_e = E\Delta t / \rho_{surf} \tag{8.15}$$

   where E is the erosion rate and ρ_{surf} is the bulk density of the surface layer.

7. Return to step 1 for the next sediment layer.

9. MOUSETRAP

MOUSETRAP is a water quality simulator based on the MOUSE package developed by the Danish Hydraulic Institute (see Section 4 and the Appendix of the paper on "Modelling Quality of Urban Runoff"). MOUSETRAP deterministically simulates the transport of sediments and dissolved and attached pollutants, and the water quality processes in sewers. It is subdivided into four modules: Surface Runoff Quality (SRQ), Sediment Transport (ST), Advection - Dispersion (AD) and Water Quality (WQ). The

SRQ module was briefly described in the paper on "Modelling Quality of Urban Runoff".

The ST module has been described by Mark *et al.*(1995). The *sediment transport* modes in sewers are assumed to be the same as in rivers: bed load, suspended load and wash load. The *bed load* transport is controlled by the effective shear stress acting on the sand surface. As the shear stress in sewer flow is derived from the pipe wall and from the sediment bed, a side wall elimination procedure is used to calculate the bed shear stress. ST uses non-cohesive bed load formulas, such as the Ackers-White formula. The choice of formula depends on the scope of the study. The *suspended load* calculation is based on a logarithmic flow velocity profile corrected for the velocity profile distortion in partly full pipes. The *wash load* is treated as in rivers, that is by advection with the flow velocity.

The modelling approaches in ST, as quoted from Mark *et al.*(1995), are

- *"an implicit solution of the continuity equation for bed sediment on non-cohesive transport formulae i.e.: Ackers-White, Englund-Hansen, Englund-Fredsie and van Rijn.*
- *an implicit solution of the advection-dispersion equations with an explicit update of the bed based on a source/sink description for modelling of cohesive sediments.*
- *a formulation of the transport of non-uniform sediment where a range of sediment fraction can be modelled simultaneously on the basis of two models above*
- *a fixed bed model which can be used to give a first indication of the non-cohesive sediment transport capacity in the sewer system."*

The ST module has been verified using data from the Rya catchment in Gothenbug, Sweden and from experiments at Chalmers University of Technology.

The WQ module has been described by Garsdal *et al.*(1995). It includes the degradation of organic matter, bacterial fate, exchange of oxygen with the atmosphere and oxygen demand by eroded sewer sediments. The WQ module is coupled to the AD and ST modules. Thus, the transport of the suspended and dissolved components is calculated concurrently with the calculation of the effects of the biological processes. **Figure 11** shows the biochemical processes that are modelled in relation to DO, and BOD/COD. The current version assumes aerobic conditions.

The degradation of dissolved BOD/COD in biofilm is modelled as a half-order reaction with temperature dependent oxygen diffusion and removal:

$$BOD_{\deg ra.biofilm} = \Theta^{temp-20} \sqrt{2D\, k_{of}}\, DO^{1/2} A_{biofilm} / V \tag{9.1}$$

where Θ is the temperature coefficient, D is the diffusion coefficient of oxygen in water at 20°C (m^2/s), k_{of} is removal of oxygen in biofilm at 20°C (g/m^3s), $A_{biofilm}$ is the area of pipe covered with biofilm, and V is volume of water (m^3).

The degradation of dissolved BOD/COD by suspended heterotrophic organisms is modelled as:

$$BOD_{degra.susp} = K_s \cdot \Theta^{temp-20} \cdot \frac{BOD}{BOD + k_{m,BOD}} \cdot \frac{DO}{DO + k_{m,DO}} \cdot k_b \cdot BOD_{susp} \quad (9.2)$$

where $K_s = \mu_{max} / Y_{max}$ (Max growth rate at 20°C/Max yield constant) (d^{-1}), $k_{m.BOD}$ is the half saturation constant for dissolved BOD(g/m^3), $k_{m,DO}$ is the half-saturation constant for DO (g/m_3) and k_b is the fraction of active heterotrophic organism in BOD_{susp}.

The hydrolysis of suspended matter is modelled as a temperature dependent first order reaction as:

$$BOD_{hydro} = \Theta^{temp-20}\ k_{hl}\ BOD_{susp} \quad (9.3)$$

where k_{hl} is the first order decay constant at 20°C (d^{-1}).

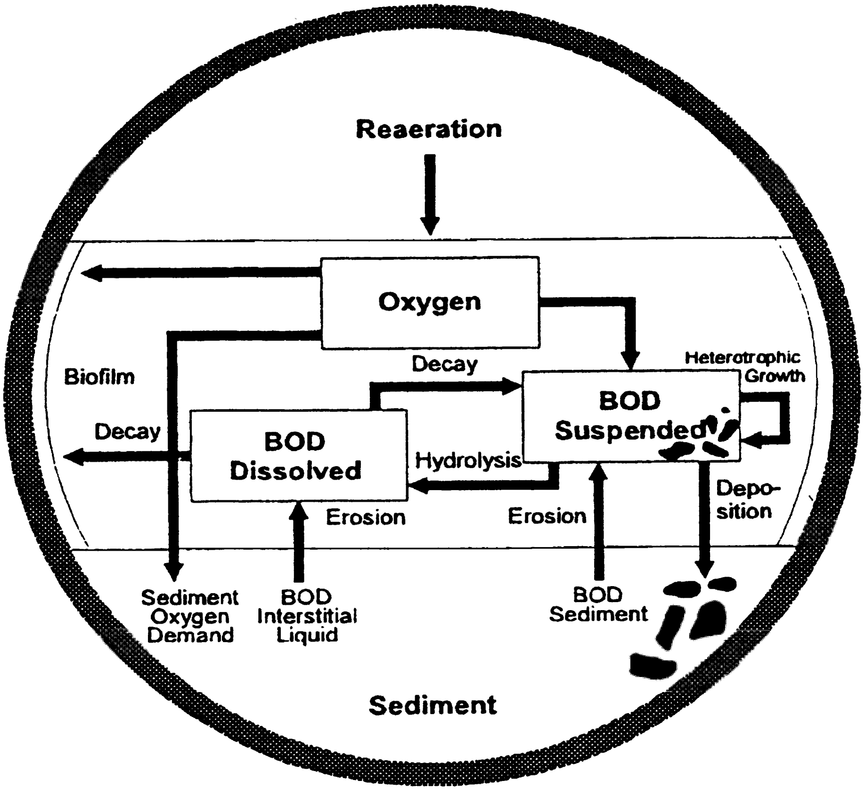


Figure 11. Water Quality Processes modelled in MOUSETRAP.
Source: Garsdal *et al.* (1995), reproduced with permission.

The growth of heterophobics is formulated as a function of the degradation of the BOD in the water phase as:

$$BOD_{growth} = Y_{\max}\ BOD_{\deg ra,susp} \tag{9.4}$$

Finally, the reaeration of the waste water is obtained as the product of the reaeration coefficient and the oxygen deficit:

$$REAR = K1\,[1 + K2\,\frac{u^2}{g\,d_m}]\,(s\,u)^{K3/d_m}\,(C_s - DO) \tag{9.5}$$

where $K1$, $K2$, $K3$ are reaeration constants, u is the flow velocity (m/s), g is the acceleration of gravity (m/s^2), d_m is the hydraulic mean depth and C_s is the oxygen saturation concentration (g DO/m^3).

10. Stochastic Transfer Functions

This section is taken essentially from Delleur and Gyasi-Agyei (1994). There is increasing concern about the sediments transported in urban storm sewers. Progress has been made on the measurement of suspended solids, and telemetry systems have been installed that permit remote access to flow, temperature and suspended solids concentration data. Using observations obtained in the main trunk sewer in Brussels, Belgium, a transfer function model for the prediction of suspended load concentration from temperature and discharge measurements was developed. This model is based on the transfer function methodology developed by Box and Jenkins (1976). It is shown that the transfer function model correctly tracks the suspended solids observations and makes reasonable forecasts. It provides a valid alternative for the determination of suspended solids in urban sewers from discharge and water temperature observations which are more easily measurable on line than suspended solids.

10.1. INTRODUCTION

The sediments in urban storm sewers cause operational costs to the cities and contribute to the pollution load of the receiving streams. The study of the origin, occurrence and behaviour of sediments has been the object of a recent international workshop (Verbanck, 1992) which emphasised the progress in the general understanding of suspended and deposited solids in sewers. In particular, progress has been made in the suspended material measurement. It is now possible to continuously monitor in real time certain parameters such as conductivity, turbidity and temperature of the flow in a sewer system. Vanderborght and Wollast (1990) obtained a linear relationship between a measure of white light absorption and suspended solids concentration. A multi-depth sampling device has recently been developed to observe the stratification of particulate pollutants (Verbanck, 1993). Kelly and Gularte (1981) showed the sensitivity of erosion of cohesive sediments to temperature. They conducted their tests with remolded grundite in a recirculating water tunnel with controlled temperature.

A method is presented for the prediction of the suspended solids concentration from observations of the flow rate and the temperature using transfer function techniques. Although the bed shear stress is the most appropriate parameter for defining erosive or depositional criteria, this parameter is not commonly available. The discharge is used instead as an index of the suspended sediment transport capacity of the sewer. The water temperature is used as an index of the viscosity, which in turn affects the Reynolds number of the flow and hence its sediment transport characteristics. A similar transfer function methodology has been used, for example by Lembke (1991), to predict suspended sediment concentration in rivers from streamflow and precipitation observations.

10.2. DATA

The data were obtained for the combined sewer system of the City of Brussels, Belgium. Brussels is located in the Senne River Basin, a subtributary of the Scheld. The Senne crosses the city from South to North with a mean annual flow of 12 m^3/s. The Senne and its tributaries form an important part of the drainage and wastewater collection system of the City of Brussels. The slope of the Senne is very flat, approximately half a metre per kilometre; its left bank tributaries are also very flat but its right bank tributaries, in general, have steep slopes with a maximum change in elevation of approximately 70 metres.

The data used in this study pertain to the Brussels main trunk sewer, located in the centre of the city. It is a combined sewer system draining an area of 3520 ha of which 60% is impervious. There are 11 overflow points and 8 storage reservoirs with a capacity of about 100,000 m^3. The receiving streams are the Senne River and the Maritime Canal. The sewer system is equipped with a computerised telemetric system. The sewer system serves approximately 400,000 people. The dry weather flow is of the order of 100,000 m^3/day or 1.2m^3/s, the infiltration flow is approximately 40% of the dry weather flow. The dry weather flow exhibits a diurnal variation with a minimum of about 0.8 m^3/s at 05:00 hrs and a maximum of about 1.6 m^3/s at noon (Bauwens, 1992).

The data set consists of three time series at 30 min intervals: the water temperature in °C (input 1), the sewer discharge in m^3/s (input 2) and the suspended solids in g/L (output) measured near the sewer outfall. The total length of the series was 200 time steps, but only 180 observations were used to estimate the parameters and the remaining 20 were used to prepare forecasts from time step 180. **Figure 12** shows the evolution of the water temperature and the discharge.

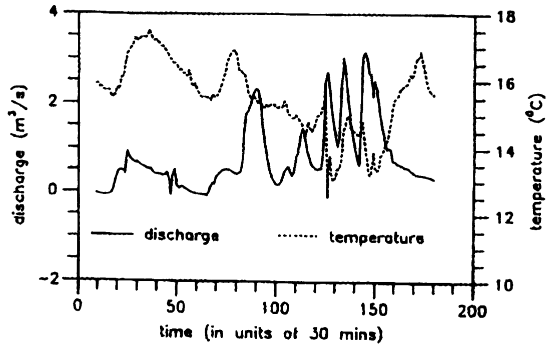


Figure 12. Observed discharge and temperature.
Source: Delleur and Gyasi-Agyei (1994)
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10.3. TRANSFER FUNCTION MODEL

For single input, the general form of the transfer function model is

$$Y_t = v(B)X_{t-b} + N_t \tag{10.1}$$

where Y_t is the output series at time t, $v(B) = v_o + v_1B + v_2B^2 + \ldots$ where B is the backshift operator such that $B^sX_t = X_{t-s}$ and X_{t-b} is the input time series delayed by b time steps, N_t is the disturbance or noise of the process. Without N_t equation (10.1) is of the same type as the unit hydrograph, where Y_y is the direct runoff at time t, X_{t-b-s} is the antecedent rainfall excess at time $t-b-s$ and the $v_o, v_1, v_2, \ldots$ are the unit hydrograph ordinates, or the impulse function responses weights. Equation (1) is thus a shorthand for

$$Y_t = v_o X_{t-b} + v_1 X_{t-b-1} + v_2 X_{t-b-2} + \cdots + N_t \tag{10.2}$$

Equation (10.1) is overparametrised and a representation requiring fewer parameters is obtained by expressing the impulsive response as the ratio of two polynomials (Box and Jenkins, 1976)

$$v(b) = \frac{W(B)}{\delta\,(B)} \tag{10.3}$$

where $W(B) = (W_o - W_1B - \ldots -W_sB^s)$ and $\delta\,(B) = (1 - \delta_1 B - \ldots \delta_r B_r)$ in which s and r are the orders of the polynomials. The disturbance N_t in general is not a white noise, and can be represented by an autoregressive-moving average, ARMA (p,q) process, (Box and Jenkins, 1976).

$$\phi(b)N_t = \theta(B)A_t \qquad \text{or} \qquad N_t = \frac{\theta(B)}{\phi(B)}a_t \tag{10.4}$$

where $\phi(B) = (1 - \phi_1 B - \phi_2 B^2 - \ldots - \phi_p B^p)$ and $\theta(B) = (1 - \theta_1 B - \theta_2 B^2 - \ldots \theta_q B^q)$ and a_t is a white noise, p and q are the orders of the autoregressive and moving average components, respectively. The ARMA (p,q) model (3) is rewritten as

$$N_t = \phi_1 N_{t-1} + \phi_2 N_{t-2} + \cdots + \phi_p N_{t-p} + a_t - \theta_1 a_{t-1} - \cdots - \theta_q a_{t-q} \tag{10.5}$$

The final transfer function model obtained by combining (10.3) and (10.4) becomes

$$Y_t = \frac{W_1(B)}{\delta(B)} X_{t-b} + \frac{\theta(B)}{\phi(B)} a_t \tag{10.6}$$

For two inputs X_1 and X_2, the model is generalised as

$$Y_t = \frac{W_{1(B)}}{\delta_1(B)} X_{1,t-b} + \frac{W_2(B)}{\delta_2(B)} X_{2,t-b} + \frac{\phi(B)}{\theta(B)} a_t \tag{10.7}$$

Typically the flow series, Z_t, is highly skewed and a transformation is used to normalise it. A logarithmic transformation is often used. A more general Box-Cox (1964) transformation given by

$$X_t = \frac{Z_t^\lambda}{\lambda} \quad , \lambda \neq 0 \qquad \text{or} \qquad X_t = \log Z_t \qquad \lambda = 0 \tag{10.8}$$

can be used. This transformation was used for the flows, but the temperature and suspended sediments did not require transformation as their skewness coefficient was less than 0.05.

10.3.1. *Model Identification*

Using the identification method of Liu and Hanssens (1982) and the SCA software (Hudak and Liu, 1989), the time delay b was found to be zero and the numerator polynomials were identified to be of order 2 and 3 for water temperature and discharge, respectively, and the denominator polynomials of order 1 in both input series. For the error N_t, the autocorrelation and the partial autocorrelation functions identified an AR(1) model. **Figure 13** shows the cross correlations between inputs and output and the 95% confidence interval. It is seen that the cross correlation of the discharge is more significant than that of the temperature. The discharge-suspended solids cross correlation exhibits a significant positive peak at the origin, indicating the importance of the discharge and confirming the zero time delay.

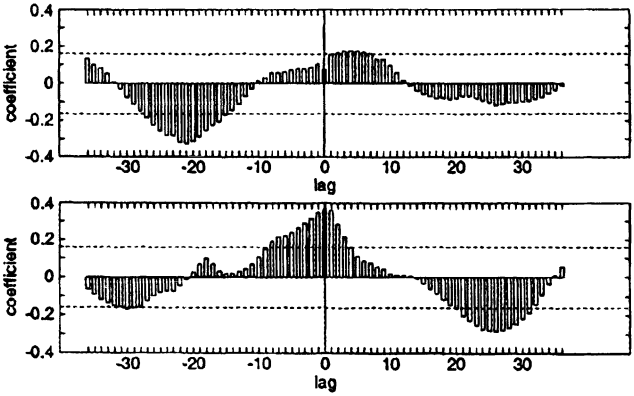

Figure 13. Cross correlations between temperature and sediment Concentration (top) and between discharge and sediment concentration (bottom). Source: Delleur and Gyasi-Agyei (1994), reproduced with permission.

The cross correlation between temperature and suspended solids is somewhat weaker than that between discharge and suspended solids. This slight asymmetry in the temperature-suspended solids cross correlation indicates that the suspended solids tend to lag the temperature [note that $\rho_{yx}(k)$ in SCA is the same as $\rho_{xy}(k)$ in Box-Jenkins, 1976)].

10.3.2. *Estimation of the Transfer Function Model*

The combined transfer function and noise models identified above were tested. Insignificant parameters were removed one at a time and for each model diagnostic checks were performed. The final model selected is

$$Y_t = \frac{W_{1,0} + W_{1,1}B}{1-\delta B} X_{1,t} + W_{2,0} X_{2,t} + \frac{1}{1-\phi B} a_t \tag{10.9}$$

where Y_t is the sediment concentration in g/L, $X_{1,t}$ is the water temperature in °C, and $X_{2,t}$ is the discharge in m^3/s, after a Box-Cox transformation with $\lambda = 0.9$. The parameters obtained with the SCA software are listed in Table 3. The expansion of equation (10.9) and the introduction of the parameter values from Table 3 yield

$$Y_t = 1.5423\, Y_{t-1} - 0.5795\, Y_{t-2} - 0.1657\, X_{1,t} + 0.3198\, X_{1,t-1} - 0.1535\, X_{1,t-2}$$

$$+ 0.0718\, X_{2,t} - 0.1107\, X_{2,t-1} + 0.0416\, X_{2,t-2} + a_t - 0.6478\, a_{t-1} \tag{10.10}$$

TABLE 3. Parameters values of the selected model

Parameter	$W_{1,0}$	$W_{1,1}$	$W_{2,0}$	δ	ϕ
Value	-0.1657	0.1716	0.0718	0.6478	0.8945
Stand.Error	0.0157	0.0152	0.0200	0.0903	0.0343
T-Value	10.55	11.31	3.60	7.17	26.04

The residuals a_t are approximately normally distributed with zero mean and variance of 0.0048. The observed sediment concentration and the fitted values (one step ahead forecasts) are plotted in **Figure 14.** It shows that the model tracks the observations correctly. The twenty forecasts made from time step 180 are shown in **Figure 15.** The forecasted and observed series are generally in good agreement and all the observed values lie within the 95% confidence interval of the forecasts.

The transfer function model provides a valid alternative for the determination of suspended solids in urban sewers from discharge and water temperature observations which are more easily measurable on line than suspended solids. However it is recommended that the model be tested on more extensive data sets before actual field implementation.

10.3.3. *Other Applications*

Capodaglio (1994) used the same techniques to predict the one day ahead daily flows at Fusina (Venice, Italy) and Green Bay (Wisconsin, USA) sewage treatment plants from respective daily rainfall time series. Similar results were obtained for the first set of data by Zheng and Novotny (1991). Tan *et al.* (1991) used a multiple input / single output model to predict dry weather and wet weather sewer flows in Melbourne, Australia. The parameters were estimated recursively.

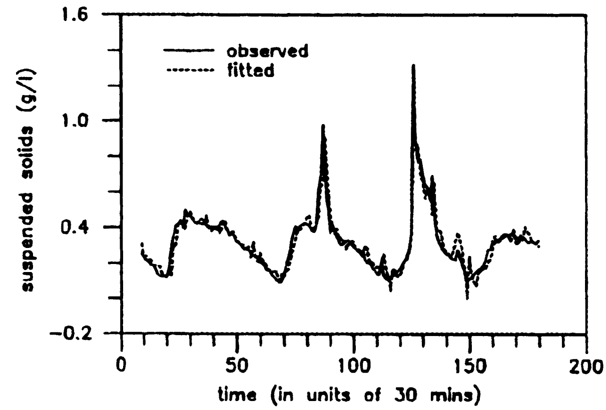

Figure 14. Observed and fitted suspended solids concentration.
Source: Delleur and Gyasi-Agyei (1994). Reproduced with permission.

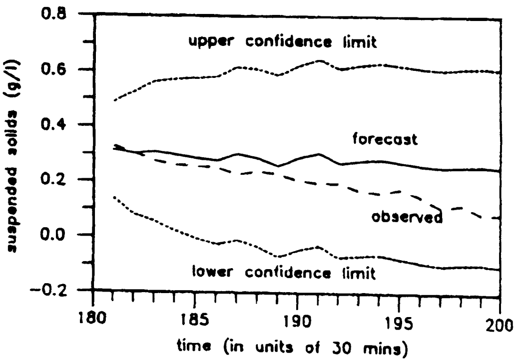

Figure 15. Forecast and measured suspended solids.
Source: Delleur and Gyasi-Agyei (1994). Reproduced with permission.

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OPTIMISATION MODELS FOR URBAN RUNOFF CONTROL PLANNING

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1. Introduction

Given the significance of urban runoff impacts on many receiving waters and the massive costs of future investments in drainage infrastructures, the design of urban runoff quantity and quality control systems should be cost-effective. Achieving cost-effective design requires that the entire range of runoff control alternatives be investigated in the planning stage so that the most cost-effective runoff control systems can be identified for the design level analysis.

In order to analyse the runoff control performance of different runoff control alternatives in the planning stage, efficient screening and optimisation models are acutely needed. Since early 1980's, researchers at the University of Toronto have developed analytical probabilistic models [1,15] that provide planning estimates of runoff control performance. The probabilistic models are closed-formed solutions of the control performance equations that are highly efficient in both a conceptual and computational sense. Optimisation models [15] have also been developed to determine the cost-effective combinations of control measures for achieving various levels of control. These models allow engineers to develop the cost-effectiveness curve for achieving various levels of runoff control.

This cost-effective design approach has been demonstrated in the combined sewer overflow and storm runoff control studies for the Ontario Great Lakes Areas of Concern (AOCs), sponsored by Environment Canada [5,6]. In the combined sewer overflow studies, the cost-effectiveness curves of centralised and satellite treatment systems were developed and compared. In the stormwater control studies, the cost-effectiveness curves of storm runoff control were developed for each AOCs.

This paper reviews the modelling concept of the analytic probabilistic models and focuses on the optimisation models for runoff control planning.

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2. Literature Review

Howard [11] and Flatt and Howard [8] introduced a theory to analyse the performance of storage/treatment systems for controlling combined sewer overflows. By assuming rainfall event volume and interevent time to be independently and exponentially distributed, they derived the probability distribution of overflow volume so that the frequency of overflows and the average magnitude of overflow volume could be estimated. However, their analytical model does not account for the random nature of storage level in the downstream storage reservoir.

Di Toro and Small [7] modelled runoff characteristics (flow, duration, and interevent time) using independent gamma probability distributions. These, in turn, were routed through a storage and treatment system and transformed into a long-term fraction of runoff loads controlled by integrating the joint probability distribution of runoff duration and runoff flow. The long-term fraction of runoff loads controlled could not be solved analytically and required numerical evaluation. However, the probability distributions of runoff characteristics are usually difficult to determine as long-term runoff records are seldom available.

Other contributions to the probabilistic modelling of runoff control systems include Chan and Bras [4], Loganathan and Delleur [16], and Segarra-Garcia and Loganathan [21].

Sullivan *et al.* [22] incorporated cost and performance analyses in the assessment of the magnitude and significance of pollution loadings in urban runoff from Ontario cities. Heaney *et al.* [9], Nix *et al.* [18], and Nix and Heaney [19] also employed the production theory of microeconomics to determine the least-cost combinations of storage-release strategies.

3. Analytic Probabilistic Models for Runoff Control Performance Analysis

3.1. DERIVED PROBABILITY DISTRIBUTION THEORY

The analytical probabilistic models [1,15] were developed by derived probability distribution theory in which the probability density functions (pdf's) of input rainfall characteristics are transformed into the pdf's of output system performances.

The basis of derived probability distribution theory is that the probability distribution of a dependent variable is fundamentally related to, and may be derived from, those of the independent random variables using the functional relationship between dependent and independent variables [3]. For instance, if a random variable Z is a function of two random variables, X and Y, given by

$$Z = g(X,Y) \tag{1}$$

the pdf of Z $(f_Z(z))$ can be derived by integrating the joint density function of X and Y $(f_{X,Y}(x,y))$ and differentiating with respect to Z as follows:

$$F_Z(z)=\int_{-\infty}^{+\infty}\int_{-\infty}^{+z} f_{X,Y}\,(g^{-1}(z,y),y)\,|\frac{\partial g^{-1}}{\partial z}|dzdy \tag{2}$$

$$f_Z(z)=\frac{\partial(F_Z(z))}{\partial z}=\int_{-\infty}^{+\infty} f_{X,Y}\,(g^{-1}(z,y),y)\,|\frac{\partial g^{-1}}{\partial z}|dy \tag{3}$$

where $F_Z(z)$ is the cumulative distribution function of Z, $f_Z(z)$ is the probability density function of Z, and $f_{X,Y}(g^{-1}(z,y),y)$ is the joint density function of X and Y.

Success of the above derivation is dependent upon the determination and integration of the joint probability distribution of the independent variables. Thus, simplifying assumptions regarding the joint probability function of the independent variables may be needed to derive a closed form solution.

3.2. MODELLING CONCEPT OF THE ANALYTIC PROBABILISTIC MODELS

A typical layout of a combined or storm sewer system is illustrated in Figure 1. Part of the rainfall becomes runoff that is finally collected at a potential downstream storage site. If the storage capacity (S) and the runoff treated at a constant rate (Ω) over the rainfall event is less than the runoff volume, the excess runoff volume overflows uncontrolled to the receiving water.

The rainfall-runoff process is modelled by a simple runoff coefficient method in which initial rainfall fills the depression storage (S_d) and a portion of subsequent rainfall (ϕ) becomes runoff from the catchment. Adams *et al.* [2] analysed long-term rainfall records across Canada and found that rainfall characteristics such as rainfall event volume (v), duration (t), and interevent time (b) could be described by pdf's which are exponentially distributed. With the exponentially distributed rainfall event volume, the pdf of runoff event volume can be derived using the derived probability distribution theory.

Long-term quantity and quality control performance of a storage-treatment system in Figure 1 can be specified by: (1) the average annual percent of runoff volume controlled (C_r) and (2) the average annual number of overflows (N_s). Both C_r and N_s can be determined after the pdf of an overflow event from the storage-treatment system has been derived. The overflow volume is determined by the amount of runoff and the storage contents within the storage reservoir.

Volume balance relationships of rainfall-runoff-overflow can be derived between the rainfall characteristics (v, b, t) and the catchment and drainage system characteristics (ϕ S_d, S, Ω). These relationships are then transformed onto the joint probability space of v, b, t which are assumed to be independent. The probability of overflow per rainfall event can then be derived by integrating the joint probability space of v, b, t defined by the volume balance relationships. The control system performance measures, C_r and N_s are then derived in terms of rainfall, catchment, and drainage system characteristics (v, b, t, ϕ S_d, S, Ω) and are given by

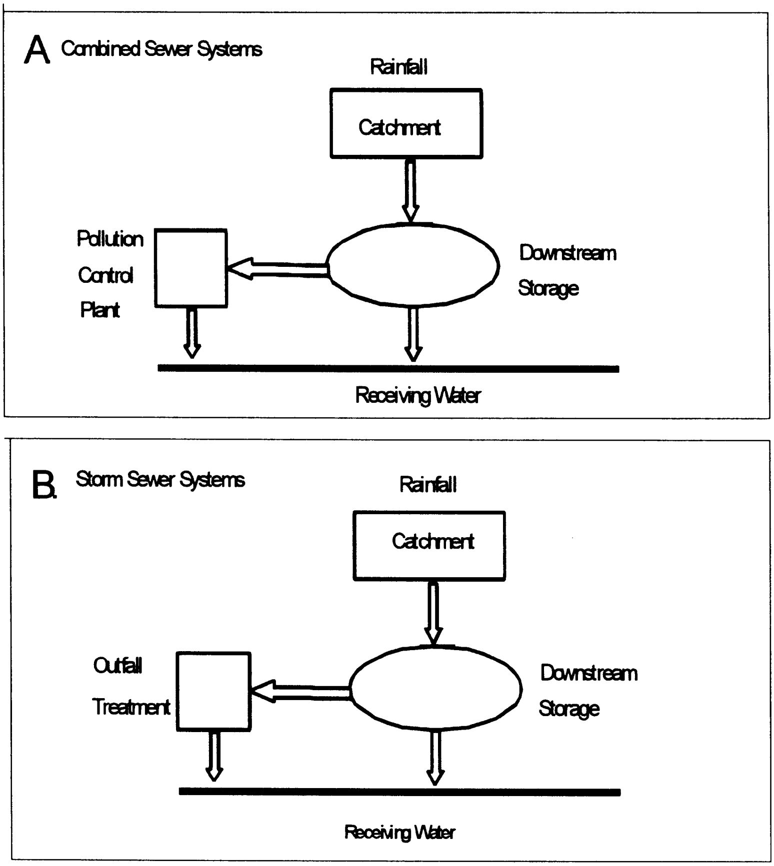

Figure 1. Typical layout of combined and storm sewer systems.

$$C_r = 100\%\left[1 - G_P(0)e^{\zeta S_d}\right] \tag{4}$$

$$N_s = \theta\, G_P(0) \tag{5}$$

where

$$G_P(0) = \left[\frac{\dfrac{\lambda}{\Omega}}{\dfrac{\lambda}{\Omega}+\dfrac{\zeta}{\phi}}\right]\left[\frac{\dfrac{\Psi}{\Omega}+\dfrac{\zeta}{\phi}e^{-(\frac{\Psi}{\Omega}+\frac{\zeta}{\phi})S}}{\dfrac{\Psi}{\Omega}+\dfrac{\zeta}{\phi}}\right]e^{-\zeta S_d} \tag{6}$$

and λ is the reciprocal of the mean t, Ψ is the reciprocal of the mean b, and ζ is the reciprocal of the mean v, and θ is the average annual number of rainfall events. The depression storage S_d can be used to model upstream storage and runoff coefficient ϕ can be used to model upstream runoff reduction by such a measure as an infiltration facility. The storage capacity S can be used to model downstream storage reservoir volume and the controlled outflow rate Ω is used to model satellite or centralised treatment systems. The control performance measures, C_r and N_s, estimated by these analytic probabilistic models, had been compared favourably with those simulated by the continuous STORM model [12,13].

4. Optimisation Models for Urban Runoff Control Planning

Traditionally, cost-effectiveness analysis of urban runoff control at the planning stage has involved comparison of the cost of a few selected alternatives and determination of the least-cost alternative that achieves the design performance. With the development of the analytic probabilistic models, the least-cost combination of control measures that achieves the design performance can be determined by a constrained cost minimisation procedure.

4.1. PRODUCTION THEORY

In microeconomic theory, the production theory of a firm relates the quantity of output as a function of the quantities of variable inputs. A rational firm will maximise the quantity of output for a given cost level or minimise the cost of producing a required quantity of output. Combinations of input levels which are used to produce a given output level can be summarised by a production isoquant. With knowledge of the input cost functions, the least-cost mix of inputs that achieve the required production level is determined as illustrated in Figure 2. The tangency point between the isoquant and isocost curve is the least-cost mix of inputs which achieves the required isoquant level. The line connecting these tangency points is the expansion path of production. A cost-effectiveness curve can be developed from the expansion path by plotting the least-costs to achieve various levels of production [10].

Production theory can be applied to the planning of urban runoff control systems. Inputs to the production of runoff control are the different types of control measures such as storage and treatment facilities while outputs are the control system performance measures such as C_r and N_s. Combinations of control measures that can provide a certain level of runoff control can be summarised by a performance isoquant, while combinations of control measures that require the same total cost can be summarised by an isocost curve. The tangency point between the performance isoquant and isocost curve represents the combination of control measures at which the rate of technical substitution is equal to the rate of transformation between control measures. The required performance is then satisfied. The corner points represent the combinations of control measures in which one of the control measures is constrained due to existing conditions of catchment and/or drainage system. For convex isoquants, the least-cost mix of control measures is the tangency point between the isoquant and the isocost curve. By determining the least-cost combinations of control measures that achieve various levels of control system performance, a cost-effectiveness curve of runoff control can be developed.

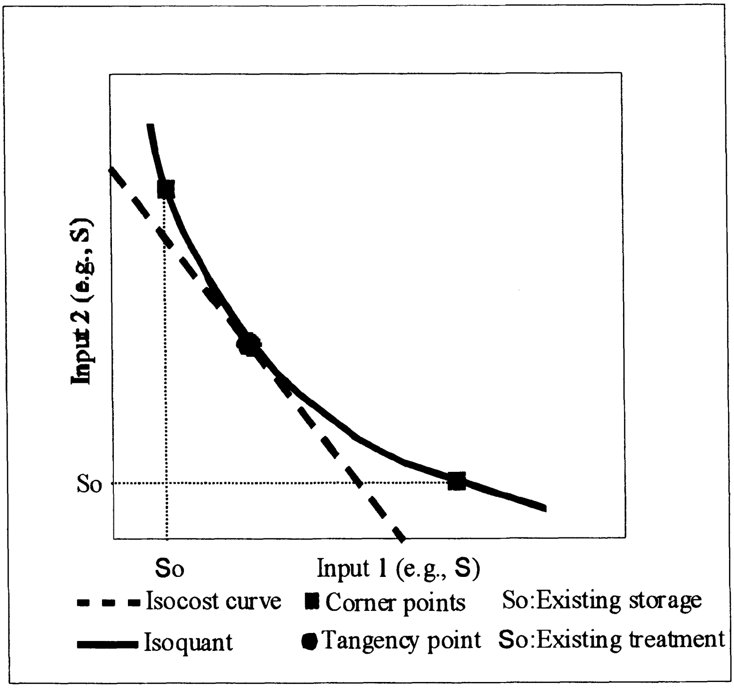

Figure 2. Illustration of the determination of least-cost combination of control measures.

4.2. CONSTRAINT COST MINIMISATION

A constrained cost minimisation technique is employed to determine the least-cost mix of control measures that can achieve a certain level of runoff control. The formulation of the least-cost analysis is as follows:

$$\text{minimise } C_T = f_c\,[X_i,\ldots,X_n] \;;\; i = 1,\ldots,n \tag{7}$$

$$\text{subject to } Y_k\,[X_i,\ldots,X_n] \le Y_{ko} \;;\; k=1,\ldots,m \tag{8}$$

in which C_T is the total cost of providing the required control measures; $f_c\,[X_i,\ldots,X_n]$ is the cost function of providing n types of control measures (e.g., S, Ω); n is the total number of feasible control measures; $Y_k[X_i,\ldots,X_n]$ is the control performance measure k (e.g., C_r) as a function of control measures X_n; Y_{ko} is the required control performance measure k; X_i is the control measure i; m is the total number of required control performance measures.

The above minimisation problem can be solved by employing the Lagrange multiplier method. The objective function is combined with the constraint equation(s) to form the Lagrange function Z given by

$$Z = f_c(X_i,\ldots,X_n) + \sum_{k=1}^{m} E_k(Y_k(X_i,\ldots,X_n) - Y_{ko}) \tag{9}$$

in which E_k is the kth Lagrange multiplier or the marginal cost of an additional unit of Y_{ko} at the optimal mix of control measures. The Z function is then differentiated with respect to each types of control measures X_i and E_k to form a system of algebraic equations as follows:

$$\frac{\partial Z}{\partial X_i} = 0 \quad ; \quad i = 1,\ldots,n \tag{10}$$

$$\frac{\partial Z}{\partial E_k} = 0 \quad ; \quad k = 1,\ldots,m \tag{11}$$

Numerical algorithms such as the Newton Raphson methods can be applied to solve for the unknowns X_i and E_k. The solution to this set of equations is the tangency point between the performance isoquant and isocost curve. Since the performance isoquants of C_r and N_s possess a shape convex toward the origin, the tangency point is the least-cost combination of control measures.

5. Demonstration of Cost-Effectiveness Analysis

5.1. CASE STUDY SITE

A catchment in the Barrington area of the City of East York, Ontario is used to demonstrate the cost-effective analysis. The study area consists entirely of single family residential housing served by a storm sewer system. Its area is 17.4 hectares and the estimated percent of imperviousness is 60%. A tipping bucket rain gauge was installed by the East York Engineering Department during 1974-75 and the rainfall data recorded during the field study period were found to be comparable to the closest permanent rain gauge located at the Toronto Bloor Street. Stormwater quantity and quality data were recorded in this catchment by the Ministry of Environment [17,14]. The purpose of this case study is to determine the most promising combinations of control measures that achieve the long term control performance measures C_r and N_s.

5.2. OPTIMISATION OF DIFFERENT MIXES OF CONTROL MEASURES

Forty-three years of continuous rainfall record at the Toronto Bloor Street gauging station were first analysed to determine the probability density functions and statistics of rainfall characteristics such as storm event volume, duration, interevent time, and average intensity. It was found that exponential distribution could be used to describe these characteristics.

Four different types of control measures, namely upstream and downstream storage (S_d and S), runoff reduction facilities (ϕ) such as infiltration facilities, and downstream treatment rate (Ω), were investigated. There are many combinations of these four types of control measures that can achieve the required performance measures. The combinations of upstream runoff reduction (ϕ), downstream storage volume (S) and downstream treatment rate (Ω) are examined in this chapter. Other combinations of control measures had also been investigated [15]. The cost functions for downstream detention basins (S), microscreening facilities (Ω) and upstream runoff reduction facilities (ϕ) were estimated from the reported cost functions [20,23].

Quantity performance is modelled by C_r which ranges from 10% to 98%, while quality performance is modelled by N_s which ranges from 1 to 50 spills per year. The total costs of providing C_r using different combinations of control measures are shown in Figure 3. Similar plots for N_s were also developed [15]. It is noted that the total cost increases rapidly when C_r is higher than about 70%. For instance, the total cost to provide 98% runoff control is about \$3.0 per square metre of catchment. The least-cost combination of S and Ω to provide this performance is 6.6 mm and 1.2 mm/hr, respectively. Other least-cost combinations of control measures are also determined and compared with those of S, Ω, and ϕ. The most promising combinations of control measures for achieving the C_r control requirement are the S, Ω, and ϕ combination and S and Ω combination.

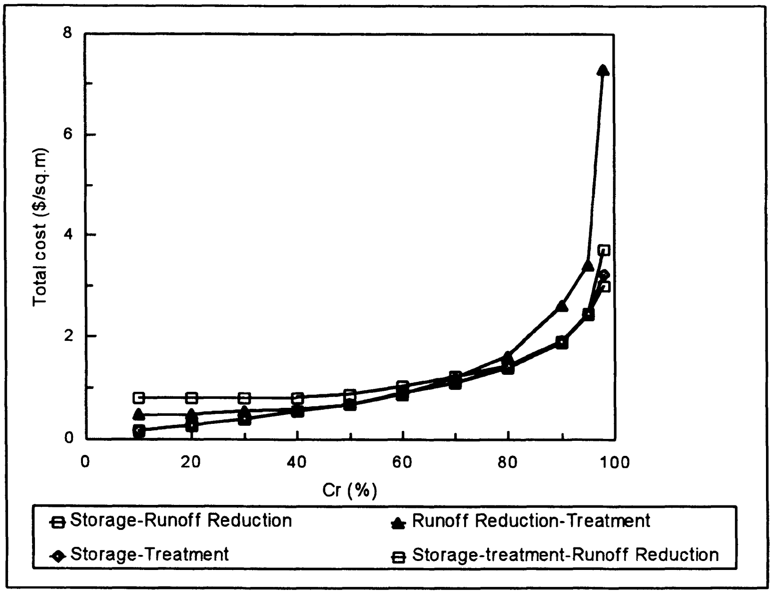

Figure 3. Comparison of total cost-effectiveness curves for achieving C_r among various combinations of control measures.

6. Conclusions

With the development of analytical probabilistic models, preliminary prediction of urban runoff control system performance can be efficient. Although the assumptions of the analytical models may not be perfectly satisfied in every application, these models may still be useful for preliminary evaluation of urban runoff control system alternatives.

Least-cost analysis of urban runoff control systems provides important information such as cost-effectiveness relationships and expansion paths for achieving various levels of runoff control. This information is useful for specification of the target design performance and subsequent design level analyses of control system alternatives. Since cost and performance of control systems are explicitly taken into consideration, the runoff control planning extends conventional engineering analysis, which emphasises an approximate performance analysis of runoff control with rather arbitrary design levels.

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MODELLING WASTEWATER TREATMENT PLANTS

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1. Introduction

Sewerage systems have become more complex and difficult to operate due to development and changes in urbanised areas, and more stringent quality requirements for effluents discharged to the environment. These trends have rapidly raised the investment, operation and maintenance costs of wastewater treatment plants. The minimisation of costs of treatment processes must be one of the main objectives in wastewater systems planning and management. It is not an easy task as treatment systems are very complex.

One way of searching for feasible solutions is to simulate the system with mathematical models. These models are representations of the knowledge we have about the system and help in decision making by integrating and processing the existing information in such a way that it becomes easier to draw the conclusions needed. If we can build a model that is a good representation of the real system, we can use it to conduct experiments that would otherwise be prohibitively expensive or even impossible to carry out. We can run the model under conditions that would be harmful or dangerous in the real system. These models can serve as "prototypes" of the systems we need to analyse. By exploring the prototype, we can gain better understanding of the real system.

Development of a mathematical model involves compromises to balance conflicting needs. On the one hand, a model must incorporate major processes occurring within a system in a manner that is consistent with established knowledge about the system. On the other hand, model equations must be solvable with a reasonable degree of effort (computational time). Difficulty of solutions increases markedly as the number of processes increases. Fortunately, observed progress in computers makes this problem less critical. A modeller should include only those processes that are essential to a realistic solution and must select process rate expressions that allow the use of simplified solution techniques without detracting from the applicability of results.

In wastewater engineering, there has been rapid progress in development of models for the processes used in a typical municipal treatment plant [1]. The conceptual understanding of processes has expanded sufficiently to enable formulation of physically

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based models that seek to account for major processes occurring within the treatment plants. These models are powerful tools because they allow extrapolation of the design space beyond that experienced in a physical model. In this way, many potentially feasible solutions may be evaluated quickly and relatively inexpensively, thereby allowing only the more promising ones to be selected for actual testing in a physical model. Dynamic nature of the mathematical models allows the inclusion of unsteady conditions that occur in the plant as a result of the loads continually changing over time scales comparable to the plant residence time. The design of new or modified plants as well as the operation and control of existing wastewater treatment plants can be greatly facilitated by computer simulations. Simulations allow exploration of alternative wastewater treatment system configurations, inputs, and operational strategies, and expand engineer's experience and decision-making abilities. Once calibrated to a particular wastewater treatment system, a model allows the engineer to quickly screen many potential designs and eliminate those that are inefficient from either a process or an economic perspective. Furthermore, once a system configuration is selected, a model allows individual units to be sized in such a way as to minimise total system costs. Finally, after a plant is constructed, a model can be used to investigate alternative operational strategies to minimise the impact of new waste loads.

2. Activated Sludge Plant

Activated sludge plant usually consists of three main components, namely, a primary settler (which can be omitted), an activated sludge bioreactor (BR) and a secondary settling tank (SST). Each of them can be successfully modelled. The submodels of the individual elements together with the model of inputs (influent flow rate, concentrations) comprise a model of the treatment plant. Only carbonaceous and nitrogenous pollutants are of concern in this paper. Phosphorus removal processes are beyond its scope and can be found elsewhere [2]. Screens and grit chambers do not contribute much to the input modifications, and can be usually neglected in modelling.

2.1. INPUT MODEL

Concentrations of pollutants in wastewater are structured according to their nature (organics and nitrogen) and transformations they undergo in treatment processes. The items of the structure are then state variables in these process models.

Chemical oxygen demand (COD) is a superior measure of organic material in wastewater for modelling purposes. It provides a link between electron equivalents in organic substrate, biomass and oxygen utilised by the biomass. The organic matter in wastewater can be subdivided into four categories [3]:

- inert soluble organics (S_I, mg COD/L)
- inert particulate organics (X_I, mg COD/L)

- readily biodegradable organics (S_S, mg COD/L)
- slowly biodegradable organics (X_S, mg COD/L)

The inert organics, (S_I and X_I), do not undergo transformations, and leave the treatment plant unchanged in form. The readily biodegradable material, (S_S), consists of relatively simple molecules that can be taken in directly by heterotrophic bacteria and used for growth of new biomass. The slowly biodegradable organics, (X_S), consist of relatively complex molecules, and before heterotrophic bacteria can use them, they must be hydrolysed extracellularly and converted into readily biodegradable substrate. For purposes of activated modelling, the X_S fraction is treated as if it were particulate.

Nitrogenous matter in wastewater can be divided into five categories:

- N-NH_4 nitrogen (S_{NH}, mg N/L)
- biodegradable soluble organic nitrogen (S_{ND}, mg N/L)
- biodegradable particulate organic nitrogen (X_{ND}, mg N/L)
- inert soluble organic nitrogen (S_{NI}, mg N/L)
- inert particulate organic nitrogen (X_{NI}, mg N/L)

A content of inert nitrogen, (S_{NI} and X_{NI}), in wastewater is usually very low and may be excluded from modelling. Proportions of the COD and nitrogen fractions in a given wastewater are characteristic for the wastewater, and supposed to be more or less constant over a time. Typical values for unsettled domestic sewage are given in Table 1.

TABLE 1. Typical characteristics of unsettled domestic sewage [4, 5, 6]

Symbol	Fraction of total COD	Fraction of total nitrogen
S_I	0.05	-
X_I	0.13	-
S_S	0.22	-
X_S	0.60	-
S_{NH}	-	0.60
S_{ND}	-	0.10
X_{ND}	-	0.25
S_{NI}	-	0.01
X_{NI}	-	0.04

The ultimate BOD, BOD_5 and TKN concentrations can be calculated in terms of the above classification:

$$BOD_u = S_S + X_S \qquad BOD5 \approx 0.68\, BOD_u \qquad TKN = S_{NH} + S_{ND} + X_{ND} \qquad (1)$$

The diurnal and stochastic components of the treatment plant influent flow rate and concentrations can be modelled in a way that enables generating them within a treatment plant simulation program.

2.2. PRIMARY SETTLING TANK MODEL

In the primary clarifier, settleable matter is removed by gravity. It lowers the load to the biological reactor. As a result the reactor can be designed smaller than the one without the primary settler.

There is no doubt that the fundamental nature of particle settling in the primary settling tank is extremely complex and essentially stochastic. Fortunately, it has been proved that relatively simple mechanistic lumped-parameter models applied to real-world problems are reliable, and give fairly good predictions of pollutants concentrations in the primary settler effluent [7]. As an example we can use a dynamic model which conceptually divides the primary settler into three separate functional elements: a solids-liquid separator, a sludge zone at the bottom, and a series of imaginary completely-mixed compartments to account for hydraulic mixing [8, 9]. Suspended solids (SS) removal efficiency is determined from an empirical equation relating the suspended solids removal rate to their concentration. It is assumed that only suspended solids are removed and no other reactions take place in the settler. An illustration of predictive power of the model is given in Fig. 1.

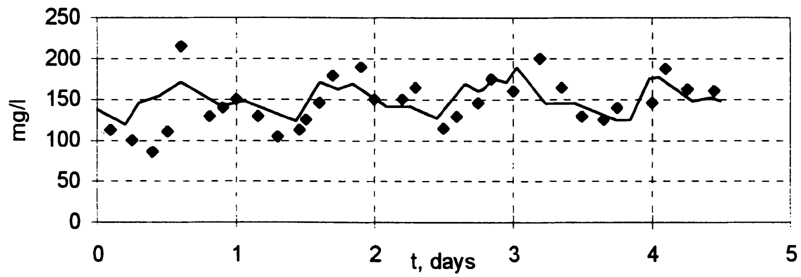


Figure 1. **Comparison of simulation results (solid line) with experimental data (black points) for full scale primary settling tank effluent SS concentration [9].**

2.3. ACTIVATED SLUDGE MODEL

Most designs of biological wastewater treatment systems (especially those with nitrogen removal) incorporate long solids retention times (SRT). Because of that, differences in effluent concentrations of soluble biodegradable organics with respect to different system configurations are generally small. Conversely, large differences in activated sludge concentrations and electron acceptor (oxygen or nitrate) requirements are common. Furthermore, good design practice requires providing sufficient quantity of electron acceptors in response to diurnal and seasonal changes in demand, and final settlers capable of handling all anticipated concentrations of solids. Thus, models describing substrate removal should be mostly directed to the settler's impact upon activated sludge concentrations and electron acceptor requirements.

The first good example of such a model was the IAWPRC Activated Sludge Model 1 [3]. The model incorporates carbon oxidation, nitrification and denitrification. The fate of the carbonaceous and nitrogenous components of sewage in bioreactors is schematised in the model as shown in Fig. 2. For modelling purposes, the readily biodegradable material, (S_S), is treated as if it were soluble, whereas the slowly biodegradable material, (X_S), is treated as if it were particulate. A portion of the readily biodegradable organics taken in by heterotrophic bacteria, (X_{BH}), is oxidised either by oxygen (oxic process) or nitrates (anoxic process, denitrification) as terminal electron acceptors. The rest of these organics, in the form of organic molecules, is incorporated into the biomass. The energy released during the oxidation process covers the energetic expenditures of the biomass synthesis. The biomass synthesis is associated with assimilation of some nitrogen and phosphorus.

The slowly biodegradable material, after entering bioreactor, is supposed to be quickly enmeshed into the activated sludge flocs structure, and then to be acted upon extracellularly (hydrolysed). The hydrolysis process is slower than the utilisation of readily biodegradable substrate. The extent to which the X_S fraction in the sludge flocs is hydrolysed (before the flocs leave the system) depends mainly on the process temperature and the time allowed for hydrolysis in the system (sludge age). The higher the temperature and the greater the sludge age, the more complete is the hydrolysis.

Non-biodegradable organic and mineral matters pass through the activated sludge system unchanged in their forms. Inert soluble organic mater, (S_I), leaves the system at the same concentration as it enters. Inert suspended organic mater, (X_I), becomes enmeshed in the activated sludge and is removed from the system through sludge wastage. The same holds true for mineral suspended solids, (X_{min}).

Under aerobic conditions, ammonia nitrogen, (S_{NH}), serves as the energy source for autotrophic nitrifying bacteria, (X_{BA}). They oxidise ammonia nitrogen to nitrate nitrogen (S_{NO}) (nitrification). Nitrate formed during this process may serve as terminal electron acceptor for heterotrophic bacteria under anoxic conditions, yielding nitrogen gas. Usually nitrate nitrogen content in raw municipal wastewater is nearly zero.

Soluble biodegradable organic nitrogen, (S_{ND}), is converted by heterotrophic bacteria to ammonia nitrogen (ammonification). Biodegradable particulate organic nitrogen, (X_{ND}), is enmeshed in the sludge flocs and hydrolysed by heterotrophic bacteria to soluble organic nitrogen, (S_{ND}), in parallel with hydrolysis of slowly biodegradable organics, (X_S).

Heterotrophic and autotrophic bacteria decay slowly releases both biodegradable matter, (X_S), which re-enters the cycle and is further hydrolysed, and non-biodegradable particulate organic matter, (X_P), which stays enmeshed in the sludge flocs and leaves the system with sludge wastage. Nitrogen present in the X_S fraction, (X_{ND}), undergoes hydrolysis in parallel. The greater the sludge age, the more decay products are released.

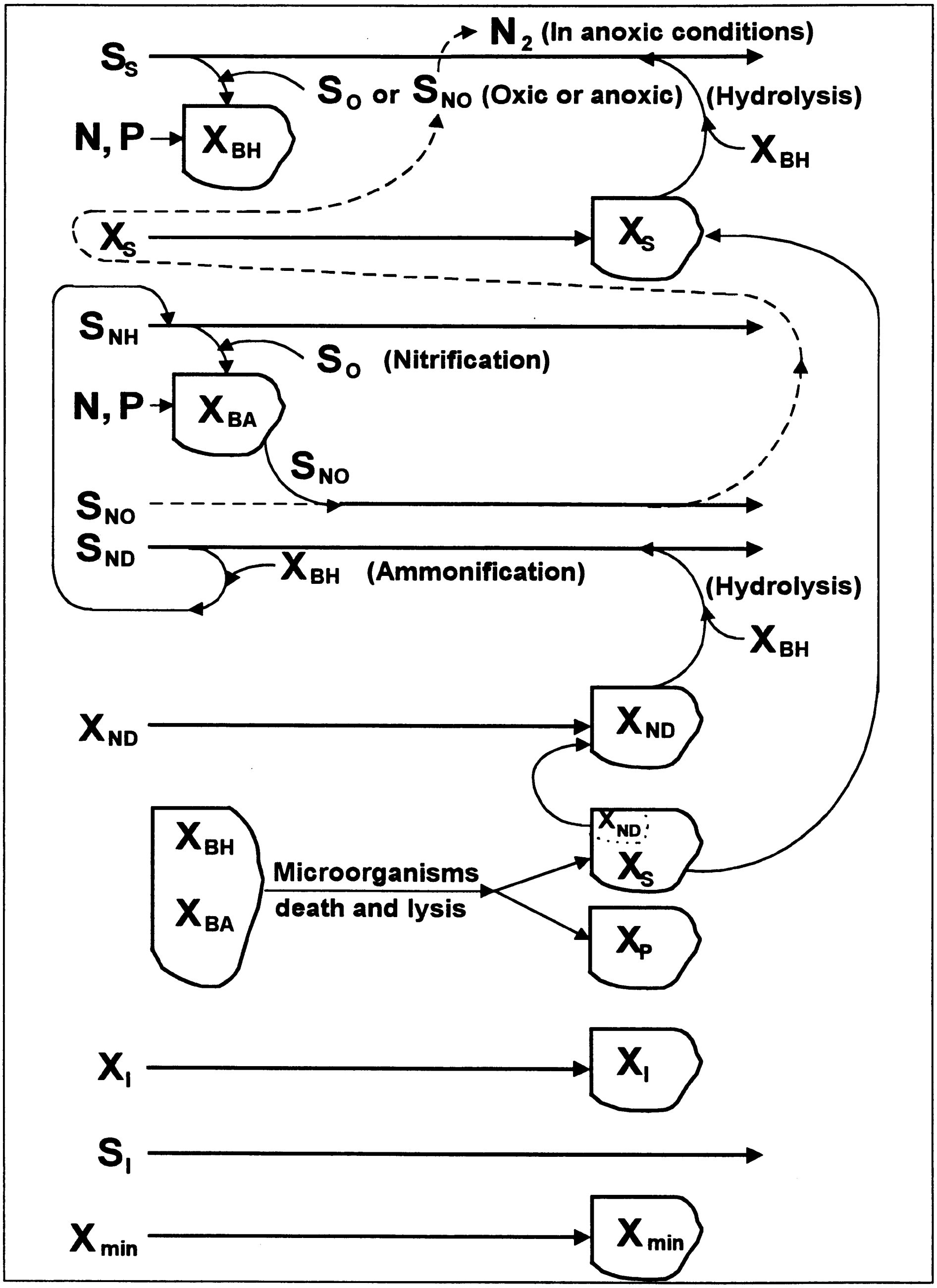
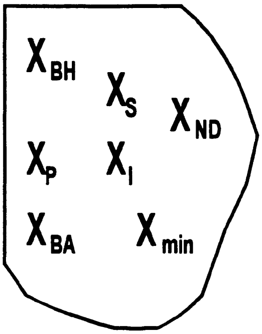


Figure 2. Scheme of sewage pollutants transformation in the activated sludge process.

The activated sludge flocs are comprised of all the insoluble sewage components and the particulate products of the biomass decay. The flocs that are carried over from the final clarifier contribute to the total organic and nitrogen pollution in the secondary effluent, as illustrated in Fig. 3.



$X_{COD} \approx X_{BH} + X_{BA} + X_P + X_S + X_I$, mg COD/L

$X \approx (X_{BH} + X_{BA} + X_P + X_S + X_I) / 1.5 + X_{min}$, mg SS/L

$X_{VSS} \approx (X_{BH} + X_{BA} + X_P + X_S + X_I) / 1.5$, mg vss/L

$f_{BOD/SS} \approx (X_{BH} + X_{BA} + X_S) / X$, g BOD_u/g SS

$f_{COD/SS} \approx (X_{BH} + X_{BA} + X_S + X_P + X_I) / X$, g COD/g SS

$f_{TKN/SS} \approx [0.12 (X_{BH} + X_{BA} + X_P + X_I) + X_{ND}] / X$, g N/g SS

Figure 3. Carbonaceous and nitrogenous forms of pollution associated with the components of the activated sludge flocs ($f_{BOD/SS}$, $f_{COD/SS}$, $f_{TKN/SS}$ are respective fractions of BOD, COD and TKN in the activated sludge solids).

The model can predict the sludge composition and its production in a given system as a function of sludge age and pollutant concentrations in treated wastewater. Suspended (sludge) as well as soluble organics and nitrogen concentrations can be calculated under steady state and dynamic conditions in each reactor of variously configured systems. The same holds true for spatially and temporally changing oxygen uptake rates. The illustrative results obtained for typical domestic wastewater treated aerobically in completely mixed reactors are shown in Figures 4 and 5.

An illustration of the predictive power of the model under dynamic conditions is given in Fig. 6. The results were obtained for unsettled municipal wastewater treated in a single reactor, which was an aerobic completely mixed pilot system under daily cyclic square wave flow and load conditions (12 hours feed / 12 hours no feed) [10].

2.4. SECONDARY CLARIFIER-THICKENER MODEL

The overall efficiency of the activated sludge process is largely dependent on the satisfactory performance of the secondary clarifier. This unit serves as a thickener (to concentrate biological solids which must be recycled to sustain the processes in bioreactors) and clarifier (to keep the final effluent suspended solids concentration as low as possible). A good settler model should be able to predict: concentration of solids in the underflow (return sludge concentration), solids blanket height, solids concentration profile in the settler and concentration of suspended solids in the overflow [9, 11, 12].

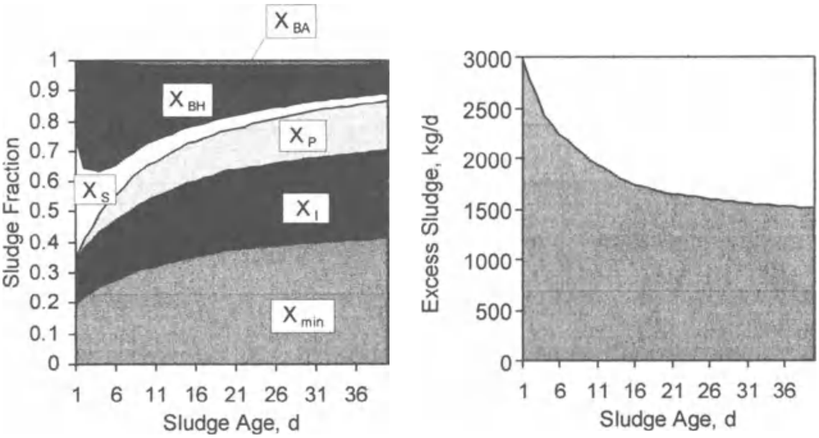


Figure 4. **Activated sludge composition and excess sludge production for typical domestic sewage (Q = 10,000 m^3/d, temperature 15°C).**

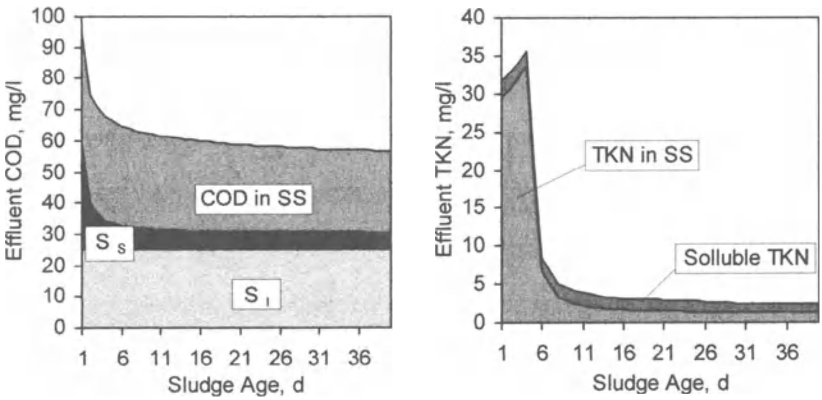


Figure 5. **Secondary effluent COD and TKN for typical domestic sewage treated aerobically (SS_e = 30 m/L, temperature 15°C).**

Much research has been carried out on many aspects of sedimentation in the activated sludge process, yet there is no coherent theory to describe the dynamics of the sludge particle settling. The behaviour of secondary settler is generally viewed in terms of two separate functions: clarification and thickening. Most models of clarification are empirical and usually relate the results to the concentration of SS in the mixed liquor, flow rate and sometimes the sludge blanket level in the secondary clarifier.

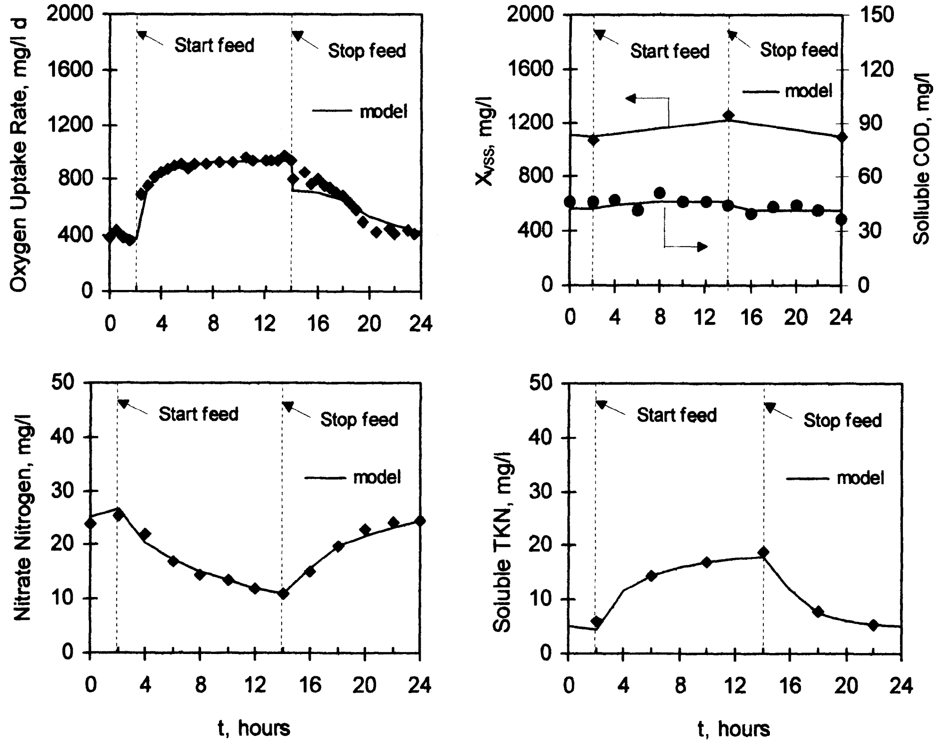


Figure 6. Comparison of experimental results and the activated sludge model predictions under daily wave loading conditions [10].

Models of the thickening function of the settler divide the sludge blanket into several completely mixed layers (Fig. 7), and apply the limiting flux theory to describe the solids transport between the layers. These models can predict behaviour of the sludge blanket under different loading conditions. An illustrative example is given in Fig. 8, which describes a full-scale final settler under high loading conditions [11].

3. Application of the Activated Sludge Plant Model to Design

The dynamic models of the activated sludge treatment plant can be very convenient tools for designers and operators. On the following pages, an example of possible model applications is given. The example demonstrates upgrading a nitrifying activated sludge system to enable biological nitrogen removal. The objective is to obtain average effluent concentrations not exceeding these limits: $BOD_5 \leq 30$ mg/L, SS ≤ 50 mg/L, $N_{total} \leq 30$ mg/L, $N\text{-}NO_3 \leq 30$ mg/L, $N\text{-}NH_4 \leq 6$ mg/L.

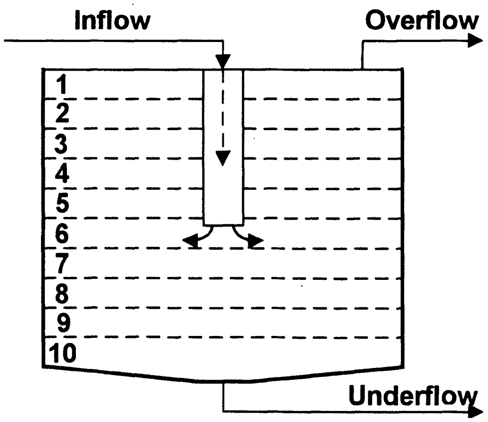


Figure 7. Layered final settler model.

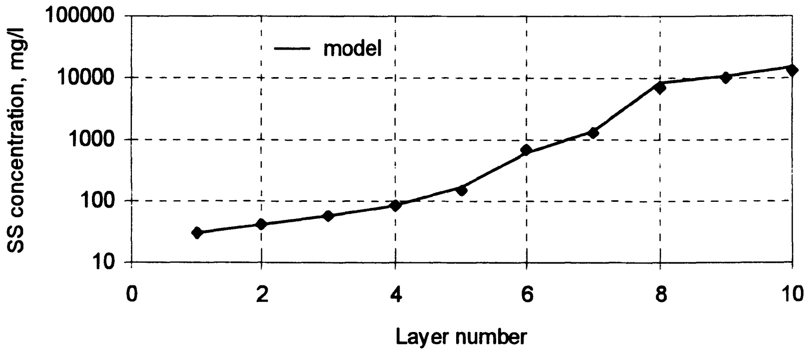


Figure 8. Comparison of observed and simulated SS concentration profiles in a full scale secondary settling tank [11].

A hypothetical treatment plant that is subject to analysis is shown in Fig. 9. The average influent flow rate to the plant is 10,000 m^3/d, $BOD_5 = 280$ mg/L, SS = 250 mg/L and TKN = 60 mg/L. The analysis was carried out assuming constant or diurnal loading conditions. Assumed input time series of BOD_5 and TKN are shown in Fig. 10. The first day of the series is characterised by constant loading; the days from 1 to 4 by diurnal loading, and the next 16 days by diurnal loading plus a first order autoregresive stochastic component [9]. By adding the stochastic component, one can account for noise associated with real inputs when they are stationary. The simulated outputs for the system operated at the sludge age (Solids Retention Time - SRT) of 6 days, and the reactor temperature 15°C, under loading conditions given in Fig. 10, are shown in Fig. 11. One can notice that the system provides nitrification, but the resulting concentration of total nitrogen in the effluent is too high ($N_{total} \geq 30$ mg/L, N-$NO_3 \geq 30$ mg/L) as compared with the stated objective. It is obvious that denitrification of nitrates is necessary.

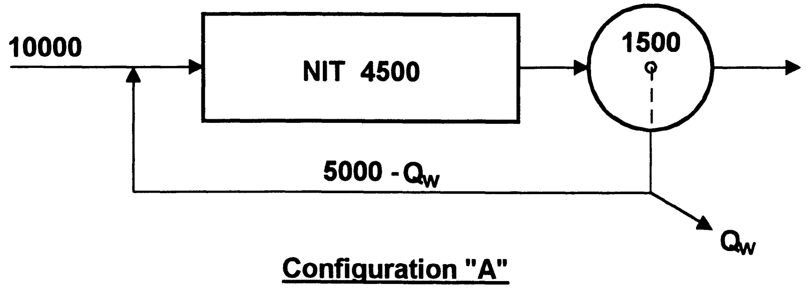


Figure 9. Configuration of the treatment plant to be upgraded. The numbers inside the shapes indicate volumes (m^3) of the respective units, while those along the arrows indicate the respective flow rates (m^3/d).

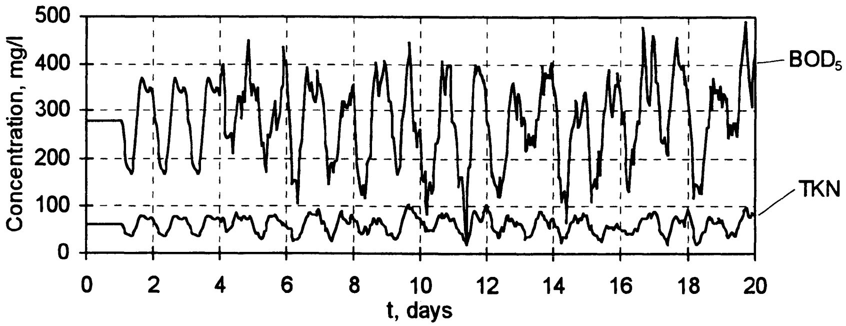


Figure 10. Influent BOD_5 and TKN time series to the plant.

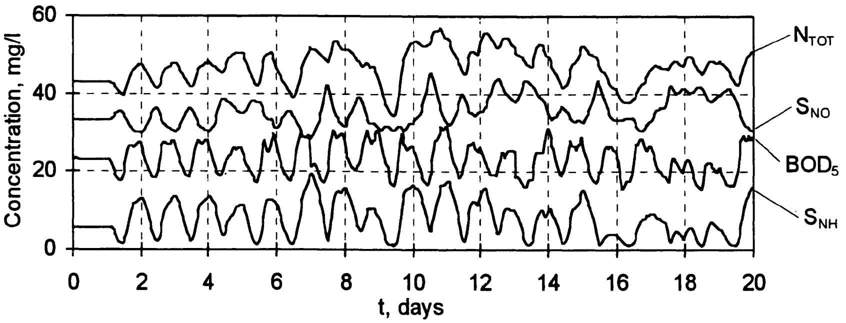


Figure 11. Effluent concentrations for plant Configuration A.
($V_{NIT} = 4500$ m^3, SRT = 6 d, temperature = 15°C, resultant $MLSS_A \approx 3000$ mg SS/L)

As a first trial solution of the problem, the plant was modified as shown in Fig. 12.

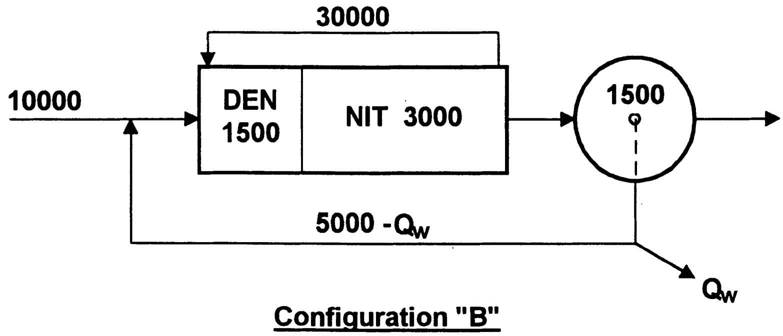


Figure 12. Configuration B of the upgraded plant.

Configuration B consists in assigning one third of the aeration tank to the anoxic reactor. Feasibility of this configuration was checked by simulation. It was assumed that inputs to the plant were constant and equal to the mean load values. The initial conditions for the simulation were the steady state values for Configuration A. The simulation results for 19 days, after starting Configuration B, are shown in Fig. 13.

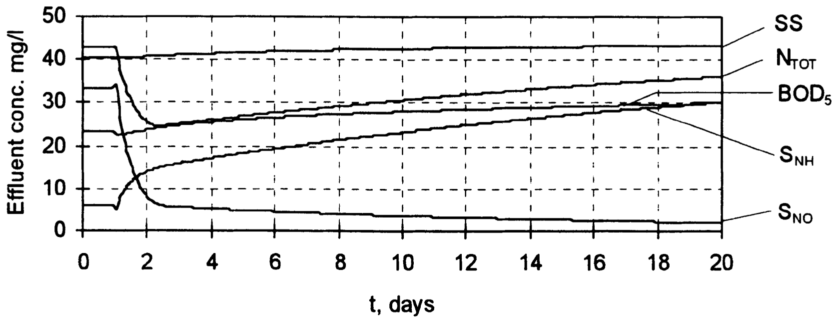


Figure 13. Transition from Configuration A (V_{NIT} = 4500 m^3, MLSS$_A \approx$ 3000 mg SS/L, SRT = 6 d) to Configuration B (V_{NIT} = 3000 m^3, V_{DEN} = 1500 m^3, SRT = 6 d). Temperature = 15 C, resultant MLSS$_B \approx$ 3200 mg SS/L.

It can be seen that Configuration B did not produce satisfactory results. The most spectacular effect was cessation of the nitrification. It is so because the applied SRT (6 d) appeared to be too short to sustain the nitrifiers in the activated sludge (at SRT = 6 d, the effective aerobic solids retention time in the system is much shorter - SRT_{AER} = (3000/4500)$\cdot$6 = 4 d.). To extend the time for nitrifiers to grow in the system, the SRT was increased to 9d. The simulation results are shown in Fig. 14.

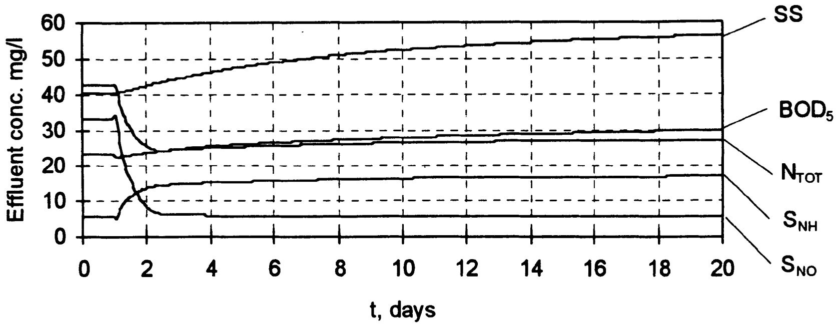


Figure 14. Transition from the Configuration A (V_{NIT} = 4500 m^3, $MLSS_A \approx$ 3000 mg SS/L, SRT = 6 d) to Configuration B (V_{NIT} = 3000 m^3, V_{DEN} = 1500 m^3, SRT = 9 d). Temperature = 15°C, resultant $MLSS_B \approx$ 4100 mg SS/L.

As can be seen the sludge age is still too short to assure satisfactory nitrification results.

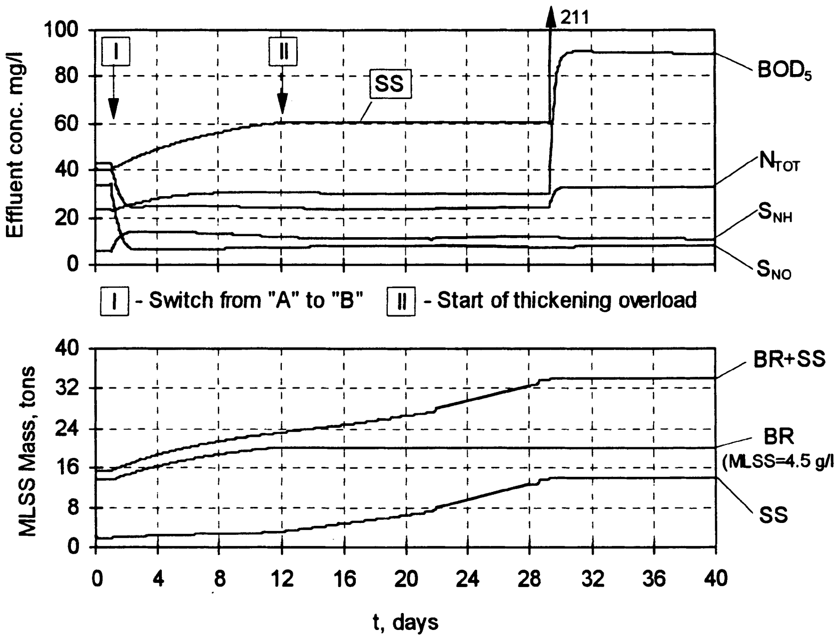


Figure 15. Transition from Configuration A (V_{NIT} = 4500 m^3, $MLSS_A \approx$ 3000 mg SS/L, SRT = 6 d) to Configuration B (V_{NIT} = 3000 m^3, V_{DEN} = 1500 m^3, target SRT = 12 d). Temperature = 15°C.

A further attempt to increase the SRT to 12 d was not successful, as obvious from the simulation results shown in Fig. 15. On the 12th day (11 days after switching to Configuration B) the MLSS concentration reached 4500 mg/L. At this concentration, the thickening function of the secondary settling tank was overloaded. From this moment on, the mass of the MLSS in the bioreactors could not be increased above ~20 tons. All the remaining mass that was not removed as the excess sludge, built up in the secondary settler. Under these conditions the effective SRT of the sludge in the bioreactors could not be longer than ~9.6 d, and this did not guarantee the needed degree of nitrification. If the settler overloading lasts, the sludge build up in the settler finally reaches the overflow weirs and it is massively discharged with the effluent. Conclusion from the carried out simulations is that the upgrading of the plant requires an extension of the bioreactor and/or the final clarifier and/or building a primary clarifier.

Following the conclusions from the above analysis, Configuration C (Fig. 16) consisting in enlargement of the bioreactor was checked.

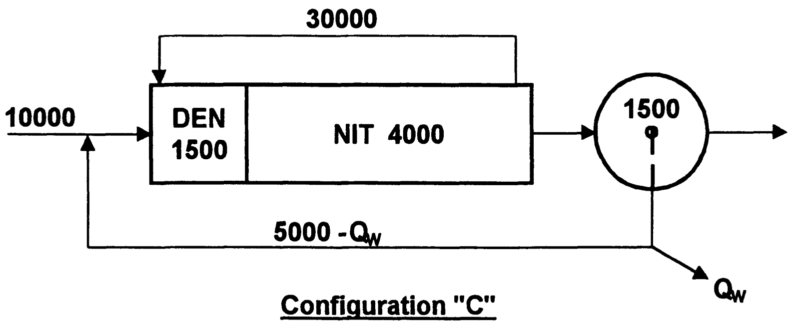


Figure 16. Configuration C of the upgraded plant.

The results of simulation are shown in Fig. 17. As can be seen, the results of BOD and nitrogen removal are satisfactory, but SS exceeds the limit of 50 mg/L. It is so because of the high MLSS (~4300 mg SS/L) causing high solids load to the clarifier, resulting in high effluent SS.

Instead of increasing the bioreactor volume, it seems reasonable to try to increase the final clarifier (Configuration D in Fig. 18) or to add a primary clarifier (Configuration E in Fig. 19).

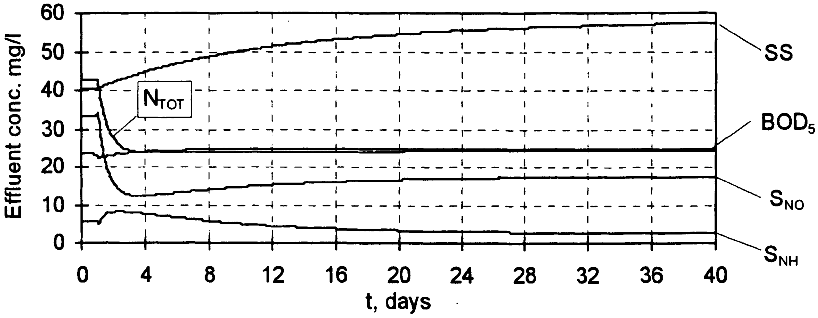


Figure 17. Transition from Configuration A (V_{NIT} = 4500 m^3, $MLSS_A \approx$ 3000 mg SS/L, SRT = 6 d) to Configuration C (V_{NIT} = 4000 m^3, V_{DEN} = 1500 m^3, SRT = 12 d). Temperature = 15°C, resultant $MLSS_C \approx$ 4300 mg SS/L.

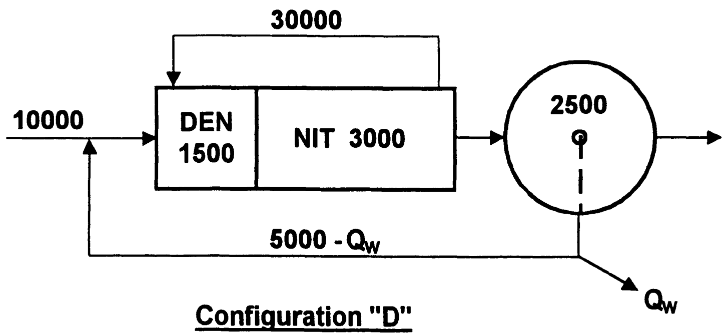


Figure 18. Configuration D of the upgraded plant.

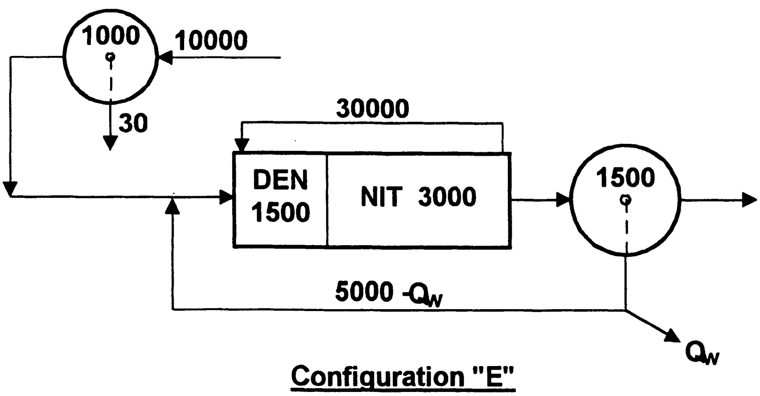


Figure 19. Configuration E of the upgraded plant.

The simulation analysis for the two options was carried out. The construction costs involved in each option are more or less the same, because of the same additional volumes assumed. The simulation results are shown in Figures 20 and 21. In both cases the results appear satisfactory. In some respects, the results are different. If there is no primary clarifier in the system, the resultant MLSS is very high (~5000 mg SS/L). This can make the final settler in the system prone to overloading at peak flows and/or in periods of poor sludge settleability. The presence of the primary clarifier results in much lower MLSS (~3200 mg SS/L), but at the same time, brings about poorer denitrification as a result of the lower BOD/TKN ratio in the primary effluent.

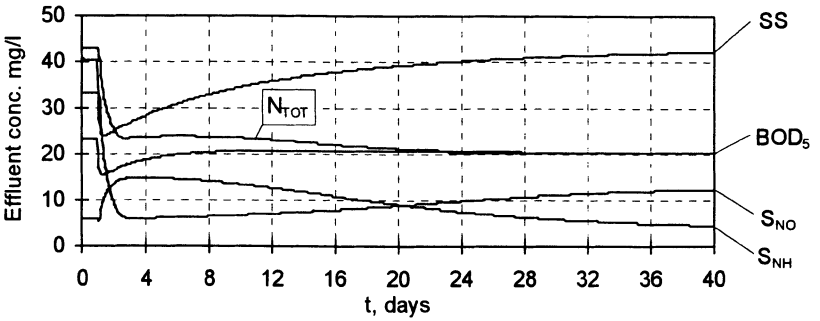


Figure 20. Transition from Configuration A (V_{NIT} = 4500 m^3, $MLSS_A \approx$ 3000 mg SS/L, V_{SST} = 1500 m3, SRT = 6 d) to Configuration D (V_{NIT} = 3000 m^3, V_{DEN} = 1500 m^3, V_{SST} = 2500 m^3, SRT = 12 d). Temperature = 15°C, resultant $MLSS_D \approx$ 5000 mg SS/L.

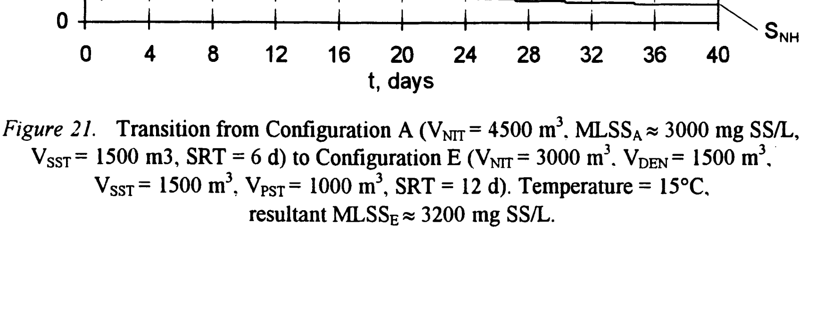


Figure 21. Transition from Configuration A (V_{NIT} = 4500 m^3, $MLSS_A \approx$ 3000 mg SS/L, V_{SST} = 1500 m3, SRT = 6 d) to Configuration E (V_{NIT} = 3000 m^3, V_{DEN} = 1500 m^3, V_{SST} = 1500 m^3, V_{PST} = 1000 m^3, SRT = 12 d). Temperature = 15°C, resultant $MLSS_E \approx$ 3200 mg SS/L.

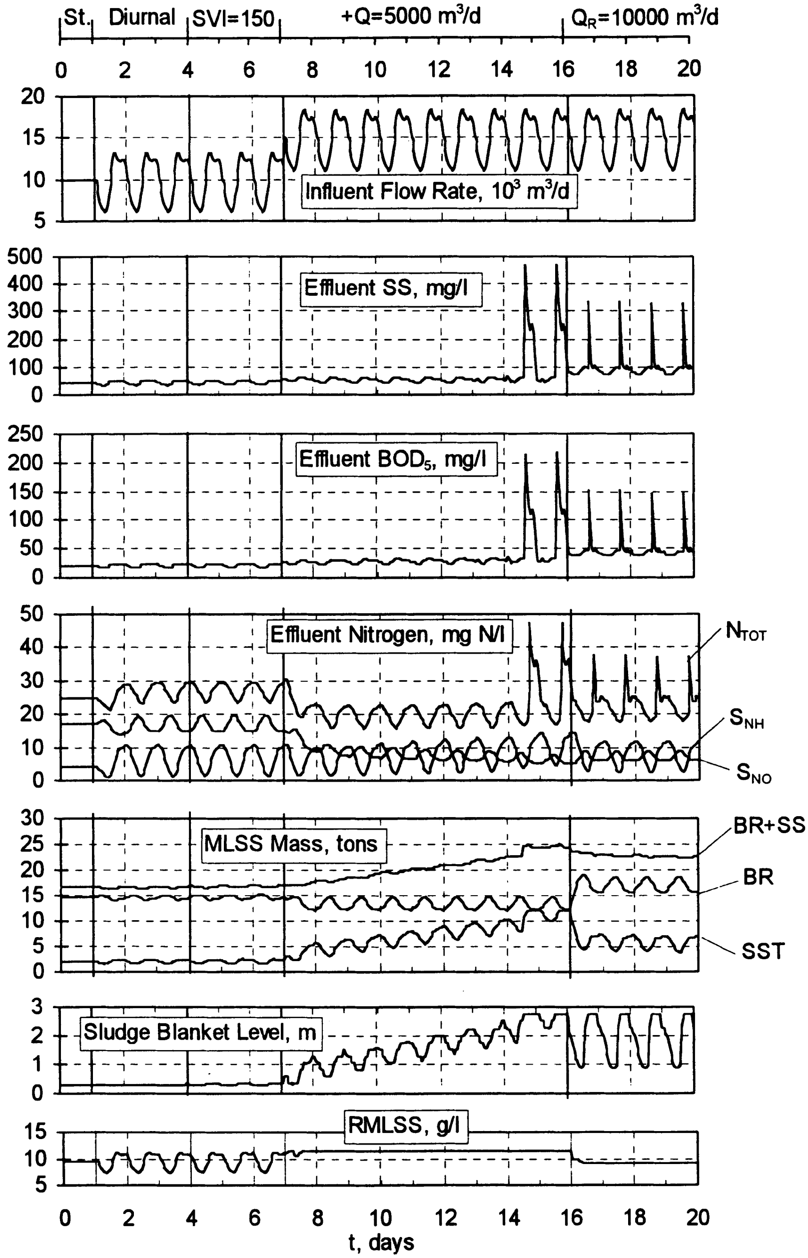


Figure 22. Simulation results for Plant E under various operating conditions.

The performance of the Plant E under diurnal loading as well as in periods of poor sludge settleability (SVI = 150 cm^3/L) and wet weather conditions (150% Q) sustained for 9 days has been analysed. The results obtained are shown in Fig. 22. The first day series is representative for Plant E at steady state, while the days from 1 to 4 represent the plant under diurnal loading conditions. It is obvious that the plant performs well during this period. The same holds true for the next three days when the activated sludge settleability deteriorates (SVI rises from 100 to 150 cm^3/g). The days from 7 to 16 represent wet weather conditions (flow rate rises by 50%), together with poor sludge settleability.

As can be seen, the increased hydraulic load overloads the secondary settler, with respect to sludge thickening. Solids accumulation in the settler results in the rise of the sludge blanket level (SBL). In terms of the effluent quality, the results of the overloading are not dramatic for about one week. One would observe only some increase in the effluent SS and BOD_5, and even a noticeable decrease in the total nitrogen concentration (but not in the load). Symptoms of severe overloading can be noticed only if the SBL is monitored. Otherwise the plant operator would be unaware of the problems to come. If the overloading lasted for more than one week, the plant would collapse as a result of massive solids discharge. The massive solids discharge starts after the accumulating sludge blanket reaches the final clarifier overflow weirs.

The results shown in Fig. 22 for the last four days (from 16 to 20) relate to a remedial action performed by the operator. The action consists in doubling the sludge recirculation ratio (from 5000 to 10000 m^3/d) to transfer the sludge from the settler to the bioreactor. As can be seen, this action proves to be unsuccessful.

To make the system less sensitive to the overload, Configuration F (Fig. 23) has been tried out.

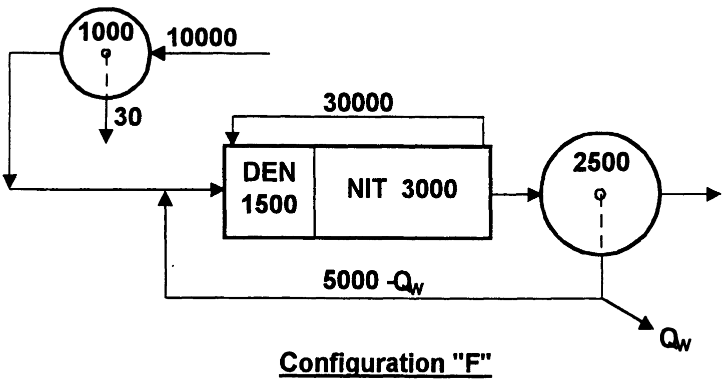


Figure 23. Configuration F of the upgraded plant.

This configuration consists in enlarging the secondary settler, which appeared to be a bottleneck of Configuration E. The simulation results for Plant F, under the same loading conditions as assumed for Plant E, are shown in Fig. 24. It is obvious that, owing to the enlargement of the final settler, the system can successfully sustain the imposed overload for more than two weeks without violating the effluent limits.

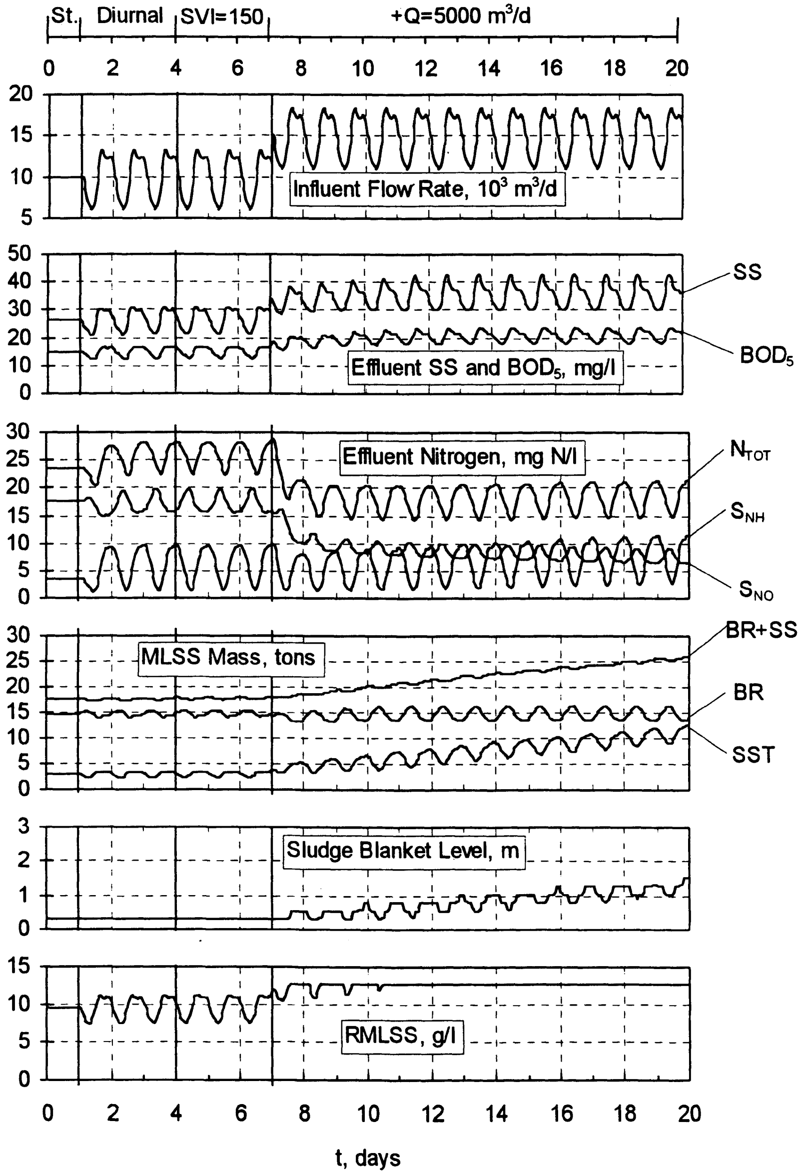


Figure 24. Simulation results for Plant F under various operating conditions.

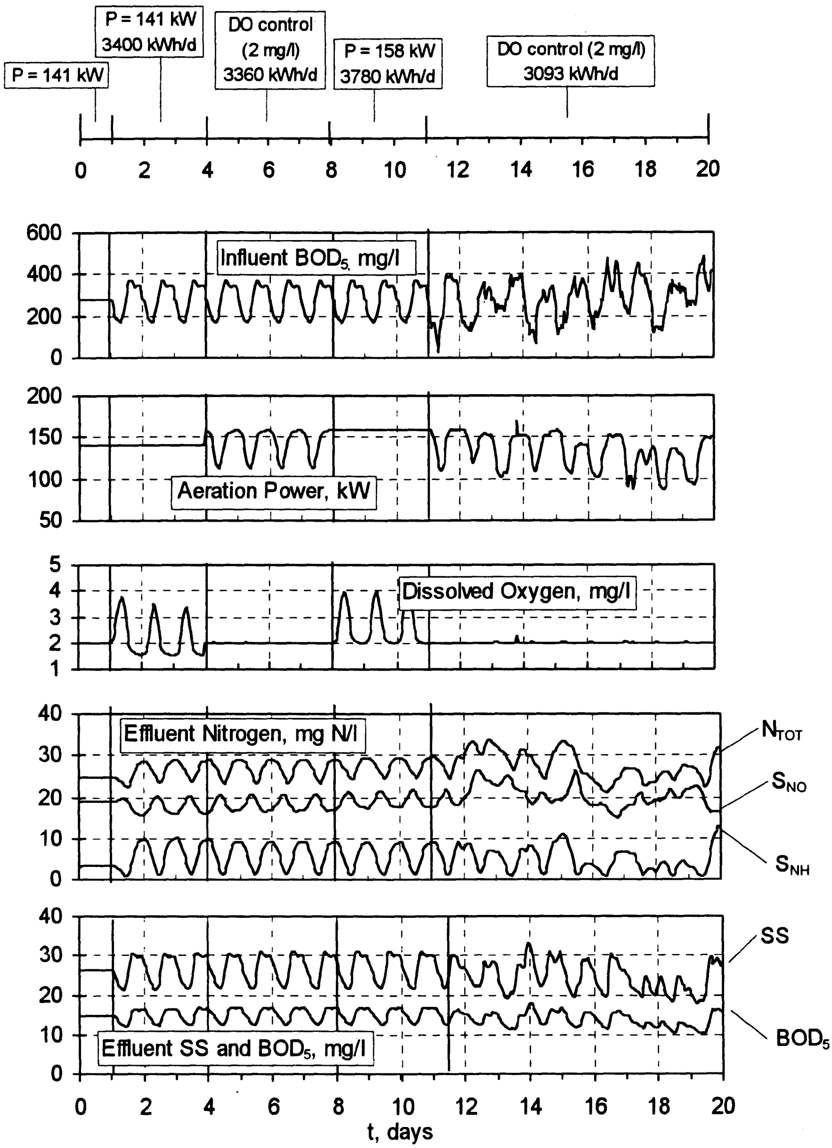


Figure 25. Simulation results for Plant F under various aeration control strategies.

The dynamic model enables the evaluation of an aeration system and its control as well. An example of such analysis is illustrated in Fig. 25, for Plant F under constant loading (day 1), diurnal loading with constant aeration power (days 1 to 4) or constant DO in the aeration tank (days 4 to 8), diurnal loading with constant aeration power assuring that DO is never below 2 mg/L (days 8 to 11), diurnal plus stochastic loading, and constant DO control. The simulation enables easy comparison of various aeration system control strategies, and provides good estimates of the oxygen demand variability and power consumption, thus greatly facilitating the aeration system analysis.

4. References

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MODELLING RECEIVING WATERS

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1. Introduction – Mathematical Models of Open Channels for Hydraulic and Environmental Engineering

A wide range of water quality problems, including deterioration of ecological conditions in rivers, are caused by direct discharges from sewers and/or combined sewer overflows (CSO) (Falconer 1992). This practice started to create major problems in the first decades of the twentieth century, especially with respect to oxygen conditions in rivers and reservoirs. Suspended solids discharged from sewer systems cause sedimentation in rivers, usually at not well known locations, affecting the riverine habitat. In a small creek, a sudden increase of discharge, resulting from a storm overflow, will wash out organisms and plants.

The EC directives reflect the increased public awareness of the water quality and ecology of rivers in urban areas. The ever-stricter standards and regulations in EC member states, and also in many other countries in Europe, emphasise the need for modern methods that will establish the relation between the varying sewer effluent quantities and qualities and the resulting water quality and ecological status in the receiving waters. These demands can be met by applying mathematical models which relate the time varying effluents to the environmental condition of the river, taking into account the hydrological and hydraulic conditions of the selected loading scenarios.

Since a user of such models would also like to use these models for prediction purposes, it is more appropriate to use deterministic models than stochastic ones. Using mathematical models that are based on steady uniform flow conditions in rivers would be obsolete in many cases, and it could lead to unrealistic results.

When a mathematical model is successfully set-up and calibrated, the user will be able to simulate not only the hydrodynamic behaviour of aquatic systems in river channels and creeks, but also other physical, biological and chemical processes. This will be done on the basis of characteristics and parameters obtained from the hydrodynamic sub-models. Such characteristics are essential for all other processes. For this reason this

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paper deals mainly with the development of 1D and 2D mathematical models and focuses primarily on their hydrodynamic 'hearts'.

1.1. UNSTEADY FLOW IN RIVERS

Water flow in open channels is classified as steady or unsteady. The flow of water in river channels and through related structures (drains and conduits with free surface flow, tunnels and culverts, reservoirs, weirs etc.), for which the velocities change with time at a particular location, is classified as unsteady flow. Water flow through river reaches in urbanised areas is almost always unsteady. When the change of discharge with time is very gradual, the flow is more or less steady, and a quasi-steady approach can be accepted (the flow is steady or nearly steady for a certain period of time).

The mathematical treatment of unsteady flow in river channels is an important but relatively difficult problem as explained by Mahmood and Yevjevich (1975). Basically, these difficulties exist because many variables enter into the functional relationship and their differential equations cannot be integrated in a closed form, except under very simplified conditions. On the other hand, it can be assumed that the velocity distribution and friction terms in unsteady flow are generally well represented by those derived under steady flow conditions. Therefore, the reliability of computed results for unsteady flow must be assessed by using the theory of errors and a sensitivity analysis of the solutions, based on estimated errors of individual variables included in the basic governing equations.

Water waves. The item wave has to be understood in a broad sense. Thus, a surface wave is a temporal, spatially propagated change in the water surface. A water wave in a river channel generally means any change of discharge, velocity or water stage with time.

1.2. CHOICE OF MODELS

Unsteady flow in rivers or man-made channels is a complex phenomenon and cannot be understood in every detail. Abstraction, e.g., replacing the real system under consideration with a model of similar but simpler structure, is necessary if important aspects of the phenomenon are to be understood or controlled.

Two distinct classes of models can be specified: material and symbolic (Abbott 1991).

A material model is a representation of the real system by another material system that is assumed to have similar properties, although it is usually not so complicated or difficult to work with. Scale models and analogue models are examples of material models.

A symbolic model is a representation of the real world system by an abstract entity. It is a symbolic or mathematical description of an idealised situation that reflects some of the structural properties of the real system. Symbolic models are either empirical or theoretical. Mathematical models are symbolic models.

1.3. MATHEMATICAL MODELS APPLIED TO OPEN CHANNELS

Mathematical models have found increasing practical application in the following main categories of problems in urban drainage:

- prediction
- design
- planning strategies
- real-time forecasting and control
- other problems

Mathematical models can be classified according to several criteria. From the point of view of model users, classification according to the following problem related and user-oriented criteria may be appropriate:

1. Purpose of model application (as classified above)
2. Type of the system to be modelled
   - elementary systems (single channels, short reaches, etc.)
   - complex system (surface water system, whole river basin including groundwater)
3. Type of hydrological, hydrodynamic and aquatic processes (or related variables) to be simulated
   - soil moisture, evapotranspiration
   - ground water storage, level, discharge
   - river discharge and water levels
   - water temperature, ice condition
   - sediment transport and related processes
   - water quality
4. Degree and type of causality of the process

Deterministic models

Causality is expressed in the form of cause-effect relations. These are best reflected in deterministic models that relate given dependent variables to a set of independent variables. There are actually three types of deterministic models:

- models based on fundamental laws of physics, chemistry and biology
- conceptual models reflecting fundamental laws in a simplified approximate manner and usually involving a certain degree of empiricism
- black box models which do not explicitly take into account the governing laws but only the cause-effect relation of system inputs to outputs, in a very general and purely empirical manner

Stochastic models

Stochastic models do not take into account the principle of causality. Becker and Serban (1990) introduced two types of models in this category.

Probabilistic models are generally represented by probability distribution functions of the hydrodynamic and environmental variables of interest (such as discharges, velocities, stages and storage volumes). They are often described in terms of parameters such as averages, standard deviations and coefficients of skewness.

Time series generation models form the second category of stochastic models. These models are used for extrapolating in time a sequence of recorded variables or describing events while preserving their basic statistical characteristics.

5. Time and space discretisation in modelling

Time step.

The appropriate time step in a modelling task is mostly determined by the purpose of the model application and basic model properties. Recently developed models are designed in such a way that different time steps can be applied.

Spatial discretisation

Spatial discretisation (sometimes described as 'topological modelling'), is more difficult to select, but it is essential for the tasks under consideration. Examples of spatial discretisation are shown in Fig. 1. There are two basic categories of spatial discretisation approaches:

- distributed models
    - grid base parameters (elementary unit areas)
    - semi-distributed parameters (larger sub-areas)
- lumped models
    - statistical distribution of important parameters
    - no distribution of parameters at all

1.4. SCHEMATISATION

A river basin is subdivided into subareas of approximately homogeneous inputs and hydrological and hydrodynamic conditions. In the case of surface water systems, the discretisation procedure is very much related to the dimensionality of the deterministic model that can be one-, two- or three-dimensional. For simple flow routing through river reaches in urban areas (without extended flood plains, reservoirs, lakes etc.), a one dimensional (1D) description is generally sufficient. When there is an extended flood plain in the domain of interest, it is more appropriate to apply a two-dimensional (2D) distributed or semi-distributed model. A fine grid representation as shown in Fig. 2 (c) is only necessary in very detailed and specific investigations (water quality - spill of toxic waste, sediment transport – deposition of polluted materials in urban areas, etc.). In many other cases it is also possible to apply a much simpler but often equally efficient solution, based on a semi- or quasi-two-dimensional description of the river reach (1D+).

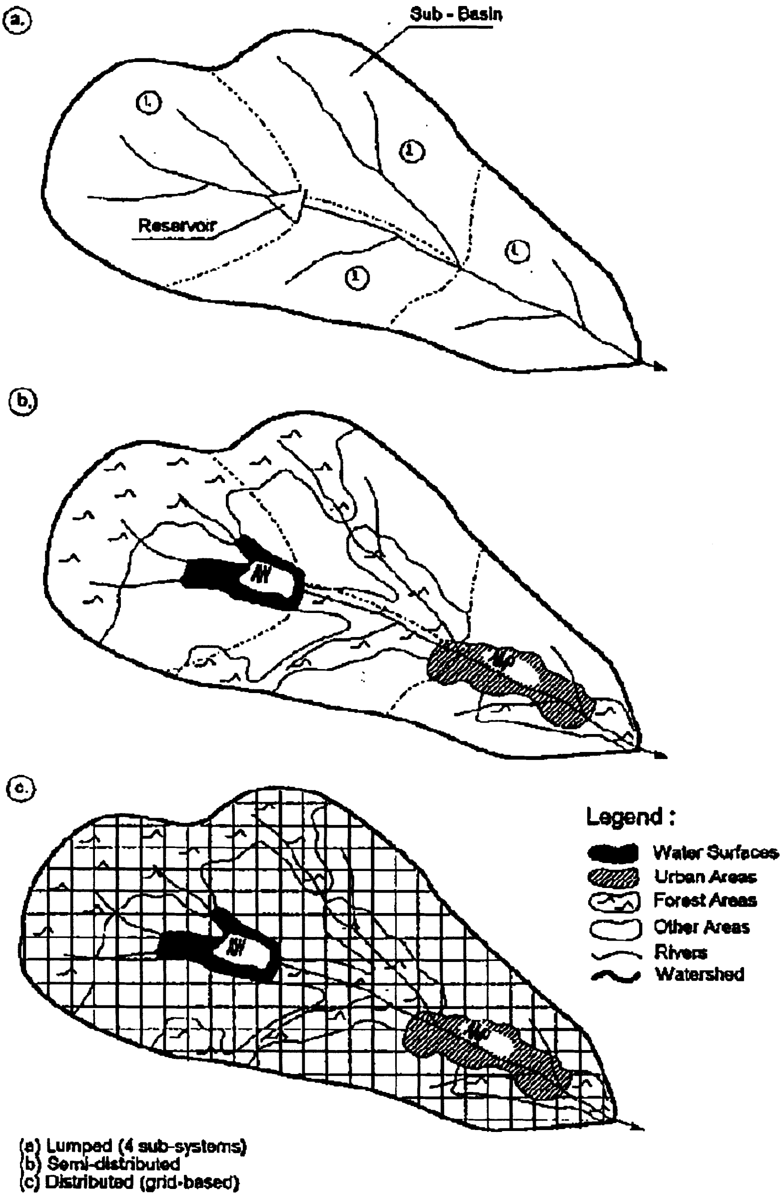

Figure 1. Representation of different spatial discretisation schemes of a river basin (a) lumped four sub-systems, (b) semi-distributed, (c) distributed - grid based.

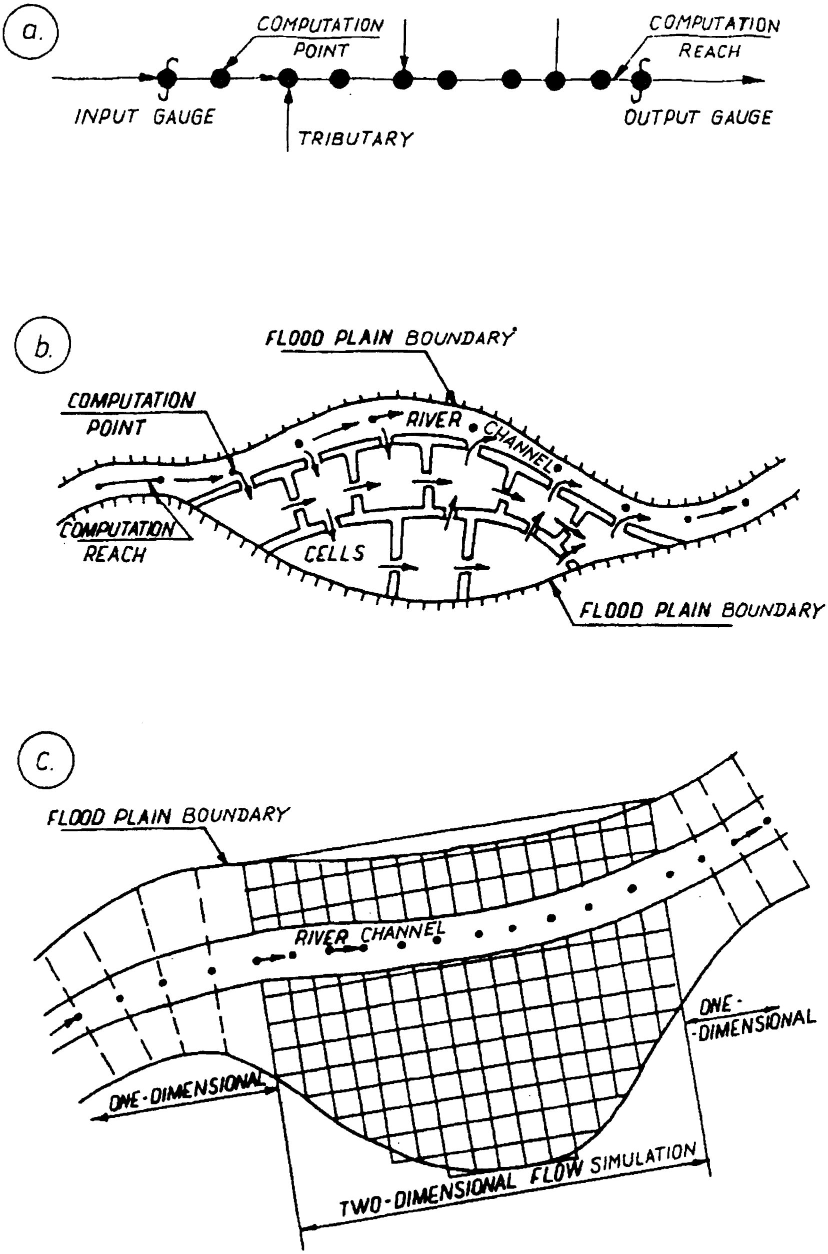

Figure 2. Spatial discretisation schemes in river modelling according to Becker and Serban (1990) (a) 1D distributed model, (b) 2D semidistributed, (c) 1D and 2D distributed.

1.5. AN INTEGRATED APPROACH

Until recently, several more or less independent elements of urban drainage have been designed, operated and maintained separately (such as sewer networks, waste water treatment plants (WWTP), and systems of river channels). The interactions and mutual influences between the components were virtually neglected in the past. Attempts to apply a receiving water based strategy to urban drainage raised the requirements for modelling the processes in rivers and receiving waters. It should be stressed that integrated urban drainage is a relevant concept for more objective evaluation and more sophisticated operation of drainage systems. In integrated systems, all relevant phases of transport processes are mutually interrelated in a dynamic form (Krejcik *et al.* 1994). The new integrated approach to urban drainage systems includes all elements of the process and provides the user with an option to reduce the pollution load to receiving waters. This option is materialised by the use of deterministic mathematical models. These models have to be accurately calibrated and verified against data, which are provided by systematic monitoring of the relevant urban area. The basic principles of 1D models applied for simulation of processes in open channels are very much the same as those for sewer networks. The numerical approximation of the governing equations is discretised in the branch or looped system, which is comprised of two basic elements: nodes, and their connections (links). The internal boundaries for the two systems (inflows or outflows schematised at a particular location in the network) are in reality just the connection between waste waters and 'clean waters' in the drainage system. It is obvious from the above that for an integrated approach to urban drainage, we must unavoidably use deterministic models based on physical laws with a certain level of schematisation, which is determined by the objectives and scale of the study. Mathematical models applied for the two elements of drainage, the *sewer network* and *open channel* flow, should both be based on the same principles, and each model should accept the results of the other one as internal boundaries at the locations where the two systems interconnect (CSOs, weirs discharging to streams, etc.) under given conditions (water level, operational scheme, etc.).

At a recent software conference, the Danish Hydraulic Institute (1992) presented a plan for integration of the MIKE11 and MOUSE packages (Lindberg 1995), which are based on the same numerical principle and a very similar computational network. Preliminary findings were also presented showing a direct interconnection between MOUSE and STOAT, a package for simulation of wastewater treatment plants processes designed by the Water Research Centre in Swindon. Price (1995) presented a similar idea of connecting an open channel package with the HydroWorks package.

1.6. MODEL OBJECTIVES

When assessing the effectiveness of applying mathematical models to receiving waters according to the integrated approach strategy, the following elements have to be taken into account: definition of goals of applying the modelling tools, type of mathematical models to be applied, and type and quality of the required data set. The models considered in this chapter provide the advantage of simulating the effect of hydrodynamic (HD), sediment transport (ST) advection dispersion (AD) and water quality (WQ) effects in receiving waters. A number of different models based on various assumptions and simplifications are discussed in this section. Four major application areas of mathematical modelling in river systems, where mainly HD, ST, AD and WQ modules are applied, are defined by the use of various conditions from the time-dependency point of view:

- constant HD and WQ conditions in time,
- dynamic (unsteady) HD conditions, constant WQ parameters,
- quasi-steady HD conditions and dynamic ST conditions,
- dynamic HD conditions and time dependent WQ parameters,

A number of difficulties are caused by the application of the specified models in urban areas. The major difficulties relate to calibration and validation of the applied models. This requires a great amount of data and measurements, which are rarely available. Most of the models applicable to some element of an integrated system were developed independently, and for this reason, they cannot be interconnected easily through boundary conditions, as discussed by Brelot-Wolff and Chocat (1993).

2. One-Dimensional Models

2.1. ONE-DIMENSIONAL (1D) HYDRODYNAMIC MODELLING OF OPEN CHANNELS

The chapter about 1D hydrodynamic modelling of open channels in urban areas deals with flows in such systems as a river reach or a channel in a town or a densely populated region, where channels and rivers have been trained and hydraulically improved. For flow in such systems, a coherent theory is available that has been confirmed by long experience. The theory is summarised by a set of non-linear partial differential equations containing some coefficients. These equations, together with definitions of the terminology, were presented earlier in this chapter ("Modelling Sewer Hydraulics" by Havlik).

At this point, we introduce some practical aspects of modelling of open channels, provided by Cunge (in Mahmood and Yevjevich 1975), and summarised in Table 1.

TABLE 1. Practical application of mathematical models in an open channel

Type of activity	Input	System	Output
Prediction	given	given	required
Identification	given	required	given
Detection	required	given	given

Prediction is a major topic in urban drainage involving rivers and channels. In this case, the system is known (i.e., the river channel exists, but some changes may be planned). The input is also known (such as the inflow hydrograph, or the required load from the catchment), but the output is not known (velocity, discharge and stage variation are not known).

Identification typically involves calibration of the model set-up. Until the model has been properly calibrated, it cannot be used for simulation. The system is never known completely and calibration has to furnish the unknown elements. For instance, the roughness characteristics are adjusted until the computed and observed hydrographs along the reach are sufficiently close to one another.

Detection is a very special case of engineering activity in urban drainage. We do not know what caused the recorded output in our known system. Our task is to find this cause.

It is useful to mention some general conclusions based on general experience with 1D mathematical modelling in rivers and open channels:

- It is impossible to reproduce a river system exactly. We have to proceed by simplification, dividing the system in an urban area into cells and assuming that the friction laws are relatively simple, but adequate to globally represent the real behaviour of the system. For these reasons it is necessary to state the objectives of the study beforehand.
- Calibration and validation of the model is always necessary.
- The role of the engineer in charge of modelling is very important and must not be neglected. His or her knowledge and experience are necessary conditions for success, and are needed to calibrate the model correctly and design all new modifications of the model set-up.

2.1.1. *Governing Equations and Assumptions in Modelling Gradually Varied Unsteady Flow*

Hypotheses and definitions. The definition of gradually varied flow was introduced by Boussinesq in the following terms:

Consider the flow in an open channel of a small slope and a slightly curved axis. The cross sections of the channel are of arbitrary shape, constant or not, along the channel axis. The elements of the channel walls, of its bottom and of the free water surface form only small angles with this axis. The flow varies only slowly with time so that its unsteady character is not much accentuated. In such flows the velocity vectors are nearly parallel to the channel axis and the curvature of streamlines is small. This description of the flow forms the basis of the hypotheses used to establish the famous Saint Venant equations for gradually varied unsteady flow.

In addition to the material provided earlier in this Chapter by Havlik, several assumptions related to river channels are summarised below:

- The flow is one-dimensional, i.e., the velocity is uniform over the cross section. The centrifugal forces due to channel curvature are negligible.
- The pressure within the cross section is hydrostatic.
- Vertical acceleration is neglected.
- The water surface is horizontal in the lateral direction.
- The effect of boundary friction and turbulence can be accounted for by introduction of a resistance force.

2.1.1.1. Independent and Dependent Variables

1D description is sufficiently accurate for a wide range of applications in river channels in urban drainage. It is necessary to define independent and dependent variables which should be used in deriving the governing equations.

Dependent variables. 1-D unsteady flow in channels, assuming that the water density is constant, can be described by two dependent variables at any given cross section. Depending on the nature of the problem, we can define several pairs of variables. The equations will differ for different pairs of variables, but the physical assumptions will be the same. The most appropriate combinations of variables are:

- velocity **u** and depth **d**,
- velocity **u** and pressure head **p**,
- discharge **Q** and water stage **h**.

The user has to take into account that the depth and velocity may vary considerably over the cross section.

Independent variables. The dependent variables define the state of the fluid motion along the water course and in time, i.e., as a function of two independent variables :

- **x** for space description,
- **t** for time.

Two basic principles for definition of the **x** axis are known. The designer of the mathematical model selected one of them, and the user has to investigate the principle of the reference system in the modelling package. The **x-axis** may be introduced as a horizontal co-ordinate, or it may follow the bottom slope. The former system is more frequently used in 1D packages.

Cross sections have their own co-ordinate system based on **y** - the distance from the bank mark, and **z** - the bottom level elevation of selected points in a cross section.

2.1.1.2. Governing Equations

Since two dependent variables are sufficient to describe 1-D flow, we need only two equations, each of which must represent a physical law. However there are three laws which could be applied: **conservation of mass, momentum and energy.** The choice of conservation laws is extremely important, both from a theoretical and practical point of view, and it is dealt with in great detail in Abbott (1979). When the flow variables are continuous, either of the two representations may be used for model design: conservation of mass and momentum or conservation of mass and energy, which are equivalent. When the flow variables are not continuous (in such cases as a hydraulic jump, bore, dam break wave, etc.), either of the two representations may be used for model design, but they are not equivalent. Only one of them is correct. The mass-momentum conservation laws are applicable to both discontinuous and continuous situations, while mass-energy laws are not, as explained by Abbott (1979). For this reason, modern packages provide a 1D simulation technique based on the mass-momentum governing equations. Cunge *et al.* (1980) provided full derivation and all the details for the integral and differential forms of the governing equations. Integral formulation is valid for both flow cases (continuous and discontinuous) and is therefore more general. Better models can handle discontinuities by introducing a special numerical algorithm (to cover such phenomena as dam break waves, etc.).

The basic equations were introduced earlier for sewer systems, so we will simply present the resulting form of the governing equations based on Saint Venant's hypothesis.
Conservation of Mass : rate of change = inflow - outflow

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t} = 0 \tag{2.1}$$

and *Conservation of Momentum* : rate of change = inflow - outflow + impulse

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{\beta Q^2}{A}\right) + g A\frac{\partial y}{\partial x} + g AI_f = g AI_o \tag{2.2}$$

kinematic wave approximation

diffusive wave approximation

full dynamic wave approximation

Contributing terms in the mass equation:

- change of mass in control volume
- flux through upstream end (boundaries)
- flux through downstream end (boundaries)
- lateral inflow to control volume

Contributing terms in the momentum equation:

- change of momentum in control volume
- flux through upstream end (boundaries)
- flux through downstream end (boundaries)
- hydrostatic pressure changes at the upstream end
- hydrostatic pressure changes at the downstream end

- changes of bed elevation
- change in width of the channel
- bottom shear force
- wind shear force
- lateral exchange to control volume

2.1.2. *Evaluation of Solution Techniques*
Differential equations such as 2.1 and 2.2 are a mathematical description of real life unsteady flow. They express the physical laws that are thought to be most important ones for the phenomena studied, but they provide no direct answers as to the values of water stages and discharges, which are functions of time and space. These equations are too complex to be solved by analytical methods. However, it is possible to find approximate solutions, i.e., to estimate the water levels and discharges at a certain number of discrete points in the domain.

Discretisation is the process of expressing general flow laws, derived for a continuous medium, in terms of discrete values at a finite number of points in the flow field. A detailed description of all methods of discretisation goes beyond the objectives of these lecture notes, which aim to supply an overview of modelling in open channels. Details and a seminal source of information discussed in this section are provided by Cunge *et al.* (1980).

There are three classes of numerical solution methods that can be found in the current technical literature:

- the FEM finite element method,
- the MCH method of characteristics,
- the FDM finite difference method.

The principle of the *Finite Element Method* is that the parameter to be computed is represented by a chain of functions. At every grid point, a function is defined, representing a parameter (flow velocity, water level). The coefficients of the function at every grid point are optimised so that all discrete functions connect smoothly. The advantages of the method are that the numerical grid can be chosen freely, and boundary conditions are taken care of automatically. This method is well suited for equilibrium problems in a 2D plane, e.g., groundwater flow.

The foundation of the *Finite Difference Method* is as follows: functions of continuous arguments, which describe the state of flow, are replaced by functions defined on a finite number of grid points within the considered domain. The derivatives are replaced by ratios of differences. Thus the differential equations, i.e., the laws describing the evaluation of the continuum, are replaced by an algebraic finite difference relationship.

A finite difference scheme expresses the way in which the integrals and derivatives are replaced by discrete functions.

Numerical schemes applied in computational hydraulics to river simulation are either explicit or implicit. Since explicit schemes have problems with stability and accuracy, implicit schemes currently dominate in professional model packages. Two implicit schemes of FDM are commonly used in mathematical models for open channel flow:

- the Abbott - Ionescu implicit scheme working on a staggered computational grid
- Preismann's implicit scheme on full extension of a computational grid

A computational grid is a finite set of points sharing the same domain in the (x, t) plane as the continuous argument functions. Since a basic introduction to numerical schemes and operators was given earlier in this chapter, this topic is not dealt with here.

2.1.2.1. Stability, Accuracy and Economy of Solution

A comparison of numerical methods used in mathematical models is extremely difficult, unless there is a formal set of objective criteria that will determine the best method. Unfortunately, this approach is impossible to follow for practical reasons. Nevertheless, we can use some indicators or at least show which elements are likely to influence the choice. The basic arguments and theory can be found in standard texts (e.g., Mahmood and Yevjevich (1975), Abbott (1979) or Cunge *et al.* (1980)).

Stability criterion. When a numerical method is unstable, a small perturbation (e.g., truncation errors) of the exact solution will grow in time. In many cases this amplification is exponential and the error grows beyond any reasonable level after a few time steps. The oscillations appear with each time step and each space interval, as a result of a basically unstable finite difference scheme. Sometimes the growth of errors is less rapid, and is noticeable only after several tens of time intervals. This type of error is characterised by long wave oscillations spread over the domain, and is usually due to unstable schematisation of the resistance term. It is obvious that an unstable scheme cannot be used.

Stable schemes of finite differences may be divided into two categories:

- conditionally stable, usually with a time step Δ **t** limited by the Courant condition

$$\Delta t \leq \frac{\Delta x}{\left(U \pm \sqrt{g\, y_s}\right)} \tag{2.3}$$

where **U** - velocity, Δ **t** - time step, Δ **x** – length step, **g** - gravity, $\mathbf{y_s}$ - hydraulic depth

- unconditionally stable (implicit schemes) which allow an arbitrary time step Δ **t**.

Apparently implicit schemes are better for practical applications.

Convergence criterion. The concept of convergence assumes that a sequence of computations with an increasingly fine mesh tends toward the exact solution. A given scheme is convergent only if the sequence of solutions converges to the true solution.

One possible estimate of accuracy consists of considering linearised equations and comparing their known analytical solution with the numerical solution provided by a given finite difference method. The solutions are usually assumed to be Fourier series, and then the components of the two solutions are compared. Thus it is possible to define (see Liggett and Cunge in Mahmood and Yevjevich (1975)) two convergence criteria:

$$R_1 = \frac{\text{damping factor of the numerical solution}}{\text{damping factor of the analytical solution}} \tag{2.4}$$

$$R_2 = \frac{\text{celerity of the numerical solution}}{\text{celerity of the analytical solution}} \tag{2.5}$$

Accuracy criterion. Accuracy refers to the magnitude of differences between the observed prototype data (such as measured hydrographs) and computed results. This criterion is obviously most important, but it is practically impossible to formulate. There might be several reasons for discrepancies or for a lower level of agreement between the results of a mathematical model and the prototype:

- inaccurate simplifications and approximations in the basic equations fail to simulate the complexity of the prototype,
- insufficiently accurate measuring techniques (such as surveying errors in channel networks)
- insufficient data (such as unaccounted for inflow or outflow – e.g., by infiltration)
- neglected phenomena (such as bed variation due to erosion or deposition, variation of bed resistance with season or vegetation growth etc.)
- poor schematisation of the open channel network

2.1.3. *Data Needs for 1D Open Channel Modelling*

Data needs are infinite. We are very fortunate if data of good quality are available when the user wants to apply a 1D mathematical model. Data collection is a time consuming and usually very expensive process. The data that are needed depend fundamentally upon the purpose of the model, and can be grouped into three classes (Cunge *et al.* 1980):

- topographical data
- hydraulic and hydrological data needed for initial and boundary conditions
- supplementary data

2.1.3.1. Topographical Data

The topographical data used in building mathematical models of rivers may be divided into:

- qualitative data
- quantitative data

Qualitative, reconnaissance type descriptions of the river, its tributaries and flood plains in the urban area involve identification of the physical conditions, which determine the flow development pattern (the existence of banks, dykes, elevated roads, local obstacles, etc.). Qualitative data may be obtained by field investigations, inquiries, satellite and aerial photographs, and from reports. The essential objective is to learn enough about the flow patterns in the domain of interest to be able locate cells, computational nodes and other model elements.

Quantitative topographical data are needed for model representation of the river and the flood plain in the urban area. There are essentially three factors in flow simulation of rivers that depend directly upon the topography:

- preferential flow directions which depend on infrastructure development
- wave celerity, which depends on cross-sectional characteristics
- the volume of water stored in the domain

The river valley may be divided into three zones: dry season open channel, flood channel, and flood plain. All data are obtained either from maps, or from special field surveys. Quantitative data are obtained from three background sources, and are usually available in the following forms:

- *longitudinal profiles* (with a defined thalweg, channel banks, natural levees, road and railway profiles)
- *cross sections* are needed at more or less equal distances for channel sections, flood plain sections and detailed hydraulic structure descriptions.
- *inundated area maps,* in which the flooded zones are modelled as a network of interrelated cells, and for each cell, the modeller must establish the elevation of dykes, roads and other obstacles around the cells, and also the storage versus water elevation curve.

2.1.3.2. Hydraulic and Hydrological Data

Hydraulic data needs fall into two general categories:

- boundary and initial conditions
- discharge and stage observations for calibration and verification purposes

Initial conditions. The user has to specify an initial condition for all variables included in the selected governing equations. Since the overall situation, and thus the magnitude of the variables, is usually not known when we start simulations using the mathematical model, an initial state of the system has been reached by an automatic procedure which can be an integral part of the model. In such automatic calculation, a steady state backwater calculation is performed using the steady state mode of the simplified Saint Venant equations. This procedure determines the water level and discharge, using the boundary condition values at the start of the simulation. An iterative procedure is usually used to obtain a correct discharge distribution between diverging branches.

A hot start is very good tool for setting up the desired, user-specified initial conditions that are taken from an existing result file. In such a case, the other conditions must of course be consistent with the former case, indeed.

Boundary conditions. All models require boundary conditions that specify the anticipated impact on the simulated domain. The resolution of the time varying boundaries has to be smaller than the time step used in the simulation. In a 1D modelling approach, we can divide the boundary conditions into:

- external boundary conditions:
    - upstream boundary condition
    - downstream boundary condition
- internal boundaries which are located inside the simulated domain.

External boundary conditions are required for all units modelled, i.e., all upstream and downstream ends of modelled branches, which are not connected at a junction. The relationships applied at these borders may consist of:

- constant values of variables (such as stage H = a constant, or discharge Q = a constant)
- time varying values such as Q(t)
- a relationship between h and Q (e.g., a rating curve)

The choice of boundary condition depends on the physical situation being simulated and the availability of data. One very important aspect is the identification of a river flow regime. As it is well-known from the theory of unsteady flow in open channels, there are three regimes in rivers which are defined by the Froude number (subcritical flow, critical flow and supercritical flow):

$$Fr = \frac{U}{\sqrt{g\, y_s}} \qquad (2.6)$$

where y_s = hydraulic depth (Area/ water surface width).

The flow regime itself defines the rate of influence of the boundary conditions from upstream and downstream ends of the domain.

Typical upstream boundaries:

- constant discharge Q (from a reservoir, an "unlimited" water body)
- discharge hydrograph of a specific event (Q= f(t)), (a hydropower station, pump station, discharge controls on locks and weirs)

Typical downstream boundaries:

- constant water level (e.g., in a large receiving water body)
- time series of water levels (tidal cycle, water level control, downstream condition in a river)
- reliable rating curve (e.g., from a stream gauging station)
- critical outflow (fall, pipe end, weir)

Internal boundary conditions are required at specific locations where the natural behaviour of the flow phenomena is influenced by hydraulic structures and their operational rules. The structures can change their characteristics during the period of simulation. The impact of internal boundary conditions could be visible throughout the domain or it could be local. The following are examples of hydraulic structures, performing as internal boundary conditions in a channel, and which can be found in the urban domain:

- broad-crested weir
- special weir
- spillway
- culvert
- bridges
- dams
- regulating structures and real-time control operational strategies applied to rivers
- CSO outflows
- junction of rivers or channels
- imposed water level or discharge as a function of time
- imposed water level or discharge as a function of another variable (reservoir operation)

Lateral inflow can enter through air (direct rainfall / runoff from related catchments), the banks and bottom of the channel (seepage, groundwater supply) and side structures (outflow devices, bottom outlets, etc.). It can be introduced as a point source (discharge or load of pollutant), or a source distributed along a channel reach (e.g., infiltration in $m^3/s/km$).

2.1.4. *1D Schematisation*

Modelling can be explained as schematisation of the real world. This is particularly true in the case of 1D modelling, because many features are simplified and many others are omitted. Schematisation is the most important aspect of modelling. A modeller has to improve the behaviour of the mathematical model by the best possible schematisation of topography, hydraulic features and numerical parameters, in order to make the response of the model very similar to that of the real world prototype. The best test of the model schematisation is the execution of the calibration and verification procedures.

Topological schematisation focuses on definition of elements and their interconnections. Most deterministic models consist of:

- branches
- nodes (crossings of branches)
- grid points (computational points)
- cross sections (at defined nodes or grid points)
- external boundaries
- internal boundaries.

Hydraulic schematisation deals with setting up parameters of the continuity equation, momentum equation, and processing of cross section data. The hydraulic discretisation of 1D models also requires that boundary conditions are correctly defined.

Bed resistance. One of the most important items in schematisation is bed resistance. Better models allow several types of bed resistance descriptions, as discussed in Chow (1959) and Engelund (1966).

The resistance radius, R_*, is calculated after Engelund (1966) as

$$\sqrt{R_*} = \frac{1}{A}\int_0^B y^{\frac{3}{2}}\, db \tag{2.7}$$

where y is the local depth and B is the width of the water surface within the same river reach. It is derived in a slightly different way than in standard texts:

$$R_h = \left(\frac{\sum_i^N \left(\dfrac{A_i^{5/3}}{r\, P_i^{2/3}} \right)}{A} \right)^{3/2} \tag{2.8}$$

where P_i is the wetted perimeter of parallel channels, i.e., zero shear, N – number of channels, A_i – cross-sectional area, and r – coefficient of relative resistance.

The choice between R* and R_h should be based on a hydraulic schematisation of a cross section. We can draw a conveyance value plot based on h, vertical elevation, and compare results. Usually R* gives a better vertical representation of conveyance for significant variations in shape, such as the cross sections with flood plains, which are typically found in urban areas.

Sudden changes in variables. Reverse flow in channels can cause problems during computation. Changes of flow direction will lead to local oscillation in the momentum equation terms, especially the term:

$$g\, A \frac{Q|Q|}{K^2} \tag{2.9}$$

for which Verwey suggests a special discretisation (Cunge *et al.* 1980).

Small depth problems. In certain situations, computational difficulties develop when physical flow depths are small, though no such problems are found in nature. This is a prime example of a problem, which should be handled by the program itself, without any action by the user. A small-depth situation usually precedes or follows a 'zero depth' situation. In a branched model, which includes only channel flow, a zero flow situation is seldom found. However, in looped models, flow in the flood plain or in certain channel branches can be nil during low flow periods. This can lead to disconnection of the computational algorithm in certain areas, and this disconnection can invalidate the solution algorithm. One remedy is to introduce a small leakage of discharge from an adjacent weir or from some other structure. Another solution was suggested by

Preissmann and is known as the Preissmann slot. The problem of low flow conditions is handled by introducing a narrow slot underneath the cross section, so that the calculation can continue when the water surface falls below the slot top. A small amount of water is automatically added to prevent complete drying out.

Weir flow oscillation. Closely related to the dry bed problem is the weir flow oscillation problem. In certain situations and for certain modelling systems, which do not use an iterative solution technique, the direction of flow over the weir can be opposite to the water surface slope. This oscillation is caused by a difference between the given magnitude of discharges and the time step.

Coefficient of non-uniform momentum distribution. The coefficient β reflects the fact that since local momentum flux is proportional to the square of the local velocity, the overall mean channel velocity is not representative of the overall momentum flux, unless it is corrected by the Boussinesq coefficient β

$$\beta = \frac{\int_0^b u_z^2\, h_z\, d_z}{U^2\, A} \tag{2.10}$$

where **u** - local velocity of the width element, $\mathbf{h_z}$ - depth of the width element, **A** - total area, **U** - average cross sectional velocity, **b** - width of the cross section at the water surface.

Storage width b_{st}. The purpose of introducing the storage width b_{st}, or the storage cross-sectional area ($\mathbf{A_{st} > A}$), is to take into account the fact that flooded zones often act as storage areas. The velocity of water in storage is nil, in the general flow direction, and thus it does not contribute to the overall momentum flux in the cross section.

2.1.5. *Calibration and Verification*

Calibration. A mathematical model is a simplified, discrete representation of a complex and continuous physical flow situation. Three-dimensional terrain features are represented by one or two dimensional elements. Model calibration is the process of adjusting the dimensions of simplified geometrical elements and the values of empirical hydraulic coefficients in such a way so that the flow events simulated by the model will reproduce, as closely as possible, the comparable natural events. A model's potential for reproducing and predicting real flow events, and the potential quality of its calibration, depend on the amount and quality of topographical, topological and hydraulic data available for the watercourse under study.

An important unknown factor in the modelling of open channel flow is the bed roughness. Relations between the flow condition, bed material and roughness have been established. The geometric and hydraulic characteristics of a river reach are usually represented, at least in 1D and 1D+ mathematical models, by three dominating functions of water stage **h** : width **b(h)** , wetted area **A(y)** and conveyance **K(y)**. These functions provide an appropriate representation of all relevant physical influences at any river cross section. These functions are assumed to represent not only one particular cross-section, but in fact the entire reach between two adjacent cross sections.

In engineering studies, calibration is usually applied as a trial and error procedure, but mathematical optimisation techniques are also available.

Verification. There are often misunderstandings about the difference between calibration and verification of a model. After successful calibration, at least one set of prototype data not used in calibration, should be available for verification. The model should represent the measured verification data within some predetermined limits. In principle, no additional model adjustments are allowed at this stage. Of course, if the verification process fails, the calibration phase should be repeated.

2.2. CASE STUDIES

There are several case studies and pilot projects, by which we can document the implementation of ideas presented in the earlier sections. Such a complex study was discussed by Zeman *et al.* (1994). The study focused on the Ploucnice river basin, where a large project, Revitalisation of the Ploucnice River Basin, was conducted. The river system in the northern part of Bohemia is presented in Fig. 3, together with basic hydrological information.

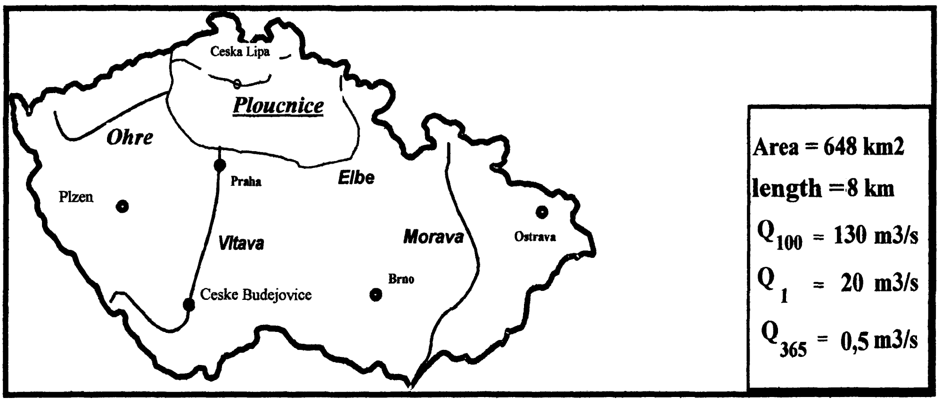

Figure 3. The Ploucnice project location

The Ploucnice pilot project began in 1991 and constitutes the first large-scale application of hydroinformatic tools in a fully integrated form in the Czech Republic. The environmental problems in this area result from uranium ore mining at Hamr and Straz pod Ralskem, without controlling discharges of polluted mining waters into the river system, over a period of more than 20 years. The processing line for ore extraction operated without any decontamination station up to 1988, and the mining wastewaters were discharged directly into the river. The European Community wanted to identify the locations of polluted sediment materials, and to provide the data required for developing an acceptable solution to this environmental problem as soon as possible, because the Ploucnice river is a tributary of the Elbe River. Several towns were included in this study (such as Mimon and Ceska Lipa)

2.2.1. *Objectives of the Ploucnice Pilot Project*

The principle objectives of the partners was to apply modern hydroinformatics tools in the Ploucnice river basin, and to demonstrate the ability of using mainly simulation techniques to solve the most difficult environmental problems. The Institutes involved insisted not only on obtaining the methodology for the tools applied, but above all, on designing technical measures in the study area, leading to an ecological stability at the final stage of the project. In order to attain these ambitious objectives, the following tasks were proposed:

- to investigate historical files and records of relevant data available at the Ohre Water Board,
- to start an intensive data collection programme and evaluation procedure for all required topographical data, necessary hydrological characteristics, hydrodynamic and qualitative parameters, by utilising the most modern technologies
- a man-made flood event was generated from October 19 to 23, 1992, by water release from the Horka reservoir ($Q_{max} = 6.0\ m^3 \cdot s^{-1}$), and all useful data were recorded by a team of more than 80 (primarily for calibration purposes),
- after setting up the domain of interest, requirements for initial and boundary conditions were formulated,
- setting up 1D and 2D models for hydrodynamic (HD), water quality (WQ) and sediment transport (ST) simulations, and links between them,
- investigation of 1D hydrodynamic behaviour in the area of interest by the MIKE 11 package to provide input files for AD, WQ and ST simulations and boundaries for 2D simulations, respectively,
- simulation of 2D hydrodynamic behaviour close to the area of the Hradcany railway bridge, where the polluted sediment material settled in the past,
- to design a set of adequate technical and agricultural measures to stabilise the contaminated materials in the area, where the concentrations of radioactive pollutants are most critical and require remediation as rapidly as possible.

The main objectives of the pilot project can be summarised in the following way:

- to verify the set of modelling tools with forecasting ability, in order to investigate the potential mobility of polluted materials in the area of interest in the river channel and adjacent flood plains,
- to provide the Ohre Water Board (supervising the Ploucnice basin) dynamic modelling tools, in an integral form, for environmental impact assessment studies, and technology transfer to other Water Boards.

The above stated aims were reflected in the procedures and actions formulated by the Ploucnice Pilot project.

MIKE 11 (DHI 1992) is a professional engineering software tool for one dimensional modelling and for detailed design, management and prediction of behaviour of river and flood plain systems. MIKE 11 was selected as a general tool for 1D simulation in the area of interest (the 1D model covers the Ploucnice river from the confluence with the Elbe as far as the upper reaches in Straz pod Ralskem). The MIKE 11 package proved to be a very important tool for various tasks within the framework of the pilot project.

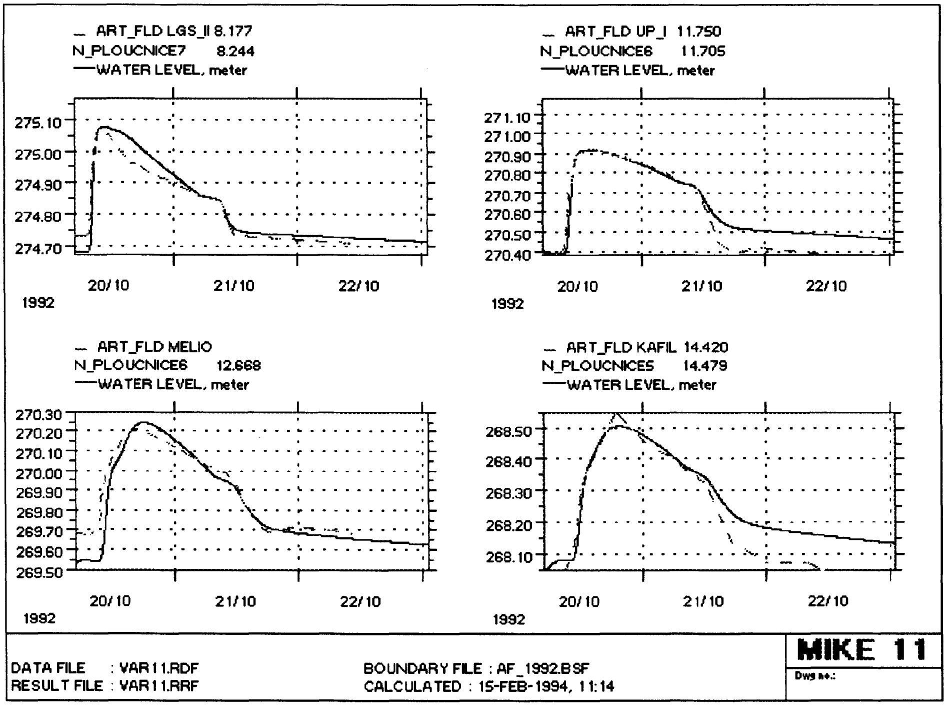

Figure 4. MIKE 11 result for the man-made flood.

The Ploucnice River has preserved its natural state as a meandering channel in flood plains, forming wide valleys in some places. Due to this fact, the length of the channel streamline is much longer there than the length of the flood plain streamlines under flood event conditions. For this reason, there are several sub-reaches that can hardly be treated as 1D cases, because during floods, the flow has at least a 2D character. Since MIKE 11 can handle both a branch system and interconnected branches in loops, the authors set up two independent models:

- The channel model: which can handle bank-full discharges flowing in the Ploucnice, taking into account only the structures located directly in the channel itself,
- The flood plain model, which describes the complex behaviour during flood events.

2.2.1.1. River Channel Model (Mike 11)

The channel model set-up serves primarily for detailed simulation of hydrodynamics, water quality and sediment transport for moderate discharges below 10 $m^3 \cdot s^{-1}$.

Since detailed information was expected from the results of the channel model, the average distance between cross sections was approximately 200 m within the total reach between Straz pod Ralskem and Ceska Lipa (44 km). The model for the sub-reach between Ceska Lipa and Decin was set up in a similar way. The average bank-full width of the channel in the reach under consideration varied from 6 to 15 m. The channel set-up contained 235 cross sections and only 6 structures, that had large impacts on the hydrodynamic and qualitative parameters. Approximately 35 % of the reach is in some way trained, while the rest of the channel consists of natural courses with a lot of singularities and irregularities. Due to the pollution caused by municipal wastewaters and agricultural fertilisers, the channel is densely filled with vegetation in the spring and summer season. Unfortunately this caused problems during stream gauging and the development of rating curves.

The Manning resistance parameters obtained from the first calibration trials showed inconsistencies in magnitude at different cross sections. Seasonal variation (vegetation growth in channel) was proven. During the man-made flood, a very intensive measurement programme was conducted to explain the unusual flow behaviour and to assist in selecting the correct resistance parameters for model applications. Calibration based on the man-made flood was successfully completed, and provided consistent results as shown in Fig. 5.

2.2.1.2. Flood Plain Model (Mike 11, 1D+)

For higher than bank-full discharges, a 1D+ flood plain model was used to develop flood protection measures in the basin, and to compare the results of 1D+ and 2D modelling in the domain of interest. The model 1D+ set-up between Ceska Lipa and Straz pod Ralskem comprised 126 cross sections and a looped and branch system was designed with 64 structures (broad-crested weirs located on the river banks, connecting channels in the flood plain, where the main stream lines are located during the floods). First simulation results were unsatisfactory, partly because calibration of a 1D+ model is rather complicated for the following reasons:

- the dynamic response through complicated connections is usually quite slow, and for this reason, it is important to have a longer event available with a sufficient number of measurement locations for calibration purposes,
- results of calibration may vary greatly according to the stage variations,
- changes of roughness in time typically occur not only in channels, but also in flood plains, (clogging of dense bushes, bending of long grass under the higher depth and/or velocity gradient),
- infiltration, evapotranspiration and other types of water losses,
- the number and extent of complicated connections of nodes was very close to the limit of what the MIKE 11 simulation package can handle. Some of the schematisations are presented in the Fig. 6,
- even this skilled team had problems in co-ordinating the project properly, which caused several delays and misunderstandings; this led to introducing a very strict hierarchy of responsibility among the co-operating groups,

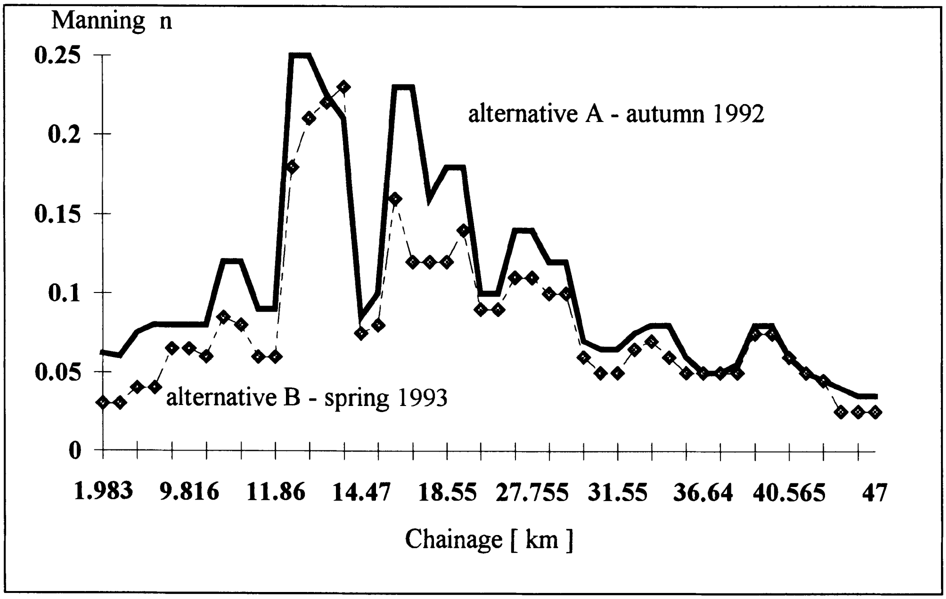

Figure 5. A comparison of seasonal roughness

The flood plain model served primarily for comparison of results between the 1D and 2D simulations for the same domain (Hradcany railway bridge) and provided very useful research materials for those interested in decision making policy and application of the computational tool to the natural courses of river systems.

2.2.1.3. Problems in Execution of the Project

Several problems occurred during the execution of the project that can be generalised, to some extent.

Surveying and measurements. When comparing the area photogrammetry with manual surveys, some differences, due to different measurement and data evaluation techniques, were observed. To obtain better data for channel cross sections, new measurements were taken at a few locations. The main reason for these new measurements was to provide more measured points on the river bed, where there had been only one point before. Good results were obtained from measurements in the flood plain areas. When comparing the topographical and manual surveys, the maximum estimated data error was in the range from 0.15 to 0.30 m, which was considered satisfactory.

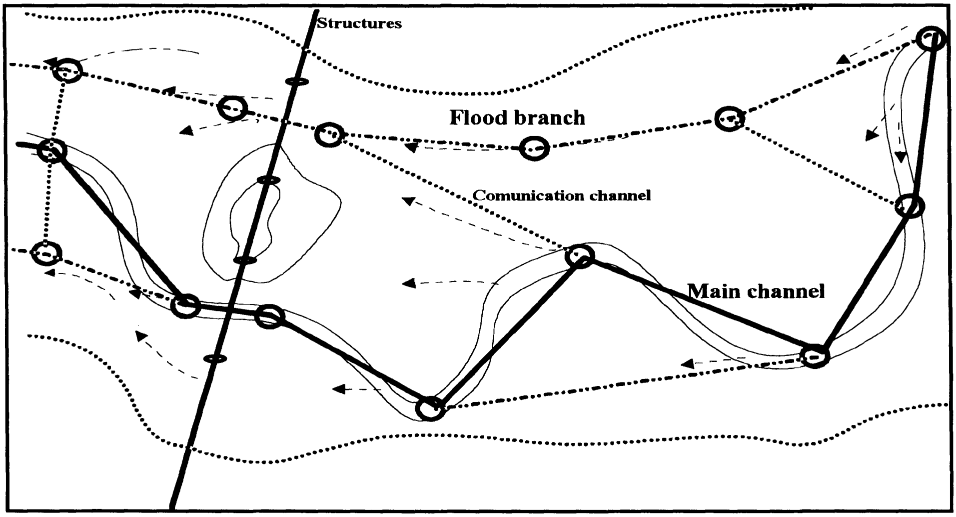

Figure 6. Topological schematisation of the flood plain 1D+ model set-up.

Hydraulic Structures. All the river structures (24) were surveyed manually. The structures in the flood plain served for good numerical representation. 1D+ schematisation, however, was estimated successfully from the GIS system.

Roughness. Based on knowledge of the terrain, standard literature values were used (Chow 1959) as a first approach for set-up and simulation. During the calibration procedure, large deviations of roughness from the literature values were discovered for this river. The Manning number values were in the range from 0.040 to 0.250, with common values ranging from 0.040 to 0.110. Extreme values obtained in calibration (range 0.120–0.250) can be explained by the measurement difficulties caused by vegetation and river meandering.

The data available for calibration and verification were not ideal. At a number of sites, the Hydrology Institute measures discharges, which are obtained from water stages substituted into rating curves. Since no records of the changing rating curves have been kept, it was not possible to calculate back the time series of water levels. Thus, the only water levels available for calibration of resistance values in the 1D+ dimensional hydrodynamic model were the measurements taken during the man-made flood. For rainfall data to be used in calibration of the rainfall / runoff model (NAM), it was difficult to find a continuous period of several years, for which a sufficiently large number of rainfall station records would be available.

The project proved to be a good opportunity for coupling different models, and using the results from one as input to another one, and vice versa. The NAM module proved to be a reliable tool for estimating discharges in sub-catchments, where no discharge or water level measurements were available. MIKE 11 was used for checking the backwater effects upstream of the flood plain area, which were caused by changes in the topography and simulated by the FLUVIUS model. Hydrodynamic results from FLUVIUS were used as input for the advection/dispersion and sediment transport simulations done by MIKE 21.

2.2.1.4. Pilot Project Summary

The work completed during the preparation stage can be summarised as follows:

- hydroinformatics tools were applied for the first time on such a scale in the Czech Republic
- co-operation between DHI , a leading European Institute, the Ohre Water Board, the Water Research Institute, the Czech Technical University, Hydrosoft and Hydroinform led to multidisciplinary research based on a combination of hydroinformatic tools (DMT, MIKE 11, FLUVIUS, SEZAM, MIKE 21, GIS MGE INTERGRAPH),
- the adopted project methodology is now available for further application by other end users,
- significant results were achieved, especially in the evaluation of the Manning roughness coefficients applied for 1D, 1D+ and 2D modelling, and their relationships (a time varying value of n during the flood event, as used in the 1D model represents approximately 80% of the value used for 2D applications in order to obtain the same results),
- the study dealt with hydrodynamic simulation methodologies based on the most advanced hydroinformatic tools (such as Remote Sensing Methods, Digital Model of Terrain, GIS, 1D and 2D hydrodynamic simulation tools), which provide a research team with a fundamental platform for a decision making policy. This platform was adopted by the local water management authorities.

3. Two-Dimensional Environmental Hydraulics Models for Urban Areas

3.1. 2D HYDRODYNAMIC MODELLING IN OPEN CHANNELS

Two dimensional (2D) mathematical modelling of unsteady flows in rivers and flood plains can be used to identify the impacts of human activities, from the hydrodynamic point of view. Modern software is designed as the so-called "full technology line", according to the current standard for the fourth generation of models, capable of communicating with GIS or database systems. These modelling tools can be considered as direct support for decision makers in the areas of water management and ecology.

3.1.1. *Introduction*

The flow in a river and adjacent flood plain system is three-dimensional, time dependent and primarily turbulent. For a given river and flood plain system, if the water movement is mainly horizontal and the effects of small scale velocity fluctuations are aggregated into shear stress, the description of flow can be simplified by a vertically integrated two dimensional unsteady model. Many existing models are historically based on published elements of the well-known Leendertse model (Leendertse 1967). The finite difference method and finite element method are mostly used for numerical solution of the system of complete non-linear equations. In the flow equations, hydrostatic pressure distribution is assumed. This section describes the models, which are based on a fractional algorithm method evaluating wave propagation by an Alternating Direction Implicit (ADI) algorithm. Flooding and draining of the flood plain, wind effect, and various forms of bed resistance, are accounted for.

3.1.2. *Governing Equations*

The two-dimensional partial differential equation for unsteady flow in shallow water can be derived using the usual Saint Venant hypotheses of vertically uniform horizontal velocity and negligible vertical acceleration (hydrostatic pressure distribution). Then the basic equation contains the terms representing conservation of mass and momentum.

The existing mathematical models usually provide numerical solutions to the vertically integrated equations of continuity and conservation of momentum under the assumption of constant water density. The set of governing equations includes the continuity equation:

$$\frac{\partial h}{\partial t}+\frac{\partial p}{\partial x}+\frac{\partial q}{\partial y}=0 \tag{3.1}$$

the momentum equation in the x direction:

$$\frac{\partial}{\partial t}(\rho u h)+\frac{\partial}{\partial x}(\rho u^2 h)+\rho g h\frac{\partial h}{\partial x}+\frac{g h^2}{2}\frac{\partial \rho}{\partial x}+\frac{\partial}{\partial y}(\rho v h u)=0 \tag{3.2}$$

and the momentum equation in the y direction:

$$\frac{\partial}{\partial t}(\rho v h)+\frac{\partial}{\partial y}(\rho v^2 h)+\rho g h\frac{\partial h}{\partial y}+\frac{g h^2}{2}\frac{\partial \rho}{\partial y}+\frac{\partial}{\partial x}(\rho u h v)=0 \tag{3.3}$$

in which $\boldsymbol{u,v}$ = vertically averaged velocity components; $\boldsymbol{h}$ = water elevation; $\boldsymbol{p,q}$ = unit discharges; $\boldsymbol{d}$ = water depth; τ_{wx}, τ_{wy} = wind stress components expressed as function of wind velocity (10m above surface).

3.1.3. Fractional Step Approach

The solution of eqs. (3.1 – 3.3) is based on a finite difference implicit scheme. The values of a variable at time (n+Δt) depend on their values, at other locations, at time (n+Δt). Therefore, computation of a set of variables requires the solution of a whole system of linear simultaneous algebraic equations. Use of the alternating direction implicit method (ADI) and application of the double sweep algorithm avoid the matrix inversion problem. Benque *et al.* (1982) have proven an approach to the special decomposition of the governing equations (fractional step approach) that may prevent the difficulties related to calculation of the acceleration terms. This follows from the fact that different numerical methods are applied at each fractional step. The particular impact of each fractional step of the algorithm mentioned above is incorporated through the superposition procedure within one time step Δt. The discrete values of variables are described in a space-staggered grid.

The ADI algorithm is defined for approximation of the basic equations in a space -staggered grid (Abbott-Ionescu type), where Δx and Δy represent the distance between "information points", e.g., between *h* and *p* point in the x-direction, and between *h* and *q* points in the y-direction.

Boundary conditions in the staggered grid, or time series h(x,y,t) or p(x,y,t), q(x,y,t), respectively, have to be specified at the particular boundary profiles. A special type of boundary condition may be the reflecting boundary, the solution of which is based on the Riemann invariant (Dirichlet type of boundary condition) (Stelling, 1984). This allows computation of unsteady flow without the proper set of time dependent downstream boundary conditions. The initial conditions are defined as the values of h(x,y,t), p(x,y,t) and q(x,y,t) at the time t=0, and are required at each grid point.

3.1.3.1. First Step - wave propagation, wind forces, bed resistance

In the first step of computation, terms containing wave propagation, wind forces and bed resistance are taken into account. The appropriate set of equations is solved by a double-sweep algorithm. The equations are algorithmically connected in the first stage in such a way that ***t*** advances ***u*** directly from nΔt to (n+1/2) Δt.

The subsequent computation connects the equations to advance *v* directly from nΔt to (n+1)Δt while advancing *h* from (n+1/2)Δt to (n+1)Δt. In this scheme, information on the *h* values only is accumulated during each cycle.

3.1.3.2. Second step - advection

The advection step is represented in the form of modified transport equations

$$\frac{\partial}{\partial t}(p)+\frac{\partial}{\partial x}(p^2/d)+\frac{\partial}{\partial y}(pq/d)=0 \tag{3.4}$$

$$\frac{\partial}{\partial t}(q)+\frac{\partial}{\partial x}(q^2/d)+\frac{\partial}{\partial y}(pq/d)=0 \tag{3.5}$$

The details of this method can be found in Abbott *et al.* (1981). The approximate terms of the given equations are solved using a modified nine-point method of characteristics.

3.1.3.3. Diffusive Interface

A diffusive interface enables us to control the rate of numerical diffusion during computations. This may be considered a great advantage in the case of extreme changes in hydrodynamic parameters. The diffusive interface is attached to the original set of governing equations in the form

$$\frac{\partial f}{\partial t}=D_{DIS}\left[\left(\frac{\partial}{\partial x}\left(\frac{\partial f}{\partial x}\right)\right)+\left(\frac{\partial}{\partial y}\left(\frac{\partial f}{\partial y}\right)\right)\right] \tag{3.6}$$

where D_{dis} represents the diffusion coefficient.

3.1.4. *Mathematical Models for Two-Dimensional Unsteady Flow*

3.1.4.1. Fluvius

The model Fluvius (Hydroinform 1994) is a 2D unsteady flow model for rivers and flood plains, possessing the modern characteristics described in Section 3.1. It is a fourth generation model with graphical post-processing and communication with GIS or database systems. It has been applied by several agencies, with minimum training efforts.

The FLUVIUS mathematical model is based on numerical integration (FDM) of the complete set of non-linear equations for 2D unsteady flow in the horizontal plane. In the flow equations, hydrostatic pressure distribution is assumed. The algorithm used in the mathematical model is based on a fractional step method, and was described in Section 3.1.3.

The computational part of the model is written for an FTN77/x86 32-bit compiler, with the maximum addressing memory limit of 2GB. It can currently be run also on workstations and mini-computers working in the Unix operating system, and a version for NT-Windows is tested. Pre- and post-processing is available on a PC platform. The maximum number of grid points tested on PC's was 1,000,000 points, requiring 64MB RAM. A direct interface to GIS was designed to enable the user to work with pre-processed data in the form of isohypses and digital planes of the terrain. A transformation unit provides the user with a free location of the computational grid in the area of interest. The graphical modules serving for presentation of the results meet all requirements with a high degree of flexibility and simplicity, including animators of hydrodynamic simulations, particle tracing methods and a results browser working under Windows 3.1 or Windows 95.

3.1.4.2. MIKE-21

MIKE 21 is a comprehensive modelling system for two dimensional free-surface flows (DHI, undated) and the related problems in the following fields of application:

- Coastal hydraulics and oceanography: tidal hydraulics, wind and wave generated currents, storm surges, hindcast and forecast, tsunamis (tidal waves).
- Waves: wave agitation in harbours, non-linear transformations, ship motions, harbour resonance, hindcast and forecast, design wave parameters.
- Sediment processes: cohesive and non-cohesive sediments, effects of waves and currents, littoral transport, impact due to dredging and reclamation.
- Environmental hydraulics: water quality, oxygen depletion, euthrophication, heavy metals, cooling water, sewer outfalls.

The hydrodynamic module simulates water level variations and flows, in response to a variety of forcing functions, in lakes, estuaries and coastal areas. The water levels and flows are resolved in a rectangular grid covering the area of interest, when provided with the bathymetry, bed resistance coefficients, wind field, hydrographic boundary conditions, etc. The system solves full time-dependent non-linear equations of continuity and conservation of momentum. The solution is obtained using an implicit ADI finite difference scheme of the second-order accuracy. MIKE 21 is available on a range of UNIX workstations and on PCs using SCO UNIX.

3.1.4.3. BOSS

BOSS is a two-dimensional surface water modelling package that can model water surface elevation, flow velocity, contaminant transport and dispersion, sediment transport and deposition, and both subcritical and supercritical flow for complex flow problems (BOSS International, undated). It is possible to model single- and multiple-opening bridge and culvert roadway crossings, wetlands, irregular flood plains, split flows and other complicated flow situations.

Along the flow boundaries and at each node in the finite element mesh, the water surface elevation, flow velocity, pollution contaminant concentration, and bed scour and deposition are computed. The software can find solutions for a single instance in time (steady state solution), or during a series of time steps (unsteady flow solutions). Unsteady flow solutions can be used to model flow fluctuations caused by inflow hydrographs, tidal cycles and storm surges. The software is available for personal computers using Microsoft Windows, Windows 95 and Windows NT, and also for most UNIX graphic workstations.

3.1.5. *Adjustment Procedure and Calibration of the Model*

During the development of a model, many numerical experiments and tests need to be undertaken. These can be subdivided into two classes.

1. Numerical experiments with simple geometries (such as static tests, steady tests, dynamic cube - type tests, swing tests). These experiments are mainly used for debugging numerical modules and/or for studying the properties of the model, including its stability, accuracy and phase errors.
2. Calibration of the model using a real topography and hydrodynamic parameters based on a simplified scheme of the river channel and adjacent flood plains. A mathematical model can be compared with the data measured in the field, with model data from (e.g., from aerodynamic models), or with data from other well-established mathematical models.

3.1.5.1. Computer Animation

Computer animation plays an important role in the development and life cycle of a mathematical model. There are three main reasons why it is so important: for debugging the model, for data (results) pre-processing and visualisation, and for understanding what is really happening inside the model. One model run usually contains several hundreds of time steps i.e., 300MB of computed data and only 2-3 time levels are usually stored on a hard disk, all other data are immediately lost. Animation is a possible way of saving these data in a very effective form, without large storage space requirements. Five hundred PCX files can be represented by a total memory of 5-6MB, and can easily be studied and visualised.

The following example (Fig. 7) shows a numerical instability of a node; the selected array of the unknown variable (water depth in this case) is drawn on the screen.

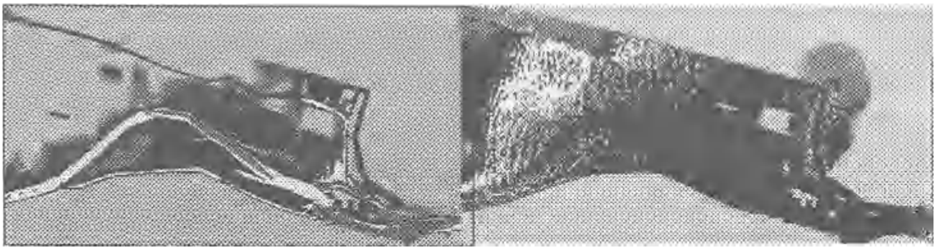

Figure 7. Computer animation displays a numerical instability of the model.

3.1.6. *Applications in Urban Areas - Flood Maps and GIS*

The Geographic Information System (GIS) is a tool for managing, analysing and displaying geographic data and other data, which can be related to geographical objects.

A GIS model can be efficiently used for data preparation and verification, and for data post-processing and visualisation. Any user may benefit from GIS as a graphical data processor for displaying results from the numerical simulator in the form of graphs, maps, thematic maps; 2D, pseudo 3D and 3D images; for browsing results; for using grid-analyst tools to work on computed data; and for geo-coding of data.

GIS commands for selecting objects, with the help of querying, selecting and SQL Select, enable us to prepare input data for the model and to analyse the results. Using SQL Select, it is possible to create query tables containing information that was only implicit in the database tables. SQL utilisation, in combination with other basic units, greatly improves the efficiency of the application of simulation packages.

GIS fulfils two main functions in the management of flood plain areas:

- to delineate the flood plain land
- to examine the impact of alternative flood mitigation and flood protection measures on flood level and extent.

The GIS application that reads data from results of two dimensional simulations can be designed to generate 2D and 3D water levels, flood inundation and flood depth maps, and flood risk evaluations. The system is mostly based on a digital terrain model.

3.2. 2D MODELLING OF WATER QUALITY AND TRANSPORT PROCESSES IN OPEN CHANNELS

Mathematical models that can be used in environmental impact assessment usually consist of integrated submodels describing hydrodynamics, sediment transport, transport-diffusion and water quality.

3.2.1. *Water Quality Modelling*

Water quality modelling may be used to investigate environmental problems caused by such pollution sources as domestic and industrial sewage, and agricultural runoff in coastal and flood plain areas. The water quality module is mostly integrated with the advection-dispersion module, which describes the physical transport processes at each grid point covering the area of interest. Other data required are concentrations at model boundaries, flow and concentrations from pollution sources, water temperature, etc.

The water quality module can be used for a large range of environmental investigations, such as studies of public health problems caused by bacteria from sewage outfalls and other sources; survival of bacteria related to various environmental conditions; oxygen conditions affected by BOD and oxygen consuming substances; evaluation of potential for euthrophication problems related to nutrient levels (nitrogen and phosphorus); decay of chemical substances, and effect evaluation based on the resulting concentration levels.

3.2.2. *Transport Processes*

3.2.2.1. Advection-Dispersion

The transport-diffusion module simulates the spreading of dissolved or suspended substances in an aquatic environment, under the influence of fluid transport and natural dispersion processes. The substance may be of any kind, conservative or non-conservative, inorganic or organic; salt, dissolved oxygen, inorganic phosphorus, nitrogen and others. The same module can be used to study heat fluxes.

The concentration of the substance is calculated at each point of a rectangular grid covering the domain under study. Information about the transport, i.e., currents and water depths at each grid point, is provided by the hydrodynamic module. Other data required include substance concentrations and discharge qualities at outfalls, together with concentrations at boundaries.

The results of a two-dimensional model of unsteady horizontal flow are used for solving the transport-diffusion equation for dissolved or suspended substances, in two dimensions. This is in reality a mass-conservation equation. Discharge quantities and compound concentrations at source and sink points are included, together with the decay rates.

$$\frac{\partial}{\partial t}(C h)+\frac{\partial}{\partial x}(C u h)+\frac{\partial}{\partial y}(C v h)=\frac{\partial}{\partial x}\left(h D_x \frac{\partial C}{\partial x}\right)+\frac{\partial}{\partial y}\left(h D_y \frac{\partial C}{\partial y}\right)-F h C+S \qquad (3.7)$$

where: **C** - compound concentration (arbitrary units); **u,v** - horizontal velocity components in the **x,y** directions (m/s); **h**-water depth (m); ρ-fluid density (kg/m^3); $\mathbf{D_x}$,$\mathbf{D_y}$-dispersion coefficient (m^2/s); **F** - linear decay coefficient (1/s); $S = Q_s\,(C_s\text{-}C)$; Q_s - source/sink discharge per unit horizontal area (m^3/s/m^2).

The system can be solved by the transport-diffusion equation using a two-dimensional form of the finite difference scheme working on a rectangular grid. The QUICKEST scheme is a third-order accurate scheme for the unsteady advection-diffusion equation; it uses a quadratic interpolation for convective kinematics with estimated streaming terms. It has several advantages over other schemes, especially because it avoids the 'wiggle' instability problem associated with central differencing of the advection terms. Also it reduces the numerical damping which is characteristic for the first-order upwind methods.

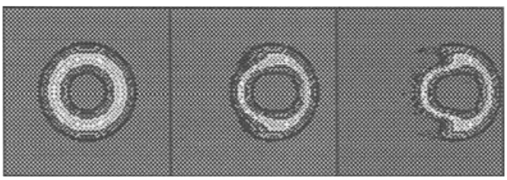

Figure 8. Transport-diffusion example.

3.2.2.2. Non-Cohesive Sediment Transport

The sediment transport module of two dimensional unsteady flow models assesses the rates of bed level changes. The sediments are assumed to be non-cohesive, i.e., sand, but may vary in grain size through the modelled area. For a given topography, water depths and flow field, the sediment transport module calculates the sediment transport capacity at each grid point. Erosion and deposition rates in the modelled area are thereby estimated. The data concerning flow field and water levels are provided by the hydrodynamic module.

The sediment transport capacity is calculated as a function of the water depth, flow and sediment conditions. The rates of bed level changes, δz/δt, are described by the equation of continuity:

$$\frac{\partial z}{\partial t}+\frac{1}{1-n}\frac{\partial q_x}{\partial x}+\frac{1}{1-n}\frac{\partial q_y}{\partial y}=0 \tag{3.8}$$

where, $\boldsymbol{x,y,t}$ - independent variables, $\boldsymbol{q_x,q_y}$ - sediment transport components in **x**- and **y**-directions, $\boldsymbol{z}$ - bed level, $\boldsymbol{n}$ - porosity of the bed.

The total sediment load is split into the bed load and the suspended load. The bed load represents the movement of particles in contact with the bed by rolling, sliding and saltation, while the suspended load represents the movement of particles with the flow. The settling tendency of the particles is continuously compensated by the diffusive action of the turbulent flow field.

3.2.2.3. Mud Transport

Mud transport describes the erosion, transport and deposition of silt, mud and clay particles under action of the flow field. For a correct description of erosion, the consolidation of sediment layers in the bed can be calculated. Mud mainly consists of cohesive sediments, which coagulate into 'flocs' in suspension and form a 'cohesive mass' on the bed. A deterministic, physically-based description of the behaviour of cohesive sediments has not yet been developed. Consequently, the mathematical descriptions of erosion and deposition are essentially empirical, although they are based on physical principles and the models are heavily dependent on field data. Extensive data over the entire area to be modelled are required with respect to bed materials, settling velocities, in situ flocculation characteristics, vertical velocity and suspended sediment concentration profiles, compaction of layers under the bed surface, effect of wave action, and critical shear stresses for deposition and erosion. The dynamic variation of water depth and flow velocities must also be known, along with the boundary values of suspended sediment concentrations.

4. Economy and Convenience of Applied Mathematical Models

When evaluating different modelling methods, a potential user always tries to find out which one consumes the least computer time. This criterion, however, should be considered within a broader expenditure context. The ratio of labour costs to computer expenses is not always simple to express. Computer time becomes an important factor at least in the following situations:

- dam break problems
- 2D problems, flood plain problems in urban areas
- a complicated network of sewers and water supply systems

In conclusion, the following guiding principles may be given to those who would like to work out the economic criteria for understanding the mathematical model application:

- use the method of characteristics for laboratory problems or for checking the accuracy of other methods,
- use an explicit scheme when developing a new programme which will be applied to a given, well defined problem requiring a small step, or when a new program is needed quickly, or when the program is to be run only a few times
- do not use an explicit method for sewer systems and for a channelled reach of the river in a urban area,
- use an implicit method for flood propagation problems, but make sure that it allows for the existence of discontinuity and that the tool works for all regimes of flow (supercritical and subcritical, open channel and pressurised flow),
- when looking for a tool, take as a criterion the applicability of individual tools to cover the whole range of problems,
- do not accept any package without knowing exactly what is in it, otherwise how can it be evaluated?

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OPERATION OF SEWER SYSTEMS

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1. Introduction

Urban sewerage and drainage systems are important infrastructures which require large financial investments. To protect these investments and ensure delivery of sewerage services, such systems have to be properly operated and maintained, with adequate financing and administrative control. Thus, the operation of sewer systems can be defined as a set of procedures designed to provide a service by collecting, transporting, and managing the quality of, urban wastewater in a cost-effective, environmentally responsible and sustainable way. More specifically, the process of operation comprises all activities conducted after placing the sewer system into operation - including the specification of system performance parameters, day-to-day operation rules, inspection procedures, short-term and long-term maintenance, financial aspects, and administrative control. Further explanations of these issues follow.

The system performance parameters need to be defined with respect to the conveyance flow rates/capacities, acceptable frequency/duration of service interruptions or damages, and the level of treatment/dilution provided. These service parameters generally evolve in time, usually as a result of increasing regulatory demands on control of stormwater and combined sewer overflow pollution. The older systems were typically designed for single objective controls (flow conveyance), but in recent years, the demands on them were increased to reflect much broader ecosystem health objectives. Consequently, the original sewer systems, which may have been fully functional for the old objectives, have to be upgraded.

Day-to-day operation rules describe the procedures followed to deliver the prescribed service. The complexity of such procedures varies with the sophistication of the operated system; for example, in gravity storm sewers there are hardly any needs for operational intervention under normal operating conditions. The other extreme is represented by a combined sewer system subject to real time control, with sophisticated sensor networks (rainfall, sewer flow and quality, receiving water conditions, including water quality), a data gathering module, a central control module and the

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implementation of operation strategies involving variable flow control at many points in the system. Most sewer systems fall somewhere between these two extremes, with some requirements on operation of pumps and gates, to control flows and their quality.

Many failures of sewer elements are not sudden incidents, but rather represent a qualitative change preceded by a gradual build-up of conditions leading to the problem. For example, the clogging of sewers generally results from a gradual build-up of sediment, or from structural failures that are usually preceded by the appearance of sewer cracks or partial collapse. Indications of the deteriorating condition of the system can be detected through inspection, which benefits from modern technology including in-sewer inspection by video cameras, or ultrasonic techniques.

Sewer system maintenance is usually classified into two categories, corrective or short-term maintenance dealing with remediation of the immediately pressing problems (i.e., trouble shooting, cleaning out a blocked sewer or an ice-blocked sewer inlet), and preventive, long-term maintenance aimed at avoiding anticipated problems. The latter category includes sewer system rehabilitation and renovations, which are fully discussed in the next paper in this Chapter.

Sewer system operation requires financial resources, and these have to secured through allocated budgets. Normal procedures of financial operation and control are applied, including the collection of funds and proper accounting. The sewer systems represent very large capital investments and as discussed later, operation of such systems appears to be generally underfunded (Lehmann 1994).

Finally, sewer system operation has to be organised administratively, usually by concentrating operation activities in a single department, which has to be properly set up, funded and empowered to deliver the expected services.

The issue of funding of sewer system operation is gaining in importance and has received much attention in recent years, particularly in connection with the privatisation of water services. Typically, public water infrastructures represent very large capital investments, as documented by two examples from Switzerland and Norway. In Switzerland, the replacement value of the public infrastructure was estimated at 250 billion SF (approximately 200 billion U.S.$); wastewater systems represent approximately 30% of this amount (60 billion U.S.$), and water supply systems represent 25% (50 billion U.S.$). However, municipalities typically budget only 3% for wastewater and the same amount (3%) for water supply systems in their operation budgets. In Norway, the replacement value of the wastewater systems is 70 billion NOK (approximately 11 billion U.S.$), but annual expenditures are just 1.5 billion NOK (240 million U.S.$). Similar examples could be found in other countries as well and indicate under-funding of sewer system operations.

Thus, the sewer systems are capital intensive systems, which are still operated in many jurisdictions under the myth of "maintenance-free" policies. Long service lives of some structural components of these systems (particularly concrete pipes) disguise the overall system ageing and deterioration. In fact, it would be hard to find another service industry that would operate such expensive systems while spending so little on their operation and maintenance. Catastrophic failures are generally handled by special financing or fund allocations from general budgets. Some changes in this attitude should be noted, for example, some municipalities in Germany are collecting annually 2% of the replacement sewer value in a special fund that can be used only for rehabilitation activities.

The discussion presented in this chapter is restricted to sewer systems only, recognising that the operation of wastewater treatment plants is rather complex and the proper treatment of that subject is beyond the scope of these proceedings. Also it should be recognised that sewer system operation is discussed here only in connection with applications of hydroinformatics to sewer systems, and no attempts are made to provide a detailed treatment of operational procedures.

2. Tasks and Challenges of Urban Drainage Operation

The main challenges in providing urban drainage by means of combined and storm sewers can be summarised in the following five points: (a) increased regulatory demands on stormwater and CSO control, (b) reduced capacities of receiving waters to accept wastewater effluents, (c) poor utilisation of existing system capacities, (d) ageing/ deterioration of sewer systems requiring rehabilitation/replacement, and (e) the rate of sewer replacement outpaced by the rate of sewer deterioration. Further discussion of these points follows.

2.1. INCREASING REGULATORY DEMANDS ON STORMWATER AND CSO CONTROL

When the first combined sewers were built (100 years ago), wet-weather flows were considered unpolluted and the small fraction of sanitary sewage escaping from sewers through CSOs was considered sufficiently diluted. However, increasing demands on sewer systems have evolved from the original single objective (prevention of flooding or water ponding in the case of storm sewers, and the protection of public health in the case of combined sewers) to much broader objectives, namely ecosystem health protection. Consequently, the older systems, designed for simple objectives, have become obsolete and require both functional and structural renovation. In fact, it is prudent to combine any rehabilitation projects addressing structural rehabilitation/renovation with a critical analysis of functional renovation and harmonise both requirements.

The objective of ecosystem health protection is not well defined, but quite often it may be replaced by effective mitigation of wet-weather discharge impacts, which depend on both the characteristics of the catchment producing such effluents and the characteristics of the receiving waters (Ellis and Marsalek 1996). The actual impacts need to be evaluated for individual sites and the issues of interest include habitat changes (particularly changes in morphology), water quality changes (dissolved oxygen depletion, eutrophication), sediment and toxic impacts, impairment of public health (by discharge of fecal bacteria), impacts on biological communities, and groundwater impacts (Ellis and Hvitved-Jacobsen 1996). In general, the complexity of processes in receiving waters and the time-varying nature of these impacts necessitates the assessment of receiving water conditions by computer modelling. In current modelling practice, there is a major knowledge gap with reference to ecotoxicological modelling, particularly when reconciling laboratory and in-situ field dose-response relationships as well as in relating biotic uptake rates to target pollutant concentrations and exposure durations and frequencies (Ellis and Hvitved-Jacobsen 1996).

2.2. REDUCED CAPACITIES OF RECEIVING WATERS TO ACCEPT WASTEWATER EFFLUENTS

Progressing urbanisation increases the total loads of pollutants on receiving waters and leads to the exceedance of self-purification capacities of such water bodies. This is further exacerbated by the cumulative nature of some runoff impacts (e.g., gradual morphological changes resulting in the loss of habitat), or synergistic action of contaminants released from various sources. Consequently, the capacities of streams and reservoirs to accept new wet-weather discharges have greatly diminished over the years, and much more rigorous regulations have to be imposed on more recent developments. In this connection, it should be realised that the urban development of even as little as 10% of the total catchment creates detectable impacts on the receiving waters (Horner *et al.* 1994) and, consequently, there are relatively few ecosystems in the world that are not impacted by civilisation. Thus, in environmental protection or remediation, the goal should be to strive for attaining a desirable quality of the ecosystem, which will, however, fall short of the pristine "natural" state.

2.3. POOR USE OF THE EXISTING SEWER SYSTEM CAPACITIES

It has been demonstrated in the literature (Schilling *et al.* 1989; Schilling 1995, 1996) that gravity controlled and operated sewer systems are generally inefficient in utilising available sewer capacities and their collection efficiency falls short of the potential of the same system equipped for and subject to real-time control operation. Low collection efficiency translates into sewage overflows and leads to the pollution of receiving waters. A gravity only (static) control cannot account for non-uniform distribution of flows and storage in the system, or ensure that system capacities (particularly storage capacity) can be utilised in response to a given storm and system loading pattern. Such benefits can be realised only by implementation of real-time control (RTC), which is particularly beneficial in systems with large static storage, uneven distribution of storage volume, and long times of travel in sewers (Jorgensen *et al.* 1995). Under such circumstances, the volume of overflows can be reduced by as much as 25%, without increasing the system's physical capacity.

2.4. REHABILITATION/REPLACEMENT OF AGEING SEWER SYSTEMS

Sewerage rehabilitation deals with three types of sewer system failures - structural, hydraulic and environmental. To manage this task effectively, Grigg (1988) recommended the use of a computerised Maintenance Management System (MMS), which should include programs for inventory control, condition assessment, rehabilitation and replacement, preventive maintenance, budgeting and decision support. The inventory can be stored in a GIS system that should include the condition assessment data consisting of such categories as capacity, safety, structural integrity, quality of service, role and age. The needs assessment should include an analysis of projected needs, alternatives, rates of deterioration and obsolescence, cost-benefits and impact studies as well as sensitivity analysis. The use of MMS must be accompanied by a long-term commitment of personnel and budget. Descriptions of failures, methods of diagnosis and rehabilitation are given in the next paper in this Chapter.

2.5. THE RATE OF REPLACEMENT OF AGEING SEWERS IS OUTPACED BY THE RATE OF DETERIORATION OF SEWERS

Opinions on sewer design life differ widely and reflect the variation observed in actual sewer systems. While some older sewers can serve, in optimal conditions, as long as 100 years, some younger sewers built just 30-40 years ago, but exposed to adverse conditions (e.g., in landfills, or areas with corrosive ground water) already require rehabilitation. For simplicity, a fixed design life is considered, usually 50 years. On the average, this design life would require annual renewal of about 2% of all sewers, but the actual rates of rehabilitation typically range between 0.25 and 1%. Thus, the gap between the rehabilitation needs and the actual rehabilitation is increasing and contributes to growing liabilities.

The above discussion points to two obvious solutions – (a) increased expenditure on sewer rehabilitation and renewal, and/or (b) optimisation of operation of existing systems for better performance and longer service life. Recognising the ever-increasing competition for dwindling public funds and the overall aims of this ASI, only the second solution is discussed in this Chapter.

3. Potential for Improving Sewer System Operation

The existing sewer systems in Europe and North America suffer from a number of operational problems, including poor sewage collection efficiency, identified by dry weather bypassing, CSOs or sewage back up into basements, and general problems of underutilisation of the sewer system capacity.

Sewage overflows or bypasses in dry weather, when the system operates below its design capacity, have been reported in many jurisdictions. Generally, they are caused by flow obstructions in sewer systems or by malfunctioning dynamic overflow regulators. For example, it was estimated that in Trondheim (Norway) about 20% of the sewage volume bypasses the sewage treatment plant via CSOs, and one half of this volume occurs in dry weather.

Sewer blockage may cause pressurisation of the sewer system and sewage may enter basements through basement drains, with concomitant large damages and public health risks. These problems are common in older combined sewer systems and are reported in many countries. Data from Trondheim indicate that in 1980, a total of 506 sewer blockages resulted in damage of private property due to backing up of sewage.

After implementing improved maintenance and regular sewer flushing, this number was reduced to 66 cases in 1993.

Some components of sewer systems are clearly under-utilised. Primary examples of such structures are CSO storage tanks, which are used widely in Canada, Denmark, Germany, Switzerland, UK and USA. In typical conditions, these tanks are designed to reduce the frequency of overflows from 50-100 to about 10 per year. A simple calculation can be used to demonstrate the poor cost effectiveness of these tanks when not equipped and operated to provide system-wide benefits. For an assumed service life of 50 years, a typical tank will overflow 500 times during this period. Since one cubic metre of tank storage costs about \$1,500 U.S., the capture of the last cubic metre of sewage costs \$1500/500 = \$3, which is an enormous cost to prevent diluted sewage from reaching the receiving waters. Obviously, these costs could be significantly reduced if the tank would be used more frequently, especially when some tanks are under-utilised while overflows occur at other sites.

Thus, there is ample evidence that most of the existing sewer systems are not operated at their optimum, and that significant pollution control benefits can be gained just by improving system operation, without large investments in the sewer infrastructure.

4. Operation - Definition Revisited

Efficient operation of any system requires the fulfillment of two conditions, which can be illustrated by an analogy with driving a car. Firstly, the car has to be in such a technical state that all vital components function properly (e.g., engine, transmission, steering, brakes, fuel supply, and ignition). This car can run, but cannot be driven until the second condition has been met. The driver must know how to drive and where to go. In other words, he has to have a driver's license, driving experience, and knowledge where to go and how to get there. Only then the car can reach its destination.

It is obvious that fulfilling the first condition but not the second results in a poor investment strategy. With reference to sewer operation, it is necessary to fulfill both conditions and define operation as a two stage process. Stage 1 includes all the measures that bring the sewer system to, or keep it in, a proper state as originally designed. These measures include regular inspections, flushing, catch basin cleansing, grease removal, root cutting, infiltration control, rat control, etc. In a conventional approach, these measures are summarily referred to as "operation and maintenance".

However, even a sewer system which is well maintained does not necessarily perform well. This is caused by temporal and spatial variations in system's loading with sewage, stormwater and pollutants. Performance shortcomings can be observed in the form of local water/sewage ponding/flooding, CSOs, or shock loading of treatment plants, occurring while there is unused storage or flow capacity in other parts of the system. All these phenomena indicate poor performance of an otherwise perfectly maintained sewer system. Hence, it is necessary to implement the second stage, by manipulating the system continuously in such a way that its capacity is optimally used in all operational situations and no damage occurs unless the system capacity is completely exhausted. This type of operation is usually referred to as "real-time control" (RTC), because the system is manipulated during the ongoing flow process. Thus, RTC increases the collection efficiency and pollution control effectiveness of the sewer system.

5. Operation and Maintenance

Public demands to improve the performance of existing sewer systems, coinciding with a general restraint on public spending, force municipal managers and politicians to look for innovative non-capital intensive solutions. Such solutions generally involve the development of an improved operation plan. In this process, the existing sewer system and treatment facilities are analysed to determine the changes in system operation which would improve system performance, as described by the collection efficiency and reduction in total pollution loads discharged to receiving waters. This approach differs from facilities planning, which simply addresses the planning of the facilities needed to meet a performance standard.

Such an operational plan determines the capabilities of existing facilities to maximise the reduction of all municipal effluent discharges into the receiving waters. To achieve this goal, some limited new facilities may be needed, but in principle, environmental impacts can be reduced in this approach without embarking on a major capital improvements program. The operational plan comprises three major elements - examination of major system components of the collection and treatment system, the administration of the system, and system maintenance. In each element, opportunities for reducing inflow, temporal storage of wet-weather flows, or increasing treatment capabilities are explored.

5.1. INVENTORIES AND CONDITION ASSESSMENT

The foundation for rational operation, maintenance and needs assessment of a sewer system and treatment facilities is a good documentation of the entire system. Where such documentation is missing or is outdated, a new inventory needs to be developed. The types of information needed include the physical features of the collection and treatment system, its administration and the maintenance program. Inventories of physical facilities are done through a process of measuring the physical condition of system components using objective criteria. The parameters include safety, structural integrity, capacity, quality of service, role and age (O'Day and Neumann 1984). This information is entered into and stored in computer databases with a geographic base file holding the system geometry as well as all other operational aspects. The inventorised physical system is then analysed, generally by computer modelling, to determine the system's strengths and weaknesses. Positive findings include spare flow conveyance, storage and treatment capacities, and documentation of good structural integrity of the system. Weak points include overloading of system components, lack of storage in critical components, excessive infiltration and inflow, and poor structural conditions (Grigg 1988).

5.2. ADMINISTRATIVE CONTROLS

An operation plan must include administrative controls specifying basic rules for system operation. Such controls start with municipal sewer ordinances, including procedures for making connections to the system, construction requirements, limits on materials discharged to the system, pretreatment, monitoring structures/programs, and rate schedules. Additional controls are introduced at higher jurisdictional levels, e.g., at the territorial or national government levels. Such controls may prescribe the quality of system effluents. For example, the Canada Fisheries Act specifies that no matter deleterious to fish may be discharged into open waters frequented by fish (Chambers *et al.* 1997). Administrative controls may be changed to improve system performance, e.g., reducing inflows to the system by forcing disconnection of roof runoff and other sources, or by enhancing pretreatment specifications and establishing administrative bodies for improved control over the system.

5.3. SEWER SYSTEM MAINTENANCE

Operational management consists of operations and maintenance. Thus, sewer system maintenance is part of the overall operational plan, and both operation and maintenance should be under a common management. The main goal of maintenance is to keep equipment and systems ready to go at any time. A maintenance program comprises three elements: maintenance tasks, schedule, and manpower needs (Grigg 1988).

Maintenance tasks can be classified into four activities: condition assessment, inventory, preventive maintenance, and corrective maintenance. Inventory of physical facilities and system functions is a continuous determination of whether the system is working properly. If it is, then operations and surveillance continue. If not, corrective or major maintenance activities must be initiated. Corrective maintenance requires a decision: is the deficiency serious enough to warrant entering the planning, programming and budgeting activity (i.e., capital requests), or is it minor enough to go ahead and fix it or even postpone the corrective action (Grigg 1988)?

Sewer system maintenance tasks are rather varied, depending on the sewer system and its components under consideration. Typical tasks deal with street sweeping, catch basin cleaning, sewer cleansing, sewer repairs and emergency response, upkeep of stormwater management systems, inspection/cleaning of siphon and overflow regulators, and maintenance of pumping stations. Further details of such tasks follow.

Street sweeping is used not only for aesthetic reasons, but also for reducing pollutant (particularly solids) entry into sewers. The frequency of sweeping varies depending on land use (typically from daily to monthly). Catch basins are designed to control the entry of coarse street sediment into sewers and thereby to reduce sedimentation in sewers. Catch basins are cleaned regularly in most jurisdictions; inspection is used to record structure conditions and schedule any repairs required.

Sewer cleansing is particularly important in those system sections that are susceptible to sedimentation. Such areas are regularly inspected, their condition recorded, and records entered into a data base (GIS based). The areas subject to sedimentation are usually further characterised with reference to the frequency of cleansing required, varying from once a week to once a month (or even longer intervals), but most sewers are cleansed at least once every five years. The most common and cheapest method of sewer cleansing is flushing; other methods used include mechanical cleansing (dragging) or use of high-pressure nozzles.

Among the methods for repairs of sewers (cracks and joints) and manholes, injections of synthetic concrete mortar and relining with plastic liners are most common. The latter option is more costly, but may be necessary in deteriorated systems.

Maintenance of storm drainage systems somewhat differs from that of the conventional sewer systems. In particular, the stormwater systems consist of concrete, earth, grass and miscellaneous structures, without much machinery or exposure to biochemical processes. Relatively few checks are performed in such systems, generally in response to public complaints usually dealing with short-duration flooding. Consequently, the general attitude toward inventory, maintenance, and upkeep of storm drainage systems is different from that for other municipal utilities (Grigg 1988).

Storm drainage maintenance is sometimes done by contracts involving three types of tasks: routine, restoration and rehabilitative. Routine tasks include vegetation mowing, trash and debris cleanup, weed control, and revegetation. Restoration tasks are more demanding and include stormwater pond mucking, trash rack cleaning, rebuilding steep rundowns, tree thinning and clearing, extending trickle channels, repairing local erosion problems, and local grading and shaping. Finally, the most demanding are rehabilitative tasks, including reconstruction of drop structures, trickle channels, reshaping channels, installing riprap and maintenance access structures, and protecting existing drainage features.

Some experience with maintaining stormwater management systems was reported by the Denver Urban Drainage and Flood Control District: (a) there was a gap between what was designed and what was actually built; (b) hardly any communities within the District had systematic maintenance programs; and, (c) stormwater management facilities degrade rapidly without proper maintenance (Hunter and Tucker 1982).

Special structures in combined systems, such as overflows and siphons, require frequent inspections, because they are critical for satisfactory operation of sewer systems. In the case of overflows, it is customary to inspect each structure regularly (the frequency varies from daily to several times a year; on average, weekly) and during rain events, when inflow to the treatment plant exceeds some threshold. Maintenance crews are typically on call to make such inspections. In general, inspections have two goals: to correct operational failures, and prevent or reduce their recurrence. Common causes of failures include clogging, silting, sticking of movable parts, parts failure, corrosion, power failure, and hydraulic pressure failure, in the case of hydraulically operated structures. The most common problem, clogging, cannot be eliminated completely. Experience indicates which structures require more attention (U.S. EPA 1990).

Recommended maintenance schedules include cleaning out regulator chambers after every storm, and during each visit, the crew should visually inspect the regulator, remove debris, operate gates to prevent seizing, lubricate and clean chains and gears, examine for corrosion and wear, check bearings and frozen parts, and verify operation of water level sensors.

Many sewer systems employ pumping stations for drainage of low-lying areas. These stations require special attention, because of the nature of mechanical and electrical equipment used at these installations. Major factors in deterioration include wear on moving parts, mechanical damage, corrosion of metal components, and physical deterioration by ageing. The functional deterioration of pumping stations usually manifests itself through reduced pumping capacities (fouling or wear of pumps) or decreased pump efficiency. The latter may lead to overloading the pump drive. The average lifetime of mechanical installations of pumping stations can be estimated from 25 to 30 years for sewage stations, and 40 to 50 years for stormwater drainage stations. The average life of the electrical installations is just 15 to 20 years, but the average lifetime of the station structural elements is 100 years or more. In general, the need for station alterations may reduce this life (Hissink 1995).

Many problems with sewer systems are identified during maintenance activities or through citizen complaints. This fragmented information about the state of the sewer system is systematically complemented by sewer inspections, which are conducted at least once a year, preferably at the end of the wet season, using remotely controlled TV cameras. Results of inspections are filed, and three types of files recognised; the structure files reflecting the "as built" conditions, inspection files reflecting the actual state, and the failure files. By comparing the structure "as built' and actual files, deviations can be identified and the need for action assessed. The system or structure failures are recorded separately in failure files. If failures of a structure are too frequent, the reliability of that structure is endangered.

The observed defects must exceed certain threshold criteria to warrant immediate action; minor defects are rejected for immediate attention. In these decisions, the input of field crews is invaluable. Typical examples of conditions requiring immediate attention include (Hissink 1995):

- severely cracked and leaking concrete sewers (as determined by TV inspection)
- internally corroded sewers, with gravel or reinforcement steel showing
- permanent loss of storage capacity, due to stagnant water, exceeding 80%
- bank revetments of drainage channels are so damaged that the embankments sag out into the drain, blocking its hydraulic profile
- drainage channels are silted up to 0.4 m or less below the mean water level
- pumping stations are damaged or the manufacturers criteria are exceeded
- pump discharge capacity or efficiency was lowered by 10%.

Good record keeping is essential for development of effective sewer maintenance programs. This is done in computerised databases (GIS based) which are used for data storage and problem documentation. Typically, the following repair needs or activities are recorded: (a) manhole repairs (major repairs, cleaning, height adjustments, minor repairs), (b) sewer lines to be televised, (c) sewer repairs (urgent, major, minor), (d) problems of intruding roots, and (e) sewer complaints. Using this information, monthly summaries are prepared and used to issue work orders. Sewer related complaints, such as sewage back-up, point to problems of sewer blockage and the need for cleaning.

The maintenance program requires sufficient personnel, which is typically organised into a number of crews that perform routine and special tasks. Crews consist of 3-5 men, one of whom always stays on the street surface. Safety concerns are of paramount importance and dictate the needs for special training, proper equipment, and development of safety manuals with descriptions of specific safety measures. The municipal area is divided into regular maintenance sub-areas, which are regularly inspected and maintained. For all these activities, the associated costs have to be established and complete records kept of all inspection and maintenance work.

It is obvious from the preceding discussion, that in large urban areas, sewerage maintenance represents a large business activity that should be rationalised by introducing a maintenance management system (MMS). MMS brings together disparate maintenance activities through a holistic approach of caring for the system, and ensures that the overall maintenance is done properly. It involves all essential management tasks, including planning, organising, and controlling, and requires an effective decision support system. With respect to maintenance functions, MMS deals with the condition assessment, corrective maintenance and preventive maintenance, and the decision support system will provide the information and data needed for these activities (Grigg 1988).

The MMS support system is used to develop and formulate effective maintenance strategies, including the following approaches or their mixtures (Grigg 1988):

- crisis maintenance only
- maintaining the most deteriorated facilities first
- performing opportunistic maintenance, when related work is scheduled
- using pre-specified maintenance cycles
- repairing the components with the highest risk of failure
- using preventive maintenance
- reducing the causes of wear and tear on the facility
- comparing the economic advantages of maintenance strategies.

The MMS should be an integral part of the overall organisation management control system. As a minimum, it should have an inventory control component, and a record system for maintenance work scheduling and completion. The records kept should include equipment data, the preventive maintenance record, the repair record and spare parts stock cards. The use of MMS is particularly advantageous when dealing with needs assessment, which links maintenance management and capital improvement processes. Maintenance decisions and strategies require proper planning. The needs assessment process involves an inventory, a condition assessment, and identification of the desired level of improvement and maintenance; all viewed in the light of present and future conditions (Grigg 1988).

Sewer maintenance helps implement quality control in sewer operations. In this respect, quality control provides assurance that the quality of a product (i.e., delivery of sewerage services) is within the acceptable limits of quality, as defined for that product. Basic elements of the quality control process, such as inspection, administration and record keeping, quality control engineering, and performance gauging, can all be well related to the earlier discussed tasks of sewer operation and maintenance (O'Day and Neumann 1984). The performance could be assessed by the absence of complaints, or the physical and functional state of the sewer system.

In a proactive approach, sewer maintenance and rehabilitation can be improved through early diagnosis and treatment of structural and geotechnical anomalies. Methods and computer tools have been developed to carry out the tasks involved in sewer rehabilitation. These include an expert system for the structural and geotechnical diagnosis of the sewer network as well as a decision aid tool for choosing the most appropriate rehabilitation technique. The geotechnical conditions addressed include steep slopes (>5%) and hydrological conditions causing earth slides. Some of these systems have been validated in actual sewer systems and have been found useful in improving the system analysed and identifying the shortcomings of the current state of knowledge and the type of information collected (MacGilchrist 1989).

5.4. SEWER SYSTEM CONTROL STRATEGY

Even without sophisticated real-time control, it is advantageous to look for improvements in the operational strategy that would reduce discharge of pollution from the sewer system, and particularly from combined sewers. Two sources of improvement are obvious: reducing incoming flows, e.g., by disconnecting roof leaders, or enhancing infiltration in the catchment; and, improving utilisation of storage in the system, e.g., by installing flow throttles in critical sections of the system. The use of such controls has to be thoroughly tested by computer simulations. The implementation of this strategy should improve the overall system efficiency, in terms of both collection efficiency and pollution loads discharge into the environment (U.S. EPA 1990).

The connection of operations and maintenance can be illustrated by a general operation model, involving data collection, operator decision, and control command sequence. Operations connect to maintenance through the condition assessment activity. In modern approaches, an expert system and computer control are incorporated into this process, as discussed in next section.

6. Real Time Control (RTC)

6.1. WHAT IS RTC ?

An urban drainage system (UDS) is controlled in **real time** if process data that are concurrently monitored in the system are used to operate flow regulators during the actual flow process. Typically, this task involves activating a number of pumps, sluice gates, weirs, etc. to allow the occurrence of adverse effects (e.g., flooding, combined sewer overflow CSO) only if the system is at capacity and only at those locations where the least damage is caused. In **static** systems this can only be achieved in the rare case when the UDS is receiving its design load. If, for example, the outflow of a detention pond is controlled by an orifice, the optimal outflow rate is reached when the pond is full. During other periods the outflow rate is smaller than the optimum and, consequently, the emptying time is longer. To activate excess storage in a large sewer, a (static) high-side weir overflow regulator can be used. The overflow opening has to be large enough to allow passage of the design overflow rate. Thus, much of the available storage cannot be used in most situations.

Operational concepts of real time control systems (RTCS) are concerned with logical ways of using process information. Since the deficiencies of static systems are well known, moveable (self-operating) regulators have been introduced to maintain a pre-set flow or water level. Many of these regulators use process measurements taken directly at the regulator site (e.g., by a float, counterweight, etc.). Therefore, such a system is termed a **local control** system. Under local control, regulators are not remotely manipulated from a control centre, even if operational data are centrally acquired. This type of setting makes the system supervision easier.

If a RTCS is more complex or if all regulators need to be operated in a coordinated manner, **global** or **systems control** is applied. Here, all regulators are operated with respect to process measurements throughout the entire system. Global control in drainage systems is required under the following conditions:

- many regulators exist that affect each other, or
- the actual loading differs substantially from the design loading (e.g., as a result of temporally and spatially varying rainfall).

6.2. EQUIPMENT AND HARDWARE REQUIREMENTS

In an RTCS there are **control loops** consisting of:

- **sensors** (e.g., water level gauges) that monitor the ongoing process,
- **regulators** (e.g., pumps or gates) that manipulate the process,
- **controllers** that activate the regulator to bring the process to its desired value (**set point**), and
- **data transmission systems** that carry the measured data from the sensor to the controller and the signals from the controller back to the regulator.

From the large variety of available **sensors** only very few fulfill the requirements for RTC in UDS. Such requirements include, among many others, the sensor suitability for continuous recording and remote data transmission (**monitoring**). The following sensors are widely used:

- **rain** gauges (weighing gauges, tipping bucket, drop counters, and radar devices),
- **water level** gauges (bubbler, air pressure, water pressure, and sonic principles),
- **flow** gauges (level-to-flow conversion, ultrasound velocity measurement, electromagnetic induction meters), and
- **limit switches** (e.g., mercury float, diaphragm).
- Rainfall intensity data can be used to provide short term **runoff forecasts**. The forecasting horizon can be extended if rainfall forecasts are included (particularly using the radar technology).

Level measurements are the backbone of every sewer monitoring system. They are indispensable for determining the state of storage devices or converting levels to flow rates (large sewers, overflow weirs, flumes, gates). Water **quality** sensors still play a very minor role in RTC of UDS because of their deficiencies.

Regulators for sewer flows include axial **pumps** (constant or variable speed) or screw pumps. **Weirs** (perpendicular, side-spill, or leaping) are used to create storage in ponds or sewers. Self-operating weirs use a counter-weight or buoyancy of a float to adjust the height of the crest. A special design is an air regulated **siphon** which functions as a weir or a siphon depending on the air supply in its crest. **Inflatable dams** are broad-crested weirs used to activate storage in large trunk sewers. **Gates** (e.g., sluice, radial, sliding) are movable plates that constrict the flow in a sewer or in the outlet structure of a tank. **Valves** are devices used to throttle pipe flows (i.e., plug, knife, butterfly). In a **vortex valve,** the fluid rotation increases head losses with increasing flow rates. It operates without external power supply or any moving parts. Other regulators used in RTCS include air regulated **inverted siphons**, movable **tidal (backwater) gates**, and **flow splitters** which separate incoming flow into two outgoing paths.

Flow or water level regulators in urban drainage systems are often very large, custom-designed devices. However, some basic design principles are common to all successful designs.

1. Regulators should be **fail-safe** designed, so that malfunction of vital parts results in an acceptable functional decline of the system. For example, sluice gates should have bypasses, and dynamic weirs should move into a safe position in case of a power failure, etc.
2. All parts exposed to sewage and the corrosive sewer atmosphere should be drastically simplified and made corrosion resistant. The material of choice for their manufacturing is stainless steel.
3. Sensitive parts should be located in an appropriate environment, i.e., a dehumidified vault for hydraulic and electric machinery, dehumidified and heated vaults for programmable logic controllers, telemetry equipment, etc.
4. All components of a regulator station (including gates, sensors, motors, etc.) should be accessible, maintainable, and exchangeable.

5. It should be possible to supervise the vital regulator functions from the control centre.

In a centralised RTCS, a data transmission system is required; both transmission by wire (e.g., public telephone) or wireless transmission are used. **Digital** data transmission is becoming more and more common. Compared to analog transmission, its main advantages include high suitability of digital data to be fed directly into digital computers, improved transmission reliability (eliminates noise), and high information transmission rates (measured in bits per second, bps).

With the development of digital computers, many analog controllers could be replaced by a single central computer. This allows more flexibility in controller calibration, interconnection of control loops, and adjustment of set points. With the advent of inexpensive microprocessors, the vulnerability of such a system could be overcome by implementing a central minicomputer and several local **programmable logic controllers (PLC)**. Typically, the PLCs control and coordinate all functions of outstations, including the acquisition of measured data; data pre-processing (smoothing, filtering, etc.); checking the status, function, and limits; temporary data storage; local controls; and, receiving data from, and reporting to, the central station. As computers become more powerful, fewer tasks remain to be done by the central **process computer**, i.e., system-wide data acquisition, long-term storage and data management, operator interfacing, interactive simulation / optimisation (decision support software), and automatic execution of control strategies.

6.3. PLANNING, DESIGN AND IMPLEMENTATION OF RTC SYSTEMS

The planning, design and implementation of RTC systems typically include the following stages:

1. Define the operational **objectives** of UDS (e.g., minimise flooding, minimise CSOs, etc.)
2. **Simplify** the UDS, but simultaneously highlight its "hot spots" (e.g., hydraulic bottlenecks, CSO sites, regulator stations, etc.)
3. Evaluate the **potential** for RTC through simulation or monitoring (i.e., do idle capacities exist during the periods when environmental damage occurs?)
4. Determine the **current** performance of the UDS (i.e., analyse historic events during which damage was observed and/or determine the statistics of damages through long-term simulation)
5. Select **locally** controlled regulators and determine the performance of locally controlled UDS (e.g., is the capacity better utilised in comparison to current operation ?)
6. Optimise control strategy and determine the maximum performance of UDS under **global** control (i.e., how large is the improvement in reaching the operational objectives as compared to local control ?)
7. Compare RTC operation with **conventional** (i.e., static) remedial measures (e.g., larger sewers and tanks)
8. Evaluate **cost/effectiveness** of alternatives and chose the best alternative

9. Develop a **control strategy** including a sensitivity analysis and the assessment of fail-safe precautions
10. Test the **automation** concept (e.g., requirements for information processing, presentation functions, operator functions, communication system, hardware specifications, etc.)
11. Develop **software** (i.e., application software, user interfaces, control software)
12. Evaluate the UDS hydraulics, controller behaviour, pollution discharges, etc. with **detailed** models
13. Define factory and site acceptance **tests**
14. **Undertake personnel resources** planning (education requirements, training, maintenance, etc.)
15. Produce a preliminary **operations manual**.

6.4. DEVELOPMENT AND ANALYSIS OF CONTROL STRATEGIES

Controllers adjust regulators to achieve minimum deviations of the regulated flow discharge or level from the **set points**. A **control strategy** is defined as the time sequence of all regulator set points in a RTCS. In almost all cases with multiple control loops it can be shown that the optimum strategy requires time-varying set points.

A control strategy has to be physically executable (feasible), i.e., flows and levels cannot exceed the physically possible rates (**static constraints**). Additionally, the control strategy has to be in harmony with the physical laws of flow in a drainage system, including continuity and energy balances (**dynamic constraints**). The dynamic constraint of a storage device is its mass balance and, for a sewer pipe, a mathematical description of flow.

Since the RTCS depends on the loading of the system (i.e., storm inflows, pollutant loads, etc.), it is obvious that a **loading forecast** represents an extremely important information for decisions on controlling flows. The further these forecasts reach into the near future, the better control strategy can be achieved. The options which help determine the inputs to a drainage system include:

- flow and level measurements in upstream sewers, which would allow to react within the time of sewage travel,
- rain measurements and applications of rainfall/runoff models which extend the available reaction time by the concentration time of the overland flow on the catchment surface, and
- rainfall forecasts to gain additional time depending on the forecast time horizon.

If none of the above listed information is available only a local control strategy (i.e., reactive strategy) can be applied.

Since measurements include **errors**, it is important to check control strategies with respect to measurement errors or failures of sensors. From a practical point of view, control strategies have to be "cautious" to avoid "surprises", which could be caused by unexpected storm developments, inflows from unmonitored tributary sewers, etc. Control strategies are usually based only on measurements. However, it might be useful to develop a better strategy by using **off-line simulation** of drainage processes or even by including an **on-line simulation** model to "interpolate" process data that cannot be measured.

The most rigorous approach to find a control strategy is based on mathematical **optimisation**. This technique allows the evaluation of the control performance on an absolute ("the best") rather than a relative ("a better") scale. Here, the problem is translated to the minimisation of an **objective function** subject to **constraints**. The objective function is usually of a mixed integer/continuous, nonlinear, and non-monotonous type. Since powerful analytical optimisation techniques are not available for this type of objectives, the function has to be further simplified (e.g., by linear programming).

Heuristic methods for finding a control strategy can be directly derived from the experience of the operation personnel. This is usually done by specifying an initial control strategy (e.g., the default, fixed set point strategy) and using multiple simulation runs to improve the initial strategy by a trial-and-error procedure. When no further improvement appears possible, it is assumed that an optimum strategy has been found.

Optimisation or search results can be translated into **decision matrices** (DMs). Each element of the matrix represents the control decision which has to be executed for a given combination of state and loading variables. DMs allow for very fast on-line execution of control strategies. Simplifications of DMs take the form of **decision trees** that are sets of "if-then-else" statements. Recent research in this field focuses on the use of expert systems and neural networks; however, no implementation of these tools in practice has been as yet reported.

6.5. ANALYSIS OF CONTROLLER BEHAVIOUR

The **control loop** defined above is a basic element of any RTCS. In a **feedback** loop, control commands are actuated depending on the measured deviation of the controlled process from the **set point**. Unless there is a deviation, the feedback controller is not actuated. A **feedforward** controller anticipates the immediate future values of these deviations using a model of the process controlled and activates controls ahead of the time to avoid these deviations. A **feedback / feedforward** controller is a combination of the two.

A standard controller used for **continuously** variable regulator settings is the proportional-integral-derivative **PID-controller** and its simplified versions (P, PI, PD). Its signal sent to the regulator is in the form of a function of the difference between the measured variable and the set point. The parameters of that function have to be calibrated unless the controller has an **auto-tuning** capability. Calibration is performed through analysis of the underlying differential equations, or through real or simulated experiments.

Two point or on/off control is the simplest and most frequently applied method of **discrete** control. It has only two positions: on/off or open/closed. An example is the two-point control of a pump used to fill a reservoir. The pump switches on at a low level and off at a high level. The difference between the two switching levels is called the **dead band**. **Three point** controllers are typically used for such regulators as sluice gates, weirs, etc. In the middle position of the controller, the output signal is indifferent, and in the other positions, it reaches either a maximum or minimum.

Once the control system has been implemented, the behaviour of a controller has to be tested. Especially the interactions of neighbouring controllers require rather involved analyses featuring a detailed hydrodynamic model with the controller functions built-in. Full-scale experiments over the whole range of control variables have to be carried out to ensure that operational malfunction such as overshoot, instability, etc. cannot occur. During start-up operation, the initially selected control parameters can be fine-tuned to ensure that the optimum controller behaviour is eventually attained.

6.6. PLANNING TOOLS

In the early planning stages, a conceptual model of the UDS system is developed and has to be able to simulate the key process variables such as flow rates in hydraulic bottlenecks, storage contents, overflow rates, times of flow travel, pollutant concentrations, etc. The model should be calibrated and verified with process data. Furthermore, the model must allow for input of control strategies, e.g., a set of if-then-else rules, a control matrix, or an optimisation routine. Examples of simulation programs used in this stage of RTC planning are FITASIM (Wolf-Schumann *et al.* 1991) or SAMBA-CONTROL (Triton 1991).

For comparison of control alternatives, the model is first used to simulate the current system's behaviour. Inputs to the model are the events of interest, for example all rainfall events of a representative period, a set of dry weather flow scenarios, etc. The performance of the system is thus analysed under the current operation conditions. If flooding and/or CSO occurs and, at the same time, spare storage, transport, or treatment capacities can be identified, a new control strategy that activates these spare capacities could be defined.

For the same set of events the system is simulated once again, this time including the new control strategy involving such structural modifications as weir and throttle adjustments, or storage, transport and treatment capacity expansions. A different performance is obtained, i.e., different flooding volumes or CSO discharges.

Each of the alternatives can be characterised by its specific performance and costs. A cost-benefit analysis is carried out to determine the optimal solution. With this step, the conceptual analysis is finished and a decision must be made whether to implement the RTC system. Whichever alternative is chosen the risk of failure should not, of course, exceed the risk level in the existing system.

If decision to implement a RTC is made, the behaviour of the proposed system is simulated in full detail with a more detailed model. These simulations involve the hydrodynamic modelling of sewer flows and flow levels, including transients; pollutant transport modelling; treatment plant simulation; and, dynamic modelling of controller behaviour. Simulation programs used in this stage of the project are for example HydroWorks (Wallingford 1994) and MOUSE (Bo-Nielsen 1993).

6.7. MAN-MACHINE INTERFACES AND OPERATIONAL TOOLS

Any RTCS that is not operated in the fully automatic mode needs a well-defined operator (user) interface. Historically, such an interface comprised analogue displays, strip chart recorders, and control switches using relays. Today, active wall panels and colour screens are used to display standard features common in a variety of application fields. Currently under development are UDS specific **simulators** that allow an animated display of the current state of the UDS, its loading, and a dynamic evolution of the UDS state.

Such RTC simulators can be used to evaluate control strategies before they are actually executed. The simulator can be run in two ways: it can be used as an on-line tool, which has to be fed with process measurements to update the state variables, or as an off-line tool, when it can be used to train operators and to analyse past events. The successful development of these systems requires extensive joint efforts of the software development engineer and the operating personnel to guarantee that the necessary information (and only that) is available and displayed.

6.8. OPERATION OF RTC SYSTEMS

Introduction of RTC requires a number of organisational changes within the urban drainage department. For smooth implementation of such changes, the key activities are communication and documentation.

Operation of a RTC system requires intensive communications among all divisions and levels of an urban drainage department. This is a management task that is extremely difficult to achieve, especially in large organisations. For adoption of RTC in larger agencies, it seems almost impossible to imbed the RTC planning and operation in either of the traditional planning and operations divisions. Often it is advisable to create a new **operational control** division instead, originating from a UDS performance monitoring group (i.e., the group operating a measurement network), and have this division to report directly to the department's management. Thus, "traditional rivalries" can be avoided and well-educated personnel can be hired according to the needs. In small agencies, creation of a new division is not affordable. Instead, it is preferable to choose such a RTC technology that can be operated with the existing operating personnel (usually the treatment plant operators).

In any RTCS under supervisory control, the operating personnel has to be advised on how to proceed in all possible operational situations (i.e., both routine and emergency). Naturally, this should be done before critical situations arise. For example, the operator has to know and understand the operational objectives and their priorities (e.g., first allow an overflow at location x, then at y, etc.). Extreme situations such as the decision which district to flood first have to be included! Operational advice should be **documented** because it reflects a consensus of all involved parties (i.e., the public, supervising agency, management, staff). Without such an **operations manual,** operators might be afraid to take any steps to avoid being blamed in cases of mismanagement. Since it is almost impossible to foresee all possible operational states of the system, the operators need to have some freedom to make "reasonable control decisions" and their adoption of such decisions has to be backed by the management.

The general purpose of RTC is to understand drainage processes, and to monitor and manipulate them, in order to achieve **maximum performance** of the existing UDS. The information gathered during RTC operation is valuable for the whole operating agency, including both the traditional planning and O&M divisions. It is an important management aspect that the new operational division will not be regarded as "big brother" but as the source of information that allows the documentation of successes. **Success** is then defined as a close match between the envisioned and actual problem solutions. Success should result in **benefits**, and the staff's successful operation be positively acknowledged by the management and / or the supervising agency.

6.9. BENEFITS OF RTC

General performance estimates of RTC systems cannot be given because of the multi-objective character of the approach and the large variety of existing systems. However, if CSOs are the problem of concern, an analysis of the current behaviour of the system can reveal whether RTC might be beneficial.

It can be shown that during wet weather approximately one half of the total system capacity (i.e., storage, transport and treatment capacities) remains unused (Almeida 1993). The potential reduction of CSOs can be obtained by comparing the "no control" scenario with a RTC scenario, for which the control strategies are optimised (Papageorgiou 1983; Schilling and Petersen 1987; Nelen 1992). Based on case studies for a variety of sewer systems, it can be concluded that with optimum RTC the CSO volume per event can be reduced by approximately 25% of the total available storage, taking all physical constraints into account (Schilling 1996).

In terms of CSO abatement, the typical RTC benefits include drastic reductions in overflows at the most sensitive locations, reduced frequencies of overflows (about by 50%), and reduced annual CSO volumes (by 10-20%). Secondary benefits include lower energy costs (less pumping), improved wastewater treatment, control of sediment in sewers, and better supervision, understanding and record keeping.

7. Sewer System Operation in the Future

There is a trend in many countries to privatise urban drainage operations, or at least to provide urban drainage departments with more freedom and independence to act. This trend is likely to lead to a more holistic way of operation in the sense that the urban drainage system will be operated to minimise total costs or, more generally, minimise the "business risk" (Price 1996). In urban drainage the business risk comprises the continuous costs of operations (e.g., energy, maintenance, repair, reporting, etc.) plus the damages caused by adverse events and the respective probability of their occurrence (e.g., flooding, sewer collapses, pollution, etc.). Both, daily operation and adverse events cause direct, indirect or social costs that must be minimised.

In terms of the physical system and its operation in real time, the trend is to include the states of both the wastewater treatment plant and the receiving water in the operation of the urban drainage system. For example, there is no good reason for "wasting" precious storage or treatment capacity on weak sewage. If the available capacity would primarily be used to capture the most polluted sewage, the potential for pollutant discharge reduction can be increased ("pollution based RTC"). If the treatment plant and the CSOs discharge into the same receiving water, it is obvious that overall pollutant discharges should be minimised, because it is not rational to reduce CSOs at the expense of a much lower efficiency of the treatment plant.

This argument can be expanded, if adverse effects on the receiving waters are considered: the whole drainage system contains numerous elements that can be utilised with respect to the prevention of water pollution. This includes not only the hydraulic storage volume in tanks and pipes, but also the time lag of processes in the wastewater treatment plant and the receiving water. For example, in the case of an acute-effect pollution (e.g., oxygen depletion in a water course), a time delay of the failure of the secondary clarifier under hydraulic loading introduces a time shift in the occurrence of the acute effect that can be assessed and used as extra capacity. The direct focus on minimising the detrimental impact to the receiving water allows the identification of the optimal control strategy for the operation of the total system under dynamic loading. Future research is focusing on such a kind of water quality-oriented aspects of "integrated" RTC.

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SEWERAGE REHABILITATION

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1. Introduction

Different types of failures of urban storm drainage and combined sewer systems need to be distinguished: structural, hydraulic and environmental. The respective failure diagnoses are based on sewer surveys and on analysis with a simulation model, such as SWMM, to evaluate the flooding and pollution load discharged into receiving waters. Rehabilitation may include repair, reinforcement, replacement, flow attenuation or reduction, overflow management, treatment, etc. The U.S. Environmental Protection Agency has established requirements for good management and rehabilitation of drainage systems.

In the past, rehabilitation of sewerage systems has often been performed on a crisis management basis. At present there is a strong tendency toward a systematic approach to rehabilitation because of the magnitude of the problem.

2. Rehabilitation Management

The magnitude of urban water resources infrastructure is enormous. Grigg (1994) stated that the replacement cost of the water supply, wastewater and stormwater infrastructure is in the order of \$2000 to \$3000 per capita in the USA. An official estimate has been made in the USA by the Environmental Protection Agency which refers only to wastewater management. This survey reported a need of \$82.61 x 10^9 (1982 dollars) for the following six categories:

1. Secondary treatment
2. Advanced Treatment
3. Infiltration /Inflow and Replacement / Rehab
4. New Collectors and Interceptors
5. Combined Sewer Overflows
6. Treatment and/or Control of Storm Sewers

Grigg (1994) proposed an estimate of \$150 x 10^9 for the national need in storm water drainage and \$185 x 10^9 if combined sewers are included.

Grigg (1988) recommended the use of computerised Maintenance Management Systems (MMS). Such systems should include programs for inventory control, condition assessment, rehabilitation and replacement, preventive maintenance,

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budgeting and decision support. The *inventory* can conveniently be based on Geographic Information Systems (GIS). The *condition assessment* typically includes parameters such as capacity, safety, structural integrity, quality of service, role and age. The *needs assessment* includes an analysis of projected needs, alternatives, rate of deterioration and obsolescence, cost-benefits and impact studies as well as sensitivity analysis. The use of MMS must be accompanied by a long-term commitment of personnel and budget.

3. Failure, Diagnosis and Rehabilitation

This section is essentially taken from Delleur (1994), Figures 1, 2 and 3 are taken from Serpente (1994). The purpose of this section is to classify the mechanisms of sewerage failures, their diagnosis and rehabilitation techniques, and to review the associated literature with emphasis on urban storm drainage and combined sewer systems. Although the initial concern usually is for structural integrity, three types of malfunction or failure must be recognised: 1) structural, 2) hydraulic and 3) environmental. The types of failure, their diagnosis and rehabilitation methods are summarised in Table 1.

A survey in the UK (Clegg *et al.* 1988) revealed that over twice as much is being spent on hydraulic and environmental upgrading (flood relief and pollution control) than on structural upgrading. This survey also indicated that in many cases structural and hydraulic problems had to be dealt with simultaneously as part of integrated solutions.

3.1. STRUCTURAL FAILURES

Structural failures usually originate with a minor initial defect such as cracking due to excessive vertical load or poor bedding, leaky joints or badly made connections, interference from other structures and excavations. Further deterioration can occur due to erosion of joint material, and corrosion of concrete by hydrogen sulfide and other chemicals.

Water flowing in and out of the sewer, or along the sewer through the bedding material, can cause soil migration resulting in gradual loss of support. This is more likely to occur in silts and sands than in clays. The loss of side support causes the cracks to develop into fractures and allows the side of the pipe to move outwards and the crown to drop, with collapse imminent, **Figure 1**.

Fine soil can be washed from under a defective joint or a defective lateral connection in precast concrete or vitrified clay sewers. This causes the loss of support under the joint or connection. The subsequent subsidence of the precast pipe causes deflection and cracking of the pipe, **Figure 2.**

In warm climates the presence of hydrogen sulfide in sewage combines with condensed water at the crown of the sewer forming sulfuric acid which corrodes the concrete pipe wall near the crown. External corrosion from corrosive soil and groundwater as well as internal corrosion from industrial wastes contribute to the deterioration of sewer pipes.

TABLE 1. Summary of Failure, Diagnosis and Rehabilitation of Urban Drainage Infrastructure

FAILURE	STRUCTURAL	HYDRAULIC	ENVIRONMENTAL
Type	Subsidence	Flooding	Combined sewer overflow
	Total or partial collapse	Surcharge	Sanitary sewer overflow
	Corrosion	Leakage	Pollution of receiving waters
	Loss of soil support	Inflow / infiltration	
	Loose brick	Increased roughness	
	Soft mortar	Water hammer	
	Other structures interference	Flow instabilities	
Diagnosis	Monitoring	Monitoring Levels	Monitoring occurrences of CSO's
	Sewer Survey	Flows	Quality in receiving streams
	Man entry	TV inspection	SWMM model calibration and verification
	TV inspection	SWMM model EXTRAN calibration and verification	
	Infra red scan		
	Penetrating radar		
Rehabilitation	Maintenance	Maintenance	Storage in line and off line
	Repair	Increase flow capacity	Treatment
	Renovation / Renewal	Flow attenuation / reduction	Inlet control
	Lining	Storage in line and off line	Flow attenuation / reduction
	Replacement	Inlet control	Aeration
		Lining	Source control
		Real time control	Real time control
		Replacement	Replacement

Most old brick sewers have problems. Man entry is necessary to observe loose bricks and soft mortar as these defects do not show up in a television inspection. The first deterioration usually is a general mortar loss. Soft mortar will not impede root intrusion or infiltration. Because of mortar deterioration ground water infiltration can occur and bring fine soil into the sewer. Then voids can form around the sewer invert causing loss of support beneath the sewer. As the invert drops into the cavity, the side walls can also drop resulting in a total loss of the structure integrity, **Figure 3**.

Pipe breaks or collapses can be triggered by events not related to the cause of deterioration, such as an extreme storm or vibration due to a nearby excavation or an unusual transient load. Contact or interference with other structures (such as installation of lamp post on storm drains), improper bedding during installation, expansive or corrosive soils, frost loads, brittle failure, live loads, leaky water mains, repeated rodding, accidents and design defects are some of the external causes of pipe rupture.

The different structural failure mechanisms have been described, among others, by EPA (1991), the Department of Housing and Urban Development (1984), WRc (1983) and Serpente (1994). Structural rehabilitation methodologies have been described, among others by Erdos (1992), Schrock (1984, 1988, 1992), NASSCO (1991, 1992), APWA & AGCC (1989, 1991) and are also presented in several articles in Macaitis (1994). Trenchless technologies are commonly used (Doherty and Woodfall, 1996).

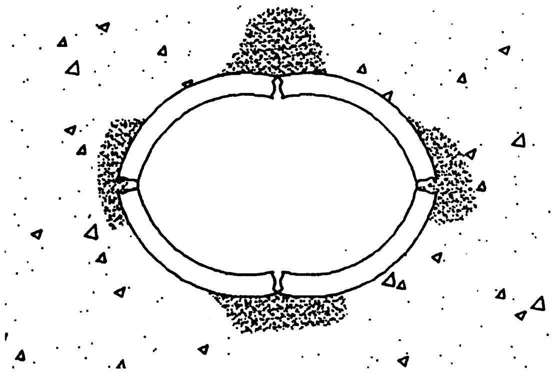

Figure 1. Concrete Pipe Fracture.
Source: Serpente (1994), reproduced with permission.

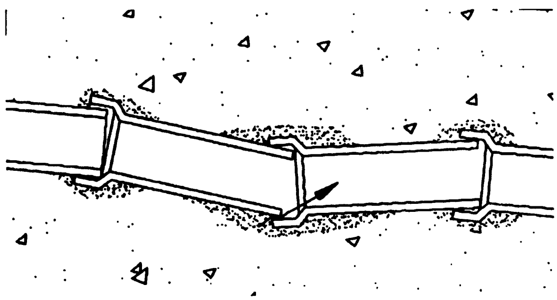

Figure 2. Precast Concrete Pipe, Displaced Joints.
Source: Serpente (1994), reproduced with permission.

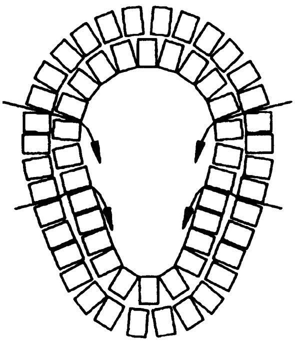

Figure 3. Brick Sewer, Mortar Loss and Infiltration.
Source: Serpente (1994), reproduced with permission.

3.2. STRUCTURAL DIAGNOSIS

Many structural defects can be detected by visual inspection when walking along the sewer right-of-way or by internal inspection. Right-of-way inspection may detect items such as sunken areas over sewers, ponding of water, deteriorating condition at stream crossings, missing or broken manhole frames and covers, and deteriorating or failure conditions of catch basin inlets and other visible structures. Visual inspection of manholes can detect the conditions of manhole shaft, structural conditions of walls, joint condition and any leakage. Larger sewers can be inspected by walking, while smaller ones are generally inspected by closed circuit television. These observations can indicate defects such as root penetration, grease, sediment, rocks and trash, horizontal deviations, sags in pipe, open joints, circumferential cracks, longitudinal cracks, and missing pipe segments.

Promising non-destructive remote sensing diagnostic methods have recently been developed. The infra-red scanner system uses the temperature differentials surrounding a sewer void. The infra-red image can be recorded on film or digitally for computer analysis (Weil 1990). The penetrating radar technique uses the emission of short pulses of electro-magnetic energy. From the wave forms of the echoes returned by the various interfaces of different materials it is possible to detect deterioration of concrete and voids. Maser (1989) suggests the joint use of radar and thermography. Further details can be found in specialised articles in Macaitis (1994).

3.3. STRUCTURAL REHABILITATION

The new technological developments that have occurred in pipeline rehabilitation during the last ten years have been summarised, among others, by Schrock (1984, 1988, 1992). Schrock distinguishes between external and internal rehabilitation methods. External methods include chemical and cement grouting. Internal methods include cement mortar lining, cured-in-place pipe insertion, mechanical sealing devices, etc. Additional details on these and other techniques can be found in specialised articles in Macaitis (1994).

3.4. HYDRAULIC FAILURES

A hydraulic failure occurs when the drainage system is unable to convey the runoff under the assumed design conditions. A free surface (i.e., non-pressure) condition is usually assumed in separate or combined sewers. The principal consequences of hydraulic failure are flooding, surcharge, excessive leakage or infiltration, blockage, flow instabilities and water hammer. Flooding is the most common result of hydraulic failure, and includes street flooding and basement flooding (Barber 1992). Surcharge is caused by the passage from free surface gravity flow to pressure flow and usually occurs when the capacity of the sewer has been exceeded.

Reduction of flow capacity can be caused by increased hydraulic roughness due to pipe joint eccentricity, ageing of the pipe material, sliming, encrustation, mortar or brick loss in masonry sewers, presence of rubble on the invert of the sewer and sediment accumulation. The hydraulic roughness of deteriorated pipes depends much more on the pipe condition than on the absolute roughness of the pipe material. For example, a vitrified clay sewer pipe which normally has a Manning's n of 0.015, can reach n = 0.033, with heavy encrustation and some debris on the invert, thus reducing the flow capacity by a factor of 220 percent.

Rapid pressure changes and pulsating flows are also a cause of pipe deterioration and failure. Rapid pressure changes can be caused by water hammer, particularly in systems involving pumping stations and check valves in which transients can happen due to power failure. Flow instabilities can occur even at constant flow. For discharges greater than a critical value, $Q / (g^{1/2} D^{5/2}) > 0.6$, both full and free surface flows are possible depending upon aeration (Zech *et al.* 1984). The coefficient of discharge can change suddenly resulting in unstable flow. For discharges in the range of $0.6 < Q / (g^{1/2} D^{5/2}) < 0.8$, air pockets appear resulting in pulsating flow. The ensuing pressure fluctuations can cause damage in pipes and manholes. The flow instabilities that occur at the transition from free surface to full flow and vice versa are very complex and depend upon discharge, head and tailwater conditions (Huberlant and Zech, 1990).

3.5. HYDRAULIC DIAGNOSIS

As hydraulic failures are usually related to insufficient discharge capacity, flow monitoring programs are essential. Methods of monitoring flows within a wastewater collection system include the use of weirs, Parshall flumes, Palmer-Bowlus flumes, dye-dilution/chemical tracers, depth and pressure sensors, ultrasonic and electromagnetic depth/velocity meters. In addition, pump stations are a logical choice for monitoring wastewater flows. These different techniques are appropriately described in manuals (ASCE-WPCF, 1983).

Simulation models can be used to detect malfunction of sewerage networks. The assessment of the hydraulic performance consists of four parts: 1) the selection of an appropriate model, 2) the calibration and verification of the model, 3) the use of the model to assess the hydraulic performance, and 4) the identification of the location of performance deficiencies and their causes.

The model most commonly used for this purpose in the United States is the Storm Water Management Model (SWMM) developed for the Environmental Protection Agency (Huber *et al.* 1984). SWMM is a comprehensive mathematical model developed for both continuous and single event simulation of the urban runoff quantity and quality in storm and combined sewer systems. All aspects of the urban hydrologic and quality cycles are simulated including surface runoff, transport through the drainage network, storage and treatment and pollutant loadings to receiving waters. The RUNOFF block generates surface runoff based on observed or arbitrary rainfall hyetographs, antecedent conditions, land use, topography and so forth. The Transport block and Extended (EXTRAN) Transport blocks combine and route the inflows through the drainage system. Dry weather flow and infiltration into the sewer system can optionally be generated using the transport block. The effects of control devices on flow quality and quantity are simulated in the storage/treatment block and finally the receiving block routes the effluent hydrographs and pollutographs through the receiving waters. Backwater effects and surcharge require the use of EXTRAN.

Wildbore (1994) states "A verified simulation model of the system is a corner stone of the strategy and is invaluable in understanding the present behaviour of the system and in developing the best upgrading option for any structural and hydraulic problems, including CSO operation. Such a simulation model is also a powerful management tool which allows an assessment to be made of the operational changes on the running cost of the system."

Maalel and Huber (1984) have shown that the use of multiple event simulation proved to be more reliable than the single event simulation and better suited for SWMM calibration and verification for continuous simulation. An expert system for the estimation and calibration of the parameters used in the quantity part of the runoff block of SWMM was developed by Baffaut and Delleur (1989). After the model has been calibrated it is essential to proceed with the verification to ensure that the model predictions are in agreement with known local performance. All available flow measurements in the network and at pumping stations should be used for this purpose. The simulation model is then ready to assess the hydraulic performance and to determine the conditions when a pipe begins to surcharge, when flooding begins at a location, when the first overflow occurs, the maximum volume of the overflow, the flow directed for treatment and storage, and so forth.

The output summaries of EXTRAN include the maximum to design flow ratios, the lengths of surcharge and the lengths of flooding. These quantities can form the basis for an evaluation parameter that quantifies the overloading of the sewers (Reyna *et al.* 1994). Any restriction of flow through a pipe such as a throttle due to partial blockage or collapse or encroaching connection will result in a recorded hydrograph with a much higher peak than the simulated hydrographs. Likewise the presence of an unrecorded overflow is revealed by the truncation of the observed hydrograph compared to the simulated hydrograph.

Properly calibrated models such as SWMM provide a very effective tool for the diagnosis of hydraulic failures. It is important, however, that the model be appropriately verified. It must be ascertained that it properly represents the physical configuration of the pipe system and that it represents correctly the response of the catchment and sewer system to rainfall events. The importance of a verified hydraulic model in sewerage rehabilitation programs has been discussed by Wildbore (1994) and by Roesner and Burgess (1992).

Other appropriate simulation models are available such as HydroWorks developed in the United Kingdom, the Dorsch hydrograph method developed in Germany and MOUSE developed in Denmark.

3.6. HYDRAULIC REHABILITATION

Rehabilitation methods of the hydraulic performance of storm sewers include: a) increase of flow capacity, b) flow attenuation and c) flow reduction. Methods of increasing the flow capacity are: 1) maintenance which can remove the effects of protruding laterals, roots, encrustation, silt, etc.; 2) lining which considerably reduces the hydraulic roughness but also decreases the pipe diameter; however, the first effect usually predominates over the latter; 3) polymer injection can increase capacity and appears to be effective for small catchments with small gradients, and may provide a temporary protection against flooding until a permanent solution can be implemented; 4) renovation or improvement of the structural condition, for example, by grouting or repointing (for brick sewers); 5) parallel relief pipe and 6) replacement, possibly with a larger diameter sewer, if the existing sewer has structural problems as well. The details of these methods and their merits are discussed in Macaitis

(1994). The second method of hydraulic rehabilitation is that of flow attenuation which is the reduction of the peak of the hydrograph so that the maximum flow during a storm event does not exceed the storm sewer capacity. To achieve this, storage is needed above or below ground. Either on-line or off-line detention tanks or tunnels can be used as well as surface detention.

A reduction in peak discharge can also be obtained by reducing the number of storm inlets, thus increasing the individual contributing areas and the overland flow travel time. This latter method is practical only for parking lots and areas with slow moving traffic.

Real time control also can make better use of the in-system storage. It includes rainfall and flow sensors linked to a central computer for real time simulation of the hydrologic response and the hydraulic behaviour of the sewer system. With the appropriate control strategies, commands are issued for the coordination and control of flow and storage devices throughout the sewer system (Labadie *et al.* 1980, Doering *et al.* 1987).

The third method of hydraulic rehabilitation is that of flow reduction. This is usually the most desirable of the three alternatives. Flow reduction can be achieved by a decrease of impervious surface, use of permeable pavements, control of infiltration into sewers, disconnection of contributing areas and overflows possibly combined with storage. Pisano (1989) reviewed the concept of storm water management (SWM) strategy which "starts at the very top of the sewershed and searches for opportunities to control and manage the storm water so that the pipes do not overload as you move downstream". In spite of the obvious advantages of SWM it requires a substantial engineering support and close observation and fine tuning, because of the many small facilities distributed throughout the sewershed. Inlet control methodologies have been successfully used in many places. Some of these are in the Province of Ontario, Canada (Wisner, 1984) and in Japan (Fujita, 1987). The Experimental Sewer System (ESS) implemented in some of the outskirts of Tokyo includes infiltration inlets, infiltration trenches, infiltration LU-curbs, permeable pavements, and storage inside the infiltration facilities, in manholes, and in circuitously laid out sewer pipes.

3.7. ENVIRONMENTAL FAILURES

The most common environmental failure is the discharge of sanitary sewer overflows (SSO) and combined sewer overflows (CSO) creating water quality violations in receiving streams. The impact of urban runoff on receiving waters has been summarised by Field and Turkeltaub (1981). The Nationwide Urban Runoff Program (NURP) performed by the U.S. Environmental Protection Agency (1983) and the U.S. Geological Survey found that some contaminated storm discharges can contribute toxics and coliforms resulting in water quality violations in receiving streams.

3.8. ENVIRONMENTAL DIAGNOSIS

The diagnosis of excessive pollutant discharge is based principally on two procedures. The first is water quality monitoring and the second one is water quality simulation. Automatic samplers are available and they can be programmed in such a way that the time elapsed between consecutive samples is shorter during the rising hydrograph than during the falling or recession part of the hydrograph.

Progress has been made in the real time measurement of suspended and deposited solids in sewers. Vanderborght and Wollast (1990) obtained a linear relationship between a measure of white light absorption and suspended solids concentration. This is the basis for the real time monitoring of suspended solids in the combined sewer system in Brussels, Belgium (Verbanck, 1992). Delleur and Gyasi-Agyei (1994) have proposed a transfer function model for the prediction of suspended solids in urban sewers. The protocols for the chemical analyses and interpretation of the data have been developed by the U.S. Environmental Protection Agency (1983) and the U.S. Geological Survey for the Nationwide Urban Runoff Program (NURP). The US EPA through its Storm and Combined Sewer Pollution Control and Development Program has developed counter measures to control overflow pollution problems.

The merits of source control, and collection system control storage and control have been examined by Traver (1982). Runoff quality simulation can be performed by mathematical models such as SWMM cited earlier (Roesner and Burgess, 1992). Baffaut and Delleur (1990) developed an expert system to assist in the calibration of SWMM runoff quality. As for the modelling of the runoff quantity, it is important that the simulation model be verified before it is used to evaluate the excessive pollutant loads from sanitary sewer overflows, from combined sewer overflows and their effects in receiving streams.

3.9. ENVIRONMENTAL REHABILITATION

In the US the Clean Water Act (CWA) of 1971 (The Federal Water Pollution Control Act), as amended in 1972, prohibits the discharge of any pollutant to navigable waters unless authorised by a National Pollutant Discharge Elimination System (NPDES) permit. The US Environmental Protection Agency (1990) published rules for the regulation of municipal and industrial stormwater discharges. Municipalities with population over 100,000 must apply for a permit to discharge stormwater. Rules concerning combined sewer system discharges were recently issued by the US Environmental Protection Agency (1993). Guidance on the development of stormwater pollution prevention plans and identification of Best Management Practices (BMP) has been given by the US Environmental Protection Agency. These documents set the requirements for good management and rehabilitation of drainage systems and state EPA's enforcement authority. Overviews of the EPA regulations have been given by Elder (1989) and Roesner (1993). Guidance for storm water sampling has been given by Smoley (1993).

Many of the methods presented for hydraulic rehabilitation are also applicable to environmental rehabilitation such as flow attenuation, on-line and off-line storage, real time control and flow reduction. The inlet control techniques (Pisano 1989), and the infiltration enhancements (Fujita and Koyama, 1990) mentioned earlier are also very effective in controlling the transport of pollutants. Many of the appropriate methods have been discussed in the ASCE book entitled "Urban Stormwater Quality Enhancement - Source Control, Retrofitting, and Combined Sewer Technology" (Torno, 1989) and in ASCE/WEF (1992). The latter reference includes a comparison of pollutant removal from urban runoff by means of an extended detention pond, wet pond, infiltration trench, infiltration basin, porous pavement, water quality inlet, filter strip and grassed swale.

The infiltration of urban stormwater has been the subject of several studies in Europe and Japan. Some of the results are summarised in Jacobsen and Mikkelsen (1993). There is a desire to increase infiltration to help maintain the groundwater levels and to compensate for the decrease of natural recharge of aquifers in urban areas. However, there is a risk of contamination of the groundwater. Hutson and Wagenet (1989) developed a mathematical model, LEACHP, to predict the fate of pesticides in soils. According to Weyer *et al.* (1993) the model is helpful in evaluating the risk of contamination from stormwater infiltration. The Institute of Hydrology at Wallingford and the Construction Industry Research and Information Association (CIRIA) are developing a manual of good practice for the U.K. (Walden 1993). In general, the feeling is that infiltration in conjunction with source control can be the foundation of an ecological design for a sustainable environment (Geldof *et al.* 1993).

New strategies have been proposed for the abatement of pollution in receiving water bodies due to combined sewer overflows. The reactive strategy, which is the one most commonly used, responds to the state of the system as the storm progresses over the catchment. Case studies in which the combined sewer overflows are minimised by means of in-system storage have been reported by Brueck *et al.* (1982) and by Ward (1982). Patry and Mariño (1984) extended the strategy to include the predictive mode in which the system is operated in response to its anticipated state prior to the actual occurrence of the rainfall, and in response to both the actual and predicted states of the system once the event has started.

3.10. SYSTEMATIC STRATEGIES FOR REHABILITATION

The National Science Foundation (1993) advocates a holistic approach for its civil infrastructure systems research. Its program is system oriented, emphasises integrating existing knowledge and technology and cutting across diverse disciplines. Several efforts already have taken place in this direction. A few of these are listed below.

Systematic strategies for rehabilitation and maintenance of urban drainage infrastructure are needed to replace the currently often used crisis management (Weissert, 1989). Karaa (1989) proposes a five-step strategy for the rehabilitation improvement and maintenance of sewer infrastructure. The first step is an inventory of the drainage infrastructure that can be conveniently performed with the help of Geographical Information Systems. The second step is the maintenance history and failure analysis. This is to track the deterioration and failure trends of several components of the infrastructure system. As discussed earlier in this paper, simulation models such as SWMM can conveniently be interfaced to analyse the hydraulic and environmental performance and reliability of the drainage system. The third step is tracking the effectiveness of the maintenance program and its cost effectiveness. The fourth and key step is the capital improvement strategy for the scheduling of capital improvements under budget constraints. The final and fifth step is the planning and scheduling of the work load. The next paragraphs will elaborate on these last two points. The sewer inventory and maintenance management systems as described by

Karaa (1989) were implemented by the Waste Water Division of the Massachusetts Water Resources Authority in 1988.

Several methods of systems analysis have been used for the scheduling of capital improvements of urban drainage facilities. Jacobs and Wright (1989) developed a strategy for the allocation of resources to infrastructure rehabilitation and repair based on the perceived reliabilities and the future deterioration expected in the rehabilitation interval. Jacobs *et al.* (1993) proposed a chance constrained mixed integer mathematical model for long-term scheduling of stormwater rehabilitation activities. It minimises the total expected rehabilitation costs and expected costs of flooding and wear. Peak discharges are estimated by the Rational formula. The pipe rehabilitation options considered are replacement and not-in-place rehabilitation. In an extension of this work Jacobs and Medina (1992) used the kinematic wave theory in their formulation of the flow propagation.

Reyna (1993) presented a multiple objective mathematical program that integrates the most significant factors in sewer rehabilitation: structural conditions, hydraulic conditions, disruptions caused by rehabilitation operations, claims costs and construction costs. The hydraulic adequacy is evaluated by means of a complete hydraulic and quality simulation using SWMM and in particular the EXTRAN module for the investigation of surcharge and flooding. The model considers different rehabilitation technologies and encourages alternatives to the use of excavation and replacement. The model was tested on a 369 acre watershed located in the city of Indianapolis, Indiana.

An expert system for the structural and geotechnical diagnosis of a sewer system network as well as a tool for the choice of the most appropriate rehabilitation technique was developed for the Val de Marne suburbs of Paris (MacGilchrist and Mermet, 1989). Expert systems are gradually being implemented to assist the facilities engineer at US Army installations in directing the repair and maintenance of facilities. Expert systems have been proposed to evaluate sanitary and storm sewer utilities. Such an expert system would aid in the determination of the effects of new facility construction on the existing utilities at Army installations and would assist in the determination of adjustments needed to the current system (Williams *et al.* 1989).

3.11. CONCLUSIONS

The rehabilitation of urban drainage infrastructure is an important problem in the United States and elsewhere. The cost of this undertaking is staggering. Failures have been identified as structural, hydraulic and environmental. New techniques of diagnosis of these failures are emerging. Databases and geographical information systems are becoming the common building blocks of strategies for diagnosis and rehabilitation. New non-destructive remote sensing diagnosis methods are appearing, infra-red tomography and ground-probing radar appear to be promising. The use of simulation models such as SWMM are facilitating the task of evaluating both hydraulic and environmental failures. New methods of repair are rapidly being developed as an alternative to the replacement of the sewers and are gaining wide acceptance. Techniques of systems analysis, optimisation theory and expert systems are becoming new promising analytical tools for the development of strategies for rehabilitation of the urban drainage infrastructure while taking into account the physical and economic constraints. The US Environmental Agency has enforcement authority to promote good management and good rehabilitation practices of drainage systems.

4. Planning and Design Framework

4.1. UK EXPERIENCE

One of the oldest integrated approaches to sewer system rehabilitation was developed in the UK by The Water Research Centre (WRc) in their well known Sewer Rehabilitation Manual (WRc, 1983). A cost-effective strategy was proposed. It gives guidelines to select new pipe rehabilitation technologies, stresses the importance of a comprehensive hydraulic model (WASP-SIM[1]) and suggests flow reduction techniques for the creation of a plan for the optimal cost-effective rehabilitation. The model considers three aspects simultaneously: sewer collapse (structural condition), flooding (hydraulic condition) and environmental impact of discharges (environmental condition). The strategy is "to concentrate on the sewers where the consequences of failure are most severe and which justify pre-emptive action in economic terms; identify structural, water quality and flooding problems at the same time; develop integrated solutions to all of the problems in the catchment and, where possible, stage these so that the most cost effective parts can be built first". As stated by Fiddes (1989) the WRc approach advocates to "retain, where practicable, the existing pipework by optimising the network performance and hence avoiding wholesale sewer replacement". Wildbore (1994) discusses new advances of WRc and mentions the policy change in urban areas from design and construction of new systems to the operation and maintenance of existing ones.

4.2. U.S. EXPERIENCE

Several organisations have addressed different aspects of rehabilitation methods. The principal ones are : The ASCE-WPCF (1983) manual on existing sewer evaluation and rehabilitation, the Department of Housing and Urban Development report on utility infrastructure rehabilitation (1984), the National Association of Sewer Service Companies report on specification guidelines for sewer collection system maintenance and rehabilitation (1992), and the Environmental Protection Agency handbook on sewer infrastructure analysis and rehabilitation (1991).

Schrock (1994), who has participated in several ASCE committees and task forces concerned with the rehabilitation of pipelines and sewers, presented the key steps in the planning, assessing and implementing rehabilitation projects. These are:

1. Planning Investigations
2. Develop System Usage Plan
3. Assess System Conditions
4. Implement the System Usage Plan

These are summarised in **Figure 4**.

[1] Now replaced by HydroWorks

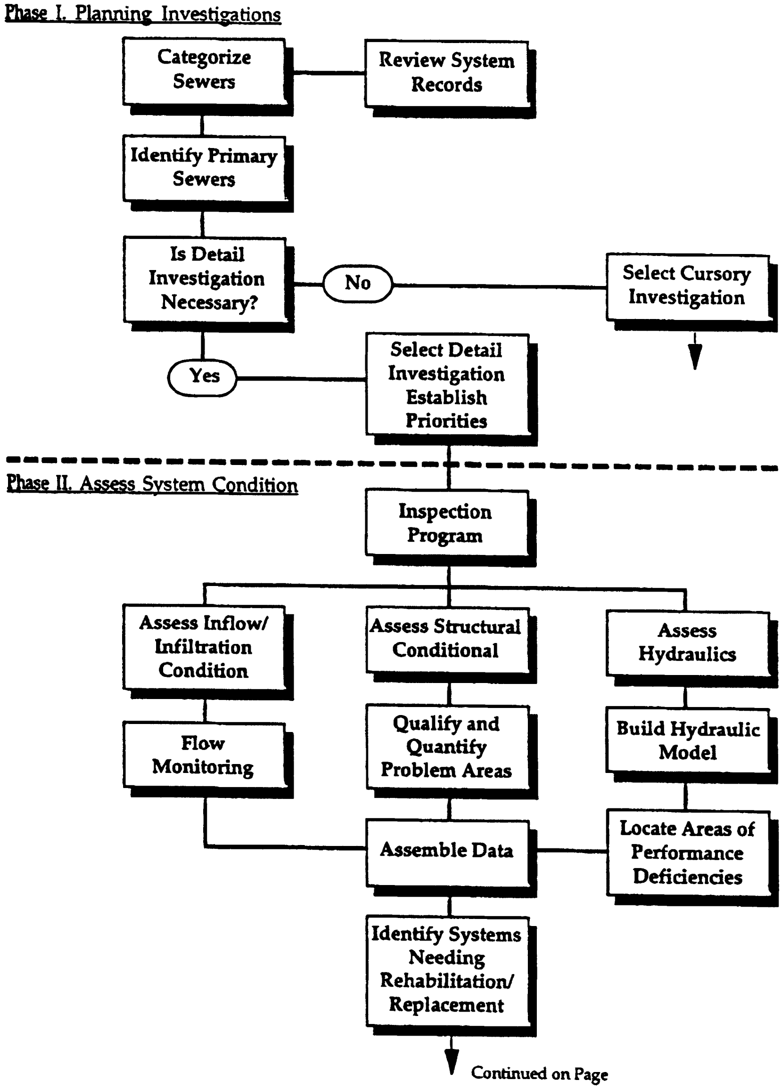

Figure 4. Investigation Flow Chart.
Source: Schrock, (1994). Reproduced with permission

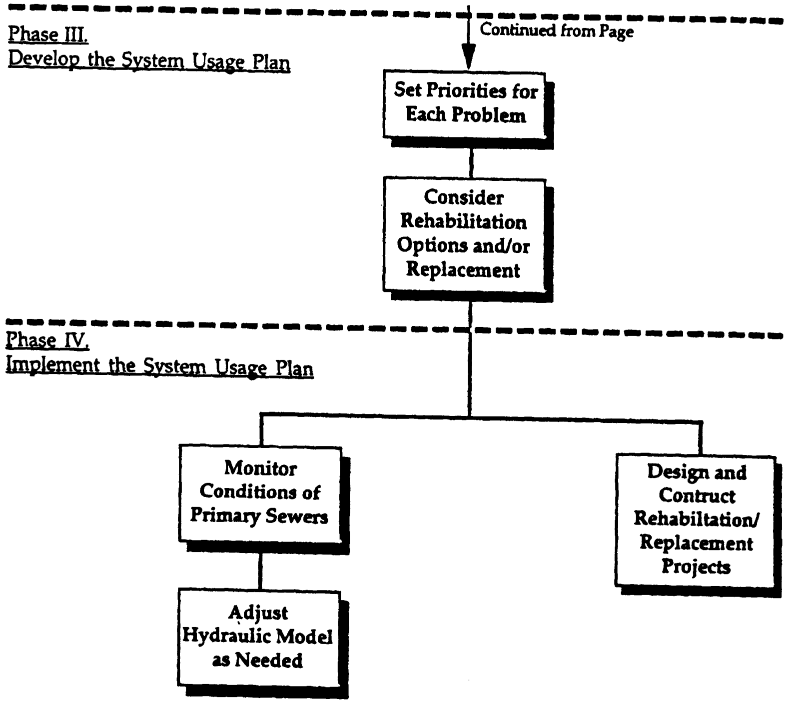

Figure 4. Investigation Flow Chart (Continuation).
Source: Schrock, (1994), reproduced with permission.

5. Rehabilitation Models

5.1. MULTI-ATTRIBUTE MODEL

This section is essentially taken from Reyna, Delleur and Vanegas (1994). The model developed by Reyna *et al.* (1994) relies on SWMM and in particular EXTRAN to model the hydraulic behaviour of the drainage system.

5.1.1. *Summary*

The need to rehabilitate many existing sewer systems to keep them operational is evident. In the past, rehabilitation has often been done on a crisis-based approach. The development of a rehabilitation plan that optimises the use of funds is the first step in a more efficient rehabilitation process. This section presents a methodology that, starting from an assessment of the structural and hydraulic conditions and data for the different rehabilitation methods, builds an optimisation model to select the segments to be rehabilitated and the rehabilitation methods to be used. The optimisation model for the planning of combined sewer systems rehabilitation, a multiple objective mathematical one, is presented as an alternative to crisis based programs. MARESS (Multi-Attribute REhabilitation of Sewer Systems) integrates several factors in its formulation. The disruption levels caused by the rehabilitation technologies employed, the overall structural condition affected by the failure impact, the overall hydraulic condition, and the maintenance and claims costs are the objective functions to be optimised. Construction costs are considered as a constraint imposed on the model. Results obtained from the application of MARESS to a case study in the city of Indianapolis are presented later.

5.1.2. *MARESS*

The deteriorated state of the urban infrastructure in the U.S. is acknowledged in the recent National Science Foundation (1993) report of the Civil Infrastructure Systems Task Group entitled "Civil Infrastructure Systems Research: Strategic Issues". According to this report, the civil urban infrastructure has not been properly maintained to meet present needs of the society. It also points out that "A holistic approach to solving deficiencies at any scale remains unresolved." It states the need for a cohesive and articulated approach that could "unravel constraints that limit constructive approaches to reviving inner-city and suburban environments." The lack of acceptance of new technologies to solve the existing problems is also addressed, giving as reasons the constraints imposed by the complexity of the urban infrastructure system and the unfamiliarity of design professionals with these new technologies.

Sewer systems face the general infrastructure ageing problems. The lack of rehabilitation increases the failure rate with the passage of time. The National Council on Public Works Improvement (Giglio *et al.* 1988) gave a grade of "C" to waste water on the report card on the national public works. According to Thomson (1991), the US should renew about 6,000 mi. of pipes yearly (a $2.5 billion job).

Considering the magnitude of the investment that is needed, it is apparent that new methodologies that help optimise the use of the funds allocated for this purpose are required to support the decision maker. The MARESS objective is to fill precisely this requirement: a holistic approach to the problem of sewer systems rehabilitation that optimises the use of funds.

Failures in sewer systems can be categorised as structural, hydraulic or environmental (Delleur, 1989). Thus, the design of a sewer rehabilitation plan needs the assessment of the physical and hydraulic conditions as the first step. The physical condition can be determined through a structural inspection program, while the hydraulic condition requires the observation of flooding and overflows, flow monitoring and the development and verification of a hydraulic model. The sewer system has to be structurally sound and hydraulically adequate. Therefore, the assessment phase will have to deal with the structural physical conditions of the system and with its hydraulic capacity conditions.

MARESS makes use of the structural data collected in the assessment phase and integrates fully the hydraulic inspection (the development and verification of the hydraulic model). This is done by making the hydraulic model an intrinsic part of the implementation.

MARESS also includes explicitly other aspects than structural and hydraulic that many times are not given due consideration. These are the life cycle costs, and urban disruptions caused by the different rehabilitation technologies.

With MARESS the selection of the rehabilitation plan to follow is done with the help of a multiobjective optimisation model. It considers the minimisation of annual maintenance and claims costs, the minimisation of disruptions, and the maximisation of overall hydraulic and structural performances (the latter is affected by a failure impact coefficient) as the objectives to be optimised. The expression of the Multi-Attribute REhabilitation of Sewers Systems Mathematical Model is given by equations (5.1)-(5.6).

min Overall Disruption Level

$$= \sum_i \sum_j DLI_j \bullet L_i \bullet x_{ij} \tag{5.1}$$

max Overall Structural Performance

$$= -\sum_i \sum_j SCI_{ij} \bullet CFII_i \bullet L_i \bullet x_{ij} \tag{5.2}$$

max Overall Hydraulics Performance

$$= -\sum_i \sum_j HCI_{ij} \bullet x_{ij} \tag{5.3}$$

min Annual Maintenance & Claims Costs

$$= \sum_i \sum_j AMCUC_{ij} \bullet L_i \bullet x_{ij} \tag{5.4}$$

subject to Annual Construction Costs

$$= \sum_i \sum_j AIUC_{ij} \bullet LCF_{ij} \bullet x_{ij} \leq \text{Maximum} \tag{5.5}$$

$$\sum_j x_{ij}\, 1 = 1 \ \forall\, i \tag{5.6}$$

where:

$x_{ij} \quad \varepsilon\{0,1\}$

i = 1 to NPIPES

j = 0 to NMETHODS (j = 0 is "do nothing")

DLI_j = Disruption Level Index

SCI_{ij} = Structural Condition Index

$CFII_i$ = Consequences of Failure Impact Index

HCI_{ij} = Hydraulic Condition Index

$AMCUC_{ij}$ = Annual Maintenance & Claims Unit Costs

$ACUC_{ij}$ = Annual Construction Unit Costs

LCF_{ij} = Length Cost Factor

L_i = Length of Pipe

The several indices are discussed in the following sections.

The model, once solved, gives as final results a sequence of lengths to be rehabilitated and the corresponding rehabilitation methods to be used (Reyna, 1993).

Hydraulic Condition. The hydraulic condition assessment requires the development and verification of a hydraulic model. The Storm Water Management Model, SWMM, (Huber and Dickinson 1988) has been selected for this task, because it is widely available and reliable. The Extran (Roesner *et al.* 1989) block of this program is able to model backwater and downstream control effects which are of primary importance in evaluating the hydraulic condition of the system.

The Runoff, Transport, and Extran blocks of SWMM are used to generate the flows and route them. (The Combine block is used to overcome size limitations). Hydrologic data, such as rainfall, imperviousness, infiltration parameters, and subcatchment geometry are entered into the Runoff block. The layout of the pipe system is entered into the Transport (no backwater or special conditions) or the Extran block.

Among the results given by SWMM, three are selected as factors to be considered to compute the Hydraulic Condition Indices (HCI_{ij}). These are: the maximum to design flow ratios, the duration of surcharge (min), and the duration of flooding (min). These indices are strongly influenced by land use. The maximum to design flow ratio indicates the possible need for increased capacity of a segment, while durations of surcharge and flooding are related to problems such as street and basement flooding. An index value in the range 0 to 10 can be assigned to each model result. For example, 0 can be assigned to no flooding and 10 to maximum flooding. For the maximum to design flow ratio a value of 0 can be assigned to a ratio less than 1 (good design) and 10 to the largest value of the ratio minus one. A weighted average of these three factors is then calculated to obtain the Hydraulic Condition Index, *HCI,* for the reach. These indices are modified, when a reach is selected to be rehabilitated. Since the HCI's are inversely proportional to the diameters, D_{ij}, and directly proportional to the Manning's roughness coefficients, n_{ij}, where the subscript i corresponds to the pipe number and the subscript j=0 corresponds to the existing sewer system and j≠0 corresponds to the j-th rehabilitation technology,

$$HCI_{i,j} = HCI_{i,0} \ x \ \frac{D_{i,0}^{8/3}}{D_{i,j}^{8/3}} \ x \ \frac{n_{i,j}}{n_{o,j}} \tag{5.7}$$

Structural Condition. To obtain the Structural Condition Indices (SCI_{ij}), a Structural Inspection Program is needed. The Program is applied to plan the inspection of the system using different available methods (i.e., man-entry, CCTV, smoke, thermography). From the inspection program the pipes can be given indices concerning their structural condition. The indices rank from 0 (perfect condition), to 10 (structural failure). A modified methodology based on these suggested by the Department of Housing and Urban Development (1984) and the Water Research Centre (1983) is used for the purpose of ranking the pipes in the system. These indices are adjusted according to the expected conditions the pipe would reach once rehabilitated with a particular method.

For the purpose of comparison, the following grades are used: 0 = perfect condition and 10 = structural failure using the following guidelines:

Grade	Implication
10	Collapsed or collapse imminent
8	Collapse likely in foreseeable future
6	Collapse unlikely in near future but further deterioration likely
4	Minimal short term collapse risk but potential further deterioration
2	Open joints due to sagging

The Structural Condition Indices, *SCI,* are calculated as follows. First an Importance Weighing Factor is obtained. For concrete and vitrified clay pipes the factors are:

Factor	Structural Problem
20	Structural cracking and deflections
15	Corrosion
10	Structural cracking without deflection
5	Open joints due to sagging

This Importance Weighing Factor is then multiplied by an Extent of Condition Factor selected as follows:

Factor	Extent
1	Slight
2	Moderate
3	Severe

The sum of the products of the Importance Weighting Factor times the Extent Condition Factor yields the Structural Condition Factor, *SCF* for each reach. The Structural Condition Index, *SCI*, is then calculated from the relation

$$\begin{aligned} SCI &= SCF/16 && SCF \leq 40 \\ SCI &= 2.5 + 3/16\,(SCF - 40) && 40 \leq SCF \leq 80 \\ SCI &= 10 && SCF \geq 80 \end{aligned} \tag{5.8}$$

SCI_o is the Structural Condition Index for the original condition and SCI_{ij} corresponds to the rehabilitated condition in reach *i* with technology *j*.

Failure Consequences Impact. Following a modified methodology of the Department of Housing and Urban Development, Consequences of Failure Impact Indices are given to each pipe ($CFII_i$). These indices intend to show the impact a broken pipe segment can cause, for example. These indices are used to adjust the Structural Condition Indices in the optimisation process. Hydraulic inadequacies are shown through the Hydraulic Condition Indices which are obtained from a simulation using SWMM.

Three kinds of consequences of failure are considered: the sewer service disruption, the traffic and local access disruption, and the disruption of other utilities. These disruptions are quantified by factors that are calculated as follows. The Service Disruption Factor, SDF_i, is calculated as

$$SDF_i = 10\ TA_i / TTA \tag{5.9}$$

where TA_i is the tributary area upstream of segment *i* and *TTA* is the total tributary area affected. The Traffic Disruption Factor, TDF_i, is calculated from

$$TDF_i = 10\ TR_i / [max(TR_i)] \tag{5.10}$$

where Tr_i is the traffic in artery *i*.

The other utilities disruption factor is calculated as the product $DUF_i \times CSF_i$ where DUF_i is a rating (from 0 to 10) and CSF_i is a Cover / Size Factor. This latter factor is calculated as

$$
\begin{array}{ll}
CSF_i = 0.5 & Depth_I \times Diam_I \leq 115 \\
CSF_i = 0.5 + (Depth_i \times Diam_i - 115)/350 & 115 \leq Depth_I \times Diam_I \leq 465 \\
CSF_i = 1.5 & Depth_I \times Diam_I \geq 465
\end{array} \tag{5.11}
$$

where *Depth* is in metres and *Diam* is in centimetres. The Consequence of Failure Impact Index, is then calculated as

$$CFII_i = weighed\ average\ of\ SDF,\ TDF,\ DUF \times CSF \tag{5.12}$$

Annual Costs. The non-rehabilitated reaches and the same reaches rehabilitated with a particular method have different Annual Maintenance and Claims Unit Costs ($AMCUC_{ij}$). These costs are a function of the diameter of the pipe.

The lengths have specific investment costs assigned when different methods are applied. To account for the widely varied life spans of the rehabilitation technologies, they are all reduced to annual costs. The Annual Construction Unit Costs ($ACUC_{ij}$) are a function of the diameters, and linear models are used as a first approach. Length Cost Factors (LCF_{ij}) are used to correct the unit costs for shorter reaches as, in the USA, unit costs are usually given for the length of 500 ft (150 m).

The unit construction cost, UCC_{ij}, corresponding to rehabilitation technology j applied to pipe segment i is approximated by a second order polynomial

$$UCC_{ij} = a_j\, Diam_i^2 + b_j\, Diam_i + c_j \tag{5.13}$$

or more simply by a first order equation

$$UCC_{ij} = a_j Diam_i + b_j \tag{5.14}$$

This is corrected by multiplying by a length correction factor, LCF_{ij}, to take care of the higher costs of short lengths (Dept. of Housing and Urban Development, 1984)

$$LCF_{ij} = \frac{A_j}{L_i^{B_j}} + C_j \tag{5.15}$$

The annual construction unit costs, AUC_{Cij}, are obtained from

$$AUCC_{ij} = \left[\frac{r(1+r)^{n_j}}{(1+r)^{n_j} - 1}\right] UCC_{ij} \tag{5.16}$$

where r is the interest rate, n_j is the expected life of the rehabilitation using technology j. The quantity in brackets is the annual to present ratio, *A/P*. This ratio can be introduced in (5.13) to obtain,

$$ACUC_{ij} = \left(\frac{A}{P}\right)_j a_j D_i^2 + \left(\frac{A}{P}\right)_j b_j D_i + \left(\frac{A}{P}\right)_j c_j \tag{5.17}$$

which is of the form

$$AUCC_{ioj} = a_j' D_i^2 + b_i' D_i + c \tag{5.18}$$

If (5.14) is used instead of (5.13), then one obtains an equation of the form

$$AUCC_{ij} = a' D_i + b' \tag{5.19}$$

Disruption Levels. Each rehabilitation methodology has an associated Disruption Level Index (DLI_j), ranked from 0 to 10, from less to more disruptive. A maximum rank of 10 is given to excavation and replacement and a 0 to no rehabilitation. This index measures the overall disruption to urban environment caused by the rehabilitation technique used. Expected levels of disruption are known from the industry for each rehabilitation method. This index gives a weight to the fact that less disruptive methods should be preferred.

The "as is" annual maintenance and claims unit cost, *AMCUC,* is assumed to be related to the original structural condition index, SCI_{io} and the original diameter, D_{io}, of the segment:

$$AMCUC_{io} = (A \bullet SCI_{io} + B)(a\,D_{io} + b)\overline{AMC}\,\overline{UC} \tag{5.20}$$

where the over score represents an average value. The annual maintenance and claims unit cost, $AMCUC_{ij}$, for rehabilitated pipe i using technology j is

$$AMCUC_{ij} = \frac{1}{C_j}\overline{AMC}\,\overline{UC}(aD_{ij} + b) \tag{5.21}$$

Computer Implementation. The overall structure of the MARESS model is shown schematically in **Figure 5**. The implementation is done using EXCEL, a well-known spreadsheet employing its workbook capabilities (to link the different spreadsheets), its database capabilities (to organise the information on the sewer network and the rehabilitation technologies), and its add-in feature (to run a linear/integer programming solver interactively). Finally, the macros capability is used especially to do the final search for the non-dominated points. Only SWMM cannot be run from EXCEL and has to be run outside of the spreadsheet environment. The resultant data have to be entered by intermediate files into the EXCEL workbook to be processed later.

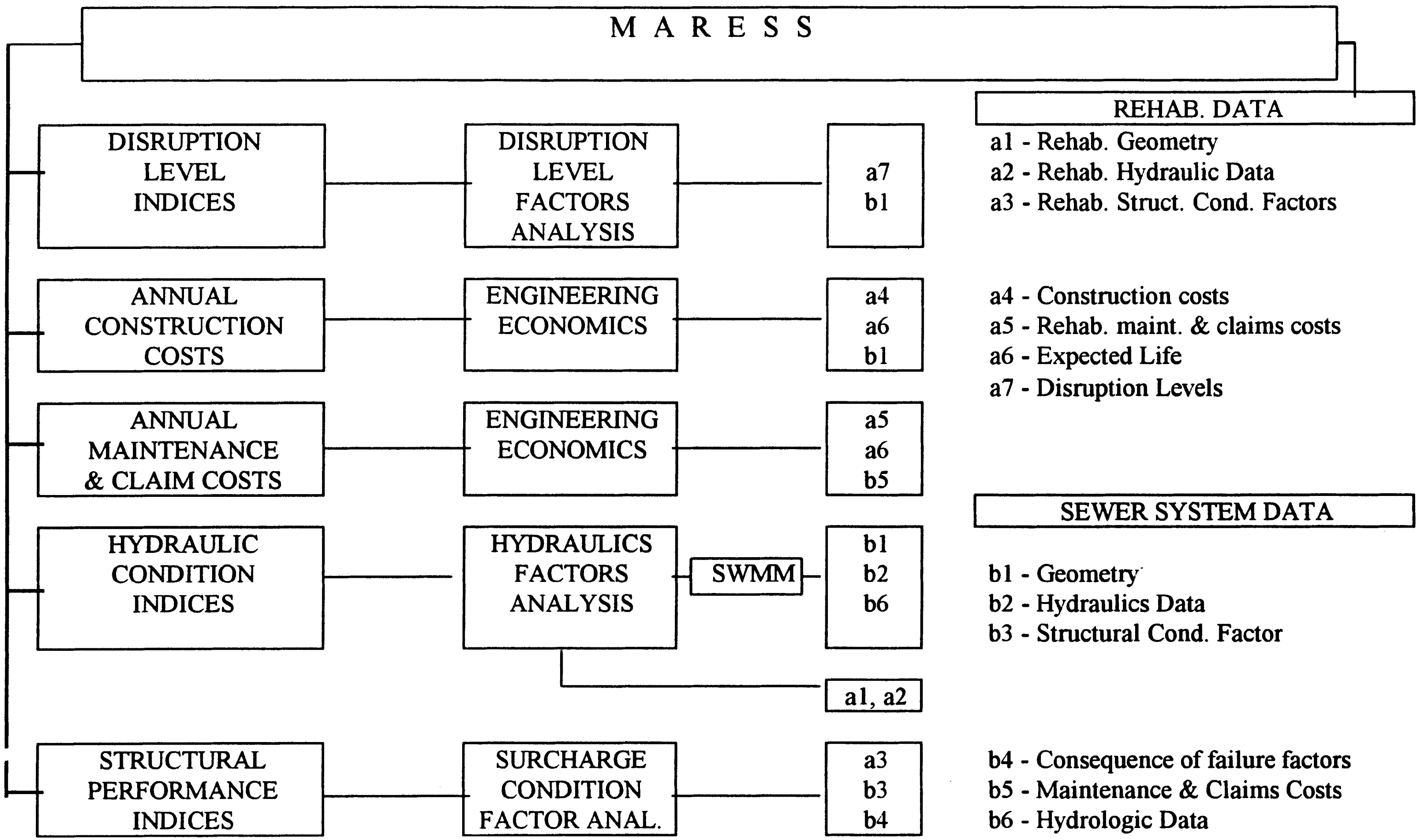

Figure 5. Structure of MARESS (Multi-Attribute REhabilitation of Sewer Systems). Adapted from Reyna (1993)

Seven different spreadsheets and two relevant macros are used in a linked form for the implementation as shown in **Figure 6**. The first spreadsheet (**Figure 7**) is the Network Database. This is needed principally by the SWMM program (both RUNOFF and EXTRAN modules). Data included in this file are conduit names, end manholes, ground and invert elevations, lengths, slopes, materials, roughness coefficients, I/I (infiltration/inflow), and subbasin characteristics such as: width, area, percent impervious, slope, roughness, etc.

The second spreadsheet, (**Figure 8**), making use of the linking capabilities of EXCEL, creates the actual SWMM input file (some data are entered directly into it). Any change done to the network database is reflected directly in the input file. This facilitates changes and analyses of different scenarios. It also avoids possible sources of errors.

The third and fourth spreadsheets (**Figure 7**) are obtained from the output of SWMM. One contains the Maximum to Design Flow ratios, and the other, the Surcharge and Flood Lengths.

The fifth spreadsheet is the Rehabilitation Technologies Database (**Figure 9**). From this spreadsheet, the mathematical program obtains: the applicability of the technologies (Yes/No matrix for the different pipes' types and structural conditions for which the different technologies are applicable), the technologies' disruption levels, the expected rehabilitated structural condition indices, the necessary coefficients to compute the rehabilitated hydraulic indices, and the annual rehabilitation construction costs and the maintenance and claims costs for each method.

The sixth and seventh spreadsheets perform computations needed to generate the multiple attribute mathematical model (**Figures 10, 11**). The final model ready for the integer programming solver is in spreadsheet seven (Mathematical Program). Spreadsheet six (Non-Rehabilitated Factors), contains parameters needed for the optimisation: the "as-is" structural condition, the hydraulic factors (Maximum/Design, Surcharge Length, Flood Length), the hydraulic condition indices, the factors needed to compute the coefficients of impact, the annual maintenance costs' parameters, and the length construction cost factors. This file obtains some of its needed input values from the Network Database; some come from the output of SWMM; some are input directly into it; and the rest are computed from the data entered, or link-entered.

The last spreadsheet of the workbook is the actual Mathematical Program in the form that must be solved by What's Best!, a Lindo Systems Inc. (1992) solver. (**Figure 11**). It contains the conduit numbers, the binary decision variables (x_{ij}), the coefficients of each one of the optimisation objectives (D_{ij}, S_{ij}, H_{ij}, MC_{ij} defined below), the coefficients of the construction costs' constraint (C_{ij}), the constraints, and the four optimisation functions (disruption level, structural performance, hydraulics performance and annual maintenance and claims costs) for each conduit.

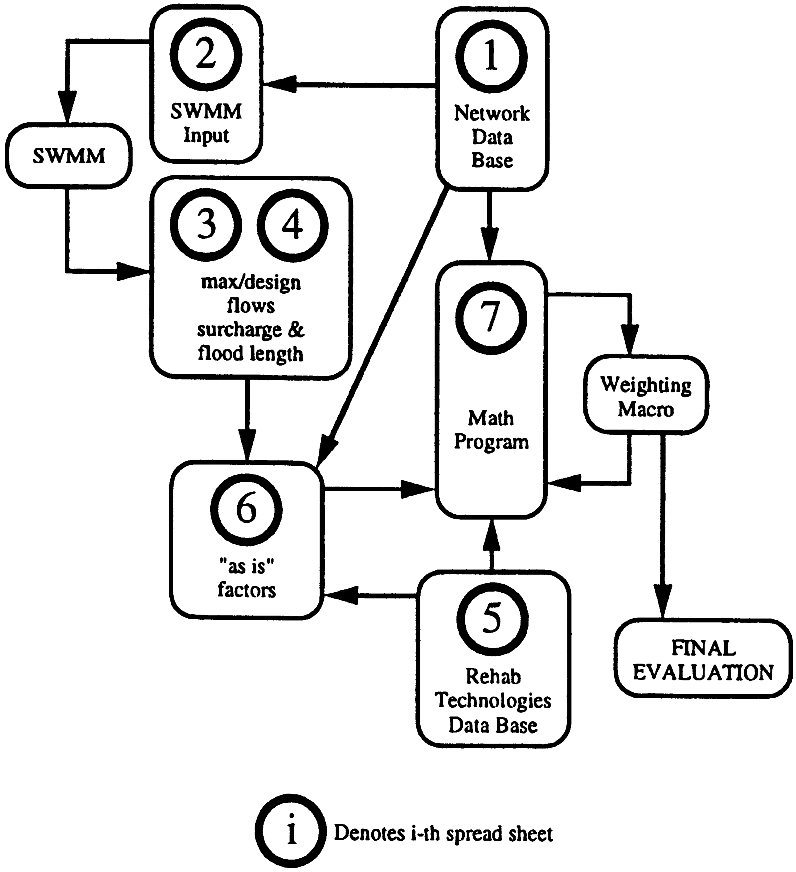

Figure 6. Network for MARESS Implementation.
Source Reyna *et al.* (1994), Reproduced with permission

First Spreadsheet Network Data Base

(1) Junction	(2) Upstream Manhole	(3) Downstream Manhole	(4)Diameter	(5) Material	(6) Up Invert Elev.	(7) Down Invert Elev
(8) Ground Up-stream Elevation	(9) Ground Downstream El.	(10) Manning's n	(11) Length	(12) Upstr. Depth Offset *	(13) Downstr. Depth Offset *	(14) Inflow/ Infiltration (I/I)
(15) Slope	(16) Tributary Area	(17) % Imperviousness	(18) Subsurface Slope	(19) Width	(20) I/I +Dry Whether Flow	(21) Structural Condition

* for EXTRAN

Third Spreadsheet SWMM output Maximum to Design Flows

(1) Conduit	(2) Design Flow	(3) Maximum Flow	(4) Max / Des Flow

Fourth Spreadsheet SWMM output Surcharge and Flood Lengths

(1) Junction	(2) Surcharge Length (min)	Flooding Length (min)

Figure 7. First, Third and Fourth Spreadsheets Column headings.
Adapted from Reyna (1993)

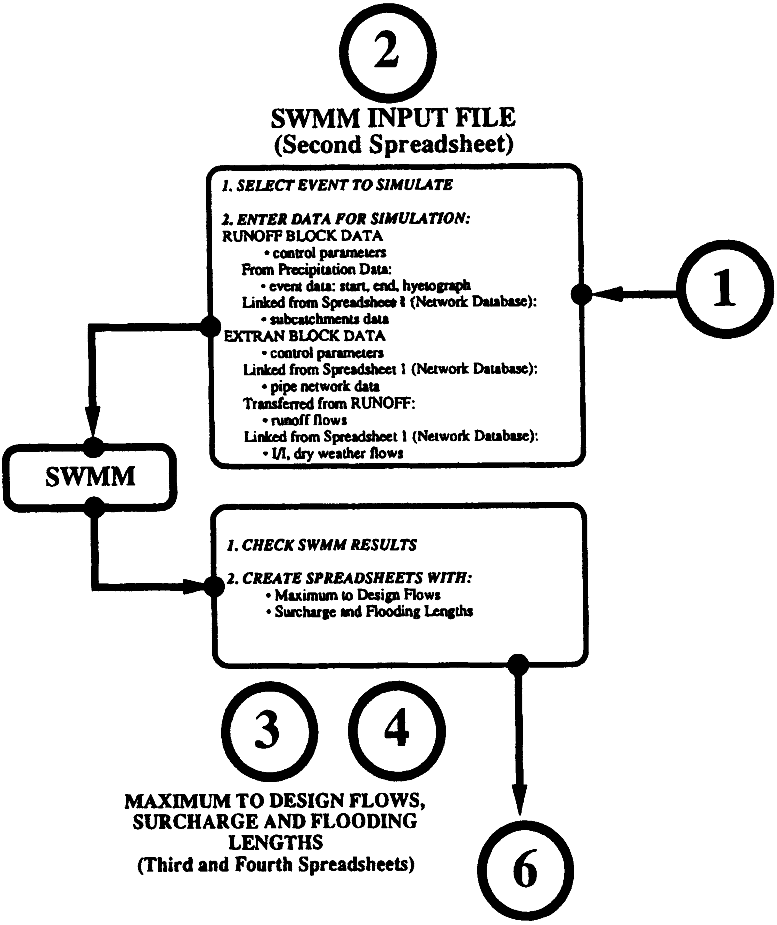

Figure 8. SWMM Input File (Second Spreadsheet) and Maximum to design flows, Surcharges and Flooding Lengths. (Third and fourth Spreadsheets). Source: Reyna *et al.* (1994). Reproduced with permission

Fifth Spreadsheet Rehabilitation Data Base

METHODS	PIPES RCP	VCP	BSB	BRK	PVC	Struct. Applic	Disrupt. Level	Struct. New	Hydr New
Excavation & Replacement	Y	Y	Y	Y	Y	10	10	0	0.00130
Polyethylene Pipe Insertion	Y	Y	Y	Y	Y	8	7	0.5	0.00146
Inversion Lining	Y	Y	Y	Y	Y	9	3	0.5	0.00138
Grouting	Y	Y	N	N	Y	4	2	1	0.00140
Cement Mortar Lining	Y	N	N	N	N	5	5	0.8	0.00156

RCP: reinforced concrete pipe, VCP: vitrified clay pipe, BRK: brick, PVC: polyvinyl chloride

Fifth Spreadsheet (Continued) Costs

METHODS	Expected Life	Rehab. Cost		Annual Rehab Cost		Annual Maintenance Cost	
		a $	b $	a' $	b' $	a $	b $
Excavation & Replacement	70	4.10	125.00	0.25	7.63	0.002	0.019
Polyethylene Pipe Insertion	60	2.65	73.00	0.16	4.52	0.003	0.024
Inversion Lining	50	4.62	94.00	0.29	5.96	0.003	0.026
Grouting	8	0.39	5.82	0.06	0.94	0.006	0.048
Cement Mortar Lining	25	0.42	18.00	0.03	1.41	0.005	0.040

Note: The Primes (') indicate that the coefficients have been multiplied by the Annual to Present ratio (A/P). Interest: 6%.

$$ACUCij = a'_j (D_i - 15) + b'_j$$

Figure 9. Fifth Spread Sheet as used in the case study.
Source: Reyna (1993)

Sixth Spreadsheet Non-Rehab Factors

(1) Conduit	(2) Initial Structural Condition Index	(3) Max / Design Flows	(4) Surcharge Length
(5) Flood Length	(6) Hydraulic Condition Index	(7) Initial Hydr. Cond. Index	(8) Tributary Area
(9)Traffic	(10) Other Utilities	(11) Depth	(12) Depth x Diameter
(13) Cover / Size Ratio	(14) Coefficient of Impact	Annual maintenance Cost (15) a, (16) b	(17) Annual maint. and claim cost
(18) Length Correction Factor			

Figure 10. Sixth Spreadsheet Column Headings
Adapted from Reyna (1993)

The coefficients of each of the optimisation objectives are defined as all the factors inside the summation signs, but an exception is made for the binary decision variables. In this way, D_{ij} is defined as $DLI_{ij} * L_i$, S_{ij} is defined as $SCI_{ij} * CFII_{ij} * L_i$, etc. D_{ij} represents the coefficients of the Disruption Level optimisation function, S_{ij} the Structural Performance, H_{ij} the Hydraulics Performance, and MC_{ij} the Maintenance and Claims Costs. C_{ij}, the Construction Costs constraint coefficients are defined in a similar manner.

The coefficients of the optimisation objectives and the cost constraint in the Mathematical Program are later scaled to have a well-formulated problem. Coefficients that corresponded to non-feasible solutions due to other constraints are made equal to "infinity". The Mathematical Program spreadsheet also includes the weights (W_k, $k=1-4$) selected and the total value of the functions (Z_k, $k=1-4$) to be optimised, and the value of the weighted optimum function (Z).

To solve the mathematical program, a weighting method is used. The technique consists in varying the weights from a minimum to a maximum (in the range 0-1) with selected steps in searching for solutions contained in the non-dominated set. Each selection of weights gives a particular non-dominated solution once it is solved with an integer-programming solver. The method gives as a result a subset of the complete set of non-inferior points (**Figure 12**).

The weighting method was implemented with macros that modify the weights in a triple loop (the fourth weight is not independent). The macros call interactively the integer program solver and write the solution into a file for that purpose.

Seventh Spreadsheet Mathematical Program

Column Headings

1. Conduit

2. to 7. Binary Decision Variables X_{i0} to X_{i5}, Initialised to Zero

8. to 35. Coefficients of Optimisation Objectives

$D_{ij} = DLI_j * L_i$

Overall Disruption Level

$H_{ij} = HCI_{ij}$

Hydraulic Performance

$S_{ij} = SCI_{ij} * CFII_i * L_i$

Structural Performance

$MC_{ij} = AMCUC_{ij} * L_i$

Maint. & Claim Costs

$C_{ij} = ACUC_{ij} * LCF_{ij}$

Construction Cost

36.& 37. Constraints

CONST. 1. Max Ann Const. Cost

CONST. 2. $-1 + \Sigma_j x_{ij}$

Adding Cells: $Max - \sum_i \sum_j C_{ij} * x_{ij} \geq 0$

38 to 41. Optimisation functions

DISRUP $= \sum_j D_{ij} x_{ij}$

STRUCT $= \sum_j S_{ij} x_{ij}$

HYDRAU $= \sum_j H_{ij} x_{ij}$

MAINCO $= \sum_j MC_{ij} x_{ij}$

Adding Cells:

$$Z_1 = ZD = \sum_i \sum_j D_{ij} * x_{ij}$$

$$Z_2 = ZS = \sum_i \sum_j S_{ij} * x_{ij}$$

$$Z_3 = ZH = \sum_i \sum_j H_{ij} * x_{ij}$$

$$Z_4 = ZMC = \sum_i \sum_j MC_{ij} * x_{ij}$$

Weighted Optimisation Function: $\sum Z_k W_k \quad ,k = 1 \; to \; 4$

Figure 11. Column Headings of Seventh Spreadsheet and Weighted Optimisation. Adapted from Reyna (1993)

Case Study. To show the application of MARESS, a full case study was done. A combined sewer network was selected in the City of Indianapolis, Indiana. The needed data were provided by the Department of Public Works, City of Indianapolis. The tributary area serves the Pogues Run Interceptor. Excessive flows are discharged into Pogues Run stream waters in CSO (Combined Sewer Overflow) 101. The network drains a fully developed area of 369 acres, with family dwellings, a shopping mall, and three schools. The sewer network has a tree structure, with the total length of the pipes in the system of 59,000 ft. Pipe diameters range from 8 to 60 in. Conduits in the system are of the following types: Reinforced Concrete Pipe (RCP), Vitrified Clay Pipe (VCP), Vitrified Sewer Brick (VSB), Brick (BRK), and PVC (Polyvinyl Chloride).

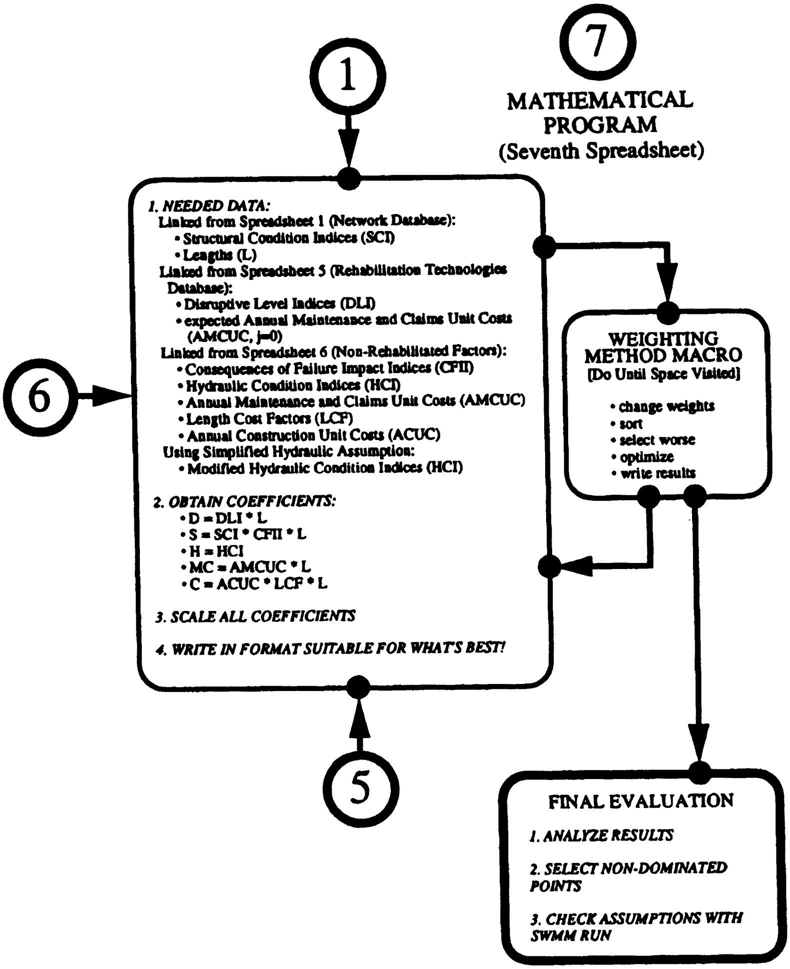

Figure 12. Mathematical Program (Seventh Spreadsheet),
Weighting Method Macro and Final Evaluation.
Source, Reyna *et al.* (1994), Reproduced with permission.

The hydraulic indices were calculated from precipitation data obtained with a raingauge close to the network, field watershed information and the results of event modelling with SWMM. Structural indices were computed from information that the City of Indianapolis has on the segment conditions. The City of Indianapolis has a GIS (Geographical Information System) database in which pipe materials, conditions and maintenance status are stored.

The following rehabilitation technologies were considered for the present case study: Excavation & Replacement, Polyethylene Pipe Insertion, Inversion Lining, Grouting, and Cement Mortar Lining. These technologies were selected considering their applicability to the network condition and pipe types. For this purpose, a preliminary analysis following the "Dallas Algorithm" (Stalnaker, 1992), was done. This algorithm is a rational and organised way to select technologies appropriate to the network segments. Furthermore, the methods selected are the ones currently being used by the Department of Public Works, City of Indianapolis.

A sample of some lines of the solution file obtained for the case analysed in Indianapolis are shown as an example in Table 2. From left to right the columns list the weights assigned to disruptions, structural and hydraulic conditions and to maintenance and claims, then the annualised cost of rehabilitation and finally the rehabilitation methods selected by the model for several pipe segments. The table is an abbreviated form of the solution file. In the complete solution there are as many "rehabilitation method/pipe segment" columns as there are segments. Only eight are shown in Table 1. There are as many lines as there are combination of weights. The annualised costs of rehabilitation are in dollars. In most cases, those costs are close to $70,000, except when it is desired to minimise disruptions, in which case the solution is "do nothing" and the cost is nil. The disruption objective ZD, the structural objective ZS, the hydraulic objective ZH and the annual maintenance costs are calculated for the whole network using equations 1, 2, 3 and 4, respectively. Z is a combination of objective functions according to the respective weights and is calculated to obtain the extreme points in the solution space. Considering the first two lines of the "Rehab. Method/Pipe Segment" columns, it is seen that segment 3604 is rehabilitated by method 2, segment 3611 by method 3 and 3603 is not rehabilitated. This shows how one can trade the different objectives from one solution to another. That is, when absolute importance is given to one objective (weight equal to 1), some importance is lost with respect to others. It can be inferred how much it costs to give too much importance to disruption levels, how one can trade between costs of maintenance, structural condition of the network, etc.

This file contains the final solution of the model. It is the information that is used by the decision maker. After pondering the different alternatives that correspond to giving more or less importance to the different objectives, the decision maker can make a rational decision on the solution to select. From this table, it can be seen how much influence the selection of one or another set of weights has on the network rehabilitation process. A weight of one given to the disruption levels (an extreme point) means simply that no rehabilitation is the solution. A weight of one to structural performance (another extreme point) will make the program tend to select excavation and replacement as the solution. If equal weight is given to structural performance and disruption, inversion lining will be the solution most frequently adopted. An actual decision will give weight to all four objectives, and the solution adopted will be a compromise between all these.

Conclusions. This section presents a computerised holistic approach to sewer systems rehabilitation. A mathematical model is shown that treats the sewer system and its rehabilitation alternatives jointly. It helps the decision maker to choose an optimal set of segments of the sewer system to be rehabilitated first according to the level of investment selected. The model shows the set of extreme points that are equivalent to optimal points for multiobjective programs. It is helpful to have a set of options from which to choose, as the problem is complex. It provides a rational analysis.

The constructed model encourages the use of alternative options to excavation and replacement for the lines' rehabilitation. It includes in its formulation (among other factors) the range of applicability of the different construction technologies considered and a database of cost factors for each method. For more detail see Reyna (1993). This makes the comparison of costs an integrated part of the model. By incorporating a database of the technologies' characteristics, the method simplifies the consideration of alternative technologies by the designer. MARESS acknowledges the need for accurate hydraulic observations and modelling. If a system is hydraulically inadequate, with sewer backups and floodings, it has been clearly seen that approximate modelling of the hydraulics is not enough. The use of hydraulics software that solves the full dynamic flow equations (EXTRAN or equivalent) is needed and it should be incorporated in the model. This model represents an alternative to the ranking methods, which by dealing first with the pipe that has been ranked worst, according to some combined index, cannot perform any value engineering analysis. It has been seen that, when the overall performance of the network is to be maximised with the available funds, often the solution is not to spend most of the funds on an expensive "cadillac" type method applied to a few segments, but to use more cost effective options on a wider selection of lines. Another problem with ranking methods is that, they either rank with respect to several conditions separately, or they combine the indices into one with fixed weights. This means that the decision maker cannot consider options according to different preferences of the objectives that need to be optimised. MARESS gives a table of "optimal" solutions for different sets of weights, and lets the decision maker select final relative weights. No judgement is involved in the selection of the weights and indices. They are used as a tool to obtain extreme points in the solution space.

The data required for the model are of the type that the public works offices usually manage. Because of the computer implementation of the model, once the data are entered, the "optimal" solutions are easily obtained. The process of implementation of the model is useful in itself by making the decision maker look at the network as what it is, a collection of mutually interacting parts. Secondary results of MARESS are a fully working hydraulics model, a system of structural ranking, a methodology to compute failure impacts, and suggested ways to analyse life-cycle costs. All these can be used for other purposes, too.

5.2. CHANCE CONSTRAINT MODEL

This section is based on Jacobs and Medina (1994). In chance constrained models one or more of the physical constraints are expressed as a probability statement. In this model it is desired to minimise the probability of failure of the system, namely the probability that the discharge exceeds the maximum permissible discharge. This discharge is obtained by kinematic routing and therefore, in turn, depends on the rainfall intensity and infiltration rate which are regarded as probabilistic. The chance constrained model is set up as follows:

TABLE 2. Sample Solution File For Case Study (Adapted from Reyna, 1993)

WEIGHTS				OBJECTIVE FUNCTIONS				N. Cost	optim	REHAB METHOD / PIPE SEGMENT							
WD	WS	WH	WMC	ZD	ZS	ZH	ZMC	C	Z	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0.25	0.25	0.25	0.25	26787	158206	43839	5996	69878	58707	2 3604	3 3611	0 3603	0 3679	3 3610	2 3605	3 3678	5 3601
0	0	0	1	45597	139743	45648	3011	69920	3011	2 3604	3 3611	2 3603	1 3679	2 3613	2 3605	3 3678	4 3601
0	0	0.5	0.5	46680	121260	43307	3369	70045	23338	2 3604	3 3611	2 3603	1 3679	2 3610	2 3605	3 3678	5 3601
0	0	1	0	52214	97243	40609	5088	70045	40609	2 3604	1 3611	2 3603	1 3679	2 3610	0 3605	1 3678	5 3601
0	0.5	0	0.5	48065	126932	46041	3054	70129	64993	2 3604	3 3611	2 3603	1 3679	2 3610	2 3605	3 3678	5 3601
0	0.5	0.5	0	57150	94688	40788	5399	70045	67738	2 3604	1 3611	0 3603	1 3679	2 3610	0 3605	1 3678	5 3601
0	1	0	0	57120	94433	40834	5397	69878	94433	2 3604	1 3611	0 3603	1 3679	2 3610	0 3605	1 3678	5 3601
0.5	0	0	0.5	13163	295920	55219	7768	68873	10466	3 3604	3 3611	3 3603	0 3679	3 3610	3 3605	3 3678	3 3601
0.5	0	0.5	0	13635	295614	48887	12366	45762	31411	3 3604	3 3611	0 3603	0 3679	3 3610	0 3605	3 3678	0 3601
0.5	0.5	0	0	19142	198472	45167	8108	69459	108807	3 3604	3 3611	3 3603	0 3679	3 3610	0 3605	3 3678	0 3601
1	0	0	0	0	545033	66088	20615	0	0	0 3604	0 3611	0 3603	0 3679	0 3610	0 3605	0 3678	0 3601

COLUMN HEADINGS:

WEIGHTS: WD: disruption WS: structural WH: hydraulic WMC: maintenance and claims

OBJECTIVE FUNCTIONS: ZD: disruptions ZS: structural ZH: hydraulic ZMC: maintenance and claims

C: Annualised construction costs Z: value of weighted optimal

REHAB METHOD / PIPE SEGMENT: Four digit numbers are segment numbers. Single digit number above segment number are rehabilitation method selected. 0: no rehabilitation

1: excavation & replacement 2: polyethylene pipe insertion 3: inversion lining 4: grouting 5: cement mortar

$$minimise COST = \sum_i \sum_j C_{ij} X_{ij} \tag{5.22}$$

$$minimise \quad p_f = 1 - \left[\prod_i (1 - p_{fi}) \right] \tag{5.23}$$

subject to:

$$p[Q_i \leq Q_{\max}(X_{ij}, \forall_i)]_i \geq 1 - p_{fi} \qquad \forall_i \tag{5.24}$$

$$Q_i, p_{fi} \geq 0 \qquad \forall_i \tag{5.25}$$

$$X_{ij} \in 0,1 \qquad \forall_{ij} \tag{5.26}$$

where X_{ij}, the binary decision variable for implementing a rehabilitation alternative, is a function of the pipe capacity, Q_{max}. The symbol C_{ij} represents the cost of rehabilitation of pipe segment i with rehabilitation alternative j. The total probability of system failure and the individual probability of failure of segment i are designated by P_f and p_{fi}, respectively. Q_i is the discharge in segment i, which is determined from the kinematic wave routing.

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Figure 13. Professors Delleur (left) and Szetela (right) at a closed circuit TV inspection of a sewer at Harrachov, Czech Republic, during the NATO ASI.

SAFE HYDROINFORMATICS

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1. Introduction

Hydroinformatics is a multidisciplinary subject that addresses the management of the aquatic environment using information technology. The task of management is complicated by the uncertainty surrounding many of the scientific processes that are inherent to the aquatic environment and by the number of stakeholders concerned to utilise and preserve the environment as a sustainable resource. Any hydroinformatics system will reflect something of the complexity of the processes and demands of the stakeholders. An important function of a hydroinformatics system is its ability to support its users in helping them to achieve their objectives, namely in managing a particular aspect of the aquatic environment. Inevitably such management involves decision making on the basis of a wide range of information of different types and from various sources. An inexperienced user can easily misunderstand historic data and textual information, mishandle the building, development and application of models, and misinterpret their results. Consequently the opportunities for incorrect conclusions and poor decision making are many. There is a need for better tools that can generate a *safe environment* in which users can be expected to achieve more reliable decisions. This can be effected by supporting the user in his or her decision making. In particular, a decision support system can provide the *framework* for accessing relevant information, marshalling and analysing the data, and making informed decisions.

Besides the need to assimilate data from different sources, the uncertain and incomplete nature of the raw data is such that there are always going to be important questions about the reliability and safety of the decision making based on the data. The problem is exacerbated by the fact that the analysis tools, algorithms and associated modelling systems also contain particular uncertainties, approximations and assumptions that can in extreme cases invalidate the results of their application. Modelling systems, in particular, are dependent for their reliability on correct and proper use in terms of model building, development and application. With the heavy dependence of a hydroinformatics system on data and models, the reliability and safe operation of the system is a crucial issue for hydroinformatics. The support for the user

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is therefore vital. Considerable assistance can be given by highly user-friendly, intuitive interfaces operating with natural language and access to extensive context sensitive help. Such features are important. But whereas these are now widely accepted as being necessary to make the software easy to use, there are still many difficulties faced by inexperienced users such that safe use of a hydroinformatics system may be prejudiced.

Inexperienced users of simulation modelling software may assume naively that the software system they are using is error free. Increasingly, however, more informed end-users have begun to ask questions about the quality of the software, its reliability and testing. Such questions are directly relevant to design failure or safety of the physical asset which has been designed or modified using the software system. This aspect of safety is being explored by software engineers and end-users working together in two major disciplines: *safety-related systems* and *safety-critical systems.* A safety-related system is one whose malfunction, either directly or indirectly, has the potential to lead to safety being compromised, whereas a safety-critical system is one in which any failure or design error has the potential to lead to loss of life.

The literature on safety-related systems and safety-critical systems focuses, by and large, on design and operation in well-publicised, potentially safety-compromising areas such as nuclear engineering, air and space craft design and operation, and health and safety at work. The umbrella term used for discussing safety issues related to IT systems is *safe information technology*. Interestingly, there is little existing literature on how to design and operate water-carrying networks *safely*. These networks on the whole may be regarded as *safety related systems*. However, given the growing public concern about threats to public health and safety from badly designed or poorly operated water-carrying networks, there are instances where some, if not all, of these networks may be classified as safety critical systems.

What we seek is a principled framework within which we can discuss how to synthesise systematically the union of human and artificial agents directed to the design of water-carrying networks and to urban drainage networks in particular. The synthesis of safe information technology methods and practices within such a framework leads to the establishment of *safe hydroinformatics*. By design we mean a knowledge based task that involves almost simultaneous access to a variety of knowledge sources, including analytical, formal, experimental and experience-based sources, and also involves a deft mixture of sophisticated computation, qualitative reasoning and extensive use of rules of thumb (or *heuristics*). This simultaneous access to a variety of knowledge sources is crucial in the design of networks that carry water, gas, oil or electricity.

Apart from major and typically rare catastrophes, water networks can be designed and operated safely and efficiently. This is partly due to the fact that networks supplying fresh water and draining away waste water were amongst the first significant engineering artefacts created by society. With access to experiential knowledge and knowledge based on local idiosyncratic conditions pre-Newtonian water engineers were able to drain water effectively from agricultural and urban land, although not as efficiently as their 20th century counterparts. But if there are so few catastrophes, how is it that the designers of water carrying networks, relying largely on heuristics, qualitative reasoning and local knowledge, produce designs that fail so infrequently? One possible answer is that the identification of the physics of fluid flow, the mathematical description of the physics, and the implementation of the description in a software simulation system, is but a small, even if vital, part of the entire decision-making process. In this sense the use of physical theories, mathematical formalisms and computer programs have to be situated in the broader context of such factors as the location and state of the drainage network, intuitive insight into system performance, the characteristics of the conurbation, the history (if any) of the network, its performance in terms of flooding and pollution, and existing management priorities and public health targets. This comprises *knowledge* of the domain. Such knowledge is the major factor in the production of safe designs. In other words, what the designer *does* with his knowledge and available tools is just as, if not more, important as the safety or reliability of the tools themselves. Given, therefore, that knowledge of the domain is crucial, there are well known ways of collecting such knowledge systematically from experts and representing it in *knowledge bases* together with relevant documents such that the knowledge can be retrieved by relatively inexperienced engineers to support them in solving specific problems.

In this Chapter we focus on how a computer can mediate in providing decision support to users of complex modelling systems. In particular we explore how to assure safety during various phases of the rehabilitation of a drainage network. We elaborate the various notions that are central to hydroinformatics, namely those of integrating IT methods, tools and techniques in a common framework to address problems related to the aquatic environment. Our emphasis is on the safe design of drainage networks through the use of software engineering techniques, and through the provision of interfaces. Such interfaces are needed to simulation engines used in the design, to expert systems containing knowledge of drainage experts, and to 'digital' libraries of relevant documents. The elaboration is conducted through the description of a recently completed SAFE Design of Networks using Information Systems (SAFE-D*IS*) project.

2. Failures, hazards and risks in sewerage network rehabilitation

A typical engineering application of sewerage network models in the UK is to rehabilitation planning. In 1984 the Water Research Centre published the first version of its Sewerage Rehabilitation Manual (SRM) that incorporates an engineering procedure to facilitate the development of 'drainage area plans'; see Water Research Centre (1984, 1986, 1995). Such plans involve the presentation and analysis of a number of options, some of which require a range of engineering judgements, some involve public health consideration, and others include cost-benefit analysis. Drainage area plans involve the recognition of priorities for rehabilitation at a regional level. These priorities are incorporated in the asset management plans of the 10 UK regional Water Service companies as required by the Office of Water, which is the UK government regulatory body that sets investment targets for each of the companies (HMSO 1991).

Phase 1

i. Initial Planning *ii. Check System Records*

Phase 2

Investigations

a. Structural *b. Environmental* *c. Hydraulic*

Phase 3

Developing an area wide plan (priorities, solutions etc.)

Phase 4

Implementing the Plan (work plans, cost-benefit etc.)

Figure 1. The Sewerage Rehabilitation Manual (SRM) method established by the UK Water Research Centre in consultation with the water industry.

Although not explicitly mentioned, the prevention of hazards and the anticipation of network failures, are amongst the principal considerations of the SRM. Consider, for instance, the key term 'critical sewer' that is used very frequently throughout much of the documentation. 'Criticality' is defined in terms of 'sewers with most significant consequences in the event of structural failure'. A related term is 'core area', which is that part of a sewer network containing the critical sewers and other sewers where hydraulic problems are likely to be most severe and require detailed definition within a flow simulation model. 'Acts of God', in their legal sense, also cause problems, so rehabilitation experts talk about 'catastrophic rainfall event', which is an event of return frequency far in excess of any sewerage design performance criteria, such as a 1 in 20 year storm. Sewer rehabilitation involves monetary expenditure and 'social costs'. Examples of the latter are 'unclaimed business losses due to road closures, and the cost of extended journey times due to traffic diversions'.

Each of the four main phases of the SRM involves a number of considerations about the environmental impact of a rehabilitation scheme. Such considerations are elaborated in terms of 'systems failure', 'hazard prevention', and so on. Tables 1a and

1b comprise the description of various tasks associated with two of the phases of rehabilitation planning. These tasks are annotated with terms like 'failure', 'hazard' and 'precaution' to illustrate the implicit safety issues.

TABLE 1a. SRM Phase 1: Initial Planning and Records

Phase 1. i	
Task: Determine Performance Requirements	
Hydraulic performance: (failure)	Operational performance: (failure)
Structural integrity: (failure)	Environmental Protection: (hazard)
Task: Assess Current Performance	
Use records of flooding (hazards)	
Task: Is full investigation appropriate?	
Full investigation (cost), Structural investigation, Rural investigation	
Task: Check regional priorities	
i) Known causes (failure);	ii) areas of imminent development (precaution)
iii)poor storm sewage overflow (hazard)	iv) system with large number of critical sewers (failure)
v) remaining critical sewers (failure)	
Phase1. ii	
Task: Check System Records	
Depth of sewer; ground quality; marginally important traffic (failure)	
Task: Identify critical sewers	
Collect information (highly impervious - roads)	
Apply screening procedure (sewer type A, B or C)	
Task: Plan records upgrading and improving access	
Produce Master Plan	

TABLE 1b. SRM Phase IV: Implementing the Plan

Task: Timing of Construction OFWAT & Company Rehabilitation Targets (failure) Unit Cost (criticality judgement)	**Task: Maintain Hydraulic Model** Audit trail must be kept for the model
Task: Timing for Hydraulic work Planned New Developments (precaution) Legislation (hazard)	**Task: Review Drainage Area Plans** Major changes New systems coming on-line OFWAT requirements
Task: Design and Construction Flooding (failure & hazard) Operational Deficiencies (failure) Structural Condition (failure) New Developments (precaution) Legislation Changes (new hazards) Pollution (hazard) External Influences (failure) Risk	**Task: Deal with system failures** If a collapse occurs; • Make it safe • Carry out repair • Monitor the area If a hydraulic problem occurs; • Develop solution • Record incident • Implement solution • Monitor solution
Task: Monitor condition of critical sewers (Failure prediction) Sewers must be surveyed and dated	

3. Hazarding safety in modelling the aquatic environment

A key aspect of sewerage rehabilitation is the use of information generated by a computational hydraulic model. Indeed, at the heart of any hydroinformatics system for urban drainage are one or more such models. These models are normally deterministic and therefore designed to replicate the physics of the flow in a particular urban drainage network. Drawing on the discussion of urban drainage modelling in Chapters 3, 4 and 5 considerable attention is given in developing commercial software codes to a sequence of physical, chemical and biological processes. These include the correct interpretation of the physics, chemistry or biology, the analytical formulation of the identified laws or principles, the numerical approximations to interpret the analytical equations, and the formal, quality controlled development of the software. But at each of these stages assumptions and approximations are made, ranging from the dissipation of energy due to the boundary friction in the Saint-Venant equations for gradually varying one dimensional flow, to the 4-point or 6-point implicit finite difference formulations of the equations requiring the introduction of a forward weighting to ensure stability of the finite difference solution and inducing an artificial numerical dispersion, to the bugs inherent in the software despite rigorous development techniques. Thus at every level the modelling software is prone to limitations and uncertainties.

The second category of uncertainty involves limitations in the input data to a model. In order to represent the complex geometry of a sewerage network considerable care is needed over what and how data is collected. The rainfall, rainfall-runoff and wastewater inflows have considerable uncertainties in their measurement or prediction. There are problems of interpretation of point rainfall over an area, estimation of domestic wastewater inflows from population density and other data, calibration of the rainfall-runoff model, and so on.

The third category of uncertainty is induced by the user's interpretation of results from the model dependent on his or her understanding of the modelling software. Whereas an expert user will know many of the pitfalls and short-cuts in building, developing and applying a model, there are still many opportunities for the introduction of errors in judgement. The situation is therefore even more precarious for the inexpert or novice user. Water services organisations have recognised this difficulty and have devised procedures such as the SRM to provide users with guidelines that help them produce better models.

Given such considerable potential uncertainties in software, data and modelling/engineering procedures, it is of concern to managers and users alike that models for particular sewerage networks may not be safe to use and their results are inaccurate and unreliable for decision making. Questions have to be asked, such as: How can such a safe model be developed? What should a manager do to ensure that a model is safe? What can be done to improve the reliable application of information generated by a model? Given the discussion above it should be obvious that there is no simple answer to such questions. Yet there remains considerable concern among users and managers that without some assurance that models and their results are safe to use, there are considerable economic risks that have to be covered through appropriate precautions. It is the minimisation of these risks that is of concern in the remainder of this chapter.

4. Approaches to improving safety

Consider first the minimisation of risk in the application of a particular simulation model. A number of studies have been made of the behaviour of complex computer programs used in high-technology industries such as aerospace, nuclear power, and air-traffic control. These studies have led many software scientists to believe that some software systems are inherently unstable. Despite rigorous development control the complexity of these systems implies that even at the design level the stability of the software implementation cannot be guaranteed. Myers (1986) estimated that there are about three software errors per thousand lines of code in large software systems connected with the US Strategic Defence Initiative (also known as the Star Wars Initiative). If this is the case for defence systems that have millions of dollars invested in them then the question arises as to the frequency of occurrence of errors in comparatively low cost, less frequently inspected and tested civilian systems.

Questions of reliability of software are what have prompted the safety-critical software programmes. Embedded in these programmes are the advocacy of standards. Bowen and Stavridou (1993) referenced 13 such standards for software systems with varied applications, including three standards for military standards in the US and two standards in Europe for software systems developed for the nuclear power and space industries. More recently Gloe and Rabe (1995) have described German National Standards for safety-related systems (see DIN V VDE 801- Principles for computers in safety-related systems published in 1990 and revised in 1994), the International Electrical Commission's standards for safety-related systems (see IEC 654 - Software for computers in the application of industrial safety-related systems published in 1991) and the International Standards Organisation Quality Standards (ISO 9000) for assessing safety-related issues in nuclear power plants, in medical systems (breathing-support machines) and in the control of railway locomotives. These authors note the existence of a number of software tools for testing software systems that help in the confirmation of a software system used as an integral component of a safety-related system.

Safety of software has therefore had to depend on quality controlled development procedures and a large, but not exhaustive, number of tests on the software. This leaves a finite if 'acceptably small' risk of failure. Regrettably, even for comparatively simple hydroinformatics modelling software there are no mathematical algorithms that can be used, even in a rudimentary way, to validate the codes.

It is necessary, therefore, to take an alternative view of how to develop safe codes. What can be deduced from the discussion is that software is more likely to be 'safe' for use if it is developed by an organisation for whom safety is important, that is, where a quality standard is in use. This should also be the case where the software is to be implemented in the design and assessment of large capital schemes, where the reputation of the organisation has direct financial consequences, and where the scientific and mathematical basis of the software is mature. The European Hydraulic Laboratories recognised the need for some means of assessing the validation of software codes for hydraulic modelling; see IAHR (1994). The approach adopted was to examine each of the main steps in the development process:

- deduction of the relevant science (physics, chemistry, biology)
- development of the mathematical algorithms interpreting the science
- production of numerical algorithms to interpret the mathematics
- formulation of the modelling structure using the numerical algorithms
- production of software code

Safety is hazarded at each of these steps. Therefore some attempt to validate the assumptions and conclusions during the process should be made. Dee (1993) recommends the development of a paper-based dossier in which the arguments in favour of the assumptions and conclusions are documented, including numerical evidence as appropriate. In no way can such a dossier be complete, but it can provide the basis for an assessment by others of the possible risks that may be incurred in using the software.

Given software that has been proven by extensive use and access to reasonably reliable data, it remains for the user to carry out the procedure of building, developing and applying a model using the data with the software. It is at this stage that the majority of problems in safety occur. No two people will produce exactly the same model. An expert modeller is more likely to produce a safer model than a novice. But even the expert modeller can make mistakes and be unaware of the risks that his or her decisions generate. Can therefore the reliability of the expert be improved still further? Also, how can the knowledge of the expert be transferred effectively to the novice? These and other questions complement the questions above.

5. Decision support systems

One way of improving safety in making decisions is through the access by engineers to information and knowledge that can support them in their work. The notion is that appropriate expertise is made available through what are loosely termed 'decision support systems'. These systems may simply provide context sensitive information. They may guide the user through a reasoned process. Alternatively, the user may have the full support of an expert system.

There are a number of examples in recent years of decision support systems in urban drainage. One of the first such systems was WIFE: the WASSP Intelligent Front End (Ahmad *et al.* 1985) where WASSP was one of the first 4th generation modelling system for urban drainage; see Price (1981). Another key example in the UK was SERPES which was developed during the UK Government Alvey programme in the mid-1980s; see WIESC (1988) and Chapter 2. This prototype expert system was focused on an advisor for sewerage rehabilitation planning. It was based on an already existing document: the Sewerage Rehabilitation Manual, produced by WRc and was complemented by extensive interviews with practising engineers. The objective of SERPES was to take the user through the complete process for what was termed drainage area planning. Advice was given on each step of the different sub-processes. Direct links were provided to modelling software which was a necessary tool for the hydraulic analysis phase of the procedure. The expert system was not finally taken up by the industry for a variety of reasons including the expense of developing the full implementation of the product, the uncertainty of the future of the industry with privatisation on the horizon, and the awareness that the tools for building expert systems were not sufficiently stable or flexible. Another important expert system for the selection of the best model to use in a given situation was developed under the EC COMETT programme; see Griffin *et al.* (1993). This involved the development of a tool kit based on an expert system to transfer knowledge and provide training with the use of several computational hydraulic models for urban drainage originating in Europe, namely MOUSE, HYSTEM-EXTRAN, WALLRUS, SPIDA and BEMUS.

A number of other decision support or expert systems have been developed to address different aspects of urban drainage modelling. For example, Liong *et al.* (1991) developed a knowledge-based calibration procedure for the SWMM RUNOFF block. The concept involves a sensitivity analysis of the calibration parameters and a strategy for parameter selection that attempts to match the simulated and observed hydrographs. Other similar calibration systems have been developed by Delleur (1991), Delleur and Baufaut (1990) and Baufaut and Delleur (1989, 1990) for calibration of rainfall-runoff and runoff quality modelling. Bowland *et al.* (1993) report on BMP-PLANNER as a decision support system and educational tool for stormwater quality management. This uses models such as XP-AQUALM and SWMM depending on the degree of complexity of modelling. Similarly, Alfakih *et al.* (1990) describe an artificial intelligence system to assist with decision making in integrating alternative solutions for urban drainage problems. A fruitful area for applying decision support systems is real time control. Jacobsen *et al.* (1993) describe the development and application of a general simulator, SAMBA-Control, for rule based control of combined sewer systems, while Lindberg *et al.* (1993) extend the concept further with MOUSE ONLINE. Khelil *et al.* (1993) has also adapted an expert system for the real time control of a sewerage network.

It is important to recognise that although decision support systems offer considerable benefits within a hydroinformatics system they are not infallible and do have some serious limitations. The chief limitation is in the structures that are available for computer-based reasoning.

Human beings reason in a variety of ways that include intuition and the exercise of feelings. Symbolic computer-based reasoning is, in comparison, very limited; see Abbott (1991). Knowledge has to be represented in a structured manner. Reasoning can at best be based on rationality; see Dreyfus and Dreyfus (1989). Therefore, any decision support given by the computer using the symbolic paradigm is limited; see Amdisen (1994) for an exploration of the use of rational reasoning in a hydroinformatics system. Although there is considerable scope here for the introduction of sub-symbolic paradigms, much can be done by recognising that a hydroinformatics system properly *includes its users*. This opens up the possibility of a symbiotic relationship between the user and the computer-based hydroinformatics system that takes advantage of the separate and distinctive ways of reasoning of both human and machine. A very good example of this relationship is shown in a recently completed project that explored the SAFE-Design of networks using Information Systems (SAFE-D*IS*).

6. Safe design of networks using information systems (SAFE-D*IS*) project

This project was a three-year (1993-1996) collaborative venture between a university (Surrey) and a vendor of specialist software systems (Wallingford Software). The project addresses safety related questions regarding the safe design, cost-effective repair and the subsequent hazard-free operation of large *in-situ* drainage networks. These networks serve large conurbations, comprise hundreds if not thousands of conduits (pipes) interconnected through a number of nodes (including inflows, outfalls, pumps, storage tanks), and require significant capital investment to effect changes in their design and subsequent repair or *rehabilitation*.

The SAFE-D*IS* project was joined by the SAFE-D*IS* Round Table consisting of members from the private sector (Thames Water PLC., Severn Trent PLC., and North-West Water PLC), the public sector (Walsall Borough Council, which acts as an agent for the Severn Trent Water for drainage matters in the borough of Walsall, and Sheffield City Council) and a UK civil engineering consultancy, Montgomery Watson PLC. The University of Surrey and Wallingford Software helped with the organisation, execution and follow up of the Round Table meetings.

Knowledge related to the safe and cost-effective rehabilitation was acquired by the SAFE-D*IS* project team from human experts and from specialist texts. The text corpus comprises safety guidelines and procedures, transcripts of expert interviews, learned papers and technical notes, legal texts including the complete Water Resources Act 1991 (HMSO) and a 450 page book that interprets previous legislation; see Wisdom (1970). The text corpus also consists of a terminology database. All the texts relate in one way or another to the rehabilitation of drainage networks. This knowledge was structured in an information system for facilitating safe and hazard free rehabilitation of a part of the network. The structured knowledge can be used to help experts examine their own knowledge, and assist novices to a greater or lesser degree throughout various phases of the complex rehabilitation process.

The SAFE-D*IS* project has identified five distinct groups of software systems that may help in the five key functions that are essential for the safe rehabilitation of complex drainage networks; see Table 2. The integration of these systems was one of the achievements of the project.

TABLE 2. Key functions for safe rehabilitation of complex drainage networks

Function	Software Components
Access electronic documents	Full-text and hypertext management
Access rules and heuristics	Knowledge-base management
Modelling complex network	Network simulation software
Sensitivity risk analyses	Risk analysis software
Model history and audit	Report generating systems

One of the important decisions of the SAFE-D*IS* project was to use as much off-the-shelf software as was possible without compromising the high standards that are demanded for a safety critical system. Thus the information system has access to proprietary simulation, risk analysis, and text analysis software, knowledge engineering tools and data base management systems. The information system developed by the project team animates the behaviour of an experienced engineer setting a number of tasks for a less-experienced engineer to execute. This animation is based on an industry-wide rehabilitation procedure that involves over 20 specialist tasks distributed over 4 major phases; see Figure 1 above.

The first phase of development in SAFE-D*IS* resulted in a conventional software system. Much like the conventional software system, including database systems or simulation engines, the first prototype relied on the user having sufficient motivation and/or knowledge to use any of the textual archives, the simulation engine, the propositions database or the automated procedures. Thus the system reacted to a knowledgeable user quite well, but for novices and, indeed, some experienced engineers, the operation of the system was somewhat baffling.

6.1. A KNOWLEDGE-RICH, INTEGRATED, PROACTIVE SAFETY INFORMATION SYSTEM

SAFE-D*IS* is a *proactive* system, that is, a system that can execute the major and ancillary tasks outlined in each of the four major phases of sewerage rehabilitation planning. This proactive system acts in many ways like other proactive systems, for instance, an expert system, wherein the system infers new facts from old, depending on the context, looks up and presents data from diverse sources, invokes other software systems, and so forth.

This proactive system acts within the framework of the SRM method (WRc, 1986). During the execution of individual phases, and tasks within a phase, the proactive system provides *expert advice*, based on rules of thumb and other heuristics obtained from experts. Proactively, the system can access excerpts and (optionally) full-text from a 'corpus' of texts, some of which are linked through hypertext links. This provides a digital library built in close collaboration with the Round Table. Advice is supplemented by access to data bases containing details of the various components of a given network and its geographical location, and supplemented by access to a industry-standard simulation model, namely *HydroWorks*[1], developed and marketed by Wallingford Software (1994).

The system also keeps a 'diary' of advice it gives to a user and invites the user to enter his or her comments on the advice given or to justify a particular decision that is deemed by the system to be out of the norm. Risk analysis, an important tool in the safety community, can be undertaken through the information system using a low-cost, easy-to-use, and off-the-shelf system, namely, Crystal Ball™ marketed by Decision Engineering Ltd. The information system also provides access to the World-Wide Web and through the Web provides access to up-to-date information related to engineering, legal and safety aspects of the aquatic environment as and when it becomes available on the Web; see Table 3 for more details. More advanced users of the information system have access to a text analysis system, namely *System Quirk*; see Ahmad and Holmes-Higgin (1995) and Chapter 2.

TABLE 3. The functionality of the various components of SAFE-D*IS*. The user interface of the Workbench is written in Visual Basic and runs on a PC. The knowledge-bases are encoded in a variant of PROLOG.

Software Component	**Narrative**
Task Selection & Display	Enables an expert/manager to select tasks for a given project to be executed by a novice engineer.
Knowledge Management	Manages the knowledge base of the SAFE-D*IS* system and contains rules related to various rehabilitation tasks
Yellow Pages Management	Tracks the task a given user is executing and selects relevant excerpts (paragraphs and pages) from a full text-data base.
Safety Labels Management	Displays 'safety labels' during or after the execution of a rehabilitation task
Diary Management	Tracks when and how successfully each task was executed and notes it in a diary.
Report Generation	Generates an 'audit' report based on the contents of the 'diary'
Plug-In External Software	Helps to access data in proprietary data bases and acts as a front end for simulation software.

6.2. OPERATIONAL DETAILS OF THE SAFE-DIS WORKBENCH

The SAFE-*DIS* workbench offers two modes of operation: *professional* and *roster*. The professional edition refers to a mode of operation designed for experts where they can either browse through the system, add more knowledge, modify or delete existing knowledge, and select some or all the phases, and tasks within the phases, for execution by less-experienced engineers. SAFE-D*IS* can thus be configured by senior design engineers in two important respects.

[1] HydroWorks is the successor to WASSP and WALLRUS

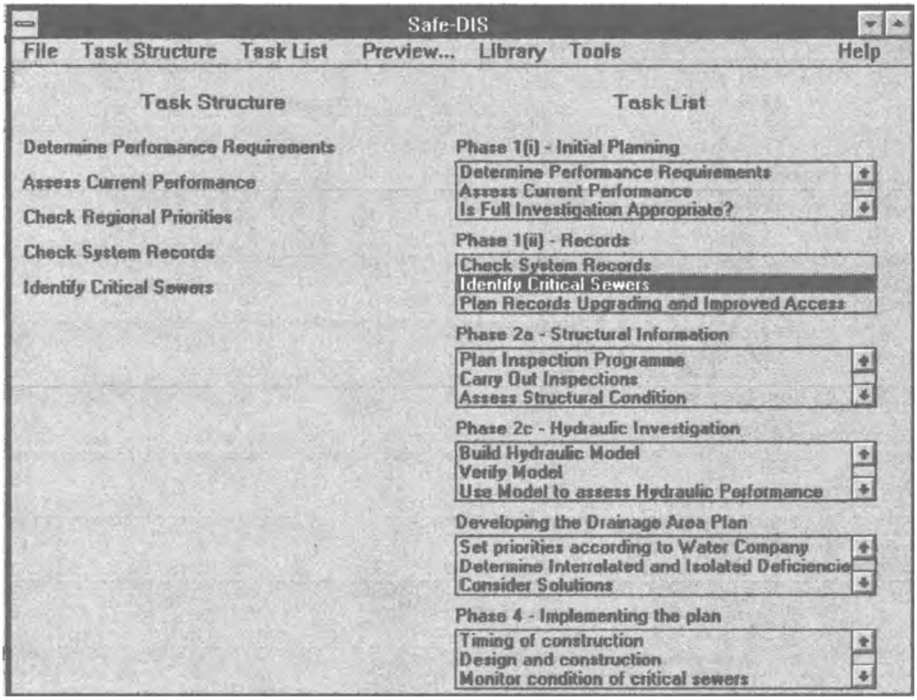

Figure 2. Working screen of SAFE-DIS.

The first level of (re-)configuration is at the knowledge-levels whereby a designated user can add or delete sub-tasks to any of the four phases of the SRM method. The second level of configuration is one where the senior engineer selects specific sub-tasks from one or all the four phases which he or she thinks should be investigated by one or more engineers reporting to him or her.

The roster edition refers to the operation of the system by novice engineers, informs where advice is provided, and allows browsing through the text corpus and access databases and simulation models. During the execution of each of the rehabilitation tasks, the user of the system is guided through a question and answer session that includes display of *safety labels* containing brief items of advice.

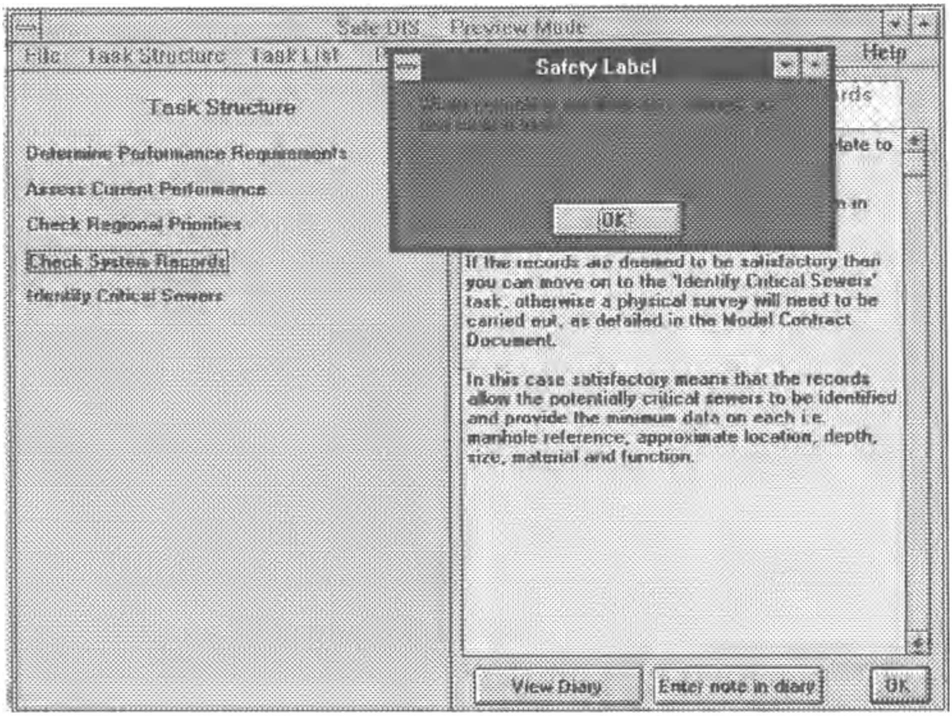

Figure 3. Safety Labels.

During the interaction, the workbench provides pro-active advice: excerpts of texts shown in so-called *Yellow Pages*. Safety labels are sometimes displayed concurrently with the Yellow Pages. The labels come in three 'colours': *red* for mandatory warnings; *amber* for potential hazards; and *green* for safety notes.

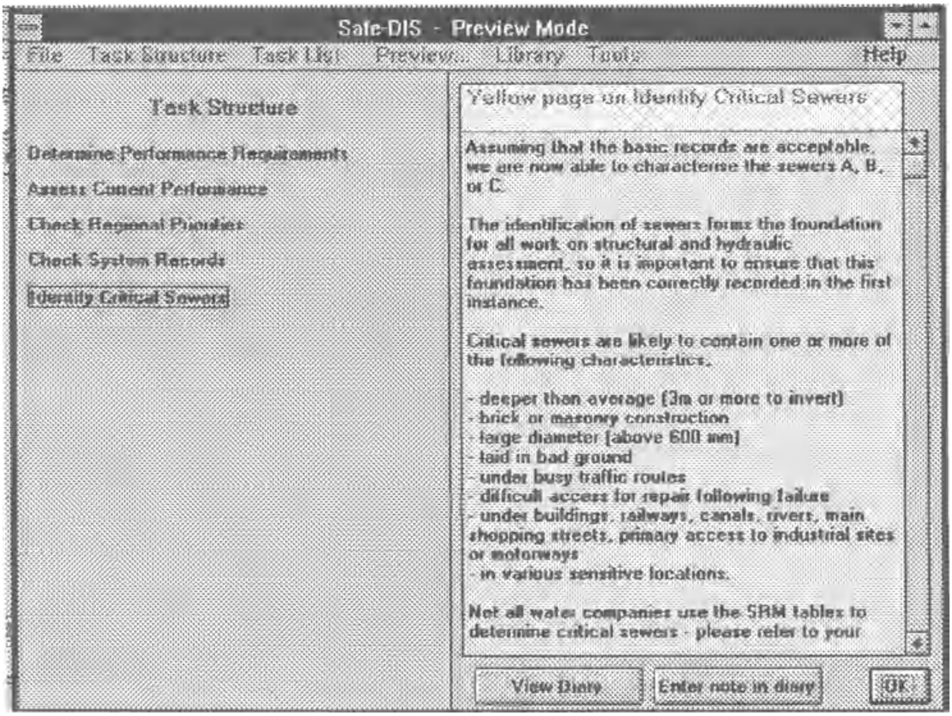

Figure 4. Yellow pages.

The access to full documentation, including Technical Notes (about 10 in number) authored by leading rehabilitation experts in the UK, together with expert interviews and legislation, is also provided by the workbench.

6.3. REPORT GENERATION AND AUDITING

The end of the interaction with SAFE-D*IS* is marked by the generation of a 'sessions report' for the end-user and, where appropriate, for his or her manager.

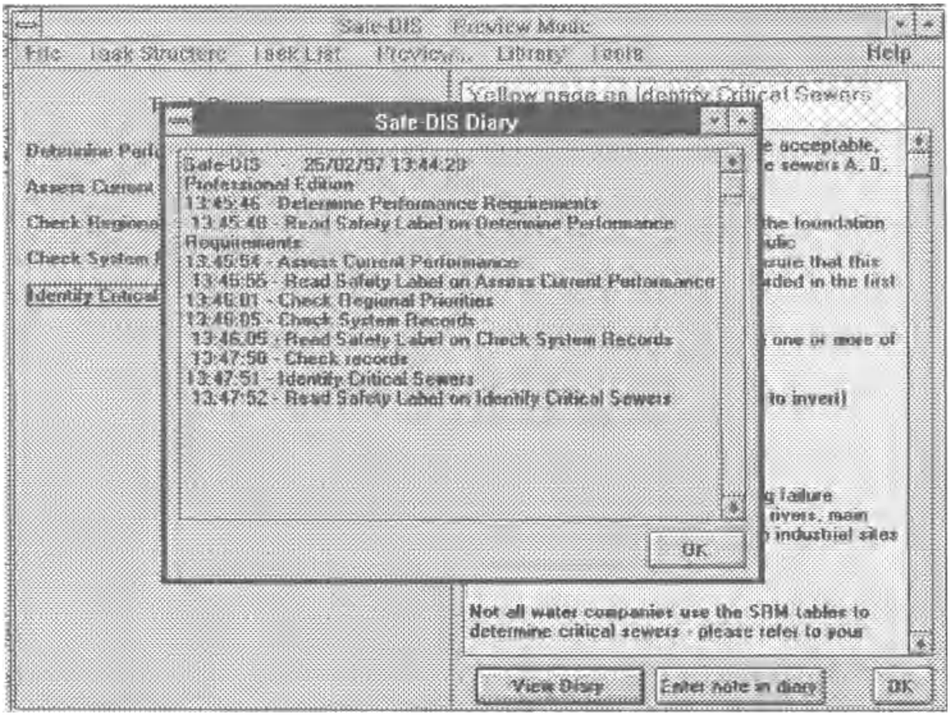

Figure 5. Report generator.

6.4. SIMULATION ENGINES

One of the subsidiary objectives of the SAFE-D*IS* project was to investigate the feasibility of intelligent front-ends and another was to investigate how heuristics and rules of thumb may be introduced in the development of simulation engines; see Ahmad (1995) for more details. In the initial stages of the project it was thought that SAFE-D*IS* will essentially provide an intelligent front-end for HydroWorks, that is, an intelligent system to aid in the selection of data for the simulation engine and in the interpretation of the output produced by the engine.

HydroWorks, like other simulation engines such as SWMM-EXTRAN and MOUSE, appears to be adapting and incorporating a number of data management features such as improved data handling and visualisation, and there is good control of software releases. The vendors of HydroWorks, Wallingford Software, and of MOUSE, Danish Hydraulic Institute, are also taking on board notions like quality management of the modelling process itself, including audit trailing and the generation of reports. This implicit development of an intelligent front-end undertaken by these vendors is a welcome development and has helped the SAFE-D*IS* project team to focus on safety-related aspects of the modelling process itself.

7. Knowledge Documentation: The role of the 'Round Table'

The project used a number of knowledge acquisition techniques reported in the artificial intelligence literature, including face-to-face video taped interviews, structured walk-throughs, questionnaires, and interactive rule elicitation; see for instance Boose (1992) and references therein. Face-to-face interviews between experts and system builders were held on topics related to the safe rehabilitation of networks based on a case study. The questions in the interview were devised by the Round Table. Each interview was video-taped and the transcript of each interview was discussed by the Round Table during brainstorming sessions. The system builders extracted specialist terminology from the interviews, and extracted heuristics and rules. The transcript was marked up such that key parts of the interview could be extracted and linked to other documents through a hypertext browser.

7.1. KNOWLEDGE ACQUISITION TECHNIQUES

A summary of the techniques used are given in Table 4.

TABLE 4. Knowledge acquisition techniques

Informal or overview interviews	These are aimed at familiarising the knowledge engineer with the domain and the particular problem which the proposed expert system is intended to solve. It is therefore likely to be the first interview session held with the domain expert and requires much preparation by the knowledge engineer in collating and learning the relevant technical terminology	This serves two main purposes, firstly, to ensure that the knowledge engineer can understand what the expert is saying, and secondly, to enable him/her to ask intelligent questions referring to an object using the correct term.
The structured interview	Structured interviews normally occur well into the knowledge acquisition phase. They are used when information is required in much greater depth and detail than the other techniques can offer and is more interrogative than conversational. The knowledge engineer will have prepared a list of topic headings rather than questions with which to conduct the interview. He/she proceeds by stating his/her understanding of a topic, his/her exact information needs and will prompt the expert to answer by asking a broad question. During the expert's answer the interviewer will regularly prompt for detail, or tactfully interrupt if the information he/she seeks is not being delivered or if the information is too detailed. In this case the interruption might be a request to briefly recap, or to repeat the description of the situation using layman's terms.	The principle outcome of the structured interviews are the details of the domain entities (tasks, rules and objects) to such a level that a decision could be made about the representation scheme or data structures required to implement them in an expert system.
"Think aloud" protocols	This technique has its origins in cognitive psychology where it was used by psychologists to study the strategies with which people solve problems. Knowledge engineers use this technique in the same way though their subjects are generally of similar intelligence and abilities and the problems they are attempting to solve are far more complex. Basically, it requires that the expert 'thinks aloud' while solving a given problem or case study.	Case studies are advantageous because the end results are already known so the expert should repeat the strategy he/she used for that problem when describing his/her solution.

7.2. BRAINSTORMING

The use of brainstorming techniques is seldom reported in the knowledge acquisition literature, yet the technique turned out to be very useful for devising questionnaires for the interviews, and subsequently for validating and verifying the acquired knowledge. In the SAFE-D*IS* Project, the brainstorming sessions were focused on the safety aspects of the specific phases of the rehabilitation procedures; see Figure 1 above. Individual members of the Round Table were given responsibility for providing knowledge related to given tasks in a specific phase; a detailed transcript of each of the sessions was prepared and circulated to the other members.

Corrections and modifications to the transcripts of the interviews and the brainstorming sessions were agreed by the Round Table as a whole. This consensus enabled the system builders to use verified and validated knowledge rather than the (un-revised) knowledge of a single expert as is the case in many knowledge-based systems' projects.

Structured walk-throughs helped in establishing the manner in which the various tasks within a phase are to be structured and in adding more knowledge for a task which the SAFE-D*IS* system could already execute.

Rule elicitation was used to develop automated/standardised procedures. These procedures, mini knowledge-bases, are particularly useful where the task is amenable to formal description, then automating according to a procedure agreed upon by experts will improve safety. During the structured walk-throughs the engineers provided rules and algorithms for various stages of the modelling process, e.g. choosing coefficients of discharge, accounting for unmodelled storage and checking for limits when doing catchment breakdown.

7.3. KNOWLEDGE VALIDATION AND USER TESTING

7.3.1. *Knowledge Validation*

The knowledge acquired from these meetings formed the main structure of the Safe-D*IS* system, and was especially used for the yellow pages, introducing each task. After initial transcriptions of the meetings were sent to the relevant experts for their comments, we received their corrections and validations. Any corrections and alterations were made, and the knowledge was fed into the Safe-D*IS* system. It is hoped that the engineers using the Safe-D*IS* system will be able to perform a second level of validation, by commenting and correcting phases which were not their own.

At Surrey we have developed a methodology for the validation of terminology and, as in this case, knowledge. Any new knowledge which is acquired is given a status of R (Red), warning anyone viewing the knowledge to *Stop* and be aware that it is un-validated knowledge. Once the knowledge has been validated by the first expert, it is given a status of A (Amber), implying that the knowledge has been checked once but that the user should *Proceed with caution*. After the knowledge has been validated by a second expert it can be given a G (Green) status, determining that the user can *Go*, as the knowledge is safe to use.

7.3.2. *User Testing and Debugging*

The SAFE-D*IS* Project team held four workshops at the offices of the Round Table members, in the final year of the Project. Each of the day-long workshops comprised a presentation of the SAFE-D*IS* project and a demonstration of the system to audiences of company personnel ranging from new recruits to senior management. The presentations and demonstrations were followed by open sessions whereby attendees could come along and get a hands-on trial of the system and speak with the SAFE-D*IS* team. The day then closed with discussions which provided further feedback from potential end-users.

Each workshop was attended by over 20 attendees. By conducting the workshops during the life-time of the project it was possible to incorporate changes to the SAFE-D*IS* workbench. Indeed, these visits convinced the project team that what was required was a proactive system, rather than a reactive system, where a user is guided by domain-specific dialogue.

System testing was concentrated on ensuring that the system was sufficiently portable to run, fully, on a variety of different machines. With the introduction of Windows 95 and the increased popularity of Windows NT, it can no longer be said that Windows 3.1 is a standard for Windows users. Thus it was felt that it was important to ensure that the Safe-D*IS* system could run properly on these platforms.

The initial testing involved a user testing the system to destruction, and noting not only any times that the system crashed, but also any strange behaviour within the system. The user involved in this stage of the testing had previously been unfamiliar with the system, and was felt to be sufficiently detached from the project to be deemed unbiased in any way. This initial testing phase ran for approximately two days, before the comments were collected and modifications to the system were made. These modifications ranged from correcting spelling mistakes in the text, to fixing problems in which the system would crash if a user performed a set of tasks in an unexpected order.

After these modifications had been made the system was tested for a further day, and again any modifications required were made. This cycle continued until it was felt that the system was working as well as possible.

8. Resume and end notes

The design of a complex artefact, like a drainage network, requires careful consideration of number of interdependent knowledge sources, engineering hydraulics and design, environmental sciences, geomorphological and financial data and legal information together with simulation and modelling heuristics. The design engineer works within a community that includes administrators, scientists, and lawyers as well as other engineers. Whether novice or expert, the design engineer should be aware of his or her limitations and seek to compensate for them by co-operating with other expert members of the community.

The implementation of hydroinformatics systems must pay more attention to ensuring that they are used safely. Emphasis should therefore be given to *how* hydroinformatics tools are used as well as to the reliability of the tools themselves. Many of the applications of computational hydraulic models that are at the heart of hydroinformatics systems may prejudice the safety of engineering decisions. Hydroinformatics systems should therefore provide decision support systems that create an environment in which the user is more likely to make informed and reliable decisions.

This issue has been addressed in the development of SAFE-D*IS*. The concepts in Safe-D*IS* emerged during a project at Wallingford Software and the University of Surrey, funded by the UK Department of Trade and Industry under their Safety-Critical Software Programme and by EPSRC. The results are being transferred into HydroWorks for safe design and analysis of urban drainage systems. However, the tools and techniques are generic and can be applied to other modelling or procedural environments. They form a basis for safety-critical hydroinformatics systems.

8.1. THE COST-BENEFITS OF THE SAFE-DIS SYSTEM

Wallingford Software has produced a detailed report of the production costs and downstream benefits of the system; see Price (1996). Here we will restrict ourselves to a brief summary of implementation costs and a short description of the implementation vehicle.

The production costs report discusses the costs involved in developing the SAFE-D*IS* further into a fully working commercial system. Table 5a summarises the costs involved for the main components required by such a system.

TABLE 5a. Summary of costs for the main components

Task	**Cost**	
	Fiscal (£k)	**Labour (person days)**
Task Management	28	85
Document Management	18	55
Expert System Shell	5	--
Risk Analysis	6	20
Model Audit facility	11.4	38
TOTAL	**68.4**	**198**

It has been estimated that in order to implement the infrastructure such that it can be used in a design the following extra costs will be incurred. These relate to (a) task objects (b) document markup / analysis and (c) expert system modules. Table 5b contains unit costs for each of the items (a) - (c).

TABLE 5b. Unit costs for task objects, documents and ES modules

Task	Cost	
	Fiscal (£k)	**Labour (person days)**
Task object	3	10
Document markup / analysis	1.5	5
Expert system module	25	85
TOTAL	**29.5**	**100**

It has been estimated that 10 task objects and 10 documents will be required together with 3 expert system modules. Thus a realistic estimate shows that to build and implement SAFE-D*IS* will require a total of around 600 person days and £200k.

8.2. FURTHER SYSTEMS DEVELOPMENT

The current system will be better implemented using object orientation; this would appear to be the most logical path for any further systems development to take. Each task in the system could be expressed as an object and contain the following slots;

- Texts
- Tools
- Database entries
- Sub-tasks

A task would have a number of texts associated with it, including safety legislation, safety labels, company manuals (or links to relevant sections of manuals), etc. Such tasks would reflect, for example, those in the SRM as described above. However, the concept of tasks is applicable at different levels. For example, a list of basic tasks is given by the pull down menus of the introductory window for the HydroWorks workbench; see Table 6.

These basic tasks reflect the need to manipulate data, fire off runs of the engine and interpret results. At a higher level there are (engineering) process tasks that include various aspects of implementing the SRM or other similar procedures; see Table 7.

TABLE 6. Tasks as defined in the current introductory window of the HydroWorks Workbench

File	Edit	View	Project	Model	Tools	Window	Help
New	Undelete	Select	New	Run new simulation	Generate rainfall	Tile	Contents
Open	Cut	Zoom in	Open	Results	Wastewater generator	Cascade	Search
Close	Copy	Zoom out	Close		Capital costs model	Arrange icons	How to
Close all	Paste	Centre	Files				Tutorials
Save	Delete	Reset	Information				Engineering guide
Save as	Auto insert mode	Options					File reference
Export to DXF	Edit field	Replay					About
Audit	Edit record	Graph					
Print	Insert before	Key					
Print Setup	Find	Find					
Exit	Replace	Select Gauges					
	Validate	Clear Labels					
	Next error	Edit Long Section					
	Previous error	Reverse Long Section					
	Goto	Refresh view					

TABLE 7. Tasks at the process level

Model Construction	Model Development	Model Application
Asset data acquisition	Runoff calibration	Base performance analysis
Critical sewer identification	Asset data confirmation	Performance optimisation
Storage compensation	Sensitivity analysis	RTC design and analysis
Network simplification	Uncertainty analysis	Trade waste analysis
Rainfall-runoff parameterisation	Prototype testing	CSO analysis
Historical performance assessment	Training and implementation of neural network sub-model	Infiltration analysis
Performance requirements determination	Dry weather flow calibration	Flooding analysis
Above ground data acquisition		Sedimentation analysis
Structural condition assessment		Prescriptive network design and analysis
		Storage tank/pond design and analysis
		Capital costs analysis

It is our opinion that the implementation of these and other, similar tasks as objects in frameworks, such as those provided by CASE management tools, will lead to significant developments in *safe* hydroinformatics systems.

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INTEGRATED URBAN DRAINAGE MANAGEMENT

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1. Introduction

Traditional urban drainage planning and operation is aimed at ensuring public health and flood protection by completely collecting and quickly removing wastewater from the settlement area, during both dry and wet weather periods. As an "end of pipe strategy", the wastewater treatment plant (WWTP) was dedicated to improving the receiving water quality, mainly during dry weather periods, by treating wastewater prior to its discharge. Optimisation was carried out separately for both wastewater collection and treatment. In particular, the sewer system was improved by increasing its collection efficiency and minimising the volume of combined sewer overflows (CSO). In many cases, this strategy provides insufficient and cost-ineffective protection of water resources, particularly of receiving waters.

Reducing the CSO volume by storage causes an extended wet weather load on the WWTP, and may subsequently reduce its effluent quality. Therefore, for optimum protection of the receiving waters, it is essential to take into consideration the loads from CSOs as well as from the WWTP effluent, and relate them to the conditions in the receiving water. Depending on the site specific conditions, different pollutants or different dynamic characteristics may be controlling the receiving water quality. In the future, urban drainage planning and operation should thus be guided by a site specific and problem-oriented approach, which requires extension of the system boundaries. The development of an appropriate strategy should be based on comprehensive analysis of the entire urban drainage system and requires **integrated urban drainage management.**

1.1. THE CASE STUDIES

After introducing the urban water management system, a description of "urban drainage" components, and the requirements for an integrated approach, three case

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studies, that were carried out at the Swiss Federal Institute for Environmental Science and Technology (EAWAG) between 1978 and 1994, will be are presented.

In the first case study (Section 2), sources and the fate of various pollutants within different systems and remedial measures are discussed. It is shown that depending on the conditions, different measures may be appropriate or superior. A cost-effectiveness analysis leads to the conclusion that for the same investment, the effect with respect to load reduction is generally stronger when the money is invested in the WWTP rather than in storage tanks.

The second case study (Section 3) concentrates on water and pollution dynamics during a storm event. The water and various pollutants are followed from the source to the sink, which may be the receiving water or the groundwater. It is pointed out how the dry-weather and wet-weather situations interact, and how different controls of the pollution load during wet or dry weather are effective with regard to different pollutants.

The third case study (Section 4) examines the effects of CSOs on receiving waters with regard to physical, chemical and biological indicators. The receiving water ecosystem may well be improved by modifying the morphology of the creeks or raising the groundwater level rather than improving the drainage system.

The case studies demonstrate the need to examine urban drainage as an integrated and interacting system, in order to identify the problems and to select appropriate remedial measures. Emphasis is given to illustrating the advantages of a problem-specific approach with regard to the underlying cause-effect relationships.

1.2. URBAN DRAINAGE SYSTEM

The urban drainage is a part of the entire urban water system that consists of the following sub-systems:

- urban settlement area (e.g., buildings, inhabitants, industry, roads),
- water transport networks and their elements (e.g., water supply pipes, sewers, pumping stations, storage tanks),
- treatment plants (drinking water treatment, wastewater treatment), and
- water resources and receiving waters (creeks, rivers, lakes, and groundwater).

Figure 1 gives an overview of the urban water system, the links between the subsystems, and some important processes within their domains.

1.3. PREREQUISITES FOR AN "INTEGRATED APPROACH"

The application of an integrated approach includes a radical change in scope of the planning and operation procedure. Important prerequisites for applying an integrated approach are the following:

- better knowledge of processes and transfers among the individual elements.
- interdisciplinary cooperation (especially between the engineers and the scientists),

- suitable working tools (numerical models for both specific purposes and for the integration),
- consistent data sets (input data, system data, model parameters and data for calibration and verification of models),
- knowledge of design and operation of remedial measures,
- a hydroinformatics system for the management.

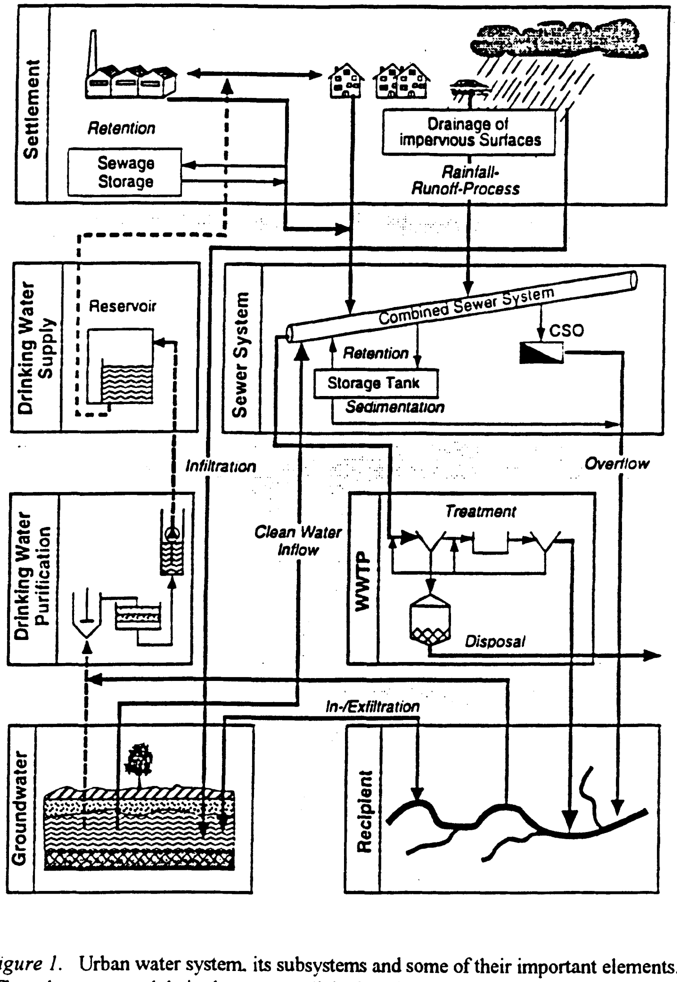

Figure 1. Urban water system, its subsystems and some of their important elements. The subsystems and their elements are linked via important relations and processes.

A major problem for the introduction of this approach in practice is that the ecological research of rivers, lakes and groundwater, and engineering activities in urban drainage have had few common interests in the past. Contacts between scientists and engineers are not well established and, consequently, many planning decisions are based solely on incomplete knowledge and insight. Only a few studies have been carried out which include the entire system, from the source of wastewater and rainfall-runoff to their final ecological manifestation in the receiving water or groundwater (Gujer and Krejci, 1987).

Integrated urban drainage management should take into account human activities within the system boundaries. The working assumption is that the solution of the problems of planning and operation requires site-specific definitions of the problems, analysis of the cause-effect relationships, and the selection of appropriate remedial measures that take into account the entire system as shown in Figure 1. This assumption is made even though such measures may be taken only in a single sub-system.

2. Case study 1: Priorities in pollution abatement in urban drainage

2.1. INTRODUCTION

The Glatt is a small river in the north-east of Switzerland. It has an average discharge ranging from 3.3 m^3/s at the outflow from lake Greifensee to 8.7 m^3/s at the confluence with the Rhein River. The watershed of the Glatt has an area of approximately 260 km^2, and is heavily urbanised (about 900 inhabitants/km^2). About 40 km^2 (16% of total area) are covered by impervious surfaces. The combined sewer system (70%) dominates over the separate system (30%). There are 13 wastewater treatment plants in the Glatt catchment which provide secondary treatment for practically 100% of the wastewater (Figure 2).

In spite of nearly complete wastewater treatment, the water quality in the Glatt River is poor. The Glatt River does not satisfy the standards set by Swiss legislation with respect to aesthetic, biological and chemical criteria,.

2.2. OBJECTIVES OF THE CASE STUDY

In response to this situation, a two phase study was initiated in 1977 to determine pollution control options. The first phase of the study (EAWAG, 1978) centred on upgrading dry weather treatment to an advanced level (nitrification, chemical precipitation and coagulation, partly also filtration). The second phase (EAWAG, 1979) focused on control measures for stormwater runoff pollution, and attempted to answer the following four major questions (Krejci and Gujer, 1985):

- What is the volume and duration of stormwater runoff discharged into the Glatt River and its important tributaries ?
- What are the pollution loads from the point and non-point sources during the periods of stormwater runoff ?
- What are the effects and the costs of stormwater pollution control measures, especially of overflow storage tanks in a combined sewer network ?
- What are the priorities of different water pollution control measures (dry weather and storm weather) in this area ?

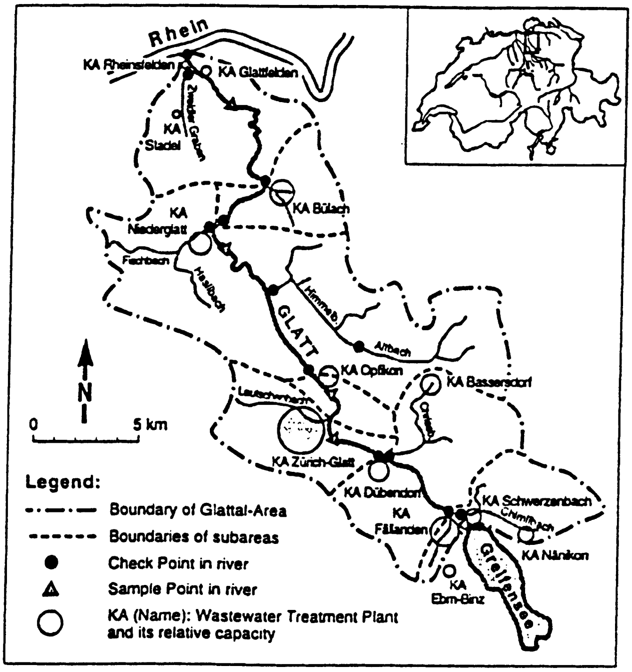

Figure 2. Glatt River watershed (Switzerland). Overview of catchment of the Glatt River, with the important tributaries and wastewater treatment plants (WWTP).

2.3. IDENTIFICATION OF EXISTING PROBLEMS

The effect of stormwater runoff on the Glatt River is evaluated by analyses of

- basic hydrologic conditions,
- combined and separate sewer networks,
- wastewater treatment plants in the area, and
- chemical and biological measurements in sewers, treatment plants and receiving waters.

The costs of remedial alternatives and the resulting pollution loads were calculated with a simulation model developed at EAWAG, and the results were verified against existing data where possible. The verification and the sensitivity analysis proved the feasibility of the simulation model to answer the four questions listed above. The following discussion of various effects gives a summary of this study and its pertinent results.

2.3.1. *Physical effects*

As a consequence of agricultural drainage, channelisation of the Glatt River and its tributaries, and of the development of urban and traffic corridor areas, the discharge and flow velocity in the Glatt significantly increase during wet weather periods, compared with the antecedent situation. Figure 3 shows the discharges and flow velocities in the Glatt River during 1974. Several times, the river bed is at least partly eroded to the extent that the river biota habitat and the conditions for infiltration of riverine water into the groundwater are significantly changed.

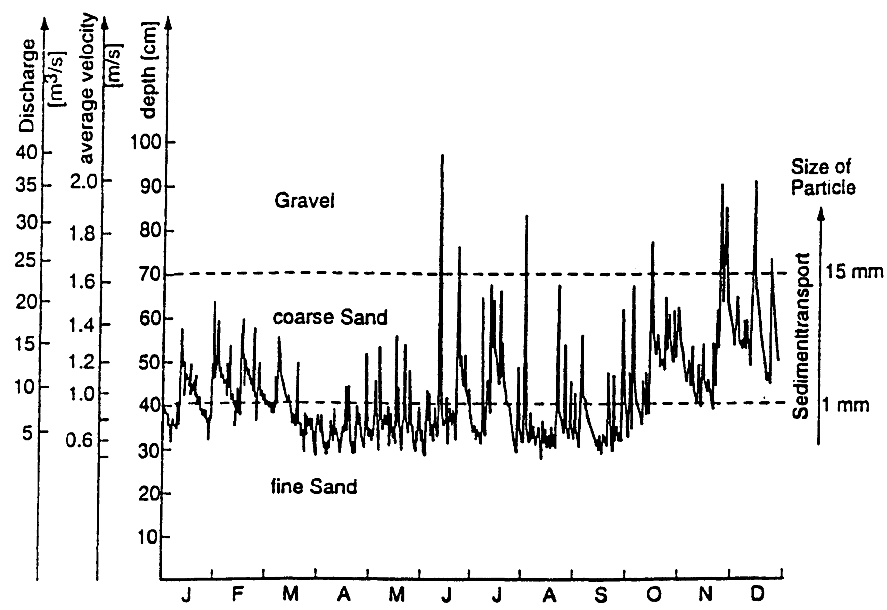

Figure 3. Depth, flow velocity, discharge and approximated sediment transport in the Glatt River in 1974.

The relative significance of the sources of flow and turbidity in the Glatt River during wet weather is shown in Table 1. Clearly, the greater part of loads is derived from non-point sources; however, the contribution of urbanised areas (16% of the total area) is significant.

TABLE 1. Relative contribution of different sources of the flow and TSS-load (turbidity) in the Glatt River during wet weather (average annual values)

Source of load	Average load during wet weather in %	
	Flow	TSS
Wastewater treatment plants	22	< 10
Combined sewage overflows	10	15 - 25
Rainfall-runoff (separate system)	6	< 10
Non-point sources	62	> 50

It was observed that during stormwater runoff, high flow velocities in the Glatt interrupt the development of river biota and change the species distribution. High erosion of agricultural land, the runoff from the impervious urban areas, and river bed erosion cause high turbidity in the river and reduce sunlight penetration in the water column.

2.3.2. *Chemical effects*

The following chemical parameters were used to characterise the pollution load entering the receiving water:

- dissolved organic carbon (DOC) - represents dissolved chemical substances originating from sewage, urban surface runoff and non-point sources,
- ammonium - represents dissolved substances originating mainly from domestic wastewater,
- total phosphorus - represents partly dissolved substances from domestic wastewater and non-point sources (agriculture),
- lead - represents small particulate substances in runoff from impervious urban and transportation areas.

Figure 4 shows the relative load contribution from different sources of selected chemicals during 13 hypothetical rainstorms, which statistically represent the storm events observed in 1974. The relatively high contribution from point sources is striking. Figure 5 shows the annual pollution load to the Glatt River versus that of Chriesbach Creek. Chriesbach Creek was chosen for comparison, because of its relatively small watershed area (approximately 16 km^2) and heavy urbanisation. In both streams, except for lead, dry weather sources (wastewater treatment plants) represent the major sources of the selected parameters.

The observed flow and concentration gradients over time, during several summer storm events in the Glatt River, showed that the concentrations of particulate substances (represented by total suspended solids, TSS, and heavy metals associated with TSS) do strongly increase. The concentration of substances from domestic and industrial wastewaters (represented by DOC, NH_4-N and P) stayed nearly constant, but some other substances (Cl, Ca, Mg) were diluted. We can assume that these relatively small, short term changes in concentration of DOC, ammonium, nitrate and phosphorus have negligible effects on water quality. However, the concentrations of heavy metals reach significantly higher values than during the dry weather period. Although the greater part of the heavy metals load is related to particulate matter, there still may be toxic effects.

During wet weather periods many other substances are transported into the receiving water (e.g., hydrocarbons from different sources). It was not possible to estimate the amount of these substances with the simulation model used. Another shortcoming of the simulation model is its inability to accurately predict suspended solids (TSS) and particulate organic substance (POC) loads, due to the lack of basic information and data.

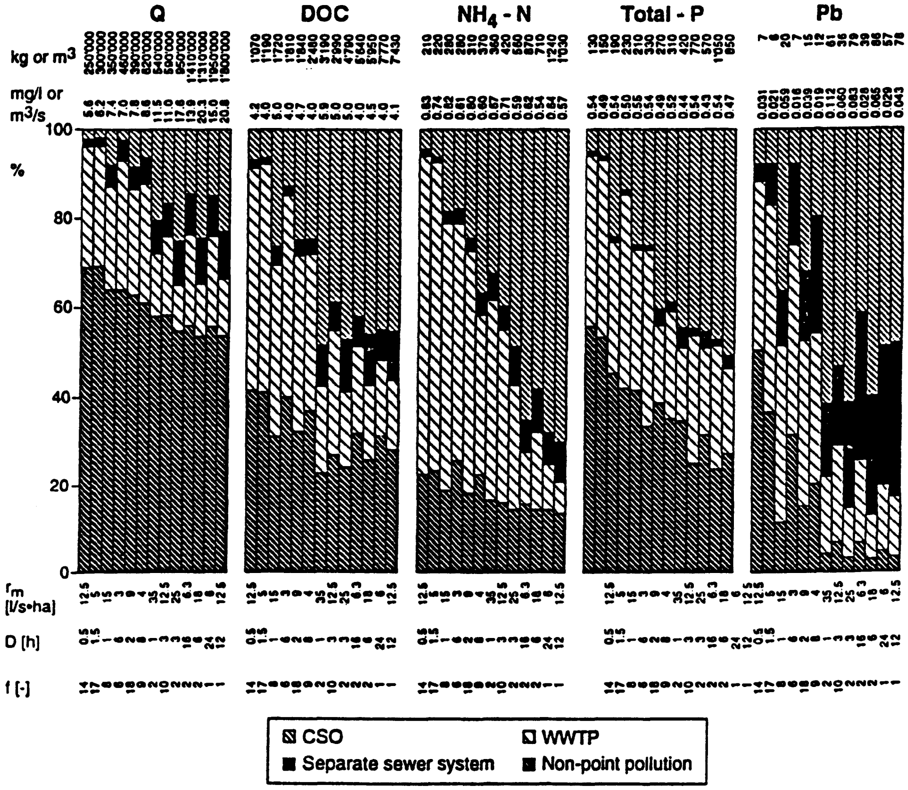

Figure 4. Calculated wet weather pollutant loads and concentrations in the Glatt River for 13 rainfalls. Explanations: total load in kg/event or m^3/event, average concentration in mg/l and average flow in river in m^3/s. The individual rainfalls are characterised by average intensity r_m in l/s.ha, duration D in hours and their annual frequency f.

Short-term changes in dissolved oxygen concentration in the Glatt River from 9-10 mg O_2/l to 4-6 mg O_2/l during storm events may result from DOC and POC loads. Two sources may contribute to this oxygen demand; many easily biodegradable substances from CSOs, and resuspension of anaerobic deposits in the river.

2.4. EVALUATION OF STORMWATER POLLUTION CONTROL STRATEGIES

The benefits and costs of the following stormwater pollution control strategies were compared to the standard measure, CSO control by storage tanks:

- reduction of pollution load from individual sources,
- changes in the characteristics of runoff,
- treatment of combined sewage overflows, and
- enlargement of the hydraulic capacity of WWTPs to accommodate wet weather flow.

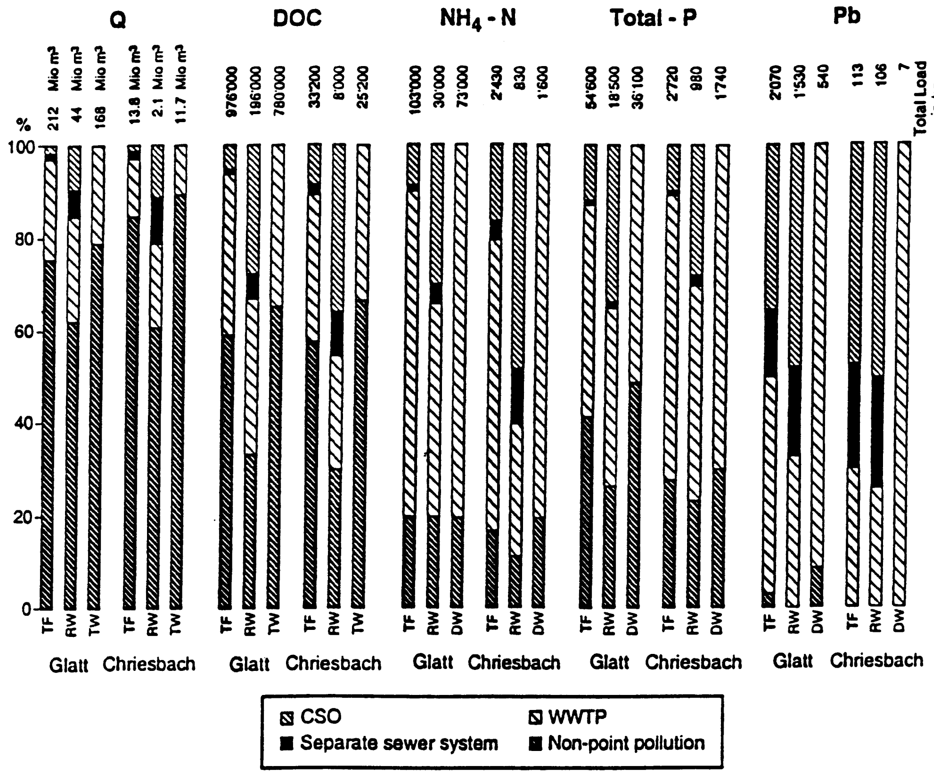

Figure 5. Calculated annual loads in the Glatt River and in Chriesbach Creek (tributary of the Glatt) from important sources. TF total annual load, RW annual loads during rainfall periods (≈ 15% of time), DW annual loads during dry weather periods (≈ 85 % of time).

2.4.1. *Reduction of pollution load from individual sources*

As examples for problem-related source control measures, the effects of the substitution of phosphorus in detergents and the removal of lead from gasoline on the load to the receiving water were investigated. The efficiency of these measures was compared with that of a non-specific measure, the storage and clarification of CSO in storage tanks with a specific volume of 3.5 mm rainfall runoff (usual volume: about 2 - 2.5 mm). Table 2 demonstrates that the substitution of phosphorus in detergents has about the same effect on pollution load from CSO as a very large increase in CSO storage. However, with regard to the lead load, the source control measure is much more effective than CSO storage.

TABLE 2. Pollutant loads during wet weather. Combined sewage overflow storage versus reduction of pollution from individual sources: example for a rainfall event with average intensity of 12.5 l/s·ha and rainfall duration of 3 hours.

Average rainfall intensity 12.5 l/(s·ha) duration: 3 hours	Load in Glatt River kg/event		Load in Glatt River kg/event	
Phosphorus substitution in detergents	NO		YES	
Volume of CSO tanks in m^3/ha *)	0	35	0	35
Total P - Load	306	281	205	191
Load from CSO	111	66	59	32
Load from treatment plants	79	100	31	43
Lead removal from gasoline	NO		YES	
Volume of CSO tanks in m^3/ha *)	0	35	0	35
Total Pb - Load	46	34	5	4
Load from CSO	34	18	2	1
Load from separate sewers	6	6	< 1	< 1

*) per ha impervious area

2.4.2. *Changes in the characteristics of rainfall runoff*

There are several possibilities for changing the runoff characteristics, e.g.:

- The impervious surface runoff can be reduced, e.g., by infiltration of stormwater from roofs and parking areas. Table 3 shows the effect of a 20% reduction in the runoff coefficient. Despite the reduction in pollution load, a relatively large part of the residual stormwater will be treated in the wastewater treatment plants.
- Flat roofs, road surfaces and parking areas that are not intensively used could be flooded during rain events, in order to detain the runoff. The effects of these changes were not calculated, but it is certain that all detention measures reduce the pollution load on receiving waters.

TABLE 3. Pollutant load during wet weather: CSO storage versus reduction of the runoff coefficient

Average rainfall intensity 12.5 l/(s·ha) duration: 3 hours		Load in Glatt River kg/event		Load in Glatt River kg/event	
Runoff coefficient		existing		reduced by 20%	
Volume of CSO tanks in m^3/ha *)		0	35	0	35
Total Load / event					
Q	(m^3)	594 000	594 000	560 000	560 000
DOC	(kg)	3 168	2 760	2 820	2 420
NH_4-N	(kg)	395	365	363	332
Total P	(kg)	306	281	285	260
Lead	(kg)	46	34	36	25
Load from CSO / event					
Q	(m^3)	92 500	59 400	70 700	37 000
DOC	(kg)	1 420	802	1 125	13
NH_4-N	(kg)	160	102	133	73
Total P	(kg)	111	66	93	46
Lead	(kg)	34	18	26	11

*) per ha impervious area

None of these measures can be realised within a short time. However, in the long run, systematic implementation of these measures during the renewal of old systems will contribute significantly to the receiving water protection.

2.4.3. *Treatment of combined sewage overflows*

There are several possibilities for the treatment of combined sewage overflows. All these alternatives tend to eliminate coarse particulate substances. Dissolved substances (DOC, NH_4-N and a large portion of the phosphorus) can only be reduced through storage and subsequent treatment at the WWTP.

For the treatment of CSOs in Switzerland, different types of storage and clarification tanks are used. These tanks can reduce the volume and frequency of CSOs (depending on rainfall event, tank volume, etc.). However, whereas increasing the storage tank volume seems effective with regard to the CSO load, the effects on the total annual pollution load in the receiving water are minor (Figure 6).

2.4.4. *Increasing the hydraulic capacity of wastewater treatment plants*

The hydraulic capacity of wastewater treatment plants in Switzerland is generally twice the maximum dry weather flow. An increase of the hydraulic capacity of treatment plants requires enlargement of their primary and secondary settling tanks. Estimations of reductions in pollution load versus the costs of enlargement using the EAWAG simulation tool showed that in rural watersheds, CSO tanks are usually more cost effective than treatment plant extensions. For large city areas, the opposite seems to be the case. In treatment plants with a primary settling tank capacity of 3 - 5 times the peak dry-weather flow, and overflow after mechanical treatment, a good removal of particulate substances can be achieved in wet weather. However, at the same time, a higher load of dissolved substances will be displaced from the primary settling tanks to the receiving water, and consequently this strategy cannot be recommended.

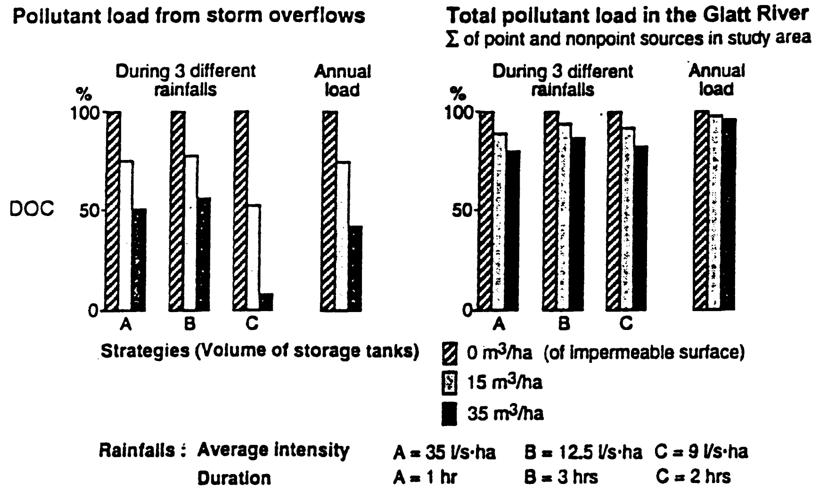

Figure 6. Effect of CSO storage and treatment in CSO tanks on the reduction of DOC-load in the Glatt River (similar effects were found for NH_4-N, Tot. P and Pb).

2.5. COST EFFECTIVENESS OF INVESTIGATED MEASURES

Table 4 shows the costs and the resulting annual load reduction achieved by WWTP and CSO tanks in the Glatt River area. The existing treatment plants are designed for removal of suspended solids and organic substances (secondary treatment). The advanced wastewater treatment provides a relatively high removal efficiency for ammonium and phosphorus. The cost-efficiency of WWTPs with respect to reduction of total annual pollution load is generally much higher than that of CSO tanks.

TABLE 4. The costs and effects of wastewater treatment at WWTP versus storage and treatment of CSO in storage and clarification tanks (cost level: 1979)

Measures	Investment cost Million SFr.	Annual cost Million SFr.	Removal in t/year TSS	DOC	NH_4-N	Tot.P	Specific costs in Fr/kg TSS	DOC	NH_4-N	Tot.P
WWTP:										
Exist. treatment	122	9.9	9000	1570	180	110	1.1	6	55	90
Adv. treatment	47	3.6	350	150	365	260	10	24	11	15
Total	169	13.5	9330	1720	545	350	1.4	7.8	25	39
CSO tanks:										
15 m^3/ha_{imp}	12	0.6	220	10	2	1.4	2.7	60	300	400
35 m^3/ha_{imp}	26	1.2	500	22	4.4.	3.7	2.6	60	300	400

3. Case study 2: Dynamics and interaction of processes in urban drainage

3.1. INTRODUCTION

The dry weather situation in urban drainage is characterised by relatively slow changes of flow and pollutant concentrations. In contrast, storm events cause highly dynamic situations with several respects, e.g.:

- rapid changes of flow rates result in large gradients of shear stress and thus erosion and resuspension of sediments from drained surfaces, in the sewers as well as from the bed in the receiving stream,
- additional pollutants from surface runoff are washed into the sewers and the receiving waters,
- combined sewer overflows discharge untreated wastewater into the receiving waters,
- increased hydraulic loads reduce the performance of treatment plants,
- elevated water levels and eroded sediments result in increased infiltration of river water into the groundwater.

These phenomena are usually investigated separately by various research groups. Due to the enormous resources needed in terms of time and personnel, it is only seldom that a research group undertakes the effort to follow the majority of event processes through the various environmental compartments. At EAWAG, it was possible to direct the interest of some 40 collaborators from different research fields to a particular goal:

"To follow the pollutants during a rain event from their source to their sink and thereby gain insight into a whole series of processes related to rain events in an entire watershed".

In the following, some of the results obtained in this study are presented; a more detailed discussion in given in Gujer *et al.* (1982).

3.2. THE STUDY AREA AND THE STORM EVENT STUDIED

The investigated area (the Glatt River watershed) was described in Section 2.1. Here, only the additional information needed with regard to the storm event study is provided. Figure 7 gives an overview of the investigated elements (urban drainage catchment Friedacker, WWTP plant Zurich Glatt, locations of sampling points along the Glatt River, and a typical river cross section with the infiltration measurement arrangement in Glattfelden). The WWTP Zurich Glatt (the treatment plant for the northern part of the city of Zurich with a capacity of approximately 100 000 PE) is the dominant pollution source in this watershed. In Table 5 further characteristics of the study area are given.

After a dry period of six warm days (the flow rate in the Glatt River was extremely low, see Table 6), a heavy storm with strong spatial variation of the rainfall intensity occurred over the Glatt River valley on July 10, 1981. Figure 7 indicates the isohyets of this event. The highest intensity was observed at the WWTP Zurich Glatt; the average intensity over 15 min amounted to 233 l/(s·ha), which corresponds to a 10-year event. Only 4 km away from the treatment plant, the rain was much less intense.

TABLE 5. Characteristics of catchment areas for individual sampling stations

Characteristics of area for different sampling stations		Unit	Small sewer catchment	Treatment Plant	River in Rumlang	River in Glattfelden
Total area		km^2	0.15	20	110	200
Sewered area	total	km^2	0.13	15	33	43
	Impervious	km^2	0.06	5.5	9.8	12
Comb. System	total	km^2	0.05	12	23	32
	Impervious	km^2	0.13	4.0	6.7	8.6
Inhabitants		1000	1.6	97	200	240
Length of area		km	0.9	≈10	≈15	≈30
Time of concentr.		hrs	0.4	4	6 - 8	15 - 20
DWF	Storm	hrs	0.1	1-2	3 - 5	7 - 10

TABLE 6. Flow in the Glatt River with respect to the investigated event

Water flow in the Glatt River (m^3/s)	Catchment km^2	Annual mean flow	Flow prior to the event	Maximum flow during the event
Effluent lake	0	3.3	1.6	1.7
Hagenholz	70	4.8	2.2	9.0
Rumlang	110	6.0	3.5	18.5
Glattfelden	240	8.3	3.9	15.0
Rheinsfelden	260	8.5	3.9	15.0

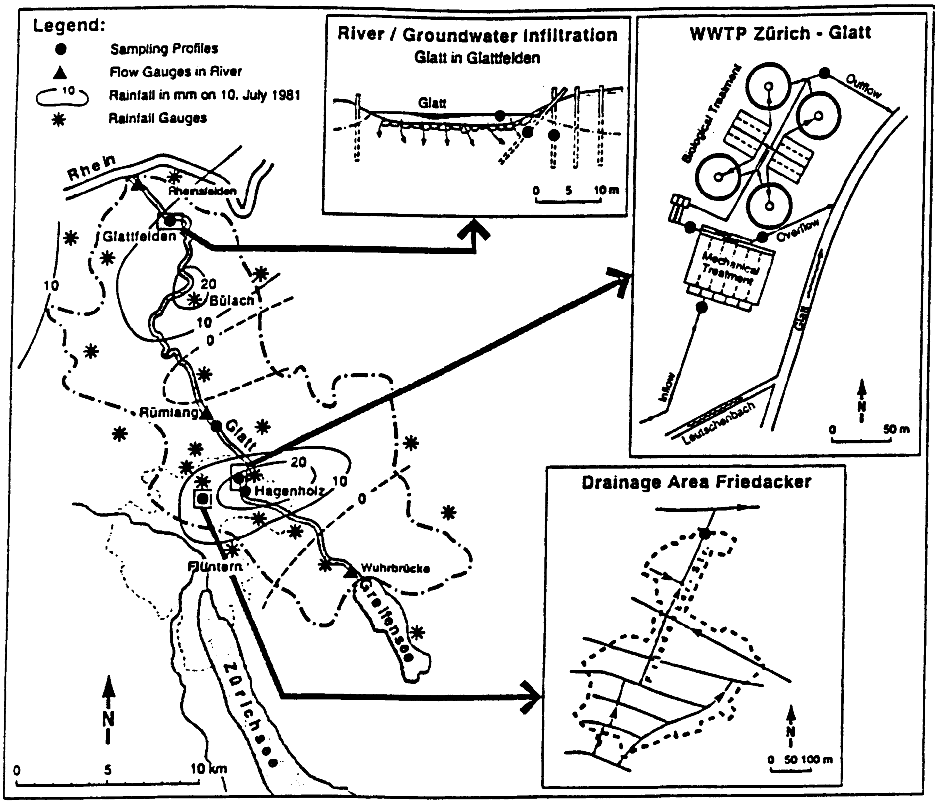

Figure 7. The study area: Glatt River Valley. Overview of investigated elements and sampling stations, rainfall and flow gauges. Isohyets estimated for the investigated storm event of 10 July 1981.

In the lower part of the river valley, the storm occurred 1 hour later, however the flow increase in the Glatt River caused by the storm in the downstream part occurred before the main wave from the upstream reach arrived. Most of the precipitation occurred over densely populated areas of the City of Zurich, whereas large parts of rural areas remained essentially dry. The results are therefore typical for runoff from urban areas only.

3.3. RESULTS

3.3.1. *Water flow and groundwater level*

Table 7 gives the maximum specific flow rate q_{max} per the contributing catchment area. at different locations, indicating strong variations within short distances. With increasing concentration time, the specific peak discharge is reduced by a factor of 100; from > 5 $m^3/(s{\cdot}km^2)$ in a small urban drainage area (Friedacker) to 0.05 $m^3/(s{\cdot}km^2)$ for the entire catchment area. The groundwater level close to the river responded with only a 1 hour delay to the increased water level in the river, indicating a significant increase of the infiltration rate.

TABLE 7. Characteristics of flood wave of the storm event at different sampling stations

Characteristics of flood wave	Beginning - end hours	Q_{max} m^3/s	q_{max} $m^3/(s{\cdot}km^2)$	V_{tot} 1000 m^3	V_{peak} * 1000 m^3
Small sewer system	17:26 - 18:30	0.8	5.5	0.73	0.70
WWTP Influent	17:30 - 22:00	7.5	0.63	48	35
River in Rümlang	18:00 - 04:00	18.5	0.17	268	142
River in Glattfelden	18:30 - 06:00	15.0	0.06	326	161

*) after subtraction of base flow

3.3.2. *Comparison of pollutant loads from different sizes of drained catchments*

The small sewered area Friedacker and the catchment area of the WWTP Zurich Glatt differ by a factor of 100 with regard to the impervious area and by a factor of 10 – 20, with regard to the concentration time. Specific peak mass flow rates of pollutants (per unit impervious area) differ accordingly during the storm event (Figure 8). However, integrated over the entire event, the total specific loads do not differ significantly. Further analysis of these results is given by Dauber *et al.* (1984) and by Krejci *et al.* (1987).

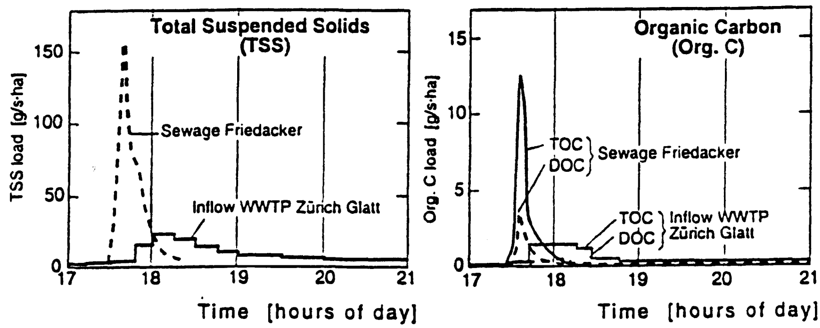

Figure 8. Comparison of pollutant runoff rate from the small drainage area Friedacker and from the entire catchment of the WWTP Zurich Glatt.

3.3.3. *Performance of the wastewater treatment plant*

The main sewer upstream of the WWTP Zurich Glatt has a relatively large hydraulic capacity (9 m^3/s), which is about 10 times the average dry weather flow. It was estimated that only about 30% of the combined wastewater was lost through the CSO structures during the observed event, while the major part of 70% was brought through the WWTP. Primary settling tanks at the WWTP Zurich Glatt have a flow through capacity of ≈ 5 m^3/s (≈ 5 times peak DWF) and 4500 m^3 of additional storage during rain event, before their effluent overflows into the river. Biological treatment has a capacity of 1.6 m^3/s and operates under nitrifying conditions during summer months. During storm events, the difference of 3.4 m^3/s was discharged to the receiving water from the primary clarifiers (during the time of this study).

Figure 9 shows the concentrations and mass fluxes of dissolved and particulate pollutants at different points in the WWTP. Since dissolved pollutants are diluted by storm water, the primary settling tanks contain relatively high DOC (dissolved organic carbon) concentrations at the beginning of the storm flow. Such DOC originated in the dry weather period. The increased flow rate now pushes the concentrated wastewater from the primary tanks over the overflow, which results in a "negative removal" of dissolved compounds by the primary tanks. This is indicated by an increased DOC load in the primary settling tanks overflow as compared to the inflow. For particulate pollutants, the primary settling tanks provide a net removal efficiency during the entire event. Although DOC concentrations entering the plant are reduced, the effluent concentrations remain almost constant and are not decreased by the biological treatment. The increase of the flow rate results in an actual load increase and, consequently, in a significant reduction of the removal efficiency.

The dynamics of the pollutant loads in a conventional treatment plant during a storm event indicates that the overall performance of the plant is poor for dissolved compounds and reasonable for particulate compounds (as long as there is no wash-out of sludge from the secondary settling tanks). This is an interesting observation in view of the large storage volumes generally provided in Swiss drainage systems. From this viewpoint, the effect of storage may be significantly overestimated, since it will cause the load to the WWTP to be at the storm water level for an extended time period, after the end of the rain. However, this study points out the importance of assessing the performance of the entire system, considering the sewer system and its CSOs as well as the WWTP.

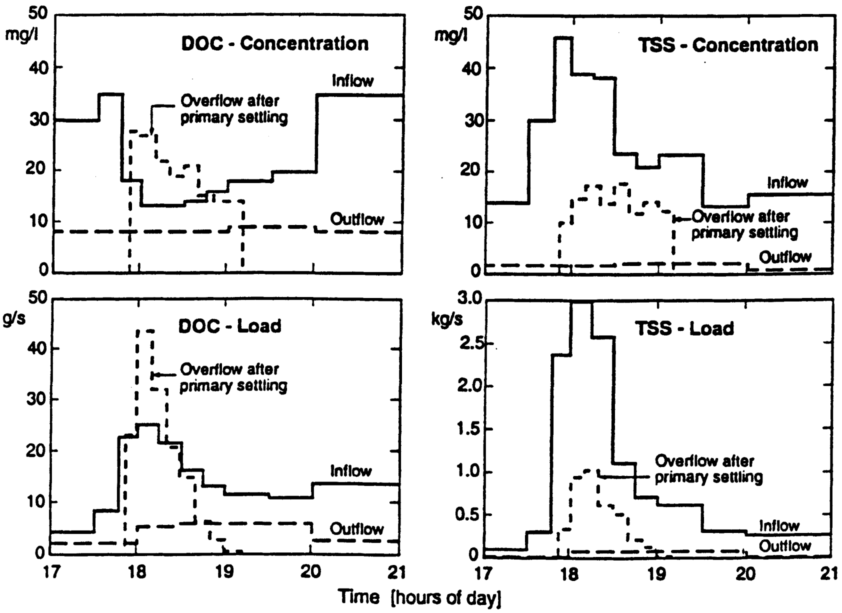

Figure 9. Pollutant concentrations and loads in the WWTP Zurich Glatt: inflow to the plant, overflow after the primary settling tanks, and effluent from the secondary settling tanks.

3.3.4. *Pollutants in the Glatt River*

Figure 10 shows the variation of the flow rate and the loads of suspended solids (TSS) and ammonium at different locations along the Glatt River. The TSS peak load coincides with the peak flow rate, which indicates that the river itself is the main source of solids. The solids load in the river originates mostly from resuspension from the river bed, rather than direct inputs during the storm event. The contribution of the treatment plant to the suspended solids balance is not significant. Particulate organic carbon (POC) behaves similarly to total suspended solids. In Glattfelden (the sampling station most downstream along the river), the accumulated POC load amounted to about 4.5 t of organic carbon. From this load, only about 1.5 t originated from inputs during the storm, and 0.5 t from the WWTP. The difference of 3 t POC is due to resuspended solids that have settled in the river during the dry weather period (6 days), prior to the event. During this period, 1 t of POC/day, in the form of suspended solids, was discharged to the river via the treatment plant effluent. The 3 t of POC are approximately equivalent to 15 g POC/m^2 of the river bed or to a layer of not more than 0.5 mm. Other pollutants sorbed to particles, such as lead and lipophilic organic contaminants, behaved similarly as POC, although the relative significance of the direct input contributions during the event was somewhat increased.

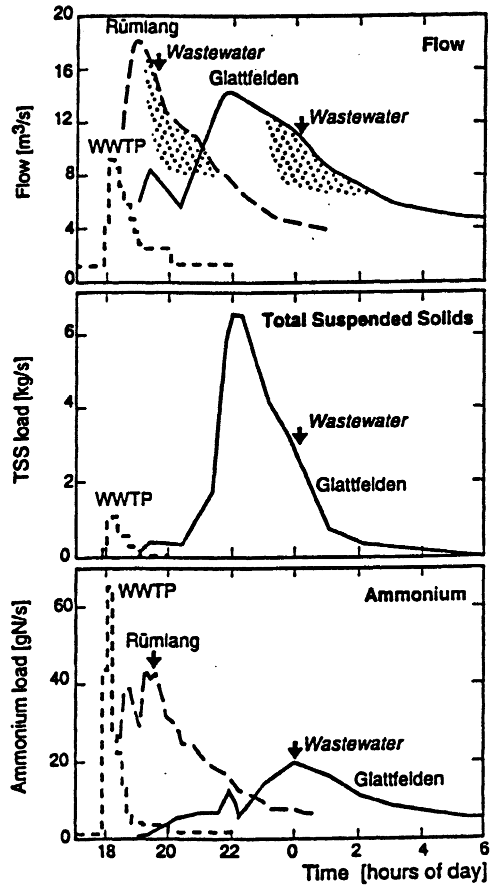

Figure 10. Flow rate and pollutant loads in the Glatt River and in the total discharge (overflow after primary, effluent of secondary settling tanks) from WWTP Zurich Glatt.

Ammonium is a dissolved pollutant that is hardly present in the river during dry weather periods, and most of the WWTPs in the study area were nitrifying even during the observed storm event. Ammonium is, therefore, a good tracer for the discharge of untreated wastewater through both CSOs and overflow after the primary settling tanks. Indeed, the contribution from overflow after the primary treatment at the treatment plant studied is significant. In addition, the peak discharge from the WWTP is delayed relative to the peak flow in the receiving stream in Glattfelden. Other dissolved pollutants such as DOC and volatile organic compounds behaved similarly as ammonium.

In summary, a survey of the dynamics of particulate and dissolved pollutants during a storm event in a river indicates that improved retention of particulate pollutants during storm events would only marginally improve water quality in the river. The dissolved pollutants, which are ecologically more crucial but also more difficult to reduce, are to a large extent discharged without much treatment to the receiving water during storm events. Improved wastewater treatment at the WWTP Zurich Glatt during dry weather period (e.g., filtration) could significantly reduce particulate pollutants which deposit on the river bottom but are resuspended during peak river flows. This effect should not be underestimated if dissolved oxygen is a concern in the receiving stream, since resuspended sediments may cause a major oxygen demand.

3.3.5. *Pollutants in the groundwater*

Intense turbulence and erosion of the river bed and elevated water levels during peak discharge lead to a higher permeability of the river bed and to increased pressure gradients between the river and the groundwater, respectively. Therefore, infiltration of surface water to the groundwater aquifer is enhanced. Since dissolved pollutant concentrations in the river are high during storm runoff, significant amounts of these compounds may percolate into groundwater. The fate of organic contaminants during infiltration is influenced by a series of physico-chemical and microbial processes, such as dispersion, adsorption, biological transformation and degradation.

Figure 11 illustrates the observations for two organic trace contaminants. 2-Heptanon, which has never been observed in the river before this event and thus does not have a history in the aquifer, appears only 5.5 hours after its detection in the river, in the groundwater at 2.5 m from the river bed. Based on the water-sediment partitioning characteristics of this substance, 2-Heptanon is transported in the liquid phase, with not much sorption to the sediment (Schwarzenbach and Westall, 1981). Contrary to 2-Heptanon, 1,4-Dichlorobenzene readily adsorbs to the solids, and any resulting changes in groundwater concentrations are hardly noticeable.

4. Case study 3: From urban drainage management to urban water management

4.1. INTRODUCTION

The new Swiss guideline for Integrated Urban Drainage Master Planning (IUDMP; VSA, 1989) introduced a radical change in both the scope and the procedure of the planning process. This guideline defines **IUDMP as a management tool** in which the urban drainage system is defined to include the urban area, soil and groundwater aquifer, drainage network, WWTP and receiving waters. Furthermore, it is proposed that in the future only site-specific and problem-related measures should be applied to solve problems in receiving waters caused by urban drainage.

However, the application of this guideline is presently not an easy task, as practical experience needs to be gained first. Additionally, as compared to the traditional master planning, the IUDMP is much more expensive in the planning phase. To support the practical application of this guideline, EAWAG carried out field investigations on integrated planning with a multidisciplinary research team. The objective of the project "Integrated Urban Drainage" was to demonstrate procedures for planning and operation of urban drainage systems with respect to technical, ecological and economical aspects. Most of the information has been derived from a case study carried out for the Swiss town of Fehraltorf (Krejci *et al.*, 1994).

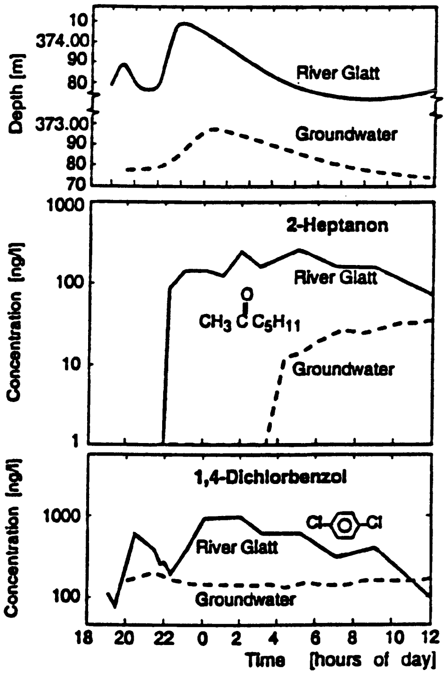

Figure 11. Variation of water depth and pollutant concentrations in the Glatt River and the nearby groundwater (2.5 m from the river bank).

4.2. STUDY AREA

The town of Fehraltorf (together with its neighbouring villages Russikon and Rumlikon) is located about 10 - 15 km east of Zurich. The urban catchment represents mostly residential land (8,000 inhabitants) with some fast developing industrial areas. The urbanised area is approximately 0.25 km^2. The drainage system can be characterised as typical for Swiss conditions: combined sewers are predominant and all drainage structures (e.g., for CSO treatment) correspond to the existing standards. Similarly, the activated sludge treatment plant fulfils the site-specific requirements for dry weather treatment (C, N and P removal). The most important receiving water is Luppmen Creek with a catchment area of 25 km^2, upstream of the Fehraltorf WWTP.

At first glance, Fehraltorf possesses a well-designed drainage system, according to the present state of the art and the current standards. However, the investigation by Krejci *et al.* (1994) revealed that the system of Fehraltorf and its vicinity is far from a technical, ecological and economical optimum.

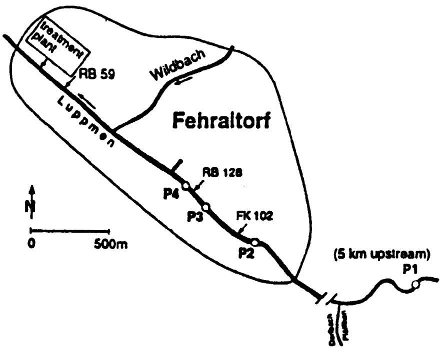

Figure 12. The Fehraltorf urban drainage area with CSO structures (RB 59, RB 128 and FK 102), the treatment plant, the receiving water (Luppmen Creek) and four sampling sites P1 ... P4 in the creek.

4.3. EXAMPLES OF EXISTING PROBLEMS

4.3.1. *Flow regime in the receiving stream*

Figure 13 shows a comparison between the flow regimes of Luppmen Creek at Fehraltorf (based on measurements in 1991 and 1992) and a reference creek (based on 20 years of measurements) in similar topographic and climatic conditions. During approximately 40% of the time, the flow rate in Luppmen Creek was below 30 l/s (1.2 l/s·km^2), which ecologists defined to be a minimum value to maintain the ecosystem functions.

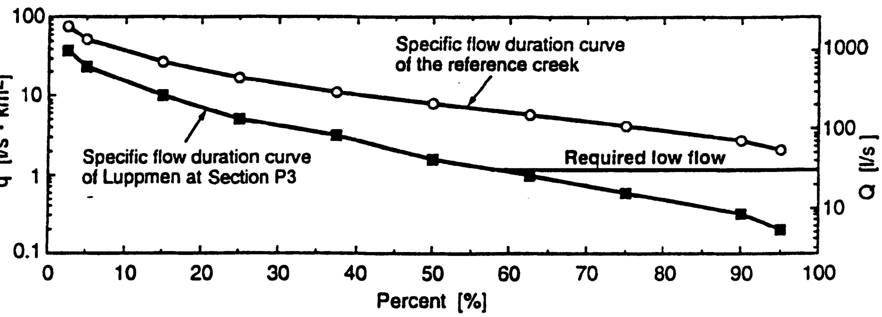

Figure 13. Specific flow duration curve of Luppmen Creek (1991/1992) and of a reference creek in similar topographic a climatic conditions.

The following reasons may be given for frequent low flows in Luppmen Creek:

- Part of the flow is diverted upstream of Fehraltorf for energy production. Until 1991, the full baseflow was diverted; in 1992, the diverted flow was reduced to 25 l/s.
- Wetlands upstream of Fehraltorf are drained (for agricultural land use) and drainage water is discharged into Luppmen Creek downstream of Fehraltorf (approximately 30 l/s).
- Groundwater is pumped at increasing rates (Figure 14) for drinking water supply. Since 1945, the population serviced has increased by a factor of three. Due to the lowered groundwater table, exfiltration into Luppmen Creek in the critical section is reduced and dry periods are prolonged. Also, the downstream end of the dry reach extends to the section downstream of the CSO structures of the storage units FK 102 and RB128 (see Figure 12).
- Groundwater recharge is reduced due to increasing urbanisation. However, this effect is considered much less important than the increased groundwater pumping.

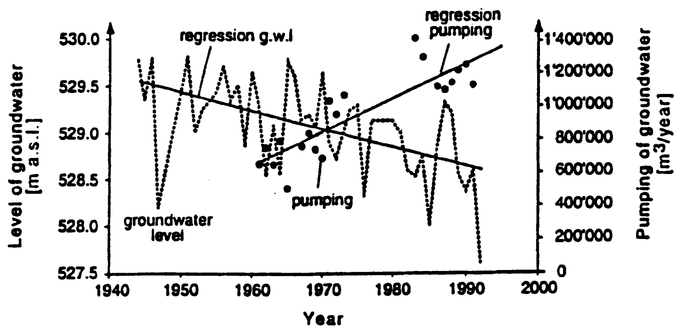

Figure 14. Groundwater pumping rate and groundwater level in the aquifer underneath Luppmen Creek.

Figure 15 shows another problem with respect to the flow regime in the receiving water. A sudden and significant increase of the flow rate in Rohrbach Creek (tributary of Luppmen) is caused by a CSO event. This unusual increase in discharge (from 10 - 15 l/s up to 1 - 3 m^3/s), caused by anthropogenic influences, is accompanied by erosion of the river bed, and occurs 15 - 25 times a year in Rohrbach Creek. Apparently, it seems to be the most critical stress (killing and drift) for the biota.

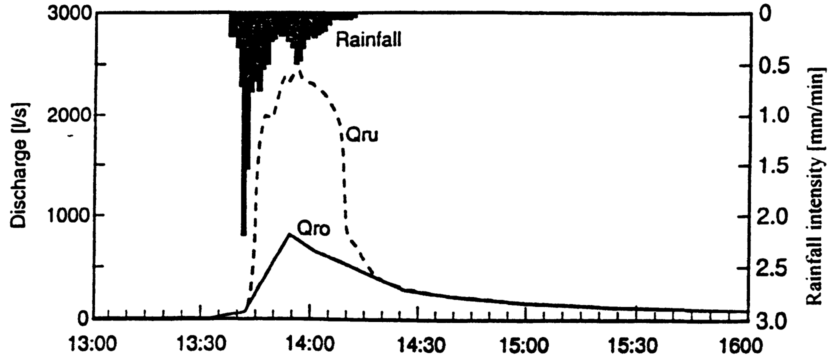

Figure 15. Observed discharge in Rohrbach Creek during the storm event of 24 June 1992. As a result of the CSO and the effluent from the separate drainage system, the discharge in Rohrbach Creek increased from ≈ 10 l/s to ≈ 2500 l/s within a few minutes. Creek bed erosion starts at a threshold discharge of 250 - 300 l/s.

4.3.2. *Morphology of the receiving water*

Figure 16 shows the morphological state of Luppmen Creek; in the vicinity of P1 the creek follows its natural course and is slightly meandering. Creek bed and banks are natural (forest area), although there are some man-made drop structures. In Fehraltorf, where the creek was originally braided, the water flows in a straight channel. The creek bed is stabilised by bed pitching and drops, the banks are protected by toe protection or (partial) bounding. Bushes and trees are restricted to the upper banks and provide only partial shading for the creek bed.

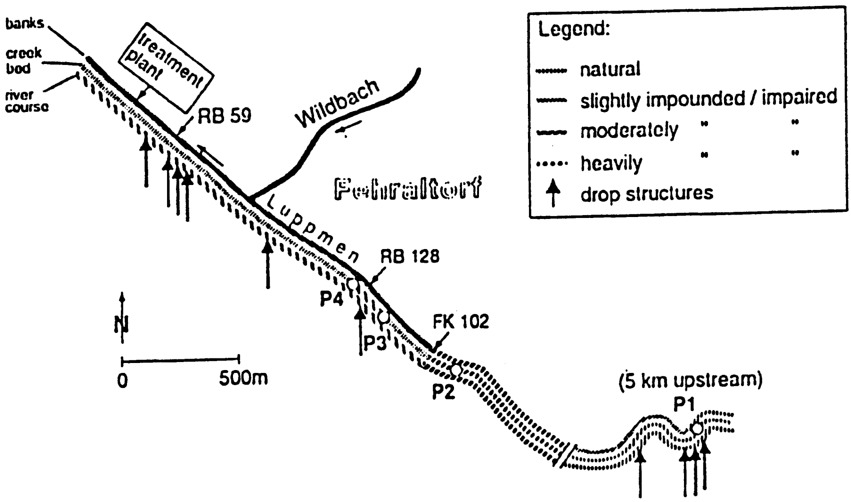

Figure 16. Morphological state of Luppmen Creek (stream course, creek bed and banks). Drop structures are indicated by arrows.

The morphological deficiencies are predominant in the sections within the town of Fehraltorf. Among others these include:

- straight stream course, no variation in width and water depth (profile), no variation in flow velocity (no pool/riffle sequences),
- sharp borderlines between banks and riparian land, no vegetated transition zones, no bushes or trees on the lower banks, little shading, input of branches, leaves, no large vegetation in the water body,
- sealed, "concrete like" bed, no access for benthic organisms,
- low flow and lack of shading yields very high summer temperature (peaks over 25°C) proliferating the growth of algae and, subsequently, high pH.

4.3.3. *Chemical pollution during wet weather*

Figure 17 shows the flow rate, ammonia and ammonium concentrations and oxygen saturation in Luppmen Creek downstream of the CSO structure at P4 (for location, see Figure 12), during the period from September 11 to 15, 1991. CSO events are indicated by arrows. After six weeks of dry weather, a short but intense storm occurred in the morning of the 11th of September. The overflow volume of about 50 m^3 was discharged into a virtually dry creek bed, and infiltrated into the interstitial spaces in bottom sediments. The high DOC values of the CSO led to oxygen depletion, both in the interstitial spaces and in the surface water ("surface water" refers to small remaining pools of creek water). Unionised ammonia concentration went up to 0.3 mg/l for ten hours, killing the remaining fish (brown trout). In the evening of September 11th, a second storm with a greater rain depth caused another CSO event that now induced a considerable flow (increased "natural flow") in the creek. As a result, CSO (ammonia) was diluted and flushed out, while an immediate increase of oxygen saturation in the upper parts of the interstitial space could be observed. The concentrations at 30 cm and 50 cm below the bed hardly changed. After the decrease of flow rate in the creek (18 hours after the second CSO event), Luppmen Creek fell dry again. It took some time to recover from the oxygen depletion in the interstitial spaces in bottom sediments, probably because particulate organic matter was infiltrated during the first CSO event. The oxygen saturation did not reach pre-event values in the interstitial space until three days after the first overflow event.

The chemical load on Luppmen Creek upstream of Fehraltorf is close to natural conditions. In the Fehraltorf town section, CSO events frequently cause violation of oxygen criteria and ammonia limits for acute toxicity in the summer period. The adverse effects are intensified by low flow rates in Luppmen Creek (< 30 l/s), high pH-values (8.0 - 8.5) and high temperature (> 20°C). In lower creek reaches (several hundred metres downstream of the urban area), where better low flow conditions (50 - 100 l/s) prevail due to exfiltration of groundwater, the acute pollution problems caused by wastewater from urban drainage occurred to a much lesser extent.

4.3.4. *Biological observations*

Figure 18 illustrates the composition of the benthic community of Luppmen Creek (summary of 1990 to 1992) at sites P1 (reference site) P2 (upstream of the first CSO structure) and P3 (downstream of the first CSO structure). While at the reference site stoneflies, mayflies and midges contribute to 76% of the individuals, their contribution at points P2 and P3 is only 17% and 9%, respectively. By contrast, crustaceans, worms and leeches contribute to only 17% at the reference site while their contribution rose to 80% and 88% at sites P2 and P3, respectively. The number of different taxa ("diversity") is significantly higher and the number of individuals per area is higher and more constant ("stability") at the reference site P1 as compared to sites P2 and P3 in Fehraltorf.

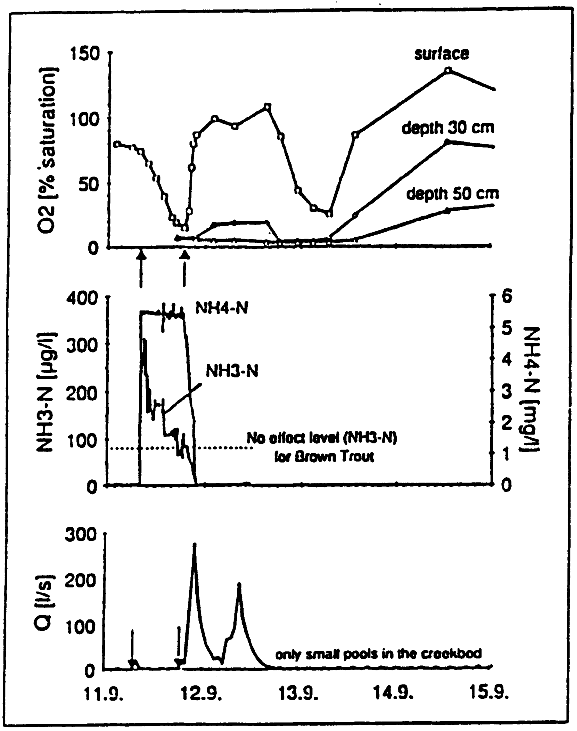

Figure 17. Flow rate, ammonia, ammonium and oxygen saturation at site P4 during and after the CSO event of September 11^{th} to 15^{th}, 1991.

The ecology of Luppmen Creek within Fehraltorf is primarily dominated by hydrological and morphological deficiencies. Oxygen and ammonia problems would be vitally reduced if the proposed minimum flows rates were not guaranteed. As no places of refuge such as a deep pervious interstitial and pools exist, hydraulic and/or chemical stresses affect all biota directly. Long term consequences caused by water quality effects of CSOs could not be identified. The unsatisfactory conditions in the creek are rather a result of poor hydrological and morphological conditions which predominantly determine the composition of biota (Frutiger and Gammeter, 1992). Only a few resistant taxa could be observed.

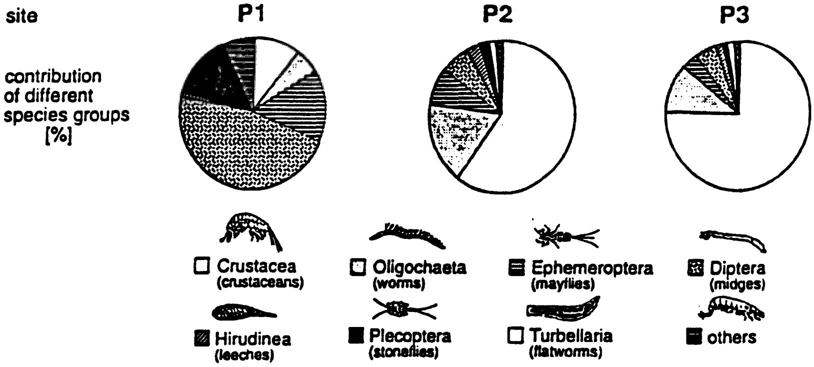

Figure 18. Summary of the composition of the benthic community in Luppmen Creek (1990 to 1992) at the reference site P1 and at sites P2 and P3 upstream and downstream of the first CSO structure in Fehraltorf.

The very low number of biota in Rohrbach Creek downstream of the CSO structure is primarily caused by physical effects, namely by frequent bed erosion by high discharges from the respective CSO structure.

4.3.5. *Operation of CSO tanks*

In Figure 19, an example of unsatisfactory operation of a CSO clarification tank is illustrated. A large portion of clean water inflow to the sewers prevents the emptying of the storage tank for an extended period after the end of the rain event. During the event of June 22-23, the tank remained filled for 45 hours in total, and the emptying of the tank could not start until 13 hours after the end of rain, presumably because of the rain-caused inflow of clean water to the sewers.

Another reason for poor usage of storage capacity may be poor coordination of the operation of storage units. The storage channel FK102 and the storage tank RB128 (see Figure 12) are partly operated in series, within the sewer network. Despite the fact that both structures were designed for the same sewage retention efficiency, the actual efficiency differs a lot. Between November 1992 and October 1993, the CSO volume from the structure RB128 amounted to 72,000 m^3 (60 overflow events with a total duration of 233 hours), whereas the overflow volume from the structure FK102 during the same period of time was only 340 m^3 (6 overflow events with a total duration of 2 hours).

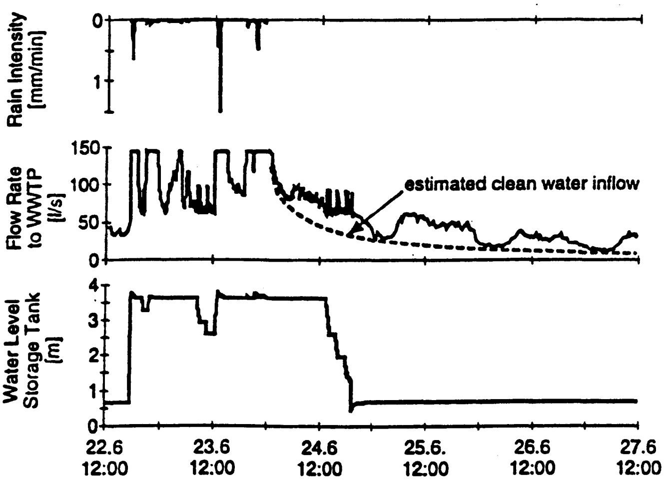

Figure 19. Example of unsatisfactory operation of a CSO tank caused by a large inflow of clean water (estimate is indicated) to the sewer network.

4.3.6. *Interactions between CSO and WWTP*

A modelling study was conducted to improve the understanding of the total load from the system, including CSOs and WWTP effluents, to the receiving water (Krebs *et al.*, 1996). Ammonium is discussed here due to its chemical equilibrium with fish-toxic ammonia (that is primarily determined by pH and temperature). The soluble ammonium comes mostly from wastewater and exhibits strong diurnal variations.

Calculations were based on data collected during rain events, a typical diurnal sewage variation and a virtual WWTP which was modelled as a nitrifying plant using ASIM (Gujer, 1990). Included in the model were several effects that were due to rain events, such as the displacement of ammonium from the primary clarifier to the aeration basin, the transfer of sludge from the aeration basin to the secondary clarifier, and the reduction of the specific activity of the activated sludge because of increased flocculation of particulate material into the sludge.

Figure 20 shows the ammonium loads in the CSO and in the effluent of the treatment plant. The first two peaks between hours 2 and 5, and hours 8 and 12, after the beginning of the rain event, are clearly correlated to runoff volumes. Another peak is observed at the 20 hour mark, despite consistently decreasing runoff volumes. This is induced by the morning peak of the diurnal sewage variation.

The ammonium load leaving the WWTP reflects the diurnal load variation, with a certain time delay. The peak values on the day after the rain event actually exceed the highest values of the day of the event itself. Due to clean water inflow, the hydraulic loading of the WWTP remains at the wet-weather value long after the end of the rain event, while the specific activity of the sludge still remains depressed due to the input of particulate material.

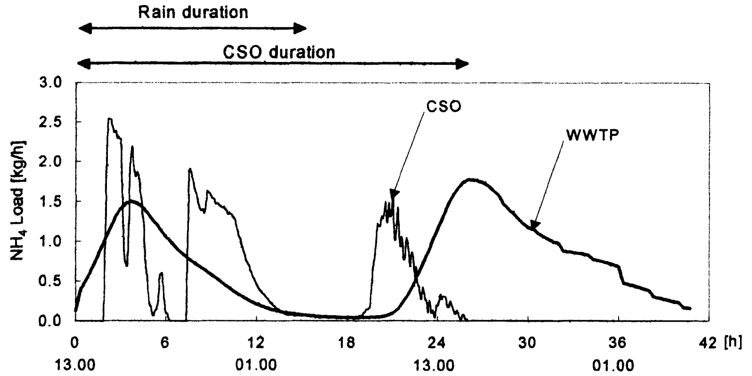

Figure 20. Modelled ammonium loads from CSO and WWTP during a long duration rain-runoff event.

Among other alternatives, the effect of reducing the rain-caused clean water inflow was evaluated. An obvious consequence was that the overflow ended some 8 hours earlier than in the original case, and the ammonium peak load from the CSO at the 20 hour mark (see Figure 20) could be avoided. Correspondingly, the storage tank was emptied earlier according to the simple condition that the inflow rate to the WWTP falls below a certain value. In this case, the storage tank content was now pumped to the main sewer while the morning peak of ammonium occurred, resulting in a serious deterioration of the treatment plant performance. Therefore, what undoubtedly appeared to be an advantage from the CSO point of view, turned into a clearly adverse effect when the impact of the entire system was considered. This example clearly shows that in order to make use of the actual capacities of the system, control needs to be driven by quality measurements, instead of only flow rate measurements. The emptying of the storage tank would then be postponed until the ammonium load into the WWTP was reduced after the morning peak.

4.4. PROPOSED MEASURES AND PRIORITIES OF THEIR IMPLEMENTATION

The low flow rate in Luppmen Creek is one of the key problems. It is primarily caused by competing use of limited resources:

- Groundwater use in Fehraltorf is not sustainable. Both, the exports to neighbouring towns and the specific consumption (average consumption in Fehraltorf was ≈ 500 l/(capita·d) in 1992) should be reduced in order to stop the lowering of the groundwater table and to attain sustainable conditions.
- The upstream diversion of water flow should be restricted to 25 l/s permanently, or even further reduced during low flow periods.
- The clean water inflow to sewers should be significantly reduced (e.g., by disconnecting overflow from all public water fountains).

- A groundwater recharge pond upstream of Fehraltorf, fed during high flow periods and mostly during stormwater runoff, could break the trend of the sinking groundwater table. The main benefits of such an infiltration basin would be an increased base flow, and reduced peak flow rates in Luppmen Creek.

Control measures in the urban catchment, such as rainwater infiltration and control of unpolluted inflow into the sewers, can contribute to reducing the volume and the frequency of CSOs as well as improving the efficiency of the WWTP. These measures are robust and should be implemented even if their immediate benefits seem small.

Additional measures in the sewer system, such as an extended CSO treatment or real time control, must be related to the site-specific situation and should be implemented only after a cost-benefit analysis with respect to identified problems. In Fehraltorf, however, the benefits of such measures without the improvements of the hydrological regime and the local morphological conditions in Luppmen Creek would only be marginal.

The recommendations introduced here have been accepted by the local authorities of Fehraltorf and further investments in the sewer system, according to the former Master Plan of Urban Drainage developed in 1979 (further CSO treatment and enlargement of sewer capacities), were partly cancelled. According to the recommendations given in the EAWAG field study, the local authorities decided in June 1996 to reduce the volume of investment in drainage network from 1.75 million SFr to 0.9 million SFr, so that about 850,000 SFr (2500 SFr/capita) could be saved. These costs indicate that even an extended study of site-specific measures may pay for itself.

5. Conclusions

It is shown by three case studies that the urban drainage needs to be addressed in its full complexity. Pollutants travel through various sub-systems, starting at the households or on the surface of the urban area, are transported in the sewer system, discharged to the receiving water via combined CSOs or WWTPs, and may end up in the river sediment or in the groundwater. An integrated view of the urban drainage system is essential for both analysis of the present situation and for development of remedial measures. The case studies revealed some crucial topics that require consideration in planning and operation as well as in research.

The travel path is highly influenced by **dynamic interactions within and between the sub-systems** during storm events, such as different response times of the urban and the natural catchments, extended wet-weather loading of WWTP when the storage in the system is increased, or infiltration/exfiltration between the receiving water and groundwater. Whatever measure is taken within a single compartment, it will affect processes and conditions in the other compartments through these dynamic interactions. For instance, in the areas with inadequate groundwater recharge, intense groundwater use for drinking water supply may be the determining factor with regard to the ecological quality of the receiving water, particularly during the periods of low flows.

The **interactions between dry and wet-weather conditions** determine to a significant extent the loads on the receiving water during storm events.

- The diurnal variation of the sewage quantity and quality ("dry-weather loading") controls the load produced by CSO and WWTP effluents during wet-weather conditions. The total impact on the receiving water strongly depends on the timing of the rain event start, i.e., the same rain event causes a much stronger impact during the morning sewage peak conditions than during the night hours.
- Particulate matter is accumulated on the catchment surface and in sewers during dry periods and is eroded and washed off during the rising limb of the runoff hydrograph. This phenomenon is also known as the so-called first flush effect.
- A similar effect occurs in the receiving water. Suspended solids are released via the WWTP effluent and may settle and accumulate on the river bed. This sediment layer may be resuspended by the increased river discharge caused by the rain event, and subsequently induce an increase in oxygen demand.

The **cause-effect relations** differ substantially for different groups of materials and chemicals. Particulate matter behaves differently from dissolved matter, and within these major groups, there is a variety of transport and transformation mechanisms. Accordingly, pollution control during storm events must target specific pollutants, depending on the relative significance of their impacts. For example, the reduction in some particulate-bound pollutants may be accomplished most effectively by improved wastewater treatment, whereas increased detention storage may be more effective for controlling some dissolved pollutants.

In many cases, the most detrimental impacts on the receiving water ecosystem may be related to the river **morphology**, rather than the short-term impacts of the drainage system. For a typical urban stream, even flow enhancement with water of drinking water quality (at CSOs and WWTP outfalls) would not help the ecosystem to approach a typical reference state. Variations of width and depth (pool/riffle sequences), shading, water exchange between the water body and the riparian zone improve the diversity of the community and its resistance against quantitative and qualitative storm water impacts.

The **hydrology** is important with regard to both minimum and maximum flow rates in the receiving water. In urban streams, the flow rate falls below the value required for a minimum ecological quality more frequently than in more natural conditions. This may be caused by one of several reasons. The rainwater may be discharged to the stream only at the downstream end of the settlement area, and there may be impacts due to various uses of the receiving waters. Alternatively, because of the lowered groundwater table, the groundwater recharge by the infiltrating stream water reduces the flow rate in the stream. Whereas factors like the annual overflow volumes are hardly significant for assessing the true impacts on a receiving water, individual peak discharges may well be important. Depending on the discharge ratio between CSOs and the river, CSOs may often cause a drastic change in the frequency of **gravel transport** and increase the stress on the river biota.

Generally, **source controls** should be evaluated as an important alternative to structural measures. Changing materials and practices, or revitalising channelised rivers, takes time, but in the long run, it may be a superior solution. The introduction of lead-free gasoline is a much more effective measure to keep lead out of the surface waters and groundwater than any sophisticated wastewater treatment process.

As shown by the examples given above, the case studies indicate the importance of a broad view when examining the urban drainage system. They also stress the important fact that problem-specific measures are not only more efficient in improving the receiving water quality, but they are also more cost-effective.

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URBAN DRAINAGE AS A PART OF RIVER BASIN MANAGEMENT

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1. Introduction

Water in urban areas, and urban storm drainage as a part of the urban infrastructure, are topics which are gaining in importance in recent years. The reasons for this are the obvious needs for an integrated approach to urban water management, and increased public awareness of the pollution caused by urban effluents, which affect both the urban areas themselves and the receiving water bodies.

Although cities are in contact with water from various origins (ground water, streams flowing through or near the city etc.), the major concern of this paper is water originating in the city area itself, that is, our concern is with water from local rainfall (urban storm runoff) and its interaction with the water originating from the rest of the river basin.

Methods for protection against flooding by local storms and for assessment of the effects of pollution transported by storms on receiving waters have been significantly improved during the past two decades with the introduction of computer based simulation, design, optimisation, real time control and management. The achievements of modern informatics (i.e., a higher level of information processing) have made a significant impact on all aspects of problem solving.

In modern societies, the status of urban drainage as a part of the integrated infrastructure system varies from one country to another, depending primarily on the level of development and public awareness of the importance of this problem. In general the importance of the system increases with the level of development, but there are also exceptions. The awareness of wet-weather pollution has rapidly increased in recent years. The drainage systems, which used to have a simple function of collecting storm water and conveying it to the nearest point of disposal, have gradually evolved and gained in importance as their role has changed. Their study now covers such important fields as urban flood protection and control, and pollution control and management.

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In densely developed countries (UK, Germany, some parts of the USA, Japan, etc.), urban drainage consumes a high proportion of the investment into the urban infrastructure. The change of the role of urban storm drainage (USD) and developments of information processing technology have created a need for new tools and products to be used in the problem solving procedure. At the level of a river basin, urban drainage is just one part (element) in the integrated water management cycle. In this paper, emphasis is placed on the interaction of urban drainage with activities and processes at the river basin level. Both technical and non-technical aspects will be discussed.

2. River Basin as a Water Resource Planning Unit

In both ancient and modern societies, the river basin has been considered an entity that determines both the range and depth of human activities with respect to water. The catchment is used as a unit for planning and management of not only water, but also of other resources, and human and economic activities. In the case of urban drainage of a particular city, the relevance of a catchment is greater the smaller the catchment and decreases as the size of the catchment increases. This is because the relative effect of the quantity and quality of runoff water generated by that particular drainage system diminishes with the size of the catchment and with the distance from the point of storm water disposal. However, the integrated effect of all storm drainage systems contributing to the balance of surface water and to the flux of suspended sediment and other pollutants has to be taken into account at the level of the river basin or the sub-basin upstream of the point under consideration. The interaction of the storm drainage system with downstream municipalities and water users is strong when the drainage peak flow uses up the capacity of the river channel, so that there is no room left for downstream runoff. In these cases, the downstream-upstream relationships and links have to be analysed to either share the existing capacity or to share the costs of its enlargement. Small river basins are therefore more sensitive to this problem and are analysed in the following discussion. On the other hand, the rivers carrying water from large catchments serve as receiving waters for both solid and dissolved pollutants. The effect of urban storm water disposal has to be analysed from the point of view of its polluting effect and contribution to the silting of downstream waters, including reservoirs.

Alterations to the natural water balance within the catchment area can have both positive and adverse effects on the upstream and downstream water users. In that respect, integrated planning and design of urban drainage systems require that both effects be analysed and an unbiased assessment be made in all phases of the planning and management process.

3. Principles of Integrated Water Management Relevant to Urban Drainage Projects

The general goal of integrated water management is a sustainable utilisation of water resources respecting the social, economic and environmental interests. The linkages between society, economy and environment are shown in Figure 1. Considering the close interrelationship between society and its economy, the first two groups are usually aggregated into socio-economic issues. It should be recognised that the goals and objectives of integrated water management are formulated at various spatial scales involving all three components.

The fundamental quality of integrated water management is holistic in nature, which recognises the system complexity and the inter-connectivity of its elements, demonstrated by the exchange of information, energy and matter, and the style of planning actions. The holistic approach also equally involves local and regional authorities, engineers and natural scientists, environmentalists and decision makers, politicians of all parties, both governing and in opposition, as well as the people affected (Geiger, 1994; Geiger and Becker, 1997). The holistic approach is sometimes called the ecosystem approach. Sustainable water management ensures that no substances are accumulated or energy lost, using recovery and reuse techniques. This approach requires novel, environmentally sound technologies.

In the context of urban and industrial water resources, the most pertinent water uses are water supply (safe, reliable and equitable), drainage and flood protection (affordable), sanitation with maximum reuse, recreation (protecting public health), aesthetic and cultural values, and ecosystem health.

The above objectives do not receive the same priorities. At different stages of social development, different objectives receive more importance. In pre-industrial society, emphasis was placed on drinking water supply, transportation and water supply for irrigation. In industrial society, generation of hydropower and waste disposal and transport are prioritised. Finally, in post-industrial society, high emphasis is placed on aesthetics and ecology. In this period, it is expected that the damage to nature (environment) in previous periods will have to be corrected in the years to come (Hedgcock and Mouritz, 1991). It is expected that the time needed to clean up the environment polluted in the industrial period will require approximately the same time and significant resources.

Water management in urban areas strives to meet the demands of economic growth by optimal development and utilisation of water resources and to create an optimal living environment, which would protect the society against harmful effect of water. These objectives emphasise two groups of interests: society and economics. Only after introducing a third group of interests (environment including air, land, water and biota, and excluding people), can a truly integrated approach be achieved. For a holistic approach, all objectives must be developed on a river basin scale by a stakeholder group broadly representing social, economic and environmental interests. Table 1 compares various water management approaches.

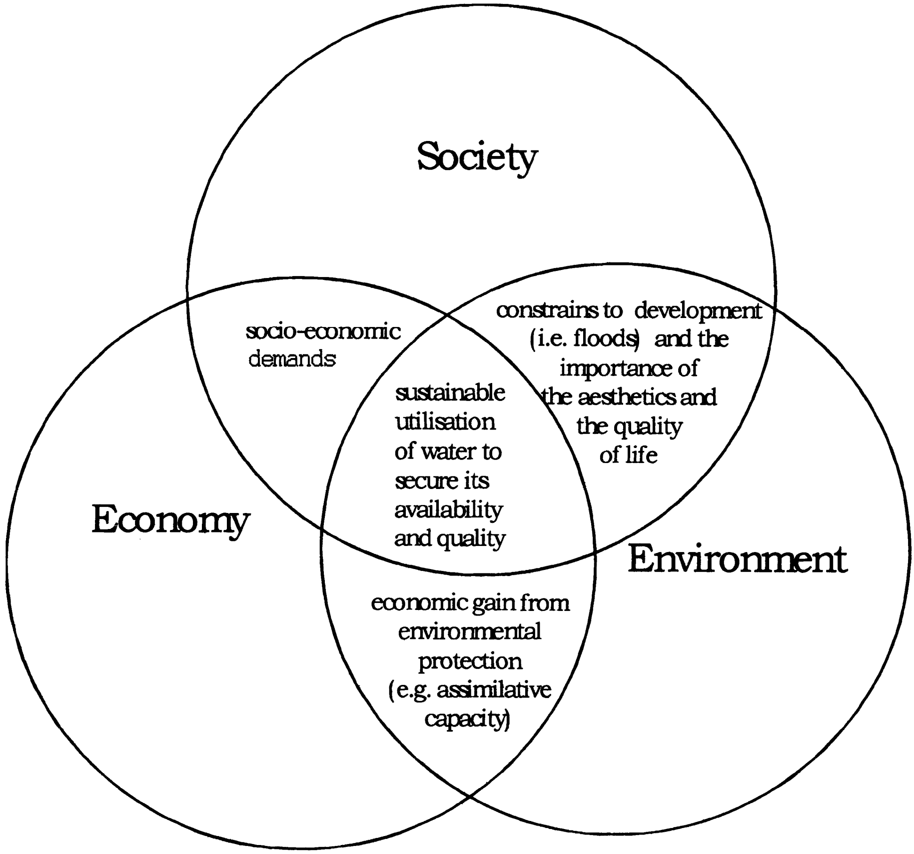

Figure 1. Linkage of society, economy and ecology (UNDP ST/ESCAP/933).

It has been widely accepted (WMO, undated) that the major driving water force for the mobilisation of resources for solving the urban flood problem at both catchment and city levels are the losses caused by floods and droughts as well as by large scale natural and man enhanced disasters. In a holistic or integrated approach to planning and management of water resources at the level of the river basin, one should aim at comprehending the following (Frieling, 1993):

a. integration of water engineering and technology, urban and rural planning with general public policy.
b. integration of planning and design of a regional (catchment in our case) level, local and microlevels (urban subcatchment), with active participation of owners and the general public,
c. integrated approach by using a common frame of reference.

A broader framework for this concept was presented earlier in Chapter 1.

TABLE 1. Different approaches in urban water projects (Geiger 1994)

Technical objective	Piecemeal approach	Environmental approach	Holistic/Ecosystem approach
Water supply	Transport water over great distance (disregarding impacts at the source)	Provide advanced treatment, no management of demands	Manage water demands by water conservation; recycling of industrial water, water reuse; provide treatment
Wastewater transport and disposal	Develop treatment regulations for various sources and individual constituents (more or less independently)	No management of generation of wastewater, develop regulations, plans for local receiving water body considering all pollution sources and constituents implement treatment	Reduce generation of wastewater through conservation, recycling; enhance self-purification of natural water bodies (by restoration of biota); provide some treatment
Protection against local flooding	Zero-runoff increase regulation - for individual developments	Develop master drainage plans, build drainage channels, pipes to convey flows	Reduce runoff generation by source controls, view storm water as resource - use in irrigation, water supply; for limited runoff flows - provide treatment in ponds and wetlands.

4. Contribution of Urban Drainage to Water Balance in Small and Large Catchments

In the analysis of contributions of urban storm drainage projects to the conflicts and uncertainty in water resources plans at a river basin level, one has to take into consideration the ways in which the existing urban structures, their features, and the newly planned drainage elements affect the balance of water in a particular urban area. A general scheme of the water balance in an area undergoing the transition from rural to urban land use is shown in Fig. 2.

The major difference between the urban and rural (or natural) part of a river basin, in terms of the water balance, is the reduced infiltration potential of urban areas and the faster response of surface runoff that is generated. A mutual interaction of urban runoff and flows in adjacent streams is shown in Fig. 3. Water running from the upstream parts of catchments flows either through the city's regulated stream, or through its system of urban drainage infrastructure. The major difference in approach to integrated solutions is indicated by the ratio of the urban peak flow to the flow in the receiving stream at the downstream end of the urban area. The forms of urban flooding caused by other man made and natural disasters (Iwasa and Inoue, 1987) also have to be taken into account.

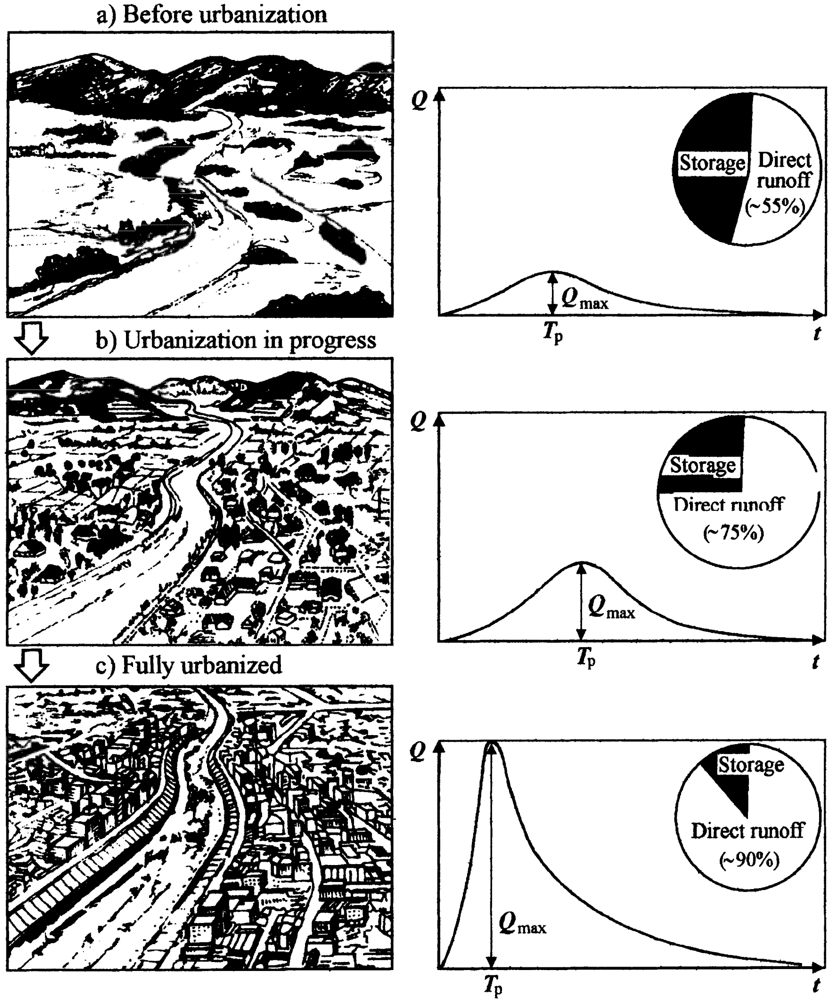

Figure 2. Surface runoff water balance in areas undergoing urbanisation (MoCoJ, 1990).

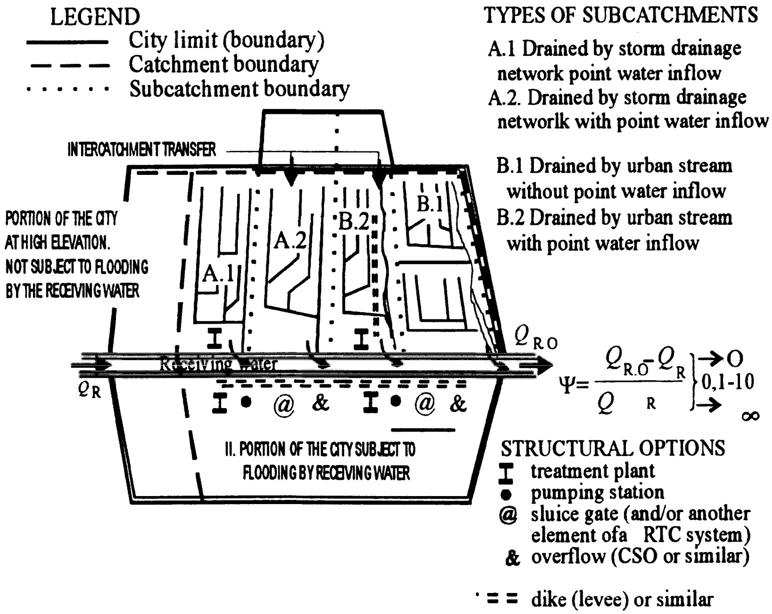

Figure 3. Classification of urban sub-catchments and interaction between urban runoff and the adjacent river.

As an example consider the Danube, as a large river flowing through the large cities of Vienna, Bratislava, Budapest, Belgrade, etc. In the most extreme events of heavy storms over these cities, the local runoff contributes only a very small proportion of the flow in the river, and one can claim that the management of urban storm drainage system in these cities will not affect the flow in the Danube. In that respect, and with reference to water quantity, these systems do not affect the peak flow in the receiving water and can therefore be designed independently. However, small streams near large cities are strongly affected by the urban runoff peak flows. In many cases, the peak flow generated by urban runoff is comparable to the conveyance capacity of the receiving stream. In these cases, the management of urban drainage has a significant effect on the receiving water and its downstream reaches. Consequently, the solution for the particular storm drainage problems has to be developed at the catchment level, and in an integrated way. However, growing concerns over the quality of surface runoff require that the interaction of particular city's pollution load is addressed in conjunction with other pollution contributions from both upstream and downstream urban areas.

5. Conflicts between the Upstream and Downstream Water Users and their Resolution by Water Management

5.1. EFFECT OF URBANISATION

A major cause of increased runoff from urban areas is the increase in impervious areas. This effect is illustrated in Fig. 2.

However, the effect of urban flooding can be considered from two points of view:

1. local floods in the cities and in small streams delivering water to large receiving waters, and
2. floods in major streams serving as receiving waters for effluents from the whole urban and suburban areas.

Two examples are shown in Figs. 4 and 5. Figure 4 depicts the increase in urbanisation of the Tsurumi River basin, in which the suburban areas of Tokyo have been fully urbanised within a short period of about 25 years. The Tsurumi River has been converted into an integral part of the urban drainage system, and proper comprehensive flood mitigation measures, such as those listed in Table 2, had to be planned and executed.

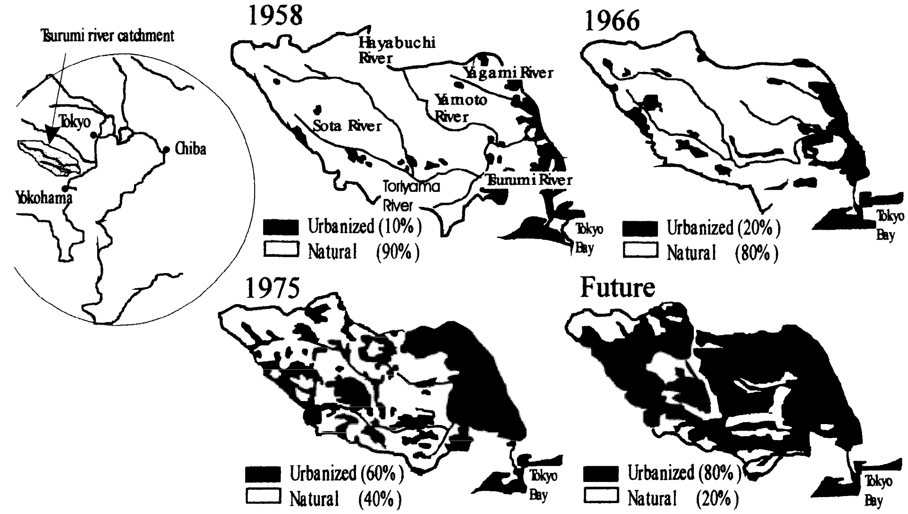

Figure 4. Urbanisation of the Tsurumi river basin.

TABLE 2. Comprehensive flood plain management of the Tsurumi river basin

<table>
<tr><td rowspan="4">River basin</td><td rowspan="3">Water retention area</td><td>Natural area
1. Preservation of urbanisation restraint district
2. Preservation of natural retention function by other acts</td></tr>
<tr><td>Developing area
1. Installation of rainfall retention facilities</td></tr>
<tr><td>Developed area
1. Installation of rainfall retention facilities in public compounds
<table>
<tr><th>Area</th><th>City</th><th>Required capacity of rainfall retention (m³)</th></tr>
<tr><td>Kanagawa Pref.</td><td>Yokohama</td><td>1,533,200</td></tr>
<tr><td></td><td>Kawasaki</td><td>274,000</td></tr>
<tr><td>Metropolitan</td><td>Machida</td><td>430,000</td></tr>
<tr><td>TOTAL</td><td></td><td>2,237,800</td></tr>
</table></td></tr>
<tr><td>Water storage area</td><td>1. Preservation of urbanisation restraint district designated by the Urban Planning Act
2. Encouragement of farming activities
3. Constraint of landfill to maintain storage function</td></tr>
<tr><td></td><td>Low-lying area</td><td>1. Suspension of pumping operation when the river water level exceeds the design high water level
2. Installation of rainfall retention facility</td></tr>
<tr><td rowspan="2">River</td><td>Riverside water</td><td>1. River improvement to control a rainfall of 50 mm/hr (upper stretch of Ochiai Bridge)
2. River improvement to control a discharge of 950 m³/s (lower stretch of Ochiai Bridge)
3. Provision of Retarding Basin</td></tr>
<tr><td>Landside water</td><td>1. Implementation of landside water drainage plan</td></tr>
<tr><td>Other measures</td><td></td><td>1. Improvement of warning and evacuation system
2. Strengthening of flood protection system
3. Publication of past flood map
4. Recommendation of flood-proofing
5. Enlightenment of awareness of flood loss mitigation</td></tr>
</table>

On the other hand, the City of Belgrade, which underwent urbanisation as shown in Figure 5, experienced only the problems caused by local storms in small streams delivering storm water to the Danube, or its large tributary, the Sava River (Jovanovic 1986).

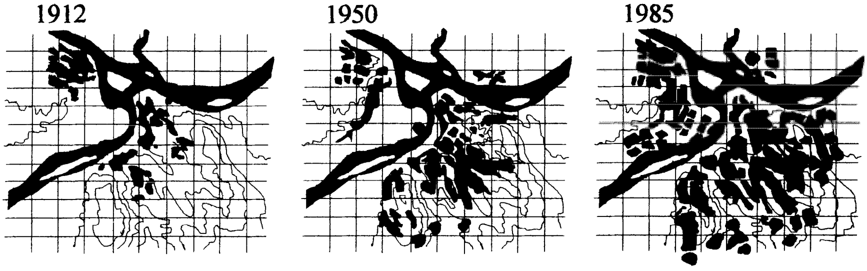

Figure 5. Urbanisation of the City of Belgrade.

The difference in the capacity of the main receiving water calls for classification of the concepts of storm drainage solutions. Such a classification depends on the ratio of the peak flows likely to occur at the point of disposal (end of pipe) to the average discharge in the receiving stream (shown in Figure 3.)

$$\psi = \frac{Q_{R_0} - Q_R}{Q_R} \tag{1}$$

where : Q_R = inflow at the upstream end of the urbanised area
Q_{R_0} = outlet flow at the downstream end of the urbanised area

5.2. FLOODS VS. WATER SHORTAGE AND DROUGTS CAUSED BY URBAN STORM DRAINAGE

Urban drainage management, as a part of integrated water management at the river basin level, requires that the changes in water balance are also addressed. Two aspects are dealt with: direct runoff and enhanced infiltration.

An increased runoff jeopardises downstream water use by the threat of flooding (due to an increase in its frequency and in water level) and this aspect is well documented. Alternatively, the application of source control measures (enhanced infiltration; Ichikawa and Maksimovic, 1988) can reduce downstream flows, but if applied on a large scale it can lead to water shortage.

5.3. WATER QUALITY

Downgrading of water quality in the receiving water bodies by urban storm drainage results from the interaction with other point and diffused sources of pollution in urban areas. In the river reaches downstream from the urban area, these effects cannot be separated unless separate monitoring of the major pollution sources is performed. Thus, the reduction of pollution, at an integrated scale (urban area), is one of the recommended approaches.

In the rehabilitation of existing storm drainage systems, an integrated approach could comprise several actions, both at the city level and in the catchment, such as:

- pollution control at source and retrofitting the existing detention facilities to include quality improvements,
- reduction of leakage from sewers,
- improvement of the network performance so that the CSO spills are reduced,
- other remedial actions that are shown in Table 3.

TABLE 3. Remedial actions and their evaluation

Remedy	Problems			
	Eutrophication	Nuisance caused by CSO's	Pond beds falling dry	Disturbance of ecosystem in pond
Treatment of low and intermediate storm - water flows in sewerage treatment plant	+			+
Moving overflow structure of combined system to less sensitive location	+	++		++
Control duck population, stop duck/fish feeding, etc.	0			0
Water treatment using macrophytes	++	+		++
Maintenance dredging and possible minor deepening	+		++	++
Phosphate fixation in bottom	0			0
Aeration in combination with dredging	+			+
Water supply for control of water level and flushing	?	?	?	?
Construction of sloping banks planted with vegetation that is adjusted to water level fluctuations, removal of barriers for fish migration				++
Biomanipulation				(++)

++ recommended measure
+ beneficial measure, but not sufficiently effective
0 not feasible nor effective

? requires further investigation
( ) possible supplementary measure to be decided upon at a later stage

6. The Possible Scenaria

6.1. NO ACTION

In many cases when the resources are limited, undertaking no action (which usually means no structural measures and projects) seems to be the only solution. However, this situation can be improved if at least a minimum program of monitoring and analysis, combined with the affordable non-structural measures, is done. For example, raising the population awareness of the potential benefits of non-structural measures helps implementation of such measures, which can keep the situation under control and reduce risk and damages in flood prone areas (Frieling 1993).

6.2. SEPARATE SOLUTION FOR EACH CITY (LARGE RIVERS)

As mentioned above, in large river basins, peak flows caused by runoff from urban areas do not cause significant changes in the receiving water level. Thus, it could be considered that in such cases the planning and design of storm drainage systems could be undertaken at the city level without taking into account the interaction with the upstream or downstream settlements. However, since the integrated solutions comprise the use of water for other purposes, water quality issues have to be addressed. Most of the existing storm drainage projects have been based on this concept.

6.3. INTEGRATED RIVER BASIN SOLUTION (SMALL CATCHMENTS)

In small catchments the capacity of receiving waters is relatively small and any significant increase of peak flows can cause flooding in downstream areas. In these cases, the measures, such as those shown in Fig. 6, have to be considered. They include two groups:

a. structural measures
b. non-structural measures

that are applied in two different environments:

a. in urban areas, or
b. in the catchment

The schematic presentation of the above measures and their evolution in time is shown in Fig. 6. It is assumed that the process of planning and construction of the flood mitigation facilities, including storm drainage, is intermittent; once the capacity of the existing system is surpassed, new construction or rehabilitation will take place. It is also assumed that peak flows (or total volumes) of surface runoff increase with time due to urbanisation.

The situation will be different when the source control measures are applied on a larger scale and when the process of urbanisation does not create an immediate increase of peak flows. In these cases, rehabilitation of the existing old systems should result in decrease of runoff flows, or at least in keeping them at the same level.

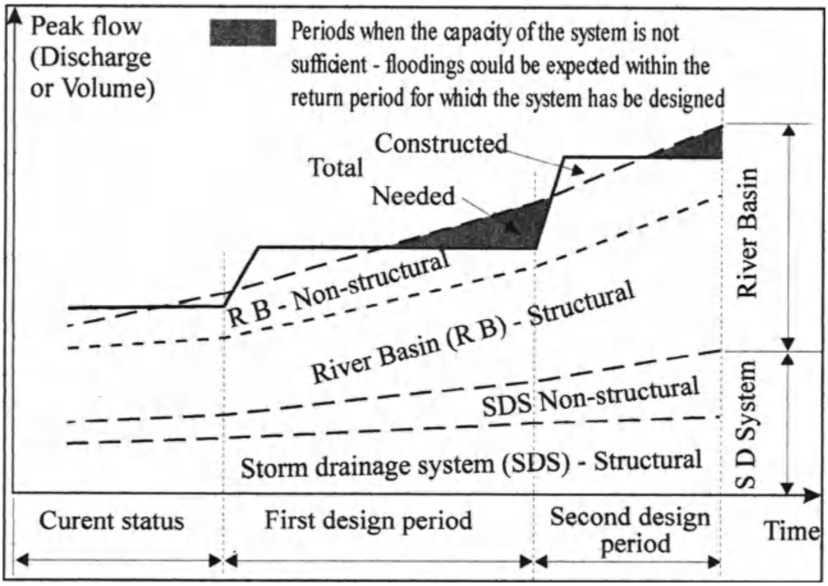

Figure 6. Redistribution of peak flows between a river basin storm drainage system and structural and non-structural measures.

The range of structural and non-structural measures applied in river basins is shown in Table 4.

TABLE 4. Structural and non structural alternatives for flood mitigation

Type of measure	Major advantages	Major disadvantages
A. **Structural alternatives** 1. Levies and floodwalls	Provides localised protection Relatively low initial construction costs Useful measure against flooding up to the design level Permits further development of existing urban areas	Usually requires land resumption Can divert floodwater to other areas and raise flood levels High maintenance costs High damage occur when structures are overtopped Can cause false sense of security in protected population Can have adverse environmental impact
2. Channel modification / By-pass flood-ways	Increases main stream capacity Provides localised protection Small land requirements Effects of construction are localised	In built-up areas land costs can be expensive Can cause problems of erosion and sedimentation Construction costs may be high if existing infrastructure has to be modified Can transfer flood problem to other areas
3. Retarding basins and flood storage areas	Natural flood storage areas are used for flood mitigation purposes Reduction of downstream discharges Flooded area used for productive agricultural purposes during dry periods Simple and cheap to construct	Areas suffer damage when flooded Local inhabitants can be at risk unless operation is properly controlled Effectiveness is limited by available storage capacity and residual drainage facilities

4. Flood mitigation reservoirs	Reduce downstream discharges Provide protection to existing development	Society as a whole bears cost while flood-plain users enjoy major benefit High construction costs Requires resumption of land for reservoir storage and possible displacement of local population Can create false sense of security within downstream communities Can cause environmental problems such as water quality degradation and siltation
5. Drainage evaluation systems	Provide protection against local flooding in areas protected by levees	High maintenance and operating costs
B. **Non-structural alternatives** 1. Land use management Zoning controls Building and development controls	Reduce potential hazard and losses Permitted land use is consistent with flood hazard Ensure that the flood problem is not worsened by new structure in flooded areas Environmental character of the area is largely preserved	Can restrict legitimate development if controls are not consistent with flood hazard Individual landowners are required to meet costs Controls can be too restrictive
2. Property acquisition and flood-way clearance	Removal of obstructions can lead to the free flow of floodwater and reduced flood levels Vulnerable development removed from flood hazard area Reduces post-flood rehabilitation costs	May not be readily accepted by residents Acquisition costs can be inordinately high if carried too far Needs to be undertaken in conjunction with a re-settlement programme if squatter populations are involved
3. Modification of catchment conditions	Can reduce environmental factors by reducing erosion and silt transport Can reduce downstream discharge by detaining rainfall	Effectiveness reduces as flood magnitude increases Individual landholders and developers are required to implement programme and meet costs
4. On-site storage	Retains floodwater at point of origin reduced downstream flood peak Relatively cheap to construct	Opportunity to implement on large scale is limited Ineffective on large catchments Landowner is usually required to meet cost
5. Flood forecasting and warning systems	Minimises damage and danger to life Alerts general public to take actions Generally cheap and quick to install Useful in conjunction with other measures	Feasible only for catchments with long response times Limited usefulness for small catchments Tends to be ignored if constantly inaccurate Needs to be integrated with other measures
6. Public information and education	Creates an awareness of flood hazard confronting communities Generates more ready acceptance of flood loss prevention measure	Can be a time consuming task for flood authority Can be unproductive if community acceptance is negative
7. Flood proofing of buildings	Reduces post flood clean-up operations Especially suited to commercial and industrial buildings	Suited only to particular types of buildings Can result in high losses in flood height is exceeded Landowner meets cost
8. Evaluation from endangered areas	Reduced loss of life Generally cheap to introduce	Requires effective flood warning system Requires detailed planning Public awareness must be maintained

9. Flood fighting	Reduces adverse impacts of flooding including casualties and damages	Requires effective flood warning system Requires detailed planning and trained personnel High cost to Government
10. Flood relief	Reduces financial burden on affected individuals Reduces impact to post flood problems	Funds are provided by entire community
11. Flood insurance	Can be helpful to cover individual flood loss or damage Can reduce the total amount of flood relief funds provided to victims Places responsibility on individuals who choose to reside in flood hazard area	Hard to obtain from private insurance companies Can encourage continued occupation of the flood-plain National flood insurance schemes require use of public funds
12. Flood adoption	Reduces potential flood losses to individuals Individuals meet their own costs	Only applicable in non-hazardous areas May not provide protection against major flood conditions

Traditionally, retention facilities have been designed so that they reduce peak flows. In recent years, a lot of effort has been put into improving water quality as well. For this purpose, use is made of aquatic plants capable of consuming most of the nutrients present in these waters and taking up some of the toxic pollutants (Moshiri, 1993). An example is shown in Figure 7.

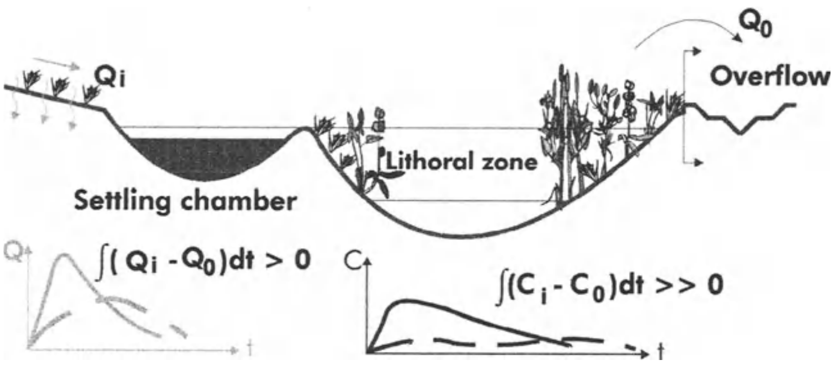

Figure 7. An example of retention basin retrofitted for storm water quality enhancement.

7. New Technologies Applied in an Integrated Solution

7.1. GIS IN RESOURCES ASSESSMENT

Defining geographic information system (GIS) today is like trying to catch a cricket: No matter how fast you move to try to keep up with it, it moves faster (Parker, 1987). However, a GIS is an information system that is designed to work with data referenced by spatial or geographic co-ordinates. In other words, a GIS is both a database system with specific capabilities of spatial-referenced data, as well a set of operations for working with the data (Maksimovic, 1992).

Most institutions dealing with water resources management live in a world of tabular data. These data are not usually linked in any significant way to other databases. A researcher cannot query the database about water quality and link the results with information about surrounding land use, soil types, or other environmental and cultural factors. This greatly limits the researcher's ability to perform sophisticated analyses in a timely fashion. There is a strong need for systems which allow for the graphic display of geographically referenced data and permit a large number of data types to be linked together without spending much time on programming. For historical reasons these systems are called GIS (Parker, 1987). The major advantage of GIS's, when compared to the traditional methods used in planning, is the remarkable speed with which the researcher is able to search through large volumes of data and generate output that matches user-specified criteria. This speed allows a decision maker to consider several alternatives based upon different criteria for a given problem.

When this general framework is applied to water resources planning at the level of the river basin and bearing in mind that this paper deals with urban drainage, several possible paths for application of GIS technologies emerge, such as:

- presentation of the existing surface and ground waters in an "easy to manipulate" form,
- performing analysis of various thematic maps and of other geo-referenced sources of information,
- quick presentation of the intermediate results and matching them with other sources of data for further analyses and presentation of final results in readable forms.

7.2. NEW INTEGRATED SIMULATION PACKAGES

Models traditionally used in urban drainage have had relatively poor graphical support and limited possibilities for inspecting their results in a suitable form. Recent developments in interfaces and gluing routines, matched to the earlier or improved versions of storm drainage simulation packages, have enabled more comprehensive analyses to be performed quicker and options for further solutions to be conceptualised and checked faster. However, further developments are needed for the analysis of urban storm drainage systems as an integrated part of the comprehensive water management plans at river basin level (Maksimovic *et al.*, 1995).

7.3. MATCHING URBAN DRAINAGE MODELS WITH SURFACE NETWORKS

In cases where storm water is not immediately discharged into a large receiving water body, but has to be conveyed through a network of surface channels, with or without interaction with other water resources, it is necessary to match the quality and quantity outputs from urban drainage systems with the transport network. In integrated solutions the mutual interaction of both quality and quantity has to be analysed. Application of both structural and non-structural measures requires that their effects are analysed as well. The incorporation of quality improvement facilities, such as retention ponds with aquatic plants, etc., calls for new integrated tools to be developed and applied in analysis of integrated effects. An example dealing with analysis of possible storm water harvesting in Israel is shown in Figure 8, and for the City of Hadera, Figure 9.

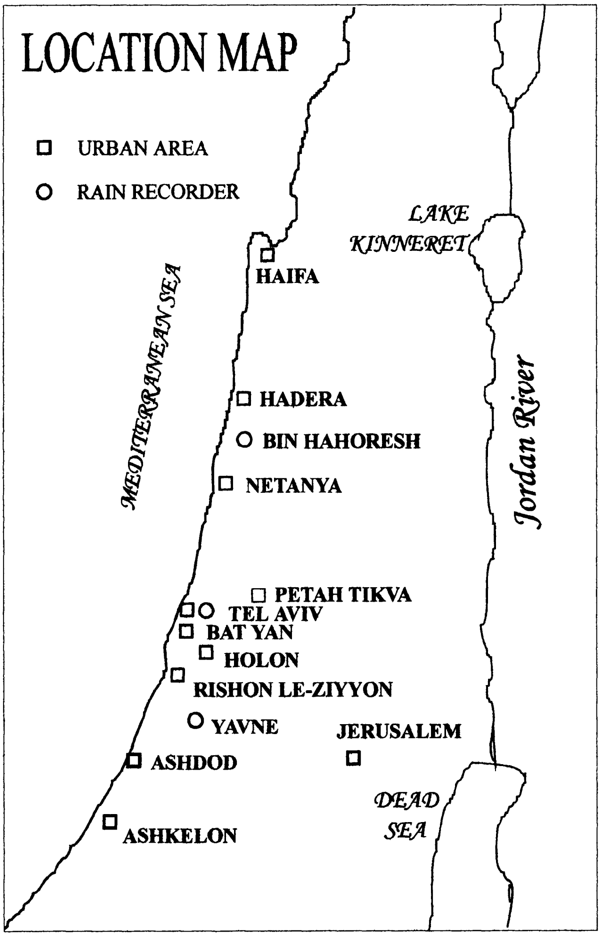

Figure 8. Location of places in Israel studied for possible storm water harvesting [Simon 1995].

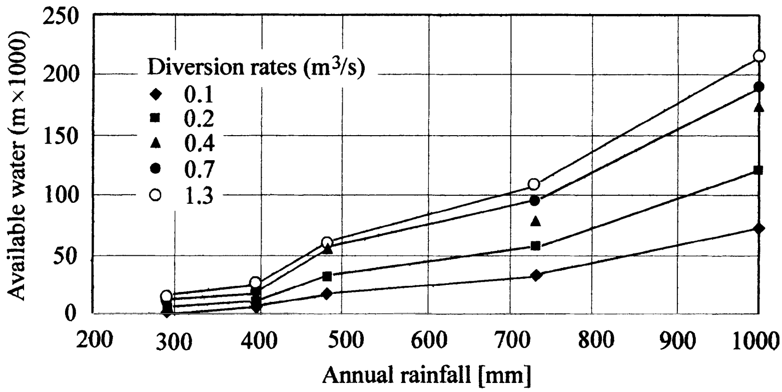

Figure 9. Available storm water for harvesting as a function of annual rainfall in Hadera [Simon 1995].

8. Third World and Developing Countries

The period before the mainframe computers were introduced was characterised by mass application of simple methods (such as the rational formula and unit hydrograph) for the assessment of peak flows. The design methodologies were based on these peak flows.

Even after introduction of mainframe and micro-computers, most of the programs have been based on the same simplified assumptions and methods which were simply embodied into computer programs. Even now, the simplified methods based on the rational formula, unit hydrograph, time area diagram, etc., continue to survive and be used in the same programs. Despite a relatively high level of development of physically-based simulation models, their application in full scale projects is far from being satisfactory, except for rare cases, such as in the UK. In general the classical methods of design without a proper application of the advanced simulation models still prevails.

In developing countries the full recognition of the potential of urban storm drainage has not yet been reached, because of lack of funding and low awareness of the pollution potential and importance of flood protection.

During the last few decades significant "bursts" of monitoring in experimental catchments have been experienced in several countries. Data collected have been used for various purposes, starting with the assessment of flooding and pollution, support for the development and calibration of simulation models, and ending with comprehensive databases serving multi-objective purposes. The UDM and UDM-Italian databases are the products of some of the monitoring programs.

9. Future Conflicts and Research and Development Needs

A number of research needs and conflicts to be dealt with in the future are highlighted:

- incorporation of urban storm drainage systems into integrated water resources planning at the river basin level,
- development of concepts and tools for analyses, planning and use of real time control in integrated solutions,
- water harvesting, recycling, rational use and other conservation measures aimed at the reduction of waste water generation,
- interaction with other surface and ground water resources, in terms of both quality and quantity,
- search for appropriate technologies for developing countries, and countries in transition with limited investment potential,
- provision for sustainability and durability,
- incorporation of a comprehensive approach to the daily routine,
- working with other specialists within common terms of reference.

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