# Optimized Control of Urban Drainage Systems

Fons Nelen

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# Optimized Control of Urban Drainage Systems

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# Abstract

*Nelen, A.J.M., (1992), Optimized Control of Urban Drainage Systems, Dissertation, Delft University of Technology, Department of Sanitary Engineering & Water Management, the Netherlands, 206 pp., 49 Figures, 11 Tables, Summary, discussion and conclusions in English and Dutch.*

This thesis describes the development and a number of applications of a model designed to assess the performance of an urban drainage system that is controlled in real time.

The conventional solution to urban drainage problems employs a system with a certain storage, transport and treatment capacity. The limited efficiency of this solution is caused by the lack in flexibility in operating the system under dynamic loading. Due to the systems dynamics and the distribution of available capacities within the system, optimum performance can only be achieved by means of real time control of the system. The potential of systems control should be considered not only after the system has been constructed, but in the design phase of the system as well, as it will influence the required capacities of the system components. The model developed can be used to assess this potential on the basis of simulation of time series of rain events.

The operational optimization problem is formulated as a non-linear programming problem. To ensure a proper systems performance for each possible loading, the objective function to be minimized is formulated as a function of the current systems state. This feature is essential when calculating time series of rain events. The non-linear problem is solved by replacing it by a succesion of linear (sub)problems. This means that at each time step of the simulated hydrograph the problem is transformed into a linear programming problem, which can be dealt with by a network flow algorithm.

The optimization routine is incorporated in a newly developed modelling package, called LOCUS, which is an acronym of 'Local versus Optimal Control of Urban drainage Systems'. The name denotes that besides the simulation of optimal controlled systems, the possibility is included to simulate local (or static) controlled systems as well, i.e. using flow regulators with a fixed stage-discharge relationship (i.e. the present way of operation of most urban drainage systems). Since both models are based on an identical system description, the difference between the results is due only to the way the system is operated and hence the effects of real time control can be quantified by comparing the results.

Four case studies conducted with the LOCUS package are presented. From the various aspects that may contribute to the potential of real time control, some have been selected for further assessment. These are possible gains concerning the required system capacities, the importance of predicting inflows and effects of prediction errors on the operation strategy, possibilities of controlling the system based on pollution parameters, effects of rainfall distribution and the possibilities of reducing the peak flow to the treatment plant. Proceeding from the results of the case studies, the role of real time control in (future) urban drainage design is discussed.

# Acknowledgements

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I would like to thank all those who supported and assisted me during this study. To begin with, I would like to express my gratitude to Professor Wil Segeren for his willingness to act as a promotor. His encouraging support, our discussions, but also the freedom he gave me in developing own ideas are highly appreciated.

The thesis has gained much from the discussions with my collegues at the University. I would like to thank Wytze Schuurmans for his valuable comments. The discussions at the University, which often were continued on our way home or even during the evening hours in Amsterdam, were not only fruitful but also most enjoyable. The comments and stimulating remarks of Elgard van Leeuwen are much appreciated. I would like to thank Frans van de Ven and Paul Ankum for proofreading the report. Betty Rothfusz is acknowledged for her practical assistance at the Department.

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Finally I would like to thank Andre Smirnov for correcting the English of the preliminary version of this report. Hopefully I did not destroy his work in preparing the final version.

# About the author

The author was born in Breda, the Netherlands, on 12 april 1960. He studied at the Delft University of Technology (DUT), Faculty of Civil Engineering, from 1978 to 1986. He graduated in the Department of Sanitary Engineering & Water Management, with the MSc theses 'Flood Control in Cebu, Philippines' as first and 'Real Time Control of Urban Discharges to Receiving Polder Waters' as second subject. In 1984 he worked 5 months in the Philippines in conjunction with the Water Resources Centre of the University of Cebu. After graduation he was employed as researcher at the Department of Sanitary Engineering & Water Management to evaluate the performance of the real time control system of the drainage system of Westfriesland. This research was carried out in co-operation with the waterboard 'Hoogheemraadschap Uitwaterende Sluizen' and the National Institute for Inland Water Resources (RIZA). In 1988 he continued his work at the DUT with the research on Optimized Control of Urban Drainage Systems, which was supported by RIZA, STOWA and DHV Water BV.

In addition to his research work he gave lectures in several courses on urban and rural water management. He participated in the development of a new lecture series at the DUT on 'Control of Water Management Systems'. He was guest lecturer in several national and international (post-academic) courses. He was co-founder of the Centre for Operational Watermanagement (COW), a non-profit organisation linked to the DUT that carries out research in the field of Operational Water Management.

Since July 1, he is employed at DHV Environment & Infrastructure BV. One day a week he continues his research and lecturer activities at the Delft University of Technology.

*To my parents*

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# 1 Scope of the study

## 1.1 Introduction

This study deals with the operation of urban storm drainage systems. A model is developed to assess the possibilities of improving the systems performance by means of real time control, i.e. where set points are derived on the basis of currently monitored process data. Although the model may conceivably have a wider application, the study is essentially based on combined sewer systems as mostly applied in the Netherlands (and other flat areas in Western Europe). This restriction is mainly due to the choice of the operational objectives that are considered and the flow model applied. However, the underlying principles of the model may be valid for all kinds of water resources systems.

To begin with, this chapter provides a glance on the development of urban storm drainage technology leading to today's situation. For a more detailed historic review reference is made to (Chow, 1962), (Koot, 1977) and (Yen, 1987). Next the situation in the Netherlands is discussed, followed by an elaboration of the operational problem.

## 1.2 Development of urban drainage technology

Since times immemorial, people have constructed hydraulic systems for the disposal of storm water in urban environment. Typical examples are the urban drainage systems which were built more then 2000 years ago in Europe during the Roman Empire and in China during the Han Dynasty. Although the constructions of these ancient systems are admirable by themselves, it can safely be stated that in those days engineering was an 'art' based on experience, rather than a 'science' based on knowledge of the laws of physics. The evolution of urban storm drainage technology may be considered to have begun around the middle of the 19th century, as a consequence of the industrial revolution and associated urbanization. Since then, several methods have been employed by engineers to determine the sizing requirements of the drainage system.

The first step of scientific advancement was the systematic quantification of the drain size, related to the area and location to be drained, using tables or a simple formula. As example we can cite the drainage table for sewer sizes and slopes prepared by a London surveyor, John Roe, the Hawksley formula and the Myers formula (Chow, 1962). The next step was a separate consideration of how much water is to be drained, a hydrologic problem, and how to drain it, a hydraulic problem. The principal example of this approach is the well known Rational Method. In American literature the method was first mentioned in 1889 by Kuichling to determine peak runoff for sewer design in New York, but the principles were expounded much earlier by the Irishman Mulvaney in 1851. In England, the method is often referred to as the Lloyd-Davis method, owing to a publication in 1906 (Chow, 1964). Despite its limitations and the many critical remarks of contemporary engineers, the Rational Method and its derivatives are still widely in use.

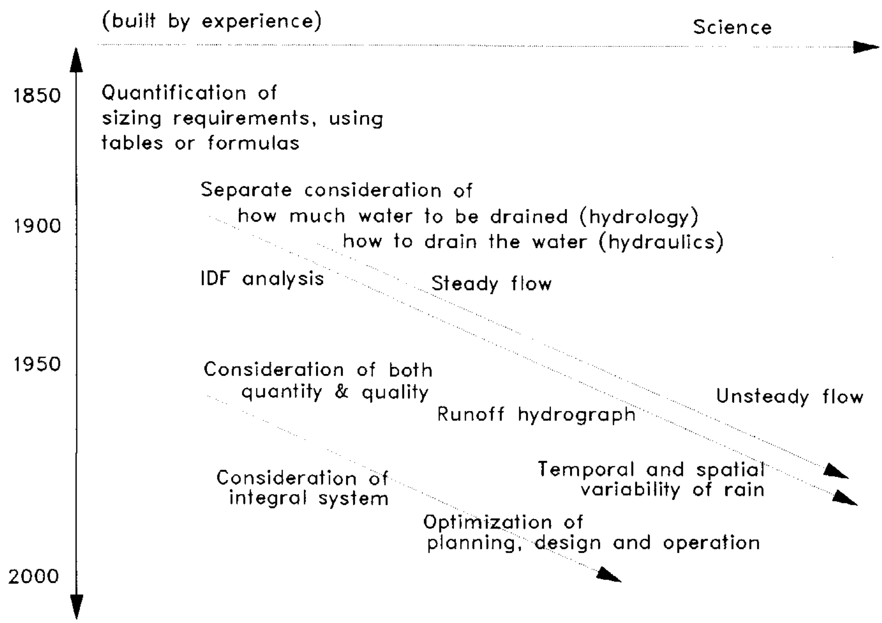


*Figure 1.1. Development of urban storm drainage technology (after Yen, 1987)*

For almost a century studies of urban drainage problems were mainly focused on techniques to determine a peak discharge for sizing sewers and other auxiliaries. For the first applications of the Rational Method observed severe rainstorms were used as the design input. Around the turn of the century, the point rainfall depth was considered as a function of the rain duration. The development of frequency analysis to establish the Intensity-Duration-Frequency (IDF) relationship for point rainfall began around 1910. The matter is still of scientific interest, e.g., (Arnell, 1982). An overview of the use of rainfall data in urban drainage can be found in the proceedings of the seminar on "Rainfall as the Basis for Urban Runoff Design and Analysis", which was held in Copenhagen in 1983 (Harremoës, 1984).

Until today the main hydrologic criterion in urban drainage design, i.e. the risk of flooding, is expressed by the return period of the design storm, which is estimated from an IDF relationship or from historical rainfall data. The basic idea is that once a drain is sized to cope with a design storm, it will be able to handle all the smaller storms. The discharge value which is calculated on the basis of this storm is assumed to have the same return period as the storm. Recent investigations, e.g., (Nouh, 1990), (Van de Ven ,1989), (Yen, 1990), have shown that this assumption is not quite correct. Catchment characteristics and the heterogeneity of real storms affect the rainfall runoff process and therefore the frequency of peak flows. In urban storm drainage design these aspects are still often neglected.

After 1960, the focus of urban drainage technology has been greatly expanded. Due to unacceptable pollution of receiving waters, the nature of the urban drainage problem changed from 'simply' draining the storm water to disposing of it in an acceptable sanitary way. In designing the system, the determination of a peak discharge only was no longer sufficient. For example, detention and retention offer possible remedies to reduce combined sewer overflows, meaning that information on the temporal variations of the storm runoff, i.e. the runoff hydrograph, was required.

Many methods have been proposed to determine the hydrograph, but none has been generally accepted as the most satisfactory. Depending on the type of project and the available catchment data various methods can be applied. The methods include modified versions of the Rational Method, the method of the Unit Hydrograph, hydrologic routing and more complex hydrodynamic (or hydraulic) routing.

Among the various aspects of urban drainage technology, the knowledge on the flow in sewer networks has advanced most. One could say that the development of sewer hydraulics is more or less in response to the need in solving storm drainage problems. In the early days, the steady uniform flow approximation was sufficient. From the numerous empirical equations for steady flow, which were developed about a century ago, some are still in general use, such as the equations by Chézy and Manning-Strickler. Later, backwater computation provided an improvement. After 1960, when water storage and flow routing were needed in solving pollution control problems, the need for unsteady flow routing was recognized. Since no analytical solution exists for the two partial differential equations for a gradually varied unsteady flow, as first published by De Saint Venant in 1871, various numerical methods have been developed for their approximation. During the last decade, many computer packages became available for simulation of surface runoff and flow through conduits, which facilitate a detailed analysis of the hydrodynamic behaviour of the urban drainage system.

## 1.3 Urban drainage in the Netherlands

### *1.3.1 Water pollution control*

Most urban storm drainage systems in the Netherlands, in the order of 75%, are combined systems, meaning that the surplus storm water and the waste water are discharged via one conduit. Before 1955, it was considered sufficient to maintain a certain dilution of the sewage that is discharged to receiving waters (Koot, 1977). For the last three decades, the main criterion in designing combined sewer systems has been the theoretical overflow frequency (TOF). The conventional method to determine the TOF is described in Chapter 2. Despite its limitations, the basic concept of the overflow frequency is still widely accepted as the main (or even only) rational base for the inter-comparison of the performance of Dutch combined sewer systems.

To control the problem of surface water pollution, treatment of waste water and storm water was required. In the Netherlands, most treatment plants were built in the period 1955-1975 (Oremus, 1990). Regulations concerning discharges of waste water to surface waters are arranged by law since 1970, when the Surface Waters Pollution Act (Wet Verontreiniging Oppervlaktewateren, WVO) was approved by the parliament. Since then water quality aspects have attracted much attention. In fact, the impact of discharges of the sewer system on the receiving water quality has become the governing factor in designing urban drainage systems. This implies that water quality standards for the receiving water are desired in addition to the hydrologic criteria.

Recent investigations have shown that large variations can occur in the concentrations of pollutants per overflow (NWRW, 1991). Therefore, the overflow frequency as the main standard for urban drainage design is likely to be abandoned (or updated) in the near future. In formulating new design standards, a possible approach would be to set criteria for each receiving water body, based on extreme value statistics for pollutants with an acute effect and annual loads for accumulating pollutants. This approach encounters two problems.

The first problem is that models which are able to predict the pollution loads with sufficient accuracy have not been evolved yet, but extensive research is being carried out in this field, e.g., (Kleijwegt, 1992). A review and discussions on the developments of the topic can be found in the proceedings of the Wageningen Conference on Urban Storm Water Quality and Ecological Effects upon Receiving Waters (1989).

The second problem is to define the permissible loads, which should be derived by considering their impact on the aquatic ecosystem. Although the available pollution transport models are far from perfect, the provoking statement could be made that they allow the prediction of the pollution load of the receiving water with an accuracy beyond our capability of ecological interpretation. These problems are not restricted to the Netherlands. Based on a literature review Gujer et al. (1987) conclude that *'today the 'mechanistic' understanding of urban drainage systems is better than the insight into the environmental impact of these systems'*.

Nevertheless it can be stated that the policies towards urban storm drainage and water pollution are changing. Contrary to the conventional design approach, which is based on a probabilistic interpretation of the loading of the system (i.e. a design storm with a certain return period), it is now recognized that the system has to be evaluated on the basis of the statistical properties of the systems states (i.e. flows, water levels) and output variables (i.e. overflows, pollution loads), in relation to the response of the environment to this output. Probabilistic design methods are not yet fully developed for urban drainage design, but with the progress of computer technology it has become common practice to simulate time series of continuous rain data to gain insight into the statistical properties of the systems state and output variables.

Modern urban drainage design considers all pathways of the water in the urban area, i.e. the sewer system, the ground water system and surface waters (Van de Ven et al., 1991). Besides improved design methods of storm water collection, transport and treatment facilities various methods of so called source controls have been developed to reduce the volume and/or the flux of surface runoff entering the storm water system. These source controls, which are all based on the principle of increasing storm water infiltration and retardation of runoff flows, may be very effective in reducing sewage discharges to receiving waters. It is obvious that their use should be restricted to less polluted storm water (e.g., from housing areas) to prevent ground water pollution.

The search for efficient solutions to urban drainage problems encouraged the concept of integral urban water management, meaning that the various components of the system should be considered as interrelated and coherently integrated into one system. Traditionally, engineers are preoccupied with the technology of the sub-system they design: the sewer system, a retention basin, the treatment plant etc., neglecting the fact that all elements of the integral system affect each other. For example, increasing the storage capacity of a combined sewer system to reduce sewer overflows means that more (diluted) sewage is discharged to the treatment plant. This might result in an increase of the pollutant loads of the treatment plant effluent. In some cases the peak flows are simply by-passed. As a consequence, the total effect of the larger storage on the pollution loads of the integral system might be negligible compared to the initial situation. Such problems have been recognized only recently. A discussion on the interactions between combined sewer systems and waste water treatment plants can be found in a.o. (Van der Graaf, 1992) and (Durchslag et al, 1991).

An important aspect that until now has attracted comparatively little attention is the problem of how to operate the integral urban drainage system under dynamic loading. In solving urban drainage problems the common approach is to provide sufficient system capacity, rather than investigating how the available capacity can be used in a more efficient way. It is only recently that the problem of lack of flexibility in the operation of the urban drainage system has emerged. The problem how to control the system in the best possible way is a main topic of this research.

Generally, it can be concluded that nowadays the appropriate solution to urban drainage problems is no longer only 'structural', i.e. resolved through construction of sewer systems, retention basins, treatment plants and other auxiliaries, but also 'non-structural', i.e. resolved through appropriate planning and operation. Hence, the next phase of scientific advancement in urban drainage technology is the development of methods for the optimal design and operation of the integral urban drainage system, taking into account the temporal and spatial variability of the system loading and other risks and uncertainties, and considering safety, economy and the environment.

### *1.3.2 Administrative aspects*

Integral urban water management is not a technical problem only. Another problem is due to the administrative borders within the urban drainage system. According to the Surface Waters Pollution Act the Provincial Governments are in charge of water pollution control. Except for the Provinces Groningen, Utrecht and Drenthe, this task was delegated to a number of water boards (waterschappen, heemraadschappen). Some of these boards were founded already centuries ago for the water quantity management. Their task was enlarged to include both water quantity and water quality control. Other water boards (zuiveringschappen) were established especially for the purpose of pollution control. They function together with a water board for water quantity control.

Concerning urban drainage systems, the water quality manager may control surface water pollution by means of construction and operation of waste water treatment plants and by the formulation of requirements and standards concerning the discharges of waste water to surface waters (like overflows). As general rules are lacking and these organisations have their own responsibility there is only a limited uniformity in the regulations and standards. The municipalities are responsible for the construction, maintenance and operation of the sewerage system. They have to meet the standards of the water board by providing sufficient storage and discharge capacity in the system. Both administrations have to make agreements on the maximum discharges to the treatment plant. Some municipalities may have to deal with more than one water board as a city can be situated within different water board districts. As the regulations formulated by these water boards, concerning the same problem, are not always alike managerial problems may occur. Besides, the border between the water authority and the municipality is not always clear, but normally the responsibility of the water authority starts at the main pumping station that pumps the sewage to the treatment plant. The cities of Amsterdam and Tilburg form an exception to this, as they treat their waste water themselves. They still have to meet the constraints concerning the discharges of waste water and treatment effluent that are set by the water board.

Both municipalities and water quality managers are facing their own specific problems which they have to solve with limited financial resources. This and the fact that the physical borders and the administrative borders of the urban drainage system do not coincide may lead to complicated discussions between these authorities and often to inefficient solutions (from a technical point of view). The search for integral solutions to urban drainage problems means that both the water quality manager and the municipality have to look outside their administrative borders. Due to existing standards, regulations and funding arrangements this seems at present rather difficult to accomplish, but hopefully the national tendency towards integral water management may stimulate the necessary changes.

## 1.4 Outline of the problem

In the early eighties, the National Working Party on Sewerage and Water Quality (Nationale Werkgroep Riolering en Waterkwaliteit, NWRW) was established in the Netherlands to investigate the relations between sewer systems and the quality of surface waters. During a period of 7 years, various projects have been carried out by engineering consultants and research institutes as part of the NWRW research programme, to monitor the different processes and to assess the relevance of the various theories developed on the subject. In its final report the NWRW (1991) indicate that '*discharges of storm water and overflows from combined sewers can often seriously impair the quality of surface waters*'.

Most urban drainage systems need upgrading since they do not meet the constraints concerning the allowable pollution outflow to the receiving water bodies. The NWRW research programme has indicated the effects of various types of sewer systems and possible measures to upgrade the systems performance have been assessed. These are discussed in bare outline in Chapter 2. One of the crucial findings was that local circumstances are very important when assessing the systems performance. Therefore, general guidelines are difficult to formulate.

Although technical progress in urban drainage design was strongly stimulated by the NWRW research programme and other investigations, it may be premature to conclude that sufficient information has been gathered for the urban drainage engineer to design and manage the urban drainage system in an optimal way. Some issues that need further research have been mentioned above. The NWRW (1991) conclude that runoff loss processes, dynamic phenomena in sewer systems (e.g., sedimentation and re-suspension of settled material) and the dispersal of pollution in receiving waters are insufficiently understood to specify design criteria in relation to a particular type of receiving water and prescribed water quality standards. In addition, the question has to be raised as to how to express and verify these design standards.

Since the differences in local circumstances and the system dynamics have been indicated, the specification of a desired systems performance seems a more appropriate approach than formulating strict capacity constraints, such as a certain amount of storage and discharge capacity that have to be available. Besides, one has to be aware that using 'proper' system dimensions that meet all the design constraints does not mean that the system performs optimally for all rains that it is exposed to. Main reasons are:

- The design method itself may be inaccurate, due to schematizations and assumptions that are necessary for the computations;
- The planned and actual drainage conditions will differ due to urban development, maintenance work, sewer construction and system failures. As a result, some sections will have more storage and discharge capacity compared to other sections of the system. These discrepancies affect the performance of the integral system;
- The systems performance is affected by the temporal and spatial variability of the system loading, leading to an uneven use of the available capacities. A design storm will never occur as a physical event. Real storms are distributed in time and space. Although they might not reach the depth of the design storm, local storms might result in combined sewer overflows, while elsewhere in the system storage capacity is still available. Theoretically, in an uncontrolled system maximum use of all available storage and transport capacity will only be achieved when the entire system is loaded with a storm greater or equal to the design storm. By definition, for every other loading some capacity will remain unused.
- The effects of the system output on the environment are variable in time and space.

Facing this situation the conclusion can be drawn that better use of the capacities of the drainage system can be achieved by actively directing and storing the flows, or by controlling the system in real time, i.e. on the basis of currently monitored process data.

The general objective of real time control is to make an optimal use of all components of the urban drainage system for all storms that the system is exposed to.

This implies in the first place that one has to define what is understood by 'an optimal use'. The general purpose of an urban storm drainage system is to keep the risk of flooding and the pollution loads to receiving waters within appropriate margins at minimum cost. Since different perceptions exist about security and appropriate cost-effective solutions, it is evident that *the* optimal operation of an urban drainage system does not exist. Any solution to the operational problem depends on subjectively chosen objectives, which therefore should be specified with care. Besides, since the objectives have to be specified for all operational conditions that may occur, they can only be formulated in terms of desired systems performance. Evidently, the use of a fixed standard related to a certain design load is not applicable here.

Once the operational objectives have been formulated, the next problem is to derive the operation strategy to achieve this desired performance on the basis of currently monitored process data. For the past few years, numerous (model) studies have been carried out to investigate the effects of real time control of urban drainage systems. The popularity of the topic is indicated in Table 1.1, which gives the number of papers related directly to real time control (RTC) that were presented at the first five international conferences on urban storm drainage (ICUD) under the heading of Planning and Management.

*Table 1.1. Number of papers on real time control (RTC) at the International Conferences on Urban Storm Drainage (ICUD)*

| | place | year | Papers on Planning & Management | Papers on RTC | Percentage |
|---|---|---|---|---|---|
| 1st ICUD | Southampton | 1978 | 13 | - | 0 % |
| 2nd ICUD | Illinois | 1981 | 13 | 1 | 8 % |
| 3rd ICUD | Gsteborg | 1984 | 44 | 4 | 9 % |
| 4th ICUD | Laussanne | 1987 | 38 | 12 | 32 % |
| 5th ICUD | Osaka | 1990 | 60 | 30 | 50 % |

Since every urban drainage system is unique and the possibilities of improving the systems performance depend on local conditions, no general rules can be formulated to quantify the potential of real time control of the urban drainage system. Therefore, an assessment of the performance of an urban drainage system that is controlled in real time has to be based on model calculations. So far, none of the above mentioned investigations has led to a general model that is tailored for this problem.

Such a model may serve two purposes: it can serve as a rational basis to assess the potential of real time control in designing or rehabilitating an urban drainage system and it can be used in practice as a tool to derive a suitable operation strategy. In this study emphasis is placed on the first aspect.

The problem to control the urban drainage system in the best possible way has to be faced in every phase of the life-cycle of an urban drainage project. At present the operational aspects are mostly being considered after the system has been designed and implemented. It is obvious that the system should be properly controlled. However, to find the most appropriate solution to urban drainage problems the potential of real time control should already be considered in the design or rehabilitation phase of the system, as this very phase provides most flexibility in choosing the appropriate type and size of the flow regulators and in implementing changes in the capacities of the various components of the system.

The key problem is the formulation of the operation strategy. To make the model suitable for design purposes one of the main requirements is that the decision making on the optimal operation of the system is consistent for every possible loading and independent of the system that is simulated. If these features are lacking it is impossible to evaluate the effects of altering the system capacities in search for the optimal design.

Recently, some existing flow models have been updated to allow simulation of automatic flow regulators. Examples are the special version of the rainfall-runoff model SAMBA, called SAMBA-CONTROL (Harremoës et al, 1989) and a special version of the hydrodynamic flow model MOUSE. Both have been developed to evaluate the effects of a predefined control scenario. The models do not (yet) incorporate a procedure to derive the (optimal) operation strategy and they do not allow external control input during the simulated process. The models are therefore not suitable for the problem under consideration.

The same holds for the German model FITASIM, which can be used as a training tool to demonstrate the effects of different ways of operation by means of animation. In developing the model emphasis is put on the graphical output. Presently, the model can be used to perform a step by step simulation in which the user may modify settings of controllable elements or to display a previous simulation (Einfalt et al., 1992).

Most studies on real time control are related to a specific catchment for which a special model has been developed. A general control model is lacking, but some authors have described a (mathematical) model that may have general application, e.g., (Papageorgiou, 1983), (Petersen, 1986), (Schilling et al., 1987), (Béron et al., 1988), (EAWAG, 1990). Evidently, the concepts derived from these and other studies can serve as a basis in developing a general control simulation model.

It is noted that parallel to this study, a computer model has been developed at the Institute for Operations Research at Zürich, Switzerland (Neugebauer, 1989), (Neugebauer et al., 1991) for a similar purpose as the model described in this report. This model, recently named NOUDS (Network Optimization for Urban Drainage), is based on the Linear Programming concept as described by a.o. (Petersen, 1986) and (Schilling et al., 1987). An efficient network flow algorithm is used to solve this problem. Although the model developed in this study also uses a network flow algorithm to solve the mathematical problem, it is mentioned here that the way the operational problem is dealt with in both models is different. In this study it is shown that better results are obtained when the operational problem is formulated as a non-linear programming problem.

## 1.5 Objectives of the study

The general objective of the study is the development of a method to assess the performance of a (Dutch) urban drainage system that is controlled in real time. A general control model, which is required for this purpose, does not exist and will therefore be developed. Proceeding from the results of a number of case studies that are performed with the model, the role of real time control and that of the model in particular will be indicated.

In developing the model it should be considered that

- In deriving the operation strategy a flexible and consistent method is required, leading to optimal performance, independently of the storms and the systems that are simulated, taking account of both water quantity and water quality aspects;
- Fast execution time is desired to allow simulation of time series of (historical) rain events. Time series calculations are needed in assessing the impact of real time control, as it is important to have insight into the statistical properties of the output variables, rather than the results of a calculation of a single event. It is noted that fast execution time is also desired to make the model suitable for on-line use in real-life applications. This, however, is not a main aim in developing the model.

To quantify the effects of an improved operation a reference is required. A conscientious approach is to compare the results obtained by the model of the optimal controlled system with the results of a model of an identical system, which is locally or statically controlled (i.e. using regulators with fixed stage-discharge relationships). Such a model will be developed as well. Moreover, to make the model generally available for planning, design and operational purposes the model should be easy to use and preferably run on a 'normal' microcomputer (AT 286/386).

From the numerous aspects that influence the performance of an urban drainage system and from which one may benefit by implementing a control system, some have been selected for further assessment. The topics investigated are

- possible gains from an improved operation concerning the system capacities required;
- importance of inflow forecast;
- possibilities of controlling the system based on pollution loads;
- impact of rainfall distribution;
- possibilities of reducing the peak flow to the treatment plant;

In this research emphasis is put on the model and its applications. However, efforts to solve the operational optimization problem do not mean that the problem of finding the optimal design is tackled. Proceeding from the results of the case studies some recommendations will be made on a methodology to take account of the potential of real time control in designing an urban drainage system.

## 1.6 Structure of the report

*Chapter 2* describes the main principles of the present methods of urban storm drainage design. The characteristics of Dutch sewer systems and the limitations of the present approach to solve urban drainage problems have to be understood for a clear conception of the contributing factors to the potential of real time control.

*Chapter 3* deals with the theoretical aspects of controlling a process. The terminology as commonly applied in Systems Dynamics is explained. The general operational optimization problem is formulated and the information required to solve this problem is specified.

In *Chapter 4* possible procedures to solve the operational problem are discussed and a suitable technique is selected. The mathematical formulation of the (non-linear) optimization problem and the unit cost functions of the systems state variables are presented.

In *Chapter 5* a modelling package is presented, named LOCUS, which is an acronym for Local versus Optimal Control of Urban drainage Systems. Attention is paid to the algorithm to solve the mathematical problem as formulated in Chapter 4. The structure of the program and some utility programs are discussed.

In the second part of the report we present four case studies that are conducted with the LOCUS model. Each topic is dealt with in a separate chapter. The first case study is of theoretical nature and is performed for an artificial catchment. The other three are dealing with real catchments.

*Chapter 6* describes an analysis of a fictitious system. The aim of this analysis is threefold. Firstly, the 'value' of real time control is indicated, expressed in terms of the amount of storage or discharge capacity that could be saved when implementing real time control. Secondly the influence of the control horizon and the effects of prediction errors on the operation strategy are investigated. The third topic concerns the possibilities of including pollutant concentrations in deriving the operation strategy, aiming at minimization of pollution loads.

The possibilities of improving the operation of the urban drainage system of Rotterdam are handled in *Chapter 7.* This case study illustrates how to use the model when in formulating the operational objectives not only minimizing overflows is important but also a distinction has to be made in the possible location of overflows.

*Chapter 8* deals with a case study of a district in Copenhagen, called Damhus. This study focuses on the impact of rainfall distribution and the extent to which one may benefit from this phenomenon by means of real time control. Three different rainfall distribution models are compared, both for a statically controlled and an optimally controlled system.

Rainfall distribution may also be important with respect to the required capacity of the treatment plant. This is illustrated in *Chapter 9*, which describes a case study of the regional waste water system of Westfriesland. For this system, real time control is applied to reduce the peak flows to the treatment plant. Three types of operation are compared for different hydraulic capacities of the treatment plant: local automatic control, based on the water level at the pumping station; systems control, using a predefined control scenario (the present way of operation) and optimized systems control, based on LOCUS.

In *Chapter 10* the findings of the study are summarized. Some recommendations for further research are formulated. The role of real time control in (future) urban drainage design is outlined.

*Chapter 11* provides a translation of Chapter 10 in the Dutch language.

# 2 Principles of urban drainage design

## 2.1 Introduction

In the Netherlands, about 74% of the urban drainage systems are of the combined type; 22% of the systems are separate systems, which mainly have been implemented in new urban developments for the past 10-20 years; 2% of the systems are so called improved separate systems, in which the storm water system is connected to the foul water system, to discharge not only the waste water but also a part of the rain water to the treatment plant; the remaining 2% are improved combined systems, in which off-line detention facilities have been implemented (VNG, 1991). Although the premises in designing the different types of systems are basically the same, it is noted that in this study emphasis is placed on combined sewer systems.

Due to the flat surface in the most of the Netherlands, most sewer systems have very low gradients (in the order of 1:500 / 1:1000) and the water is discharged by pumps. In general, the pumps are automatically controlled, based on the water level in the pump sump and using a two point controller (on/off) with fixed set points. The required pumping capacity is determined on the basis of the average dry weather flow (DWF) and a certain capacity to discharge storm water, which is referred to as the 'pump-over-capacity' (POC). It is customary to apply a pumping capacity in the order of 2-4 times DWF, meaning that the 'over-capacity' varies between 1-3 times DWF. To build bigger pumps is considered uneconomical. Besides the cost of the pumping station this would also mean that (with the present way of operation) larger treatment plants would be required.

Since the transport capacity to the treatment plant is limited, part of the storm water has to be stored. Storage is generally realized by sewer volume (in-line). Off-line detention and retention facilities are less employed. To ensure that storage capacity is available at the beginning of the next rain event, a restriction is set set to the emptying time of the system, which is defined as the storage capacity divided by the 'over-capacity'. Typical values of the emptying time are in the order of 10-15 h hours.

The (conventional) design of an urban storm drainage system is based on the principle that two criteria are to be maintained: the system must be able to discharge its design load (a hydraulic criterion) and the system should meet the required limitation of pollution outflow (a water quality criterion). The latter is presently expressed in terms of the theoretical overflow frequency. In a general sense, the search for an efficient design aims at striking a proper balance between the storage, discharge (pumping) and treatment capacity, which are required to keep the system output variables within the allowable limits and by which the costs are maintained at a minimum.

The main principles of Dutch urban drainage design and its limitations are outlined below. An extensive overview of the design methods as applied in the Netherlands can be found in (Koot, 1977).

## 2.2 Hydrologic and hydraulic criteria

The dimensions of sewers and storage facilities were traditionally computed on the basis of standards derived directly from precipitation data. Several investigations, e.g., (Van de Ven, 1989) have shown that this may lead to a significant overestimation of the peak flow, and hence, losses and the transformation from (net) precipitation to sewer inflow are not to be neglected.

The methods that may be applied to estimate the flow can be divided into two broad categories. Those which produce only an estimate of the peak flow rate and more comprehensive approaches that also provide the shape of the runoff hydrograph. The principal example of the former approach is the Rational Method. Representatives of the latter are the Unit Hydrograph and other flood routing methods. The basic idea of flood routing is to predict runoff as a function of time and place: $Q = f(x,t)$.

The methods may further be split into two major categories: simple hydrologic routing and more complex hydrodynamic (or hydraulic) routing. Hydrologic routing involves the balancing of inflow, outflow and volume of storage through the use of the continuity equation and a storage-discharge relationship. Hydrodynamic flow routing differs from hydrologic routing in that both the equation of continuity and the momentum equation are solved simultaneously.

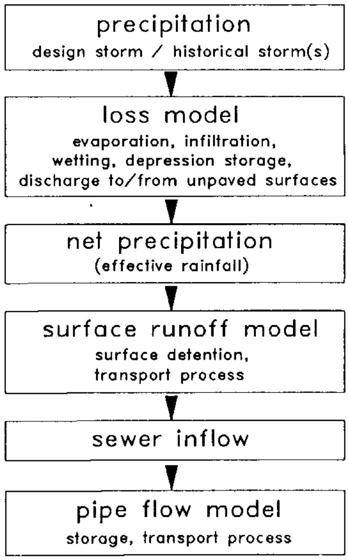


*Figure 2.1. Scheme of a rainfall-runoff model*

### *2.2.1 Precipitation*

In principle, two different kinds of rainfall data can be used in analyzing the performance of a drainage system:

1. Design storms derived from Intensity-Duration-Frequency (IDF) relationships or from historical rainfall data. The discharge value which is calculated on the basis of the storm is assumed to have the same probability as the storm.
2. Historical storms are run through a runoff model, followed by a statistical analysis of the model results.

In the Netherlands, the common approach is to check the hydraulic behaviour of the system by means of a design storm with constant intensity. This intensity is generally derived from historical rain data recorded at De Bilt, which have been assessed by a Gumbel analysis of annual maxima. The statistical methods are described in a.o. (Van de Ven, 1989). The design intensity depends on the allowable frequency and duration of inundation of streets, which is accepted for periods of abóut 10-30 minutes with a return period of 1-2 years. Typical values of the design intensity vary between 40-60 l/s.ha, for flat systems, and 70-90 l/s.ha, for steeper sloped systems.

A systems analysis on the basis of a design storm has the advantages of being relatively easy and requiring only little computing time. If the objective is to determine the appropriate size and slope of the sewers only, without significant detention storage and without flow routing, this approach is usually sufficient. However, if the project involves flow routing, storage or runoff volume estimation, information on the temporal variability of the rain is necessary. In that case the design storm derived from IDF relationships proves to be insufficient (Arnell, 1982).

The use of time series of historical storms that are run through a rainfall runoff model give in general more accurate results then a design storm, since they take into account non-linear effects in the runoff process that influence the flow statistics. A disadvantage of the method is that it requires that a large number of storms are calculated in order to allow for a statistical assessment of the system variables. Obviously, this is more time-consuming then using a design storm, but with the fast developments in computer technology this may be considered to be a minor problem.

As mentioned, storage in Dutch urban drainage systems is generally provided in-line, i.e. by sewer volume. The pipe sizes that are required to provide sufficient storage capacity may be bigger than the sizes needed to discharge the design load. The calculation method to determine the storage capacity required is based on rain records of 37 years. The method is described in section 2.4

An aspect that is mostly neglected in urban drainage design and operation is the spatial distribution of rainfall. This can be explained in the first place by the lack of sufficient data from a dense network of rain gauges (and possibly a weather radar), which are required to investigate this phenomenon. Besides, a proper description of the heterogeneity of rainfall requires more than statistics only. To explain this, meteorological knowledge and knowledge of the station history of rainfall series are essential (Witter, 1984).

In 1980 an investigation into the topic was carried out by Bakker et al. (1983) on the basis of continuous rainfall data of 8 rain gauges that are located in the Province of Twente. The measurement period was 1974-1980. They derived a relationship describing the minimum difference between the rainfall depth at two points in a period of 8 hours as a function of the greater rain depth at the two points and the distance between the respective points ($\geq$ 10 km). The relationship was used to estimate a possible reduction of the capacity of the regional treatment plant Wervershoof by using the effects of rainfall distribution (Bakker et al., 1984).

Another investigation on rainfall distribution has started recently for the West-Brabant region. A network of 16 rain gauges has been installed, covering an area of about 30x45 $km^2$, to investigate a.o. the usefulness of combining rain gauge measurements with radar and satellite observations (Van den Assem, 1989).

There are few studies quantifying the effects of storm characteristics on runoff response of a small catchment, e.g., (Niemczynowicz, 1984), (Nouh, 1990), (Watts et al., 1991). The main finding of these studies is that in determining the catchment hydrograph one may improve the results by incorporating storm movement parameters. Comprehensive research on the impact of rainfall distribution on the basis of time series of historical rain events is however still very limited.

It is noted that in designing a drainage system of a relatively large area, the effects of rainfall distribution are generally taken into account by applying an areal reduction factor (ARF) for the design discharge. The ARF is a function of duration of rainfall, return period and areal size and is used to predict the areal rainfall from point rainfall measurements. However, the degree of spatial differences in rainfall patterns in time for single storms cannot be described by using the statistical ARF. Models that incorporate phenomena like spatial distribution of rainfall and growth, movement and decay of rain cells do not yet exist (IAWPRC, 1989).

### 2.2.2 *Losses and surface runoff*

In designing and operating a storm drainage system, one of the main requirements is an accurate figure on the amount of runoff, rather than the amount of precipitation.

Input flows are difficult to predict with a great deal of confidence. During the runoff of precipitation various losses can be discerned for paved surfaces: the loss by evaporation, the infiltration loss in brick and tile pavements, the initial loss (wetting loss and depression storage) and the discharge to and from unpaved surfaces. Widely applied loss models are the runoff coefficient, in which the total loss is summarized; the exponential loss model, in which the loss intensity shows an exponential decay in relation to the cumulative precipitation depth; and the extensive loss model, in which all the loss processes are taken into account for each type of surface area. These and other loss models are discussed extensively in (Van de Ven, 1989).

In the Rational Method the runoff coefficient $C$ is applied. The method is based on the formula

$$Q = C \cdot i \cdot A \tag{2.1}$$

where $Q$ is the design peak discharge [m$^3$/s]; $C$ is the runoff coefficient [-]; $i$ is the constant (net) rainfall intensity [m/s], lasting for a critical period $t_c$ (the time of concentration); and $A$ is the size of the drainage area [m$^2$]. The method assumes that the peak discharge occurs at a time $t_c$, which indicates the period of time since the beginning of the rain until the entire basin starts contributing to the runoff. The parameters $C$ and $t_c$ are assumed to be constant.

The inability of the Rational Method to deal with catchment areas in which the rate of increase in contributing area is variable led to the introduction of the Time-Area Method, in which the contributing area is defined as a function of time (the time-area diagram). From the latter, so called Typical Storm Methods have been derived. These methods differ only from the Time-Area Method in the variation of rainfall intensity with time (i.e. the storm profile). The method generally assumes an arbitrary shape of storm profile, often constructed from the IDF relationship for a given frequency of occurrence.

The methods mentioned above are directed towards the estimation of the peak flow. When flow routing is involved, it is essential to know the distribution of the runoff volume in time to assess peak discharge, time lag and runoff duration. The Time-Area Method is in fact the first attempt to estimate a runoff hydrograph with its characteristics of runoff in time. A more sophisticated approximation is given by the Unit Hydrograph, which can be defined as the hydrograph of direct runoff, resulting from a unit depth of effective rainfall generated uniformly over the catchment area at a constant rate during a specified period of time. Other models to transform the net precipitation into sewer inflow are the linear reservoir, the non-linear reservoir and the Nash model. The methods are described by Van de Ven (1989), who concludes that at least a linear reservoir should be applied for the inflow process in order to avoid excessive errors in the hydrological modelling. The latter is used in the case studies presented in this report.

### 2.2.3 *Flow routing*

When calculating conduit dimensions the flow is normally treated as steady. For this purpose several flow formulas are available. What they have in common is the fact that they relate capacity, hydraulic gradients, coefficients of friction and the dimensions of the conduit. Some well known resistance formulas are

- Manning-Strickler: $$v = k_m \cdot R^{2/3} \cdot I^{1/2} \tag{2.2}$$

- Chézy: $$v = C \cdot R^{1/2} \cdot I^{1/2} \tag{2.3}$$

where $v$ is the mean flow velocity [m/s]; $I$ is the hydraulic gradient [-]; $R$ is the hydraulic radius (= wetted area/wetted perimeter) [m]; $k_m$ is the Manning resistance coefficient [$m^{1/3}/s$]; and $C$ is the resistance coefficient of Chézy [$m^{1/2}/s$].

For most design and operational purposes, it is necessary to know the temporal and spatial variations of flood waves through the system. For this purpose flood routing methods are used, which can be divided into two categories: hydrologic routing and hydrodynamic routing.

*Hydrologic routing* involves the balancing of inflow, outflow and volume of storage through the use of the continuity equation and an equation of motion (a storage-discharge relation). The rate of change of storage ($S$) can be written as the balance between inflow ($I$) and outflow ($O$)

$$\frac{\partial S}{\partial t} = I(t) - O(t) \tag{2.4}$$

This equation can be generalized to a finite-difference equation for two points in time, which in a general form reads

$$S(t + \Delta t) = S(t) + (I(t) - O(t)) \cdot \Delta t \tag{2.5}$$

An easy method to perform hydrologic routing is reservoir routing, in which the discharge is related only to storage. The general formula of a non-linear reservoir model reads

$$q = \left[\frac{S}{k}\right]^n \tag{2.6}$$

where $q$ is discharge [$m^3/s$]; $S$ is storage [m3]; and k [s] and n [-] are reservoir constants that have no strict physical meaning. When $n$ equals 1 then Eq. 2.6 represents a linear reservoir model.

In general, reservoir routing may be applied to e.g. flood prediction and reservoir design. Lumped hydrologic models are in general capable of describing the flow retention in sewer networks, but they fail in simulating backwater and surcharge effects.

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

- Continuity equation: $$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{2.7}$$
- Momentum equation:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left[\beta \frac{Q^2}{A}\right] + gA\frac{\partial h}{\partial x} + gAI_f = gAI_o \tag{2.8}$$

kinematic wave
diffusion wave
full dynamic wave

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

$$\beta = \frac{A}{Q^2} \cdot \int^{A} u^2 dA \tag{2.9}$$

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

Eq. 2.7 and 2.8 are valid for free surface flow only. However, they can be generalized to include flow in full pipes (pressurized flow) by introducing a fictitious slot in the top of the pipe, the so-called 'Preismann slot'. For a derivation of the De Saint Venant Equations and a description of the Preismann slot, reference is made to the various handbooks on computational hydraulics.

As was mentioned, when calculating pipe sizes the governing equations may often be simplified to a one-dimensional continuity equation and a uniform and permanent flow relationship (kinematic wave), meaning that only the frictional and the gravitational forces are considered. The resulting equation of motion can be described by Eq. 2.2 or 2.3. The diffusion wave approximation also takes the pressure gradient forces into account. In many cases the model makes a fairly good computation of backwater and surcharge phenomena. In the dynamic wave all terms of the De Saint-Venant equations are included. The main difference between the diffusive wave and the dynamic wave is that the latter is better at computing sudden changes in the runoff, e.g., the effect of a rapidly rising water level.

Since no analytical solution exists for these differential equations, various numerical methods have been developed for their approximation. For the past decade, many commercial computer packages have become available for flood wave calculations and urban drainage design. Since computer runs can be done quickly, it is possible to calculate various options within limited time. Hydrodynamic flow models are generally used for analyzing the performance of existing systems or as a checking procedure on the hydrodynamic behaviour of new systems.

Proper use of computer programs requires good preparation and fundamental knowledge of the phenomena under consideration. Even if totally misused, most programs will provide nice looking figures and graphs but in fact represent only nonsense. Obviously a model should be used that fits to the specific problem. A fully dynamic wave model is not in all cases the most appropriate approach. Furthermore a data check should be included whenever setting up a model. When using a computer model it may be suggested that a manual calculation is to be performed with one of the standard methods (e.g., the Rational method) to compare the results with the model results, using the same data set. This will lead the user through a major part of the program and gross blunders will normally be found.

A major problem in the modelling of existing systems is that the data required to verify the model are normally lacking. Most municipalities have only limited insight into the real condition and behaviour of their sewer system. Therefore it is important that rainfall and system data are collected and processed in a way that they can be used for further calculations.

Although many one dimensional hydrodynamic flow models do exist, none of them has been tailored for *controlled* water management systems. A comparative study of existing hydrodynamic flow models in the Netherlands (Schuurmans, 1988) demonstrated that in the existing flow models the hydraulic structures and the (dynamic) control of these structures in particular, are in an early stage of development. This conclusion induced the development of a model to study the hydraulic performance of controlled irrigation systems, called MODIS (Schuurmans, 1991). A similar model for urban drainage systems, which allows the simulation of automatic flow regulators and control input during the simulated process has not been evolved yet. However, since the attention paid to the operation of urban drainage systems is rapidly growing, it can be assumed that it is only a matter of time before these features will also be included in the existing urban drainage models. For example, the latest version of MOUSE (3.02) is able to simulate moving weirs and other dynamic structures, but controllers have not yet been implemented in the model.

## 2.3 Water quality criteria

In the Netherlands, the main criterion in designing a combined sewer system is the theoretical overflow frequency (TOF). The allowable TOF is set by the Water Board and is in the order of 5-10 times per year, depending on the function and the sensitivity of the receiving water body. The TOF is derived from the so called 'dots-graph' of Kuipers (Fig. 2.2). This graph contains all rain events that have been measured in De Bilt in the period 1926-1962 (37 years) and have a rain depth greater than 4 mm. An event is defined as a rain period without any dry spell. Each rain event is represented by a dot, indicating the rain duration and the associated rain depth.

It is stressed that the TOF is a theoretical value only, mentioned to facilitate a rational inter-comparison between combined sewer systems. The method has not been proposed to calculate the real overflow frequency.

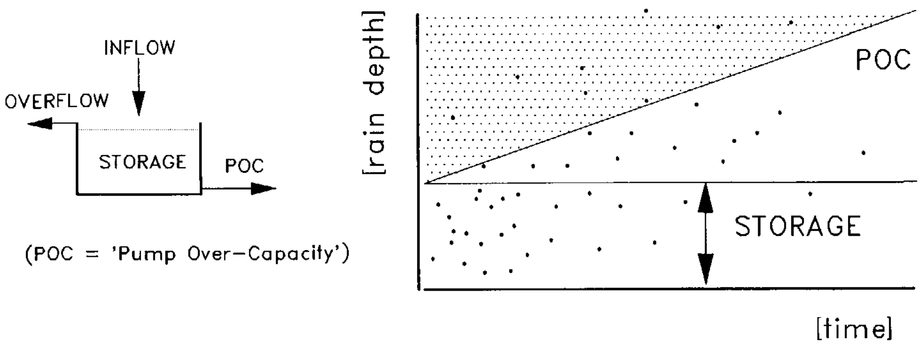


*Figure 2.2. The 'dots-graph' of Kuipers*

The principles of the method are simple. The sewer system is schematized as a single reservoir. The storage capacity of the system is represented by a horizontal line. In most cases the 'static' storage, which is the sewer volume below the lowest overflow weir, is taken into account. In some systems, with steeper slopes, some dynamic storage as a result of backwater effects is also taken into consideration. Further, the 'pump-over-capacity' (POC) is indicated, representing the pumping capacity minus the average DWF, or the capacity available for discharging the storm water. As it is assumed that the full storage and 'over-capacity' is available at the beginning of each rain storm, only the storms (dots) lying above the 'POC-line' do (theoretically) lead to an overflow. In Fig. 2.2 this is indicated by the grey shaded area. The TOF is determined by dividing the number of dots in the grey shaded area by 37 years.

It is evident that the real overflow frequency will differ from the TOF. Some aspects that are neglected are:

- The rain intensity is not constant. Intensities may occur that lead to an overflow, although the total rain volume lies below the 'POC-line';
- Full storage and pumping capacity may not be available at the beginning of the rain event, due to succeeding rain events and the long emptying time of the system;
- Losses and the transformation of rainfall to sewer inflow are not taken into account;
- The DWF varies in time, and therefore the 'over-capacity' is not a constant;
- Backwater and surcharge effects are neglected;
- Most systems consist of several sub-catchments with different storage and discharge capacities and therefore they cannot be schematized to a single reservoir;
- Pumps do not start immediately at the beginning of a rain storm.

Observe that most of the above mentioned aspects can be included by performing time-series calculations with a proper rainfall runoff model that represents the system with a sufficient degree of accuracy. With the present state of computer technology, the use of a fully dynamic wave model for such analysis would be extremely time consuming and hence not (yet) applicable. Nevertheless, there are examples in which a hydrodynamic flow model is used in combination with the 'Kuipers-graph'. In such a case, the flow model is used to get a better estimate of the 'over-capacity' line, which is derived by simulating several block rains with different intensities. For each rain it is determined at what time the system starts overflowing, leading to one point of the 'POC' line. The only advantage of this approach is that hydrodynamic effects are taken into account, but it does not solve other problems.

In general, some simplifications of the flow routing will be required to allow for long-term simulations by which the computational time is kept within appropriate limits. An example of a model that has been developed for this purpose is the Danish model SAMBA. This model is based on the Time-Area method (Johansen, 1985), by which the time-area diagram is derived from a kinematic wave model. Since SAMBA is not able to simulate reverse flow, which often occurs in Dutch urban drainage systems during overflows, the model is not considered appropriate to Dutch conditions. A lumped hydrologic model (also referred to as a multi-reservoir model) appears to be a more suitable alternative for Dutch conditions. Concerning overflows, these kinds of models will generally give sufficiently accurate results.

However, assuming the real overflow frequency can be predicted with sufficient accuracy, the question remains as to what extent the overflow frequency is a suitable parameter to indicate the pollution loads to receiving waters by overflows. The NWRW research (1991) has shown that wide variations may occur in the pollution concentrations per overflow. Due to the large and flat conduits great problems arise with sedimentation and re-suspension of fine particles and sludge. This process is considered the main contributing factor to pollution of overflowing sewage. The concentrations in the mixture of waste water and storm water vary according to the previous rain events, present sediments, size distribution of sediments, type of sediment (cohesive or non-cohesive), duration of the present rain, flow velocities in sewers etc..

Because flow velocities are important parameters in the process of sedimentation and re-suspension, attempts have been made to derive a relation between inflow intensities (and related average flow velocities) and pollution loads (NWRW 5.2, 1990). Such a model may be used to get a rough estimate of the pollution loads, but the results are not very accurate. Generally it is quite impossible to determine the actual concentrations for each rain event only on the basis of the rain intensity and the rain depth. Due to the above mentioned factors, Aalderink (1989) concludes that the degree of re-suspension is stochastic and does not relate distinctly to the type of storm.

Since the TOF does not provide an accurate estimate of the pollution loads of overflows it is likely that the use of this variable as the main water quality criterion in urban drainage design sooner or later will be abandoned. At present there is an ongoing discussion in the Netherlands about possible improvements of the conventional design standards, but none of the proposed models is accepted as satisfactory. Yet, consensus exists on the general concept that a suitable approach would be to set certain water quality standards for each receiving water, depending on the sensitivity and function of the water body. These standards should be based on extreme statistics for pollutants with an acute effect, like bacterial pollution, toxic substances and oxygen depletion, and annual loads for accumulating pollutants, like nutrients, metals and persistent organic materials. The problem is that until today no models exist that are able to predict pollution loads and their effects on receiving waters with sufficient accuracy.

As was mentioned in the previous chapter, some existing flow models are capable of handling water quality features, but with limited success. Extensive research is carried out in this field, e.g. (Kleijwegt, 1992), but these efforts have not yet resulted in accurate pollution emission models. Furthermore it is observed that comprehensive ecologic research is still lacking on the effects of urban discharges on the aquatic ecosystem, which should be the basis in formulating water quality standards. Since the overflow frequency and the overflow volumes can be predicted with a higher degree of accuracy than the pollution loads, these parameters may still be considered a useful basis to evaluate the system performance, especially when dealing with comparative studies (like in the present study), where different alternatives are compared on the basis of the same rain series to determine their (relative) effectiveness in abating pollution emissions (Wiggers, 1991). In addition, one of the existing pollution emission models may be used for an assessment of pollution loads. Due to the inaccuracies of the present water quality models they should not (yet) be considered as a replacement of the TOF.

## 2.4 Urban drainage problems

Ar present, a major concern of many municipalities is the bad condition of their sewer systems, leading to bad hydraulic performance. The bad soil conditions, the age of the system and the increased traffic loading have at several places resulted in sunken pipes, defective joints, breakdowns etc. which may cause problems like infiltrating ground water or exfiltrating sewage (depending on the ground water table), stagnant water in the system (dead storage), sedimentation problems or even the collapse of a part of the system. Several reports have been written on the condition of Dutch sewer systems and the delay in renovation. The estimates diverge, but one thing is clear: the problem is urgent. Until now, each municipality formulates its own policy and individual standards on this matter, depending heavily on financial resources. However, obstacles to the solution of this problem are not only the limited finances. The lack of a proper insight into the real conditions and behaviour of the system also plays an important role. Hence, implementation of an (automated) monitoring system may generally be considered useful, even if the monitored data is not used for control purposes.

Many water quality managers are facing a capacity problem at the treatment plant, which needs to be expanded or replaced. Increased urbanization may have led to an increased flow to the treatment plant. Further it is often considered more efficient to treat waste water at a regional level, meaning that several catchments are being connected to one regional treatment plant. Besides the hydraulic capacity, much effort is put in improving the treatment process of waste water (e.g., nitrogen and phosphorus removal).

Obviously, the storage, transport and treatment capacity of the system should be properly maintained. Based on the 'Kuipers' method it has been estimated that approximately 7 mm storage and 0.7 mm/hr 'over-capacity' should be available to keep the overflow frequency within acceptable limits. In the present discussions on water quality standards these values have been mentioned so often that some policy makers (Provinces and Water Boards) tend to interpret them as fixed standards. This tendency should be discouraged, since it might lead to inflexible regulations, that prescribe 'static solutions', thereby disregarding local conditions and the dynamics of the system.

The NWRW research programme (1991) has indicated that further upgrading of the urban drainage system is needed, since the system does not meet the required limitation of pollution outflow. Some suggestions have been made to improve the system. For combined sewer systems, these comprise (in bare outline)

- Limiting the storm water inflow by means of source controls, i.e. by increasing storm water infiltration. Although source controls may be very effective in reducing the storm water volume entering the system, and hence in reducing overflows, it is noted that the protection of ground water quality restricts their application to less polluted storm water from e.g., rooftops, courts and residential streets.
- Implementing improved overflow structures to reduce the pollution outlet by overflows. The efficiencies of three types of structures have been assessed. Storm water sedimentation tanks appear to give the best results, with a pollutant removal in the order of 60%. The swirl concentrator has a removal efficiency of about 20-40% and the efficiency of the high sided weir chamber varies between 10-25%.
- Providing sufficient storage in order to keep the storm water in the system until there is sufficient capacity to lead it to the treatment plant. This extra storage should preferably be created by means of (off-line) storage settlement tanks. Pipe sizes should be kept small to maintain a minimum flow velocity.
- Furthermore the NWRW has generated some recommendations concerning the shape and maintenance of sewer systems, the layout of the system and possible measures related to surface waters (such as reallocation of overflow structures to less sensitive waters, flushing, etc.).

What the conventional solutions to urban drainage problems have in common is that they are all based on the concept of providing sufficient system capacity. As mentioned in Chapter 1, the dynamics of the system and the lack in flexibility in operating the system are mostly neglected when investigating the possibilities for improving the system. At present, the pumps will be activated at a predefined water level at the pumping station, independently of the situation elsewhere in the system. This might for example lead to the situation that locally the capacity of the system is exceeded whereas elsewhere in the system some storage capacity is still available.

Theoretically, if no dynamic control is applied, the capacity of the system will only be fully utilized when it is loaded with its design load. By definition, for all other loadings the system will perform sub-optimally, meaning that a certain amount of storage and/or discharge capacity could be used in a more efficient way.

## 2.5 Potential of real time control

In view of the above it can be concluded that before enlarging the system it may be worthwhile to investigate the possibilities of mastering urban drainage problems by increasing the operational efficiency of the system. To obtain a comprehensive view on the performance of the system and the contributing factors to the potential of real time control to improve this performance, an insight is required into the following (dynamic) processes:

- the input to the system;
- the systems response to this input;
- the output to the environment; and
- the response of the environment to this output.

*System input*

The loading of an urban drainage system is variable in time and space, due to the heterogeneity of rainfall, the differences in the runoff characteristics of the connected areas and the temporal and spatial variations in dry weather flow. To react properly to this dynamic loading, a flexible operation of the pumps is required. Presently, most Dutch urban drainage systems are equipped with local controlled pumps, using fixed set points. Since every loading pattern is different, optimum systems performance can only be achieved if the set points are adjusted in time to the actual conditions.

In principle, there are two ways to get information on the system disturbances: by means of flow and water level measurements or by forecasting these variables by means of rainfall measurements. Rain input to a surface runoff model can be obtained by a network of rain gauges. Another way to acquire rain information is to make use of a weather radar. This seems to be a promising technique, which also provides a possibility of rain forecasting and a proper description of the spatial distribution of rainfall. A good interpretation and calibration of the radar images however is still encountering some difficulties, but research is in progress (Neumann, 1990). Possible loss models and models to transform the net precipitation into sewer inflow have been discussed above.

*System response*

As described above, several models are available to depict the flow in the system. Concerning a proper operation of the system, an important aspect that has to be considered is the fact that an urban drainage system usually consists of several sub-catchments with different characteristics. The discharge and storage capacities of these sub-systems may not be optimally attuned for several reasons, such as urban development, sewer maintenance and construction, the condition of the pumps etc. These discrepancies within the system will vary in time.

Since some parts of the system will have more capacity available compared to other sections, even a homogeneous loading will lead to an uneven use of the system capacity, meaning that some parts may be overflowing while other parts still have capacity available. Hence, proper attuning of the available capacities will in all cases be required to prevent such situations.

A comprehensive view on the system response concerning water quality parameters is more difficult to obtain. The inaccuracies of the available pollution transport models have been mentioned above. Unfortunately, there are no reliable instruments available for on-line measurements of important pollution and water quality parameters in sewer systems.

*System output and its impacts on receiving waters*

Regarding water pollution control of surface waters, it has to be considered that the effects of overflows to receiving waters are of different spatial and temporal scale. First of all, in constructing and operating the system a distinction should be made between the type and function of the receiving water body. For example, overflows to big streams are normally preferred above overflows to small stagnant waters. Monitoring the effects on receiving waters appears to be difficult and costly. This has been shown by the NWRW research programme. High rate processes such as acute fish kills by toxic matter or chemical oxidation reduction require frequent measurements on a local scale. Moderate rate processes such as BOD-oxygen depletion require measurements of a wider area and on a different time scale (in the order of hours-days). The total annual loading is usually sufficient to evaluate slow rate processes such as accumulation of nutrients and heavy metals (IAWPRC, 1989).

To evaluate the performance of the system concerning the effects on receiving waters, the effluent from waste water treatment plants should also be taken into account. In operating the system the relation between the sewer system and the treatment plant is of special importance. A better use of the available storage in the system will mean that more diluted sewage is discharged to the treatment plant, which may affect the treatment efficiency in a negative way. Therefore, the operational objective should not only be to make better use of the available storage, but at the same time to maintain optimum flow rates to the treatment plant. A discussion into the topic can be found in (Van der Graaf, 1992)

To model the total output of pollutants from a treatment plant during storm conditions requires complex models for the main unit operations in waste water treatment plants, such as primary settling, the activated sludge process and the final clarifier. In Denmark, a simple treatment plant module, called SAMBA-RENS, has been developed, which is used in conjunction with SAMBA (Harremoës, 1989). The model operates with a number of time constants which describe the treatment efficiencies during the different stages in a storm. In the model a distinction is made between the pollutant removal during dry weather conditions, rain weather conditions and the moment that sludge rise (or loss of sludge) occurs. It has also to be defined how much time it takes for the treatment plant to recover after the storm is over. The determination of proper values of these time constants appears to be difficult. In fact, the stage of development of treatment plant modules is comparable with that of the pollution transport models.

## 2.6 Concluding remarks

The conventional solution to urban drainage problems is based on the concept of providing sufficient system capacity (without investigating how this capacity is actually used). The limited efficiency of this approach is caused by the lack of flexibility of the operation of the system under dynamic loading. A dynamic operation is needed since:

- the input of the system is distributed in time and space;
- the available system capacities are not homogeneously distributed over the system;
- the effects of the systems output on the environment are of different temporal and spatial scale.

To take account of these aspects, the present methods of designing and operating an urban drainage system need some upgrading. Since the data required for a comprehensive systems analysis are mostly lacking, it may be suggested that monitoring and measuring equipment to obtain these data should be installed. Practice has shown that once the water manager has a proper insight into the real behaviour of the system, the aspiration to actively control the system will normally follow.

In Chapter 1 some administrative aspects of urban water management were mentioned. Besides the lack of insight into the potential of real time control, the present design standards, regulations and funding arrangements appear to be main obstacles to a successful implementation of real time control of urban drainage systems.

First of all, fixed design standards that prescribe 'static' solutions, such as an amount of storage that has to be available, should be replaced by more flexible standards that describe a 'desired systems performance'. Besides, the existence of separate administrations within one system might cause problems. Municipalities may like to improve the operation of the sewer system to reduce the necessary amount of storage, whereas the water boards intend to use the available storage as much as possible to alleviate flows to the treatment plant. Within the concept of integral water management, the ultimate aim is the same, namely to improve the systems performance and to minimize the construction and operation costs of the integral urban drainage system, i.e. sewer system and treatment plant.

Finally, it is noted that since all urban drainage systems in the Netherlands are equipped with automated flow regulators (pumps and valves), the step from local control towards systems control is, from a technical point of view, not very big. The main problem of the present way of operation is that the set points of the pumps are not interrelated and modified in an appropriate way at the right time.

# 3 Principles of operation and control

## 3.1 Terminology

This section provides an outline of the terminology as commonly applied in Systems Dynamics and Control Technique. For an overview of the existing variety of control systems, reference is made to (Brouwer et al., 1992).

### *3.1.1 Basic elements*

Any control system comprises the following basic elements (Fig. 3.1):

- A *sensor* to monitor the systems state and possibly variables that are used to predict the disturbances of the system (e.g. rainfall, water level, water flow). Generally, the required data depends on the configuration of the system, the specific aim of the control system and the level of control (Section 3.1.2).
- A corrective *flow regulator* (or actuator) which is able to control the process in order to reach or to maintain a desired state of the system, which can be defined by the water levels and flow rates as a function of time and place. Regulators in a water management system could be e.g., pumps, gates, sluices, movable weirs and valves.
- A *controller* which controls the flow regulator, using the receiving data from the sensor. A large variety of controllers is known from Control Technique, such as the two point controller (on/of), the Proportional Integral Differential (PID) controller, predictive controllers, and many others. The task of the controller is to verify and process the measurement data and to send instructions to the regulator. The control signals depend on the deviation of the measurement from the 'desired value' or set point. The time sequence of the set points is called the operation strategy.
- A *communication system* is needed to link the sensors, controllers and regulators in the system. It needs no argument that the reliability of the communication system is very important.

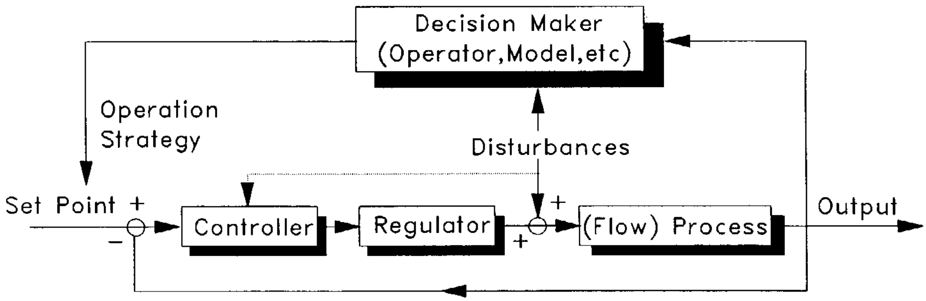


*Figure 3.1. Scheme of a controlled process*

A system is said to be statically controlled if the set points are constant in time. These set points are determined in advance, e.g., a water level or flow that have to be maintained. In this case a decision maker is not necessary to control the process. A system is dynamically controlled (or in real time) if a *decision maker* is added to the system who decides on the set points during the ongoing process on the basis of currently monitored process data and predicted disturbances. Such a decision maker can be an operator or a model.

In controlling a process it is important to define what is understood by the system, i.e. the process which lies between the input variables and the output variables and where mass transfer is controlled. Input variables can be divided into controlled input variables, which are used to control the process, adjustable input variables, which are not used in controlling the system but which should have a certain value to let the system function (e.g., a dip switch, preset levels to switch pumps on and off), and uncontrolled input variables or disturbances. The system output can be divided into controlled and uncontrolled output variables. Besides input and output variables a third type of variables can be discerned, namely the system parameters that determine the dynamic behaviour of the system (Van Straten, 1990).

### *3.1.2 Control concepts*

Control systems differ in complexity. Generally, three different levels of control can be distinguished. The simplest level is called *local control,* which consists of one 'control-loop' as illustrated in Fig. 3.1, where the controlled variable is measured at the regulator site. Most pumping stations in Dutch urban drainage systems are locally controlled on the basis of the water level at the pumping station. Local control implies that regulator action is independent of other sections of the water management system. More advanced is *regional or unit process control*, by which the control of the regulator depends on more than one measurement within the system-unit. Applying *central or systems control* means that the actual state of the total system determines the operation of every regulator. By definition, optimal performance of the urban drainage system can only be achieved when systems control is applied.

It is noted that the data collected on the performance of the drainage system can also be used for the daily management of the system, such as further data analyzing, emergency control and maintenance planning. In practice these benefits appear in fact to be the main reason for water managers to decide for implementation of a monitoring and remote control system.

A control system can be operated in different ways, which is called the mode of operation. When a system is operated in a *manual mode*, it means that the regulator itself is operated directly by an operator. This type of operation is not found in Dutch urban drainage systems.

When a control system is operated in a *supervisory mode*, the regulators are activated by local automatic controllers, but the set points of the controller (e.g., a desired water level or discharge) are specified by an operator. This type of operation is most common in automated water management systems. A typical example can be found in Rotterdam.

To perform optimal control the operators need a full understanding of the dynamics of the system. Even for experienced operators it can be very difficult, if not impossible, to predict the result of moving a number of gates or activating a number of pumps. For this purpose one might use a decision support system (DSS). This can be a simulation model, with which one can try possible strategies before actually executing them. Additionally, the operator can be provided with a decision model that can be used to suggest a strategy (e.g., an expert system, an optimization model). The advantage of this type of operation is that complete operation is under human control, which makes it flexible, as opposed to automatic operation. This might be an advantage, e.g., during extraordinary operational conditions. On the other hand, automatic operation might be preferred as you cannot expect operators to be alert 24 hours a day.

The system is operated in an *automatic mode* when the decision on the operation strategy and the execution is fully automatic. As mentioned above, most pumping stations in the Netherlands are locally automatic controlled. Central automatic control requires generally more hardware compared to the other ways of operation, reliable communication lines and a computer model that can generate a suitable control strategy. In the Netherlands, examples of central automatic control systems can be found in Wervershoof, Utrecht and Olburgen.

## 3.2 The operational optimization problem

In operating a system, a key problem is the formulation of the operation strategy, which can be defined as the time sequence of the set points of all flow regulators in the system. The operational optimization problem is aimed at finding the optimal set points, or 'the optimal path' of the system (or the process) in time. This problem should not be confused with the control problem, which deals with the determination of the required adjustments of the flow regulators, to achieve minimum deviation from the set point in order to follow 'the optimal path' in the best possible way (the 'tracking' problem). Both problems are closely related, as in deriving an operation strategy an important constraint is that the strategy has to be feasible.

In literature, both the terms 'operation strategy' and 'control strategy' can be found, often in the same meaning. It is mentioned that a clear distinction is not always possible as in some cases the operational and control problem are solved simultaneously. However, in view of the above the term 'operation strategy' may be preferred to describe the time sequence of set points (desired state), whereas the term 'control strategy' may be used to describe the necessary adjustments of the flow regulator in time (e.g., crest level of a weir, gate settings, pump rates) to reach or to maintain these set points. For simplicity it could be stated that the formulation of the operation strategy can be dealt with by the methods known from Operations Research, whereas the behaviour of various types of controllers is a topic of research within the field of Control Technique. In this research only the operational problem is addressed.

The simplest option is evidently to keep set points constant, but this means that no dynamic operation is applied. If the system disturbances showed a known periodical variability, a possibility to select set points would be to follow a time schedule (open loop control). However, since the loading of an urban drainage system shows no regular pattern at all, the desired systems behaviour, and hence the optimal set points, will differ for every storm and even change within a storm.

In a general sense, finding an optimal operation strategy means that the operation cost and the negative impact on the environment (the 'costs' of flooding, pollution loads etc.) have to be minimized, given a certain perturbation of the system, subject to the constraint that the strategy has to be feasible. In a mathematical form, this problem can be formulated as

$$minimize \; [\; \sum_{t=1}^{T} f(\underline{x}(t),\underline{u}(t)) + e(\underline{x}(T+1)) \;] \tag{3.1}$$

where $f$ and $e$ denote objective (or cost) functions of the decision variables $\underline{x}(t)$, depicting the systems state, and $\underline{u}(t)$, the control variable. The quantity $e$ $(\underline{x}(T+1))$ are the 'costs' that occur if the final state deviates from its target. $T$ is the time horizon for which the problem is optimized.

The optimal solution $(\underline{x}(t)^{*}\underline{u}(t)^{*})$ is subject to a number of constraints, namely

- initial state conditions, $(\underline{x}(0)\underline{u}(0))$ ;
- capacity constraints, describing the physical limits of the systems state variables (e.g. maximum flow, storage capacity etc.) and the upper and lower bounds of the control variables (e.g. maximum pump rate, weir flow capacity etc.);
- hydrodynamic constraints, comprising the physical laws of water motion, i.e. the equations of conservation of mass and momentum (Eq. 2.9 & 2.10);
- terminal state conditions, $\underline{x}(T+1)$, e.g. reservoir empty;
- other technical (or hardware) constraints, comprising all constraints related to the operation and dynamic behaviour of the flow regulators.

To solve the operational problem, three types of information are required, namely

- a specification of the operational objectives of the system;
- a description of the physical constraints of the problem (i.e. a model of the system);
- information on the current systems state.

If one of these types of information is lacking, no (rational) decision on the operation strategy is possible.

### *3.2.1 Operational objectives*

The general aim of operating an urban drainage system is

- to minimize flooding;
- to minimize the impact of pollution loads to receiving waters by means of
  - minimizing combined sewer overflows (differentiated in time and place, depending on the function and sensitivity of the receiving water body), while
  - maintaining optimum performance of the treatment plant (by means of controlling flows to the plant and by controlling the processes at the plant itself);
- to minimize operational costs (e.g., energy).

Ranking the operational tasks according to their priority is usually not the most difficult part of the problem. It is more complicated to indicate the relative importance of the operational tasks or to define the 'cost' of not performing these tasks. This is necessary as different objectives are considered, which may be conflicting, meaning that they cannot be fulfilled at the same time. For example, storing storm water to reduce combined sewer overflows might increase the risk of flooding. Minimizing pumping to save energy cost might increase the risk of overflows. Obviously, this problem is becoming more acute with an increasing number of operational tasks, i.e. when the operational problem is tending towards an integrated water management problem.

In principle, there are two options to make a rational decision in the set point selection: by specifying performance criteria or by formulating an objective function in terms of 'unit costs'.

Performance criteria are applied to indicate the allowable limits of the controlled output variables. In this case, solving the operational problem means that a feasible solution has to be found, where all systems state variables fall within these limits. The performance criteria determine only whether a solution is acceptable, but the optimal solution is not defined. (In mathematical terms this means that only the set of feasible solutions of Eq. 3.1 is defined, but no criterion exists by which the optimum is defined)

Optimality can only be specified by applying 'unit cost' or 'weights', which express how a deviation of a systems state variable from its desired value is evaluated. The optimal systems state is then defined by the least cost solution of Eq. 3.1. The problem is now how to find proper 'unit costs' of the variables. Obviously the specification of the actual 'costs' is difficult as most operational objectives are impossible to express in financial terms. For example, it is impossible to define the cost of $x$ m$^3$ overflow or $y$ m$^3$ flooding. Moreover, these 'costs' may be variable in time and space as they depend on the current systems state. How to deal with these problems is explained below.

Since every party concerned will have other priorities it is impossible to define the best solution unambiguously. The term 'optimality' could be misleading, but it only refers to the least cost solution of the general problem as formulated by Eq. 3.1. Since the operation strategy is based on the evaluation of the objective function, the usefulness and validity of the chosen objective function should be carefully investigated.

### *3.2.2 Physical constraints*

The operation strategy has to be physically achievable. Pumping rates, flow through conduits or structures, water levels in reservoirs, etc., are limited by the capacities of the particular elements of the water management system. These are called the static or capacity constraints of the problem. Other static constraints which may be less obvious are the initial and final state of the system.

Additionally, the operation strategy has to be consistent with the physical laws of water motion, i.e. the continuity and energy balances. These form the dynamic constraints of the operational problem. It is these constraints that make the problem of finding a good strategy generally difficult. Whereas the formulation of capacity constraints is rather straightforward, the hydrodynamic constraints usually imply a simplification of the governing physical laws, as approximated by the De Saint Venant Equations (Eq. 2.7 & 2.8). In solving the operational optimization problem, flow routing is usually performed by linearized equations or a lumped storage approach (Eq. 2.5 & 2.6), depending on the technique that is applied to solve the problem (Chapter 4).

Other dynamic constraints of the problem are related to the characteristics and behaviour of the controlled flow regulators, e.g., it takes a certain time before a pump reaches its desired discharge, the frequency of switching pumps is limited, a valve should not be closed too suddenly to prevent water hammer, etc. These aspects are referred to as the technical (or hardware) constraints.

### *3.2.3 System disturbances*

It is evident that information on the actual systems state is required to make a decision on how to operate the system. Depending on the response time of the system, the ability of the system to correct a certain perturbation of the system, and the accuracy of the forecast, information on future disturbances may be beneficial in improving this decision. In principle, there are three options to predict the (future) behaviour of an urban drainage system:

- Flow and water level measurements in upstream sections of the system;
- Rainfall measurements in combination with a rainfall-runoff model (section 2.2), which extend the reaction time by the surface-runoff time of the catchment;
- Rainfall forecasts, which allow to gain additional time depending on the forecast horizon.

It has to be considered that measurements include measurement errors. Model calculations are inaccurate due to unknown input, unknown parameters and model simplifications. Rainfall forecasting, using radar images, is a field into which great efforts are being put, but this technique includes still great uncertainties. Moreover, a rainstorm develops within a few (2-3) hours, which restricts the possible time horizon of a reliable forecast. Therefore, in determining the operation strategy, it is advisable to perform a sensitivity analysis to determine the extent to which the results are affected by these uncertainties.

## 3.3 Elaboration of the problem

Consider an urban drainage system consisting of $i$ sub-systems (all of the combined type), which discharge to one treatment plant (Fig. 3.2). Flow regulators that are commonly used in Dutch urban drainage systems are pumps and valves, which are controlled by an automatic two point controller (on/off) or a Programmable Logic Controller (PLC), in which a certain control algorithm has been programmed.

As was mentioned above, most urban drainage systems are locally controlled on the basis of fixed set points, meaning that each sub-system functions on its own regime. To meet the operational objectives as described in Section 3.2.1 in the best possible way requires a dynamic operation, where set points are considered interrelated and modified in time to the actual conditions. The problem is how to determine the best set points.

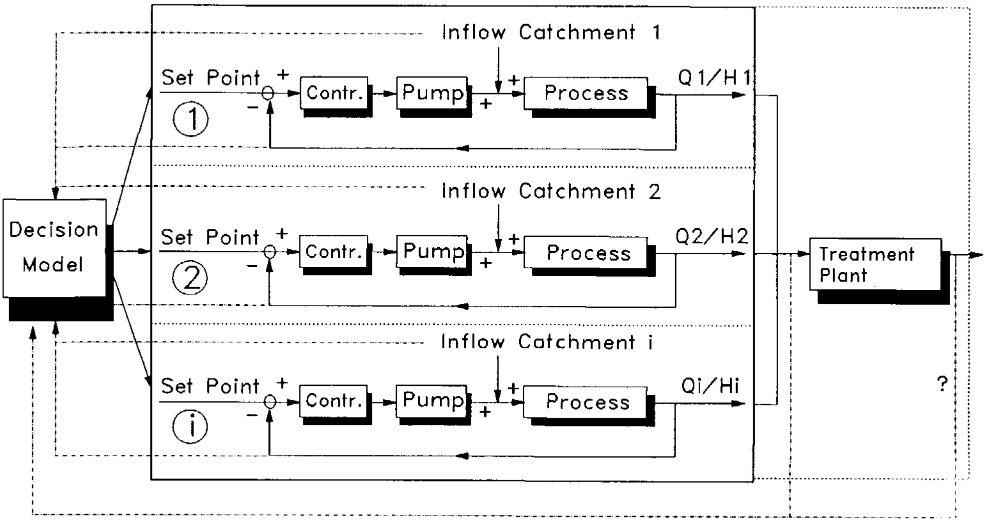


*Figure 3.2. Scheme of a controlled urban drainage system*

First, the system bounds have to be defined. An obvious choice is either to consider the integral urban drainage system, i.e. sewer system and treatment plant, or to deal with the sewer system and the treatment plant as two interrelated systems. The first approach is rather complex as it requires a model describing the dynamics of the rainfall-runoff process, pollution transport and treatment processes. Although conceptual models exist which incorporate all these processes, e.g., (Durchslag, 1991), they involve too many simplifications and uncertainties to be useful for the problem under consideration. Furthermore, in (Nelen, 1992b) it was concluded that concerning the interrelation of the sewerage system and the treatment plant the main problem is to maintain optimum flow rates to the treatment plant depending on the actual conditions at the plant.

Therefore, it has been decided to define the system as the part of the urban drainage system which lies between inflow and outflow of the sewer system. In this research, the allowable discharge to the treatment plant (as a function of time) is considered a known input variable (constraint) to the decision model.

The following input and output variables can be discerned. Controlled input variables are the set points of the flow regulators, expressed by a desired systems state (water level, flow). Disturbances or uncontrolled input variables are the surface runoff and the dry weather flow. Controlled output variables can be water levels ($H_i$) or flows ($Q_i$). In urban drainage the latter is commonly used. Note that sewer overflows may be uncontrolled output variables (e.g., in case of a fixed weir) or controlled output variables (e.g., in case of a valve, a moveable weir or a pump).

The control action depends on the deviation of the (measured) system variable and the set point. In determining set points (or the operation strategy) it is therefore important to specify what is understood by the desired systems behaviour or, in other words, what is understood by a perturbation of the system. This determines the objective function (Eq. 3.1) to be minimized.

For simplicity only one operational objective is considered here, namely to minimize the total combined sewer overflow volume of all *i* sub-systems. Once it is understood how to deal with this problem, it is easy to understand the modification required to solve the general problem. The minimum overflow volume is reached if the storage capacity of all system elements and the discharge capacity to the treatment plant are used to their maximum. The latter can be achieved by maintaining maximum flow rates to the treatment plant, depending on the operational conditions at the plant. In optimizing the use of storage we have to consider that (see section 2.5)

- the loading of each sub-system is different, due to rainfall distribution, differences in runoff characteristics and differences in dry weather flow;
- the available storage and discharge capacity of each sub-system is different; and
- the sub-systems are interrelated, i.e. one sub-system may discharge to another sub-system.

In view of the above a plausible solution is to control the system so that during the rainfall runoff process the stored volume in each sub-system is kept at a relatively equal level. The amount of water to be stored in a sub-system should be related to the available storage capacity of the respective sub-system, as this may differ from the storage capacity of other sub-systems. If this system behaviour can be realized then the probability of an overflow is equal at all sub-systems, meaning that the system will overflow at the same time at all places and only if all storage is used.

In this case a perturbation of the system can be defined as a deviation of the filling degree of a sub-system from the average filling degree of all sub-systems. Hence, the objective function should be formulated so that these deviations are minimized. Including other operational objectives will lead to a different definition of a desired behaviour of the system, and thus to a different objective function. For example, in case a distinction is to be made in location of overflows (depending of the sensitivity of the receiving waters) then the filling degree at the more sensitive places should be kept at a lower level compared to the other sub-systems. The difference in filling degree that should be maintained depends on the controllability of the system and the accepted probability of failure. A discussion on the topic will follow in Section 4.5.

As was mentioned in Section 3.2.3, the response time and the ability of the system to correct a perturbation of the system determines the extent to which one may benefit from predicting disturbances. In the case where the system is able to return to its desired state within acceptable time limits, currently monitored process data are sufficient to derive a suitable strategy. If it takes a longer time to correct the process then information is desired on the behaviour of the system for a certain time horizon to make a proper decision on the operation (and control) strategy. Obviously to predict this behaviour information on future disturbances can be helpful.

## 3.4 Concluding remarks

In analyzing the operational problem of an urban drainage system it is important to define first the system bounds and its input and output variables. In this study, the system is defined by the physical bounds of the sewer system that lie between inflow and outflow. The desired flow to the treatment plant is assumed to be known and considered as a boundary condition to the problem (and thus an input variable).

Next the desired systems behaviour (or a definition of a system perturbation) has to be formulated by which the operational objectives are met in the best possible way. Besides, information is required on the physical constraints of the problem (i.e. a model of the system) and data describing the current systems state. For example, if the objective is to minimize the total overflow volume only, without making a distinction in location of possible overflows, then the desired systems behaviour can be described by keeping the relative use of available storage at all sub-systems at an equal level. (It is noted that depending on the controllability of the system this desired behaviour is not necessarily the only solution.) The desired systems behaviour determines the objective function of the operational optimization problem.

Besides a specification of the operational objectives, a description of the physical constraints of the problem (i.e. a model of the system) and information on the current systems state is required. The extent to which one may benefit from predicting disturbances depends on the response time and the controllability of the system (or the ability to correct a certain perturbation).

The solution of the operational problem comprises the set points of all flow regulators in the system or the desired systems state as a function of time and place. There are several methods to deal with this problem, each having its own restrictions concerning the formulation of the objectives and the constraints of the problem. The various methods will be discussed in the next chapter.

# 4 The operation strategy

## 4.1 Solution techniques

A large variety of optimization problems and related solution techniques are known from Operations Research. Possible procedures that can be applied to solve the operational problem as formulated in Chapter 3 may be divided into three broad categories:

1. *Heuristic methods*: a solution is found by experience (gained by 'trial and error');
2. *Rule based scenarios*: a solution is found through comparison and evaluation of a finite number of feasible solutions. A decision tree is a tool for organizing the enumeration process;
3. *Mathematical optimization techniques*: the operational objectives are expressed by an objective function, which is minimized by means of mathematical calculus.

Each method has its advantages and disadvantages concerning the flexibility in formulation of objectives and constraints, computing time and computer resources needed, robustness of the control performance, etc.. Obviously, the suitability of the technique depends on the specific application.

### *4.1.1 Heuristic methods*

When the operational optimization problem is solved heuristically (i.e. based on experience), it is not common to formulate the objectives of the system in terms of 'costs', but certain performance criteria are obviously required to be able to evaluate the systems performance. An advantage of heuristic methods is that any kind of information can be used in the decision making, such as actual process data, intuition, rain likelihood, experience from previous events, etc. An experienced decision maker (i.e. an operator) may solve the problem effectively, by disregarding all options that are possible but not advisable. In formulating the dynamic constraints of the problem (the system description), the experienced operator in fact uses the best model available, i.e. the actual system.

A disadvantage of heuristic control is that the reasoning behind the decisions is not always clear. As a result solutions may be inconsistent and difficult to evaluate. Moreover, the experience, gained by trial-and-error, will be lost once the operator leaves his job. His successor will make (the same) mistakes all over again. A Decision Support System (DSS), which could be a flow model, an optimization model, an expert system or a combination of these models, is a helpful tool in reducing these problems. Heuristic control is extremely application oriented, since the experience gained at one catchment is just not transferable to another area.

An *expert system* or knowledge base system is a computer model in which it is programmed how experience is gained and how this knowledge is applied to solve a problem. Simply stated, it is a knowledge base containing different kinds of information in which a search pattern (rules) is programmed that should lead to a good or an acceptable solution. It is not an optimization of the problem. In fact, their name implies the idea that the solutions are not found through a model of the physical system, but through a model of the expert (here: the operator).

An expert system is said to be self-learning if it is able to accumulate information. This means that the model is not only programmed to make a decision, but also to evaluate the rules that lead to this decision, analyze them in relation to the operational situation and eventually come up with more appropriate rules. They will often be used interactively with an operator as a Decision Support System, e.g., to diagnose the actual systems state, to identify 'faults' in the system, to recommend emergency procedures etc., (Graillot, 1990), (Babovic, 1991).

An expert system is usually considered less suited to make a decision on the operation strategy itself, since this would require a huge number of rules which are difficult to oversee and to evaluate (this problem will be discussed below). Nevertheless, attempts are being made at the University of Hannover to adapt an expert system for real time control of the sewerage network of Bremen (Khelil et al., 1990). Another possibility would be to use an expert system interactively with a decision model (e.g., a mathematical optimization model) as a pre- and post-processor. In such a case the task of the expert system is to define the actual operational problem (i.e. the objectives and constraints, which both may vary in time), and to interpret the model results. A research into this topic started recently at the Delft University of Technology.

### *4.1.2 Rule based scenarios*

Rule based scenarios can be described as a hierarchy of 'if..then..else..' statements, which relate input variables to output (control) variables by means of boolean logic (true/false). Decision trees (or matrices) may be used to organize the enumeration process. In deriving the rules, the most common approach is to evaluate the system on the basis of performance criteria.

Scenarios require extensive development work. Based on a careful analyses of the water management system, the output variables have to be specified in advance for all possible states of the system. The number of operational objectives is generally limited, in order to keep overview. Obviously it is impossible to include every single input variable and current state variable in the decision tree, meaning that they have to be divided into certain classes. As a consequence, one is never sure whether the best solution is found for each possible situation (unless only one objective is considered). Moreover, the set of rules has to be modified when system parameters have been changed.

A scenario is not necessarily a set of fixed rules that relate an input variable to an output variable. The rules may be formulated as a function of systems state variables, including some adjustable control parameters to manipulate the rules. A typical example of such a scenario can be found in Wervershoof, which is described in Section 9.2. The advantage of this approach is that the decisions are related to the current systems state. Besides, the rules can easily be adjusted to the actual drainage conditions by a proper attuning of the control parameters. The extent to which the rules can be modified to achieve different objectives (which may vary in time) depend on the structure of the algorithm, but generally this flexibility is limited as the rules are formulated to meet a predefined aim.

An important features of control scenarios is that they allow for fast on-line execution. Besides, most technical constraints of the problem can easily be incorporated in the decision tree. The structured enumeration process may facilitate an evaluation of rules, given the number of rules is limited. Like heuristic methods, rule based scenarios are in the first place application oriented. In fact, all present examples of central automatic control are using a rule based scenario.

### *4.1.3 Mathematical optimization*

The methods described above are basically using two separate models: a decision model to formulate an operation strategy and a (mathematical or conceptual) model of the system to check whether the various criteria are maintained. By employing the mathematical tools of Operations Research, these problems can be solved simultaneously.

Finding a solution of the operational problem by means of a mathematical optimization model requires the formulation of an objective function in which the operational tasks are specified in terms of 'unit costs'. This objective function is minimized, subject to a set of constraints, which represent the system. Since optimality is defined for the complete system by the least cost solution of the objective function, the model will produce an operation strategy by which the objectives are met in the best possible way, within the limits of the constraints.

Main advantages of a mathematical optimization model are its flexibility and consistency in decision making. The model determines the optimal strategy, which is unique for each situation, depending only on the applied objective function. The rationale behind the solutions is clear and consistent. Obviously, these are crucial features when investigating the potential of real time control of a water management system. Therefore it can be concluded that, in view of the objectives of the study (Section 1.3), mathematical optimization is the most suitable approach in developing a model to assess the performance of an urban drainage system that is controlled in real time.

In selecting a suitable optimization technique the following criteria are applied:

- The ability to handle many variables to facilitate analysis of systems of a realistic size;
- Fast execution time to facilitate simulation of time series of rain events;
- Robustness, meaning that the decision model should end at a global optimum to avoid the need for an 'external' decision maker.

To guarantee a global optimum, the operational optimization problem should be formulated as a convex problem, meaning that the set of feasible solutions is convex, and the objective function to be minimized (maximized) is convex (concave). A set of points is a convex set if a straight line segment joining any two points of the set lies completely within the set. In other words, every line that is not a boundary line of the feasible region meets the polygon in only two points. A function $f$ is convex (concave) if a straight line, joining any two points on the graph of $f$, lies completely above (below) or on the graph.

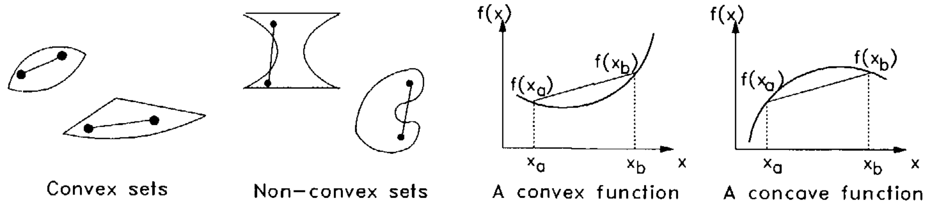


*Figure 4.1. Illustration of convex and non-convex sets*

Examples of classical optimization techniques are optimization without constraints, by which the optimum may be obtained by differential calculus, and the method of the Lagrange multipliers, which transforms an optimization problem subject to equality constraints into an equivalent problem without constraints. Due to computational requirements, the methods are not suitable here.

Some well known optimization techniques, which can be applied to solve convex problems, are linear programming, quadratic programming, dynamic programming, and gradient search procedures (note that the term 'programming' does not refer to computer programming but to planning in a general sense). The discussion in the next sections is confined to the way these techniques require the objective function and constraints to be formulated. For details on the mathematics behind the techniques, reference is made to the several handbooks on applications of Operations Research, e.g., (Wagner, 1975).

## 4.2 Dynamic programming

Dynamic programming (DP) is a multi-stage decision process (and not an algorithm as often assumed) based on the concept of decomposing and solving the optimization problem by a sequence of sub-problems. The key concept upon which optimal policies are obtained is "The Principle of Optimality", which originates from Bellman (1957).

This principle may conceptually be comprehended as follows (Fig. 4.2): Given an optimal trajectory I-II from point A to point C, the portion from any intermediate point B to point C must be the optimal trajectory from B to C. The proof by contradiction for this case is immediate: Assume that some other path, such as II′ is the optimum path from B to C. Then, path I-II′ has less cost then path I-II, which contradicts the fact that I-II is the optimal path from A to C. Hence, II must be the optimal path from B to C.

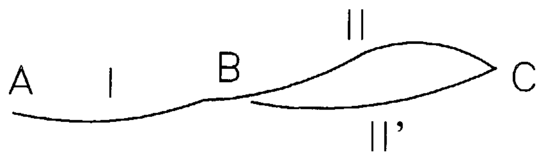


*Figure 4.2. The Principle of Optimality*

Applying this principle to the determination of operation strategies means that each decision at a certain time must be optimal, independent of decisions at former time steps. This leads to the following process:

- Start at the end of $n$ control decisions, i.e. the last time step of the control horizon. Determine the optimum set points for each possible situation at this time step and store the results.
- Next, time step $n$-1 is being considered. Again determine the optimum strategy for each possible situation, but this time by taking into account the strategy as determined at $t = n$. Store the results.
- Repeat this process, until $t = 1$. As the initial state is known, it is possible to determine the optimum strategy for all time steps 1,..,$n$.

(This process is illustrated by a simple example in Appendix B.)

Simply stated, the methodology comprises trying out all possible combinations and find the optimum one. Since all results at each time step have to be stored a lot of computer storage capacity is needed. Bellman called this phenomenon *'the curse of dimensionality'*. Although the method may be very flexible concerning the formulation of the objective function and the physical constraints (theoretically there is no restriction at all), practical applications of DP to solve control problems appear to be restricted to a few state variables due to the fact that today's computers cannot handle more data. Therefore, DP is not considered to be a suitable approach for the control problem under consideration.

It is noted that, despite this dimensionality problem, DP and its many derivatives are successfully applied in the field of water resources management in solving various optimization problems, such as the determination of the optimum design or to find the best policy in solving water allocation problems. An overview of these applications can be found in (Kularathna, 1992).

## 4.3 Linear programming

A very popular solution technique is linear programming (LP), which deals with minimization (or maximization) of a linear objective function $F$, subject to a set of linear constraints. Due to its robustness and its capability to deal with relatively large problems, the procedure has found applications in many different fields. Moreover, basic knowledge of its main principles is sufficient for a successful use of the available standard software.

A general LP problem reads

$$\text{minimize } F = \sum_{i=1}^{m} \sum_{j=1}^{n} c_{ij} x_{ij} \tag{4.1}$$

subject to a set of $m$ linear constraints

$$\sum_{j=1}^{n} a_{ij} x_{ij} \le b_i \; ; \quad \text{for } i = 1,2,\dots,m \tag{4.2}$$

with all variables subject to the non-negativity constraint

$$x_{ij} \ge 0 \; ; \quad \text{forall } i \text{ and } j. \tag{4.3}$$

The $x_{ij}$ are the decision variables, the $a_{ij}$, $b_i$ and $c_{ij}$ the parameters of the model. In this case, the variables and parameters can be explained as follows:

$x_{ij}$ systems state variable, e.g. stored volume, discharge, overflow, etc.;

$c_{ij}$ unit cost of the particular state variable;

$a_{ij}$ coefficient by which the particular state variable is multiplied, (= the coefficients of the system matrix);

$b_i$ upper capacity constraints (and rain input).

For the past few years, various examples of applications of LP to real time control of urban drainage systems can be found in literature, e.g., (Petersen, 1987), (Schilling et al., 1987), (IAWPRC, 1989), (Neugebauer, 1989), and several authors in (EAWAG, 1990). This stimulated various institutes, such as the universities of Denmark and Hannover and the EAWAG in Zürich, to develop their own LP model. It is noted that also this research was started with a LP model, but soon it was found that the problem under consideration cannot be handled properly by a strict linear model. In this section the behaviour of the LP model and its limitations are discussed quite extensively to facilitate an understanding of the modifications required in formulating a more appropriate (non-linear) objective function.

A prototype example shows how the operational optimization problem can be formulated as a LP problem.

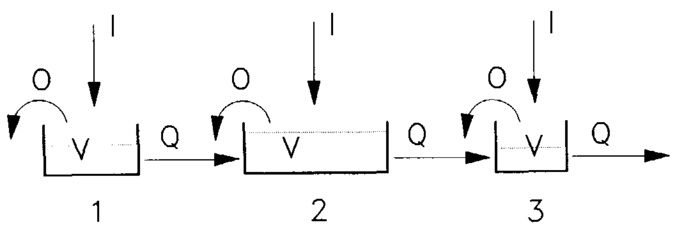


*Figure 4.3. Schematization of a simplified system*

Consider an urban drainage system, which is schematized as a number of reservoirs in series. Each reservoir (node) has an inflow ($I$), which can be stored ($V$), discharged to a downstream node ($Q$) or discharged out of the system by an overflow ($O$).

Using the symbols of Fig. 4.3 the LP problem reads

$$\begin{aligned} minimize F = \sum_{t=1}^{T} [ & cv_1^t V_1(t) + cq_1^t Q_1(t) + co_1^t O_1(t) + \\ & cv_2^t V_2(t) + cq_2^t Q_2(t) + co_2^t O_2(t) + \\ & cv_3^t V_3(t) + cq_3^t Q_3(t) + co_3^t O_3(t) ] \end{aligned} \tag{4.4}$$

subject to a set of capacity constraints

$$\begin{aligned} & 0 \leq V_i(t) \leq V_{i,max} \ ; \\ & 0 \leq Q_i(t) \leq Q_{i,max} \ ; \quad \textit{for all } i \textit{ and } t \\ & 0 \leq O_i(t) \leq O_{i,max} \ ; \end{aligned} \tag{4.5}$$

and a set of dynamic constraints (the system equations)

$$\begin{aligned} & I_1(t) = V_1(t+1) - V_1(t) + Q_1(t) + O_1(t) \ ; \\ & I_2(t) = V_2(t+1) - V_2(t) + Q_2(t) - O_2(t) - Q_1(t-t_{12}) \ ; \quad \textit{for } t = 1,2,...,T \\ & I_3(t) = V_3(t+1) - V_3(t) + Q_3(t) - O_3(t) - Q_2(t-t_{23}) \ ; \end{aligned} \tag{4.6}$$

where $V_i$ is the stored Volume in node $i$ [m$^3$]; $Q_i$ is the discharge of node $i$ [m$^3/\Delta$t]; $O_i$ is Overflow of node $i$ [m$^3/\Delta$t]; $cv_i^t$ is the unit cost of $V_i$ at time $t$ [-]; $cq_i^t$ is the unit cost of $Q_i$ at time $t$ [-]; $co_i^t$ is the unit cost of $O_i$ at time $t$ [-]; $I_i$ is the Inflow to node $i$ [m$^3/\Delta$t]; $t$ is time; $t_{ij}$ is the flow time from node $i$ to node $j$ [$\Delta$t]; and $T$ is the control horizon, for which inflow is specified [$\Delta$t].

As mentioned above, the LP problem, as described by Eq. 4.4, 4.5 and 4.6, can be solved with standard software. A well known algorithm is the Simplex method. This algorithm (in its standard or a revised form) is mostly applied. Neugebauer (1989) showed a more efficient way to solve the operational problem by formulating it as a network flow problem. The solution algorithm itself will be discussed in the next chapter. First, the applicability of the LP model is investigated.

The least cost solution of Eq. 4.4 consists of the optimal values of $V_i(t)$, $Q_i(t)$ and $O_i(t)$, for $t$ = 1,2,...,$T$. If the complete inflow hydrograph $I_i(t)$ is known for all $T$ time steps, the LP problem has to be solved only once to determine the optimal strategy for the complete event. The assumption however that the complete event is known in advance is not only unrealistic, but it will also lead to a huge optimization problem (and the use of big time steps), which may be a problem regarding computer memory requirements. It will certainly restrict the application of the LP model to smaller systems.

Therefore, a better and more realistic approach is to re-formulate the LP problem at each time step of the simulation. This means that an inflow hydrograph $I_i(t)$ consisting of $n$ time steps ($n > T$) requires a succession of $n$-$T$ LP problems to be solved. At $t=1$, the problem is solved for the first $T$ time steps. The calculated values of $V_i(1)$, $Q_i(1)$ and $O_i(1)$ are the initial state for the next time step. Next, the problem is solved for $t=2$ to $T+1$, with a new set of dynamic constraints (Eq. 4.6). This procedure is repeated $n$-$T$ times, until the whole hydrograph is simulated. Obviously, a fast algorithm is required to keep the computational time within acceptable limits.

All applications of LP to operational problems of urban drainage systems are based on a similar concept as described above. Although these studies have demonstrated that LP may be applied in a systems analysis, it is noted the method has some major limitations. Those concerning the linear objective function are discussed in the next section. Section 4.6 deals with the flow routing in the model.

### *4.3.1 Limitations of a linear objective function*

The flexibility of LP concerning the formulation of the objective function is limited, since the function has to be linear. On the other hand, a linear function may have the advantage that, with basic knowledge of the solution routine, it is relatively easy to predict the behaviour of the model and thus to determine the best set of unit costs with respect to the desired systems performance.

To illustrate the behaviour of the LP model only one operational objective is considered, namely to minimize the total overflow volume of the three reservoirs. This is usually considered a main task of the system. Other operational tasks are discussed in Section 4.5.

For the given example, the unit costs of the variables can be determined as follows. If the complete inflow hydrograph is known (consisting of $T$ time steps) then the problem can easily be solved by applying unit costs of overflows ($co_i^t$) equal to 1 (or any non-negative value) and all other unit costs ($cv_i^t$, $cq_i^t$) equal to zero. In this case the problem reads

$$\text{minimize } F = \sum_{i=1}^{3} \sum_{t=1}^{T} O_i(t) \tag{4.7}$$

subject to the same constraints as described by Eq. 4.5 & 4.6.

By definition, multiple optimal solutions exist for $V_i(t)$ and $Q_i(t)$ as no unit costs have been specified for these variables. Concerning the systems behaviour, Eq. 4.7 only ensures that all storage capacity of the three reservoirs is used before an overflow may occur. Depending on the capacity constraints (Eq. 4.5) several options may be possible to achieve this.

As mentioned above, the whole inflow hydrograph $I_i(t)$ can generally not be foreseen meaning that the operational problem consists in fact of a succession of LP problems (with a limited control horizon $T$). This implies that we have to make a decision on how to use the storage and discharge capacity of the system elements, taking account of this uncertainty.

In Chapter 3 we described a desired systems behaviour which provides a suitable solution to this problem. Here it is investigated whether this behaviour (or any plausible solution) can be translated into a proper set of (constant) unit costs.

To minimize overflows the unit costs of overflows should obviously be given a value much greater than the costs of storage and transport. Besides, the unit costs of storage should be given a value greater then the unit costs of transport, in order to discharge the water out of the system. Hence, in this case the unit costs should meet the following criterion

$$co_i^t > cv_i^t > cq_i^t \; ; \; \textit{for all } i \textit{ and } t. \tag{4.8}$$

Furthermore, the unit costs of an overflow should slightly decrease in time to avoid multiple optimal solutions (meaning that an overflow at $t$ and an overflow at $t+1$ would both lead to minimum cost). As inflow predictions are subject to errors, a decision to overflow should be postponed as much as possible until overflows cannot be avoided any more. Moreover, if overflow structures are uncontrolled (as in most cases), it is even physically impossible that overflows could occur as long as the current water level is below the crest level of the overflow weir. In formula, this criterion reads

$$co_i^t > co_i^{t+1} \; ; \; \textit{for all } i \textit{ and } t. \tag{4.9}$$

The unit costs of storage and discharge are usually chosen as a constant in time, or

$$cv_i^t = cv_i^{t+1} \textit{ and } cq_i^t = cq_i^{t+1} \; ; \; \textit{for all } i \textit{ and } t. \tag{4.10}$$

In (Petersen, 1987) and (Schilling et al., 1987) it was demonstrated that the LP model is quite sensitive to inaccurate inflow predictions. Furthermore, it appeared to be difficult to determine the best set of unit costs, yielding optimal performance of the system. This set was derived by trial-and-error. Although a basic knowledge of the principles of LP is sufficient to derive some criteria, such as Eq. 4.8 and 4.9, it can be concluded that a sensitivity analysis will mostly be required to verify the validity of the chosen set of unit costs.

From Eq. 4.4 it can be read that the optimal solution is independent of the value of the state variables at $t = 0$, or the initial systems state. In fact it does not matter whether the objective function is minimized for $t=1$ to $T$ or $t = 0$ to $T$ as the state variables at $t = 0$ are constants. The least cost solution of the LP model is determined by the constant unit costs of the state variables and the specified inflow during the time horizon $T$ for which the problem is optimized. As the unit costs are constants, this implies that the best set of unit costs is depending on the inflow hydrograph that is calculated (which explains the above mentioned findings of Petersen and Schilling). In the LP model this problem does obviously not occur if the time horizon covers the whole inflow hydrograph. In that case the problem can be formulated by Eq. 4.7 in which in a strict sense no unit costs have to be specified.

In Chapter 3 it was concluded that concerning the problem of minimizing overflows a plausible solution is to keep the use of storage at all sub-systems at a relatively equal level (related to the storage capacity of the particular sub-system). When using fixed unit costs of storage and transport, however, it is impossible to control the filling (and emptying) behaviour of the system in a satisfactory way. This may be illustrated by a simple example.

### *4.3.2 A simple example*

To keep overview, the operational problem of the three reservoirs in series, as illustrated in Fig. 4.3, is further simplified by assuming that the optimal values of $Q_2(t)$ and $Q_3(t)$ are known, say equal to their maximum for all $t$. This means that only the optimal values of $Q_1(t)$ have to be determined. The inflow is assumed constant. To illustrate the filling (and emptying) behaviour of the model the inflow volume is chosen so that, within the control horizon, no overflow occurs, no matter what strategy is being applied.

Since $Q_2(t)$ and $Q_3(t)$ are known, and $O_i(t)$ are equal to zero, the objective function $F$ to be minimized reads (using the same symbols as in Eq. 4.4)

$$F = \sum_{t=1}^{T} [cv_1^t V_1(t) + cq_1^t Q_1(t) + cv_2^t V_2(t)] \tag{4.11}$$

In this example, the following parameters are applied:

| | |
|---|---|
| Control horizon: | $T = 3$ ; |
| Capacity constraints: | $V_{1,\max} = 400$; $V_{2,\max} = 500$; $Q_{1,\max} = 10$ ; |
| Dynamic constraints: | $I_1(t) = 20$, $I_2(t) = 30$, $Q_2(t) = Q_{2,\max} = 20$, *for* $t = 1,2,3$ |

Since the inflow and the unit costs are constant for the whole control horizon, only two strategies have to be considered, namely $Q_1(t) = Q_{1,\max}$ or $Q_1(t) = 0$ for $t = 1,2,3$, as these form the extremes of the problem. Due to the linearity of the problem, the total cost of any other strategy will fall between the costs of these solutions. Table 4.1 shows the results of 3 illustrative cases:

| | | |
|---|---|---|
| Case 1: | $cv_1^t = cv_2^t (= 10)$ | ; *initial state*: $V_1(0) = V_2(0) = 10$ |
| Case 2: | $cv_1^t (= 10) > cv_2^t (= 5)$ | ; *initial state*: $V_1(0) = V_2(0) = 10$ |
| Case 3: | $cv_1^t (= 10) > cv_2^t (= 5)$ | ; *initial state*: $V_1(0) = 10$, $V_2(0) = 440$ |

In the first case the unit costs of $V_1$ and $V_2$ are given an equal value. As a result the least cost solution is found for $Q_1(t) = 0$, as every $Q_1(t) > 0$ will increase the cost (negative discharge is not allowed here). With this set of unit costs, no water is transported from Node 1 as long as it has to be temporarily stored at Node 2. If the inflow continues, water will be stored until an overflow can be foreseen within the control horizon. At that moment water will be transported, since the unit costs of overflow are generally given a value greater than the costs of storage and transport. (It should be noted that in this case the objective function, Eq. 4.11, should be extended with the variables $O_i(t)$).

To ensure that water is transported from Node 1 to Node 2 at an earlier stage, the unit costs should be given a value so that the cost of storing one unit at Node 1, is greater than the sum of the unit costs of $Q_1$ and $V_2$. This is shown in Case 2, where $Q_1(t) = Q_{1,\max} = 10$ is the least cost solution.

*Table 4.1. A numerical example (the least cost solutions are grey shaded)*

| var. | CASE 1 | | | | | CASE 2 | | | | | CASE 3 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Q1 = 0 | | Q1 = max | | | Q1 = 0 | | Q1 = max | | | Q1 = 0 | | Q1 = max | |
| | unit cost | value var. | cost | value var. | cost | unit cost | value var. | cost | value var. | cost | unit cost | value var. | cost | value var. | cost |
| Q1(1) | 1 | 0 | 0 | 10 | 10 | 1 | 0 | 0 | 10 | 10 | 1 | 0 | 0 | 10 | 10 |
| Q1(2) | 1 | 0 | 0 | 10 | 10 | 1 | 0 | 0 | 10 | 10 | 1 | 0 | 0 | 10 | 10 |
| Q1(3) | 1 | 0 | 0 | 10 | 10 | 1 | 0 | 0 | 10 | 10 | 1 | 0 | 0 | 10 | 10 |
| V1(0) | | 10 | | 10 | | | 10 | | 10 | | | 10 | | 10 | |
| V1(1) | 10 | 30 | 300 | 20 | 200 | 10 | 30 | 300 | 20 | 200 | 10 | 30 | 300 | 20 | 200 |
| V1(2) | 10 | 50 | 500 | 30 | 300 | 10 | 50 | 500 | 30 | 300 | 10 | 50 | 500 | 30 | 300 |
| V1(3) | 10 | 70 | 700 | 40 | 400 | 10 | 70 | 700 | 40 | 400 | 10 | 70 | 700 | 40 | 400 |
| V2(0) | | 10 | | 10 | | | 10 | | 10 | | | 440 | | 440 | |
| V2(1) | 10 | 20 | 200 | 30 | 300 | 5 | 20 | 100 | 30 | 150 | 5 | 450 | 2250 | 460 | 2300 |
| V2(2) | 10 | 30 | 300 | 50 | 500 | 5 | 30 | 150 | 50 | 250 | 5 | 460 | 2300 | 480 | 2400 |
| V2(3) | 10 | 40 | 400 | 70 | 700 | 5 | 40 | 200 | 70 | 350 | 5 | 470 | 2350 | 500 | 2500 |
| F: | | | 2400 | | 2430 | | | 1950 | | 1680 | | | 8400 | | 8130 |

Although water is transported now, the problem remains that the optimal strategy is independent of the systems state, (until overflows can be foreseen). The consequences are shown in Case 3, where at $t = 0$ Node 2 is almost filled to its maximum whereas relatively much storage is still available at Node 1. Even this case, the strategy $Q_1(t) = Q_{1,\max} = 10$ is the least cost solution.

Concerning the above formulated operational objective, to minimize the overflow volume, the applied strategy in Case 3 is evidently not the optimal one. If, at $t = 4$ the inflow volume at Node 2 is greater than the discharge capacity $Q_{2,\max}$, it will be impossible to prevent an overflow from Node 2, although storage capacity at Node 1 is still available.

Note that extending the model by incorporating a terminal state condition, like reservoirs empty at $t = T$, will not change the performance as described by table 4.1.

### 4.3.3 Discussion

At a first glance, the example might look rather awkward. Nevertheless it illustrates some basic properties of a LP model (with a limited control horizon), as often applied in solving operational problems of urban drainage systems.

For the given example, it can easily be seen that another set of unit costs would have resulted in a better strategy. The question is which set and how to determine it ? Even this simple problem incorporates many possibilities to manipulate the process, e.g., we may vary not only the ratio between the values of the unit costs $cv_i^t$ and $cq_i^t$, but also vary these costs in time (as indicated by the superscript t). Note that in the latter case $Q_1(t) = 0$ and $Q_1(t) = Q_{1,max}$ (for all $t$) are not necessarily the extremes of the problem. For larger systems with more decision variables and with temporal and spatial variations of the systems loading, the determination of an optimal set of unit costs can become very complex, if not impossible.

The operational optimization problem is in fact transferred into a new optimization problem, i.e. the formulation of the optimal objective function. Usually, an acceptable set of unit costs has to be derived by 'trial-and-error' (i.e. heuristically). This means that the objections, which were raised against heuristic methods in section 4.3.1, concerning the lack in consistency in decision making, are in a strict sense also valid when applying a standard LP model.

The specification of the best set of unit costs is further hampered by the fact that each loading of the system demands a unique set of unit costs, meaning that a sensitivity analysis will be required for each event that is simulated. Conversely this implies that the inflow to the system (in time and place) has to be known in advance to determine the optimal set of unit costs. Therefore LP cannot be considered an appropriate approach to simulate time series of rain events, as each event would require a different objective function.

Generally it can be concluded that a desired systems performance, e.g. as described in Chapter 3, which should be maintained in the best possible way, independently of the systems loading, cannot be expressed by a linear objective function with constant unit costs. This approach is therefore not applicable in developing a model to assess the impact of real time control of an urban drainage system.

## 4.4 Non-linear programming

The above mentioned limitations of the LP model can be overcome by applying an objective function with unit costs, which are depending on the state of the system, and thus by formulating the operational problem as a non-linear programming (NLP) problem.

As was concluded in Chapter 3 a plausible solution to minimize overflows is to keep the use of storage at all sub-systems at a relatively equal level (related to the storage capacity of the particular sub-system). This desired systems behaviour can be expressed by an objective function in which the unit costs of the variables $V_i$ are determined as a function of the filling degree of the system. In formula this function reads (using the same symbols as above)

$$cv_i^t = \frac{V_i(t)}{V_{i,max}} \cdot \kappa_i \tag{4.12}$$

where $\kappa_i$ is the maximum unit cost of $V_i$. This function can also be written as

$$cv_i^t = \frac{\kappa_i}{V_{i,max}} \cdot V_i(t) = constant \cdot V_i(t) \tag{4.13}$$

Substitution of this unit cost function in the objective function (Eq. 4.1 or 4.4) yields a (convex) Quadratic Programming problem, which, in a general form, can be formulated as

$$minimize\ F = \sum_{i=1}^{m} \sum_{j=1}^{n} c_{ij} x_{ij}^2 \tag{4.14}$$

with the same linear constraints as required by LP.

Quadratic programming is the only NLP technique for which standard routines have been developed. Other examples of techniques that can be applied to solve convex non-linear optimization problem are gradient search methods. These are stepwise procedures, in which the solution is improved in the direction of the gradient of the objective function until the optimum is reached with an accuracy according to the width of steps. As convex problems contain no local optima, the optimal solution will be a global optimum.

In general, the algorithms for solving non-linear convex optimization problems are less powerful than the available LP procedures. The search pattern is more complicated and time consuming than LP routines, thus limiting the application to smaller problems. For a time series analysis the required computational time might be too big.

A possibility would be to approximate the objective function as formulated above by applying a constant basic cost for each node $i$, depending on the value of $V_i(0)$, and a linear increase depending on the variable $V_i(t)$. In a strict sense, such a cost function is a concave function. Since the cost function is to be minimized, the resulting optimization problem is non-convex. Accordingly, the problem may contain local optima. However, specialized algorithms have been developed that are able to determine an accurate approximation of the least cost solution, thereby avoiding the solution process ending at a local optimum. Examples are mixed integer programming and the (more effective) branch-and-bound algorithm as described by Dehnert (1974), who replaces the problem as a succession of linear sub-problems that have to be solved in an iterative manner. However, the disadvantage remains that computer storage requirements and computing time will drastically increase, compared to a LP approach.

It has to be considered that finding the exact least cost solution of Eq. 4.14 is not the most important issue here. The main aim is to find a suitable objective function which expresses the desired systems performance in a satisfactory way.

As mentioned above, since the time horizon for which inflow can be specified is limited the operational optimization problem is in all cases to be solved by means of a succession of sub-problems (no matter whether the problem is formulated as a LP or a NLP problem). This means that at each time step of the simulated inflow hydrograph the optimal values of the systems state variables at $t=1$ have to be approximated as accurate as possible. These form the initial systems state at the next time step, for which a new optimization problem is formulated. Inaccurate predictions may be corrected in the following time steps of the simulation.

This consideration may help us in formulating a more effective way to solve the operational optimization problem. The fastest way is to transform the non-linear problem at each time step of the simulation into a linear (convex) problem with constant unit costs, which are determined on the basis of the current systems state variables. Evidently, the linearized problem should give a sufficiently accurate approximation of the actual problem. A first order approximation of the cost function as described by Eq. 4.12 reads

$$cv_i^t = [\frac{V_i(0)}{V_{i,max}} + \frac{\partial V_i}{\partial t} \cdot t]\ \kappa_i = [\alpha_{i,1} + \alpha_{i,2} \cdot t]\ \kappa_i\ ; \quad (cv_i^t \leq \kappa_i) \tag{4.15}$$

where $cv_i^t$ is the unit cost of $V_i$ at time $t$; $\kappa_i$ is a constant, denoting the maximum unit cost of $V_i$.

Eq. 4.15 denotes that the value of unit cost $cv_i^t$ depends on the current filling degree at node $i$ ($= \alpha_{i,1}$) and the rate of increase of the variable $V_i$ ($= \alpha_{i,2}$). The constant $\alpha_{i,1}$ is known and determined by the value of $V_i(0)$. The value of $\alpha_{i,2}$ can be approximated by the average increase of $V_i$ during a relatively short horizon $T^*$

$$\alpha_{i,2} = \frac{1}{T^*} \cdot \sum_{t=1}^{T^*} [V_i(t) - V_i(0)] \tag{4.16}$$

Note that Eq. 4.15 and Eq. 4.12 yield exactly the same value for $t = 0$, namely $cv_i^0 = \alpha_{i,1} \cdot \kappa_i$. The value of $\alpha_{i,2}$ can be approximated by Eq. 4.16, using the predicted values of $V_i$ of the optimization problem of the preceding time step. Since the value of $\alpha_{i,2}$ can only be approximated with sufficient accuracy for a relatively short horizon (in the order of 0.5-1 hour), constant unit costs may be applied for $t > T^*$. This means that $cv_i^t = [\alpha_{i,1} + \alpha_{i,2} \cdot T^*]\ \kappa_i$ *for* $t > T^*$.

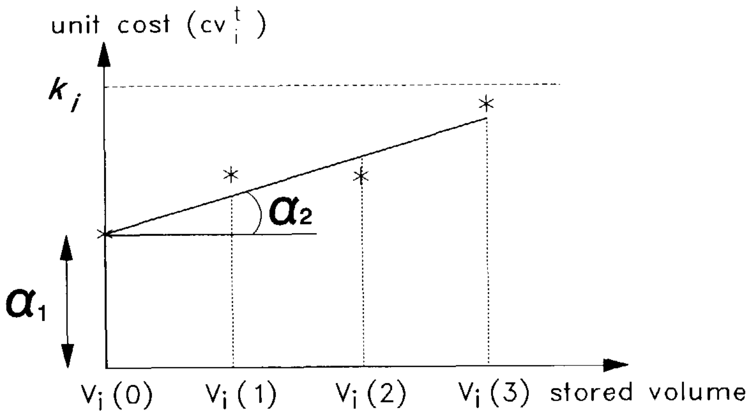


*Figure 4.4. Definition sketch for the unit cost function of* $V_i$

Tests have indicated that the objective function as described by Eq. 4.15 & 4.16 yields a systems performance by which differences in the filling degree of the sub-systems are kept within acceptable limits, which indicates that this first order approximation of Eq. 4.13 is sufficient for the problem under consideration. The use of storage within the system can be manipulated by applying different values of $\kappa_i$ for the different nodes. This will be discussed below.

## 4.5 Formulation of the objective function

Until now, minimization of overflows by means of optimizing the use of the available storage has been considered the main operational task. The main aim of operating an urban drainage system is to minimize the negative impact of pollution loads on receiving water bodies (see Section 3.2). To achieve this, minimizing overflows is obviously the first thing to be considered, but it is generally not sufficient to realize this aim in the best possible way. In this section it is discussed how to formulate an appropriate objective function.

### *4.5.1 Unit costs of storage*

The way the storage capacity of the system is used is determined by the unit cost functions of $V_i$, or the coefficients $\kappa_i$ of Eq. 4.15. If the operational objective is to minimize the total overflow volume, without making any distinction between the place of possible overflows, the coefficients $\kappa_i$ should be given the same value for all nodes. This means that the filling degree at all nodes (storage elements) of the system will be kept relatively equal (if physically possible). As a result, overflows will not occur until all available storage is used to its capacity.

It is noted that contrary to the standard LP approach there is no need to decrease the unit cost of $V_i$ in downstream direction in order to transport the water (Eq. 4.8). Since the unit cost are determined by the actual filling degree and the rate of increase of the stored volume at the nodes of the system, water will be transported if a downstream node has relatively more storage capacity available than an upstream node.

Possible differences in sensitivity of receiving water bodies may be a reason to apply different values of $\kappa_i$. For example, overflows on ponds or small streams in urbanized areas with a low through flow are in general less preferable than overflows on big rivers with a higher self cleansing capacity. The ratio between the coefficients $\kappa_i$ express the allowable differences in filling degrees of the different nodes.

For example, applying $\kappa_1 = 1.25 \cdot \kappa_2$ means that the optimum solution of the objective function is found if the filling degrees at Nodes 1 and 2 are in the proportion of 80 to 100 (as $80\% \cdot \kappa_1 = 100\% \cdot \kappa_2$). This means that at the moment that Node 2 is filled to its maximum, Node 1 will still have 20% of its storage capacity available. This remaining 20% will only be utilized if necessary to prevent an overflow at Node 2, as the unit costs of overflows are generally given a much higher value then the costs of storing and discharging the water. However, the probability of an overflow at Node 1 will be smaller than the probability of an overflow at Node 2.

The location of a possible overflow is further determined by the unit costs of the variables $O_i$. Obviously, the unit cost of an overflow to the most sensitive receiving water should be given the highest value.

### 4.5.2 *Unit costs of overflows*

If no information is available on the current pollution concentrations of the sewage (in time and place) the only rational decision that can be made concerning overflows is where to overflow. This implies that the unit cost of overflows ($co_i^t$) should be given a constant value, depending on the type and sensitivity of the receiving water body.

Note that possible overflows will only affect the systems behaviour if they can be foreseen within the time horizon $T$, because their unit costs are generally given a much higher value than the costs of storage and discharge. In Chapter 3 it was concluded that the response time and the ability of the system to correct a perturbation of the system determines the extent to which one may benefit from predicting disturbances. In the case where the system is able to return to its desired state (here: a certain filling degree, expressed by the values of $\kappa_i$) within acceptable time limits (in the order of a few time steps) currently monitored process data are sufficient to derive a suitable strategy. If this is not the case, then the control horizon will be an important factor in the decision making. A discussion into this topic will follow in Chapters 6 and 7.

A further refinement of the operation strategy, aiming at optimal water pollution control, requires a pollution transport model, which calculates the pollution concentrations, depending on the decision variables of the optimization model. The problem is that until today no water quality model has been developed which is able to determine the actual pollutograph with a sufficient degree of accuracy. However, as was noticed in section 2.3, it is a topic into which much research effort is being put.

Suppose a sufficiently accurate pollution transport model is available, then the problem is how to use this information to minimize pollution loads to receiving waters. A plausible solution is to express this objective by the unit cost function of overflows. Similar to the cost function $cv_i^t$, a unit cost function $co_i^t$ can be applied with a basic cost, depending on the function and sensitivity of the receiving water, and a linear increase, depending on the pollution concentration at the respective node. In formula, this unit cost function reads

$$co_i^t = \beta_{i,1} + \beta_{i,2} C_i(t) \tag{4.17}$$

where $co_i^t$ is the unit cost of $O_i$; $C_i(t)$ is the pollution concentration at node $i$; $\beta_{i,1}$ is the basic unit cost of $O_i$, which should be chosen according to the type and sensitivity of the receiving water; $\beta_{i,2}$ is a factor, which determines the 'weight' of $C_i(t)$ in determining the unit costs.

To ensure that the model will always first optimize the use of storage and transport, before decisions may be influenced by possible overflows and their pollution loads, $\beta_{i,1}$ should be given a value which is greater than the maximum unit cost of $V_i = \max\{\kappa_i\}$.

The behaviour of the optimization model, based the unit cost function 4.17, will be investigated in Chapter 6, using an ideal mixed reservoir approach to describe the pollution transport. Purpose of this investigation is only to indicate the potential to improve the operation strategy concerning minimization of pollution loads. In practice, a more accurate pollution transport model is obviously required. It is noted that in all other case studies that are described in this report, the coefficient $\beta_{i,2}$ is set equal to zero.

### *4.5.3 Flow to the treatment plant*

The treatment efficiency is affected by a.o. the intensity and duration of the hydraulic loading (Harremoës, 1989). A better utilization of the storage capacity of the system means that more water is led through the treatment plant. This can result in a decrease of effluent quality since the hydraulic load will be higher for a longer period. Due to dilution of the sewage the pollutant concentration of an overflow can become close to (or even less than) the concentration of the treatment plant effluent. In determining the operation strategy of the integral urban drainage system this phenomenon should, as far as possible, be taken into account.

Theoretically it is desirable to control the influent to each part of the treatment plant as all units have their design limitations. To model the total output of pollutants from a treatment plant during storm conditions requires complex models for the main unit operations, such as primary settling, the activated sludge process and the final clarifier (IAWPRC, 1987). As mentioned in Chapter 3, it has been decided to define the process to be controlled as the part between inflow and outflow of the sewerage system, where the optimum discharge to the treatment plant can be considered as an input to the problem (i.e. a capacity constraint that may vary in time).

It is noted that it would be quite difficult to express the relation between sewer system and treatment by a unit cost function of the discharge to the treatment plant. Unlike the unit costs of overflows where the calculated pollution concentration is used only to facilitate a comparison between the possible places of overflows, such a unit cost function would require an accurate pollution transport model.

Instead of using variable unit costs, a better approach would be to relate the upper bound of the flow to the treatment plant to the current conditions at the plant. For example, if sludge rise occurs at the treatment plant (or can be predicted) as a result of a long hydraulic peak load, the upper bound of the flow to the treatment plant should be decreased to an acceptable level. Similar to the SAMBA-RENS module (Harremoës, 1989), which was mentioned in Section 2.5, such an approach would only require a number of time constants, which describe the treatment efficiencies during the different stages of a storm. It should be noted that within the framework of this research, this concept has not been worked out in detail.

## 4.6 Flow routing

A general disadvantage of mathematical optimization is that a simplification of the flow routing through the system is required, which in most cases involves linearization or a lumped storage approach. Ideally, the hydrodynamic constraints of the problem are described by the full dynamic wave approximation (Eq. 2.7 and 2.8), but due to computational requirements, no mathematical optimization technique is available which allows for such detailed flow routing.

The hydrodynamic constraints of the operational problem are approximated by a lumped storage approach, meaning that the system is described by the (linear) continuity equations of a number of storage units (e.g., Eq. 4.6). It should obviously be investigated whether linear flow routing does not oversimplify the system behaviour, which might lead to unrealistic results.

Most Dutch urban drainage systems are very flat and nearly all systems are supplied with pumps. In this case, the governing equation is the continuity equation, meaning that linear flow routing may perform well. In the Netherlands, several studies have shown that in flat areas the results of a lumped hydrologic model are sufficiently accurate, certainly in predicting overflow frequencies and volumes. However, in steeper sloped systems hydrodynamic phenomena, like backwater and surcharge, may be important, meaning that a lumped storage approach might not be sufficient to describe the system.

A possibility to tackle this problem in practice is to minimize the inaccuracies of the optimization model by supplying the model with a new systems state before continuing the calculation of the operation strategy for the next time step. This systems state has to be derived from currently measured system data.

Another possibility is to use a detailed hydrodynamic flow model, which runs parallel to the optimization model from which it receives the control commands. The latter approach has been applied by Petersen (1987) to verify the results of a LP model of the Bremen catchment (a flat area). The differences between the flooding and overflow volumes, computed by a special version of the hydrodynamic flow model HYSTEM-EXTRAN and the optimization model are in this case relatively small, which indicates that simple flow routing may perform well.

A drawback of separate flow routing is the computational requirements. The method is time consuming, and therefore not suitable when simulating series of rain events. Besides, a hydrodynamic flow model is needed, capable of simulating controlled flow regulators, receiving external control input during the simulation. As mentioned in Section 2.3.3, the options in the existing flow models to simulate controlled flow regulators are limited.

The concept of non-linear (and time variant) unit costs, which are related to systems state variables and minimized by linearization, can also be applied to improve the flow routing model. Since the non-linear optimization problem is solved by means of a succession of optimization problems, which are re-formulated at each time step of the simulation, the capacity constraints are allowed to vary in time. This means that the upper (and lower) bounds of the flows can be related to the current systems state.

The flow at the new time level can in principle be computed explicitly out of the values of the systems state variables at the preceding time level, e.g. by a formula like

$$Q_{ij}(t+\Delta t) = K_{ij}\ [h_i(t) - h_j(t)] \tag{4.18}$$

where $Q_{ij}(t+\Delta t)$ is the flow between nodes $i$ and $j$ at the new time level, $K_{ij}$ is a flow coefficient and $h_i(t)$ is the water level at node $i$.

The problem is that the optimization model assumes that all flows are continuously controllable over their full range, i.e. between the upper and lower bound of the variable. However to simulate gravity flow between two connected nodes or a static controlled flow regulator we want the flow to be determined on the basis of the current water level.

The principle is then to employ a variable upper bound of the flow, which is related to the current systems state (= the water levels at the two particular nodes), and to make sure that the optimal value of this flow is 'forced' to take its maximum. The latter can easily be achieved by applying a unit cost $cq_i^t$ of the particular variable equal to zero, or by setting the lower and upper bound of the variable to the same value.

The flow between two nodes can in principle be determined by means of any equation of motion, e.g., a stage-discharge relationship or weir formula, to simulate a fixed weir or diversion structure; a (non)linear reservoir (Eq. 2.6) to simulate uncontrolled outflow of a reservoir; or a resistance formula like Manning-Strickler (Eq. 2.2) or Chézy (Eq. 2.3) to simulate pipe flow. Note that the optimization routine automatically maintains continuity. The concept as described above is presently being further developed.

## 4.7 Concluding remarks

There are three options to determine the operation strategy: heuristic methods, scenarios and mathematical optimization. The latter appears to be the most suited for a systems analysis. It has been shown that the use of a standard LP model, with fixed unit cost is generally not the most suitable approach to solve the operational optimization problem. A simple, yet illustrative, example of a LP model shows that the actual operational problem does not fit a strict linear approach.

A non-linear programming (NLP) model is proposed that solves the main shortcomings of the LP model. In formulating the objective function of the problem, the unit costs of the systems state variables are determined on the basis of the current systems state. On the next page, the general optimization problem is summarized in a mathematical form.

The NLP problem is replaced by a succession of linear problems, meaning that at each time step of the simulation the problem is transformed into a LP problem (with constant unit costs). Possible procedures to solve the linear sub-problems are discussed in the next chapter.

As the problem is re-formulated at every time step of the simulated hydrograph it is possible to apply variable bounds to the system state variables. This concept may be applied to simulate 'uncontrolled' flow in the system, meaning that the flow at the new time level can be computed explicitly out of the values of the systems state variables at the preceding time level.

No examples are known of practical applications of real time control, based on mathematical optimization. In the Netherlands, a typical example of heuristic control can be found in Rotterdam, which is described in Chapter 7. The few automatic control systems that have been implemented in urban drainage systems are all based on a rule based scenario. Examples of automatic control systems in the Netherlands can be found in Wervershoof (Chapter 8), Olburgen, and Utrecht.

The fact that no examples of on-line optimization are known, does not mean that this technique could not be applied in practice. It is the author's belief that this is mainly due to ignorance of the techniques of Operations Research, and the non-existence of a flexible model that has been tailored for the problem.

As opposed to mathematical optimization, rule based scenarios are said to have the advantages of fast execution time and the structured process of decision making, which would be more understandable for the operator(s). This even induced the concept of formulating a rule based scenario by which the rules are derived beforehand using mathematical optimization, e.g., (Almeida, 1992). It is, however, a misunderstanding that optimization routines would be too slow for on-line use in practice. Besides, decision trees can become quite complex. In many cases, the use of unit costs (or weights) may be more suited to facilitate an understanding of the reasoning behind the decisions on the operation strategy.

The operational problem of a system of $n$ nodes, with each node $i$ having two outgoing flows $Q_i$ and $O_i$ (obviously the number of flows can be increased), can be formulated as

$$\text{minimize } F = \sum_{i=1}^{n} \sum_{t=1}^{T} [cv_i^t V_i(t) + cq_i^t Q_i(t) + co_i^t O_i(t)] \tag{4.19}$$

subject to

$$\sum_{i=1}^{n} [V_i(t+1) - V_i(t) + Q_i(t) + O_i(t)] - Q_{i,up}(t) - I_i(t) = 0; \; \text{for } t = 1,2,...,T \tag{4.20}$$

and

$$\begin{aligned} &0 \leq V_{i,min}^t \leq V_i(t) \leq V_{i,max}^t; \\ &0 \leq Q_{i,min}^t \leq Q_i(t) \leq Q_{i,max}^t; \quad \text{for } i = 1,2,...,n \\ &0 \leq O_{i,min}^t \leq O_i(t) \leq O_{i,max}^t; \quad \text{and } t = 1,2,...,T \end{aligned} \tag{4.21}$$

where

| | | |
|---|---|---|
| $n$ | = | number of nodes (storage elements); |
| $T$ | = | number of $t$ for which inflow is specified (control horizon); |
| $V_i(t)$ | = | stored Volume at node $i$ at time $t$ [m3]; |
| $Q_{i,up}(t)$ | = | inflow from upstream nodes to node $i$ at time $t$ [m3/Δt]; |
| $Q_i(t)$ | = | discharge from node $i$ at time $t$ [m3/Δt]; |
| $O_i(t)$ | = | outflow (e.g., overflow) at node $i$ at time $t$ [m3/Δt]; |
| $I_i(t)$ | = | rain Inflow to node $i$ at time $t$ [m3/Δt]; |
| $V^t_{i,min}$ | = | lower bound of $V_i$ at time $t$ [m3/Δt], (in most cases equal to 0); |
| $V^t_{i,max}$ | = | upper bound of $V_i$ at time $t$ [m3/Δt]; |
| $Q^t_{i,min}$ | = | lower bound of $Q_i$ at time $t$ [m3/Δt], (in most cases equal to 0); |
| $Q^t_{i,max}$ | = | upper bound of $Q_i$ at time $t$ [m3/Δt], (may be formulated as a function of the systems state at $t$=0); |
| $O^t_{i,min}$ | = | lower bound of $O_i$ at time $t$ [m3/Δt], (in most cases equal to 0); |
| $O^t_{i,max}$ | = | upper bound of $O_i$ at time $t$ [m3/Δt], (in case of gravity overflow mostly equal to ∞); |
| $cv_i^t$ | = | unit cost function of $V_i$, which (in its basic form) reads |

$$cv_i^t = [\frac{V_i(0)}{V_{i,max}} + \frac{\partial V_i}{\partial t} \cdot t] \; \kappa_i \; ; \quad (cv_i^t \leq \kappa_i) \tag{4.22}$$

$cq_i^t$ = unit cost function of $Q_i$, which reads

$$\begin{aligned} &cq_i^t = \gamma_i(=constant) \; ; \; \text{if } Q_i \text{ is controlled;} \\ &cq_n^t = 0 \qquad\qquad\qquad ; \; \text{if } Q_i \text{ is uncontrolled.} \end{aligned} \tag{4.23}$$

$co_i^t$ = unit cost function of $O_i$, which reads

$$co_i^t = \beta_{i,1} + \beta_{i,2} C_i(t) \tag{4.24}$$

$C_i(t)$ = pollution Concentration at node $i$ at time $t$ [m3/Δt];

*Figure 4.5. The general mathematical problem*

# 5 A numerical model: LOCUS

## 5.1 Solution algorithm

The non-linear programming problem as formulated in Chapter 4 can be solved by linearization. As explained above, at each time step of the inflow hydrograph the problem is transformed into a linear problem that approximates the original problem with sufficient accuracy. This means that in determining the operation strategy a succession of linear problems has to be solved. An advantage of this approach is that it allows the use of powerful linear programming routines.

For solving the linear sub-problems different procedures are possible. To begin with, they could be dealt with by the related standard techniques, such as the Simplex Method and its variants. However, by exploiting the special structure of the problem, i.e. by formulating it as a network model, major efficiencies can be obtained, both concerning computing time and the number of variables and constraints that can be handled by the model. Using a network model for the problem in consideration is not only worthwhile, but in most cases even a practical necessity.

Recall the general Linear Programming problem

$$\textit{minimize}\ F = \sum_{i=1}^{m} \sum_{j=1}^{n} c_{ij} x_{ij} \tag{5.1}$$

subject to a set of $m$ linear constraints

$$\sum_{j=1}^{n} a_{ij} x_{ij} \leq b_i \ ; \quad \textit{for}\ i = 1,2,\ldots,m \tag{5.2}$$

with all variables subject to the non-negativity constraint.

The principle of the Simplex algorithm may be illustrated by a geometrical interpretation. The linear constraints (Eq. 5.2) form a region of feasible solutions in the $n$-dimensional space, a so-called Simplex. Due to the linearity of the problem, the minimum of the objective function (Eq. 5.1) falls at one of the corners of the Simplex. The algorithm starts at any corner and selects step by step a neighbouring corner with an improved value of the objective function. Since the Simplex has a finite number of corners and no local optima (since it is a convex optimization problem), the iterative procedure will reach the optimum by a finite number of iterations.

How this procedure can be translated into a number of matrix (or tableau) operations is described by various handbooks on applications of Operations Research, e.g., (Wagner, 1975), (Müller-Merbach, 1971), (Hillier et al., 1980).

Solving this problem by a standard procedure, which is based on the Simplex algorithm, requires the specification of a matrix containing the values of $a_{ij}$ of Eq. 5.2, and one or two vectors (depending on the procedure) containing the lower and upper bounds $b_i$ of the variables. In Fig. 5.1, the matrix is shown for the problem of section 4.3 of three reservoirs in series, as formulated by Eq. 4.4, 4.5 and 4.6. The matrix represents Eq. 4.6 for a time horizon of 3 time steps, and a flow time between the nodes equal to 0.

| Var. | V1(t) | Q1(t) | O1(t) | V2(t) | Q2(t) | O2(t) | V3(t) | Q3(t) | O3(t) | | Inflow |
|---|---|---|---|---|---|---|---|---|---|---|---|
| time | 1 2 3 4 | 1 2 3 | 1 2 3 | 1 2 3 4 | 1 2 3 | 1 2 3 | 1 2 3 4 | 1 2 3 | 1 2 3 | | |
| Node 1 | -1 1 0 0 | 1 0 0 | 1 0 0 | 0 0 0 0 | 0 0 0 | 0 0 0 | 0 0 0 0 | 0 0 0 | 0 0 0 | | I1(1) |
| | 0 -1 1 0 | 0 1 0 | 0 1 0 | 0 0 0 0 | 0 0 0 | 0 0 0 | 0 0 0 0 | 0 0 0 | 0 0 0 | | I1(2) |
| | 0 0 -1 1 | 0 0 1 | 0 0 1 | 0 0 0 0 | 0 0 0 | 0 0 0 | 0 0 0 0 | 0 0 0 | 0 0 0 | | I1(3) |
| Node 2 | 0 0 0 0 | -1 0 0 | 0 0 0 | -1 1 0 0 | 1 0 0 | 1 0 0 | 0 0 0 0 | 0 0 0 | 0 0 0 | = | I2(1) |
| | 0 0 0 0 | 0 -1 0 | 0 0 0 | 0 -1 1 0 | 0 1 0 | 0 1 0 | 0 0 0 0 | 0 0 0 | 0 0 0 | | I2(2) |
| | 0 0 0 0 | 0 0 -1 | 0 0 0 | 0 0 -1 1 | 0 0 1 | 0 0 1 | 0 0 0 0 | 0 0 0 | 0 0 0 | | I2(3) |
| Node 3 | 0 0 0 0 | 0 0 0 | 0 0 0 | 0 0 0 0 | -1 0 0 | 0 0 0 | -1 1 0 0 | 1 0 0 | 1 0 0 | | I3(1) |
| | 0 0 0 0 | 0 0 0 | 0 0 0 | 0 0 0 0 | 0 -1 0 | 0 0 0 | 0 -1 1 0 | 0 1 0 | 0 1 0 | | I3(2) |
| | 0 0 0 0 | 0 0 0 | 0 0 0 | 0 0 0 0 | 0 0 -1 | 0 0 0 | 0 0 -1 1 | 0 0 1 | 0 0 1 | | I3(3) |

*Figure 5.1 Dynamic constraints of the LP problem of 3 reservoirs in series.*

A main drawback of a standard Simplex algorithm is that the matrix (and the computing time to solve the matrix) rapidly becomes very large, with increasing number of nodes and time horizon. To describe the dynamic constraints of a system consisting of $n$ nodes, applying a control horizon of $T$ time steps, requires a matrix of $[\ ((3 \cdot n \cdot T) + n) \cdot (n \cdot T)\ ]$ entries. Moreover, the matrix is very idle. Each row of the matrix contains only a few entries that are not equal to zero. Therefore, it is not difficult to imagine that a technique which makes use of the structure of the problem and only considers the non-zero values is more suited to solve the problem.

Note that the coefficients $a_{ij}$ of the linear constraints of the problem are either -1, 0 or 1. Whenever a LP problem has this form, then it has a network equivalent. A network (or linear graph) consists of a set of nodes (points, vertices) and a set of arcs (links) connecting various pairs of the nodes. In transforming the LP model into a network structure, each relation in Eq. 5.2 corresponds to a node in the network, each variable $x_{ij}$ to an amount of flow along an arc $(i,j)$ and each $c_{ij}$ to the cost of a unit flow through the arc. Each node might be loaded with an inflow. The problem as formulated by Eq. 5.1 can be translated so as to find the least-cost path to route the flow from source to sink. Since the flow may be shipped from the source through several intermediate (or transhipment) nodes to the sink, and the flow through an arc is restricted by capacity constraints, the problem is generally referred to as *a transhipment problem with arc capacities*. Operations Research provides several algorithms to solve such a problem.

A network flow model is in fact a special structured LP model. Like the standard Simplex method, the solution algorithm is an iterative process. The procedure, in bare outline, begins with a trial solution. By each step the current paths are evaluated and extended with at least one new node, so that the bound restrictions (capacity constraints) for the arcs are met and the value of the objective function is improved. Compared to the Simplex Method, a network algorithm could be considered a more effective way to find an improved value of the objective function, which is achieved by exploiting the network structure of the problem. Some basic principles of network models can be found in appendix A. For the mathematical backgrounds of network models, reference is made to the above mentioned handbooks on applications of Operations Research. Here we confine ourselves to illustrating the way to transform the Linear Programming model into a network flow model.

## 5.2 A network flow model

In solving the operational optimization problem the flow routing in the drainage system is approximated by a lumped storage approach. The dynamic constraints of the problem are described by the continuity equations (Eq. 4.19). The continuity equation of a node $i$ at time $t$ reads

$$[V_i(t+1) - V_i(t) + Q_i(t) + O_i(t)] - Q_{i,up}(t) - I_i(t) = 0 \tag{5.3}$$

This equation can be presented in a graph, by which each variable with a negative sign corresponds to an inward arc and each variable with a positive sign to an outward arc (Fig.5.2). Note that the node itself has no storage capacity. The stored volumes $V_i$ are also represented by an arc (or a flow in the network). The values of $c_{ij}$ in the objective function (Eq. 4.18) correspond to the cost of a unit flow through the arc.

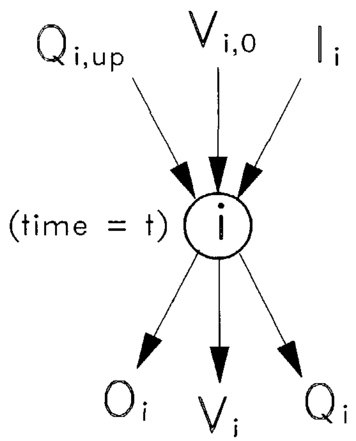


*Figure 5.2 Model of a node in a network*

The model as shown in Fig. 5.2 forms a basic element, by which any lumped storage model of a water resources system can be described. As explained in section 3.6.5, the arc capacities may be variable in time, depending on the current water levels. In (Neugebauer, 1989) the network structures of several elements of a drainage system are described, such as detention and retention basins, overflow structures, in-line storage etc.. All models of these elements, however, are basically similar to the one presented in Fig. 5.2 as they are all based on the continuity equation (Eq. 5.3). The only difference may be the number of inward and outward arcs (flows).

The dynamic constraints of the problem are formulated by a set of $T$ equations, where $T$ is the control horizon for which inflow is specified. The network model of a drainage system that has been schematized to $n$ 'storage elements' thus comprises $(n \cdot T)$ nodes. The concept is illustrated in Fig. 5.3, which shows how the problem of the three reservoirs in series is transformed into a network structure, applying a control horizon of three time steps and a flow time between the nodes equal to 1. It is noted that the method allows any flow time between two nodes that is expressed as an integer multiple of the time step $t$.

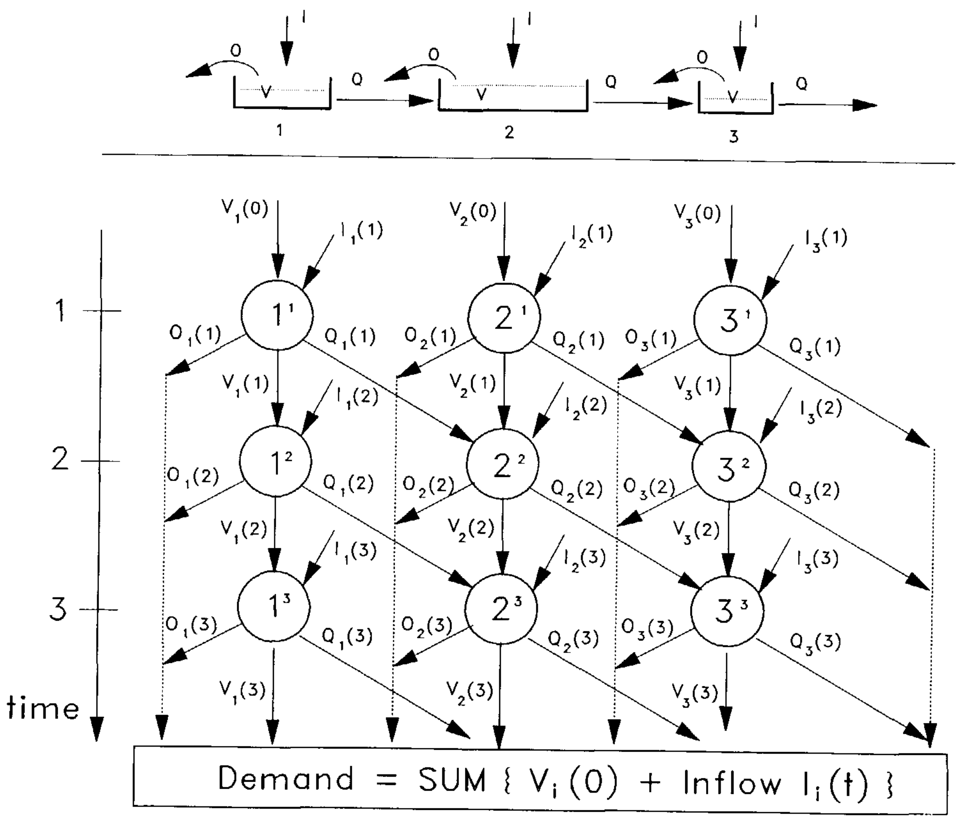


*Figure 5.3 Network structure of 3 reservoirs in series*

Each node $i$ receives an inflow $I_i(t)$. The total supply $\Sigma\ S_i$ is the amount of water that is in the system at $t = 0$ plus the sum of $I_i(t)$. The problem as described by Eq. 5.1 & 5.2 may be interpreted as to find the least cost path from source to sink (the Demand node, where all flows ultimately arrive), subject to the continuity constraint, $\Sigma\ S_i = \Sigma\ D_i$ (= total Demand)

## 5.3 LOCUS

The network flow model as described above has been incorporated in a newly developed modelling package, called LOCUS, which is an acronym of 'Local versus Optimal Control of Urban drainage Systems'. The name denotes that besides optimal controlled systems, local controlled systems can be simulated as well. The latter has been included in LOCUS to serve as a reference (Section 5.3.4).

### *5.3.1 Set up of the model*

Fig. 5.4 shows the main steps of the simulation. Each time step of the inflow hydrograph, a new problem is formulated. A so called 'supply file' is created, containing the initial systems state at the current time step $t$ and the inflow for the time horizon $T$ for which the problem is optimized. Besides the unit costs of the objective function are determined on the basis of the equations as derived in Chapter 3. If necessary, the arc bounds are modified. Next, the optimization problem is solved. Only the values of the systems state variables of the current time step ($X_i(t)$) are saved. The calculated values of the variables for $t+1$, $t+2$, ..., $T$, are used only in modifying the problem in the next time step. The use of a simple pollution transport model (which is described in Section 5.3.3) is optional.

The algorithm to solve the transhipment problem was supplied by the Department of Mathematics of the Delft University of Technology. As mentioned above, appendix A provides an introduction to network algorithms to make the basic idea of the solution technique transparent.

### *5.3.2 User considerations*

LOCUS incorporates several sub-programs to create the input-files required to run the models and to facilitate post processing of the model results. These sub-programs can be operated from a menu. Simulations may be performed for a single event or a series of events. The menu structure of the model is presented in Fig. 5.6, at the end of this chapter. This section provides a brief description of the various modules. It is noted that a model documentation and users guide is being made and will soon be available.

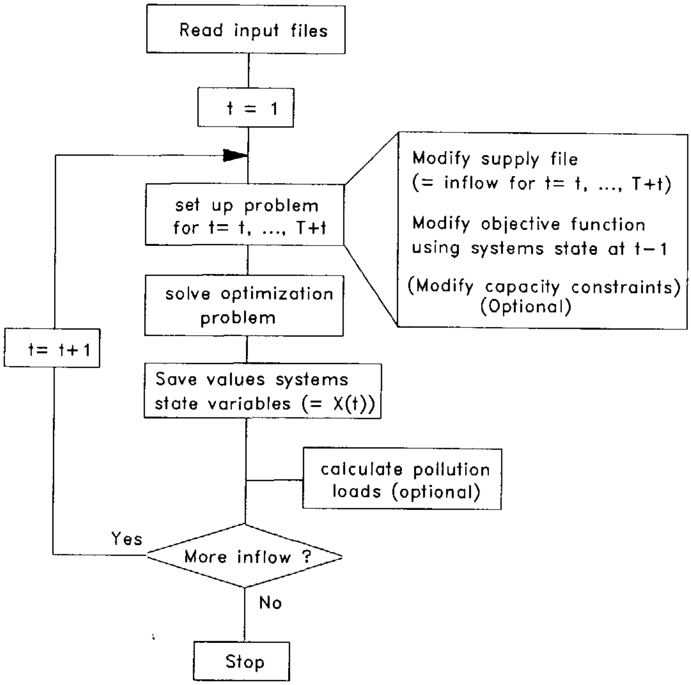


*Figure 5.4 Basic flow chart of LOCUS*

The input data are stored in four different files, which contain

1. inflow data, which have to be specified for each individual node;
2. parameters describing the arcs and their lower and upper bounds + basic unit costs corresponding to the arcs (the objective function);
3. water quality parameters: the dry weather flow to each node and pollution parameters;
4. model parameters to control the model output and parameters to simulate errors in the predicted inflow. The latter has been included to investigate the effects of an inaccurate inflow forecast on the operation strategy (Chapters 6 & 7).

*Ad. 1 Inflow data*

Two models have been developed to prepare the inflow data. One model is used to select rain events from a rain data base and another model is used to transform these data into inflow series.

Except for the case study of Damhus (Chapter 8), the rain input used in this study is abstracted from a rain series containing 15 years of rain recordings that have been measured via three ground rain gauges in Lelystad. The data used in the series are from the rain gauge which gives the greatest rain in a certain month (Van de Ven, 1989). The recordings are measured in 0.1 mm for 5 minutes intervals. For this study the Lelystad series have been transferred into a series of 10 minutes intervals. This has been done to reduce computing time, but also because a time step of 5 minutes may lead to an unrealistic operation strategy, as pumps are usually not allowed (or able) to switch to another operation every 5 minutes.

The model to select rain events from this data base has two parameters. One selection criterion concerns the minimum total rain depth. Besides the user has to define the maximum dry period after a new rain may begin, meaning that a rain event may include dry periods with a duration up to this maximum. This feature is included to take account of the effects of succeeding rain events. For example, by applying a maximum dry period equal to the emptying time of the system (in the order of 8-12 hours), it can be ensured that the system will be empty at the start of a new event.

Another model has been developed to transform the selected rain series into sewer inflow series. For this purpose, a (constant) initial loss model in combination with a linear reservoir is used (Section 2.2.2). This rather simple approach is sufficient as all studies conducted in this research are comparative studies in which the effects of different ways of operation are evaluated for the same inflow series. The use of a more sophisticated surface runoff model will therefore not affect the conclusions. The series are stored in separate files in a format that is used by LOCUS.

*Ad. 2a System configuration*

The system is schematized as a network consisting of arcs and nodes, which have to be specified by the user. No pre-defined system elements (such as reservoirs, pipes, etc.) have been included to keep the set up of the model as fundamental as possible (i.e. the structure of the problem remains visible). The model requires a specification of the upstream and downstream node of each arc, the upper and lower bound of the flow along this arc and the flow time between the respective nodes. This has to be done for only one time step. The model automatically generates the network structure for the other time steps of the control horizon that has been specified by the user. For example, the problem of 3 reservoirs in series, as illustrated by Fig. 5.3 requires the specification of 9 arcs, as each node has 3 'flows' (volume, discharge and overflow).

*Ad. 2b Objective function*

For each arc the basic unit cost have to be specified. Using the symbols of Chapter 4 (Fig. 4.5) this means that the user has to give a value to the parameters $\kappa_i$ (= the maximum of $cv_i^t$), $\beta_{i,1}$ and $\beta_{i,2}$ (= the basic cost + weighing factor of $co_i^t$) and $\gamma_i$ (or $cq_i^t$), for all $i$ nodes. Like the set up of the system configuration, this specification is required for one time step only. The model automatically generates the objective function according to the given control horizon.

It should be noted that the unit costs of a flow along an artificial arc 'outside' the physical system, e.g. an overflow which is transported to the 'Demand Node' (as indicated by the dotted line in Fig. 5.3), are automatically given a big value (e.g., 99999) to avoid multiple optimal solutions or to postpone overflows as much as possible. The arguments for this are in fact similar to the arguments that were mentioned in deriving Eq. 4.9.

According to Eq. 4.15, the value of $cv_i^t$ may become equal to zero, namely for $\alpha_{i,1} = \alpha_{i,2} = 0$. This may not be a problem when the use of storage of all system elements is treated equally. However, to express a desired systems performance where the use of a certain storage element $j$ should be kept at minimum, this unit cost function is not valid. Therefore a minimum of Eq. 4.15 > 0 is required to allow for unit costs of variables $V_j$ which are always greater than $cv_i^t$. In the model this minimum amounts to $(0.10 \cdot \kappa_i)$.

For example, in formulating the objective function of the problem of 3 reservoirs in series (Fig. 5.3) the following parameters may be applied. The simplest case is to define the desired systems behaviour so as not to allow overflows unless all available storage has been used. In this case a proper set of basic unit costs could be: $cq_i^t = 1$, $\kappa_i = 100$ (meaning that $10 < cv_i^t < 100$), and $co_i^t = 10000$, for all 3 nodes. This objective function will yield maximum transport out of the system, as the costs of discharge are low. Besides the use of storage at the 3 reservoirs will be kept as much as possible at a (relatively) equal level. Together with the high costs of overflows, this guarantees that, if physically possible, overflows do not occur unless all storage capacity is used.

Suppose an objective is added to the problem, namely to minimize the use of storage at Node 2. This can be achieved by applying $\kappa_2 = 1000$, meaning that the minimum of $cv_2^t$ is equal to the maximum of $cv_1^t$ and $cv_3^t$ (= 100). As a result, water will be stored only at Node 2 if necessary to prevent an overflow. Note that the decisions may be influenced by the rate of increase of the filling at the nodes (as expressed by the coefficient $\alpha_{i,2}$), meaning that the maximum of $cv_1^t$ and $cv_3^t$ may be reached before reservoirs 1 and 3 are actually filled to their maximum.

The possibilities to direct overflows to the less sensitive receiving water were explained in Section 4.5. The principle is that the desired difference in use of storage capacity of the various system elements is expressed by the values of $\kappa_i$. The place of a possible overflow is further determined by the value of $co_i^t$. Note that the latter will influence the decisions only if an overflow can be predicted within the time horizon of the optimization problem. Therefore this parameter will become more important with increasing control horizon.

It may be suggested to investigate first the behaviour of the model for a few events. Once a proper set of basic unit costs (and thus the desired systems performance) has been formulated, a series of rain events can be run during which this performance will be maintained in the best possible way.

*Ad. 3 Water quality parameters*

The water quality parameters of the model are a (constant) dry weather flow (DWF) for each node, and the pollution concentrations of the DWF and surface runoff. The pollution transport model is discussed below.

*Ad. 4 Results*

The main model results are the values of the systems state variables (stored volume, discharge, overflows, pollution loads, etc.) as a function of time for each specified node. These results can be presented both graphically and in tables. When simulating time series, the user can decide to store only the totals per event, instead of saving all values, to reduce the amount of output.

### 5.3.3 *Pollution transport*

LOCUS includes a simple pollution transport model which may be used to indicate the pollution loads. The main reason to incorporate this model is, however, to investigate the (theoretical) possibilities of deriving an operation strategy based on pollution loads.

The model consists of mixing of contributions of three sources: the inflow of upstream nodes with concentration $C_i 1$, domestic and industrial waste water with concentration $C_i 2$ and the sewer inflow (pollution from surfaces) with concentration $C_i 3$ (Fig. 5.5). For convenience, the most simple approach is being applied, namely to assume ideal mixing. (which is also used in, for example, the SAMBA model (Johansen, 1985)).

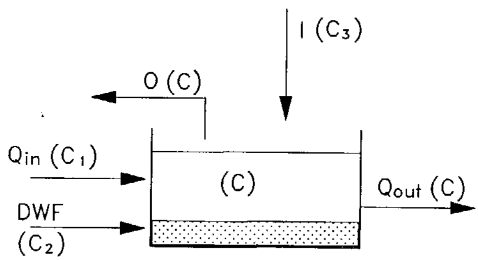


*Figure 5.5 The pollution transport model*

To investigate the possibilities of incorporating water quality parameters in the decision making it is assumed that pollution concentrations are known. Note that the pollution concentration is not a state variable in the optimization model (i.e. the variable is not part of the constraints of the problem that determine the region of feasible solutions). In determining the operation strategy, this variable is used only to calculate the unit cost of overflows in the objective function (Eq. 4.17).

For Dutch systems, it has been shown that deposits in sewers form an important source of pollution, due to the low gradient and large dimensions of the sewer systems (NWRW, 1989). Therefore, the process of sedimentation and re-suspension of these deposits are generally not to be neglected when developing a pollution emission model for design and operation purposes. However, because at present no model is available which is able to describe this process in a satisfactory way and an accurate model is not necessary for the problem under consideration, it has been decided to apply this simple pollution model

An investigation into the topic is discussed in Section 6.4. It should be noted that in the other case studies described below, the systems performance is evaluated on the basis of overflow frequencies and volumes. Partly because these can be predicted with a higher degree of accuracy than pollution loads, but mainly because these (comparative) studies are directed to problems that allow for a quantitative evaluation (e.g., impact of inflow predictions & control horizon, rainfall distribution, etc.). As the comparisons are made relatively (i.e. local versus optimal control), the variables involved are in these cases not significant for the conclusions.

### *5.3.4 Reference model*

A suitable approach to assess the effects of optimal control is to simulate exactly the same system without applying a dynamic operation, and to compare the results of both models. For this purpose, a so called reference model has been incorporated in the LOCUS package. The model uses the same input files as the optimization model. As the system is locally controlled it requires a (fixed) stage-discharge relation ($Q = f(H)$) for each node $i$, which describes the flow $Q_i$ as a function of the water level (or filling degree) at the particular node ($H_i$).

As the reference model and the optimal control model are based on an identical system description, the difference between the results of both models is due only to the way the system is operated and hence the effects of the improved operation can be quantified by comparing these results.

## 5.4 Concluding remarks

Referring to the objectives of the study, it can be concluded that a user-friendly model that can be used to assess the performance of an urban drainage system that is controlled in real time has been developed. Important features of the model are its flexibility, consistency in decision making and fast execution time, due to which it allows the simulation of time series of rain events. For example, to solve the optimization problem for a system of realistic size (10-15 elements), using a control horizon of 10 time steps, requires a computing time in the order of seconds per time interval on a 386 computer, meaning that the simulation of an inflow hydrograph of say 100 time steps (in the order of 10 minutes) takes about 1-2 minutes. A rational base to evaluate the effects of real time operation is provided by the reference model. To the author's knowledge, the model is unique of its kind.

Since the model is menu driven, it takes even an inexperienced user only a short time to learn how to handle the program. A possible negative side effect of menu supported input is that it might stimulate uncontrolled use of the model by users who are not familiar with the problem itself. Therefore, it has been decided to keep the set up of menus as fundamental as possible. The schematizations of the different elements of the system (e.g., pipes, reservoirs, etc.) have to be defined by the user itself, in terms of nodes and arcs to keep the network structure of the problem visible. Pre-defined elements are not included.

At present, the model's main limitation is the flow routing, which is based on a lumped storage approach. Simulation of uncontrolled gravity flow may be achieved by introducing state dependent capacity constraints (Section 4.5), however, at present, application of the model is restricted to systems which can be described by a lumped hydrologic model (like most Dutch urban drainage systems). Some model modifications are required to extend the flow model to simulate backwater and surcharge effects as well. As mentioned, plans to incorporate these features are being developed.

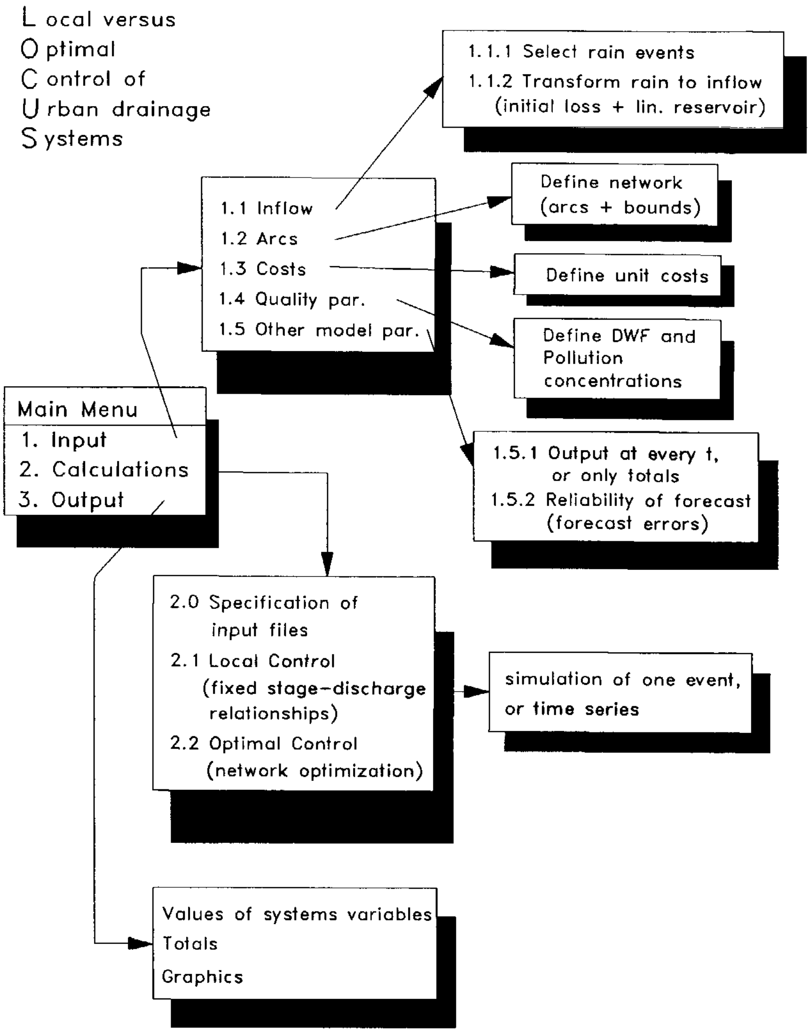


*Figure 5.6 Menu structure of LOCUS*

# 6 Analysis of a fictitious system

## 6.1 Introduction

Every urban drainage system has its own typical features and therefore no general rules can be formulated based on which the effects of real time control can be quantified. These can only be determined for a particular system by means of model calculations and on the basis of simulations of a series of (historical) rain events to obtain a statistical evaluation of an expected systems performance. For the problem under consideration an analysis of a single storm has no value, as not only the system is unique, but also the rain event that is simulated.

This chapter illustrates how to perform such an analysis for two fictitious systems, that are presented in Fig. 6.1. The system characteristics are listed in Table 6.1. All sub-catchments are identical and the rainfall is assumed to be homogeneous. Thus the effects of spatial distribution of the systems loading are excluded. A discussion on these aspects will follow in the next chapters.

The rainfall data used for the analyses in this chapter are historic events that have been recorded in Lelystad in the year 1981. To reduce computing time and because the comparisons are made concerning overflows, only major events are considered, which have a total rain depth greater than 5 mm. The rain events may include dry periods with a maximum duration of 10 hours. These criteria result in a series of 53 events. The rain data are transformed into inflow by applying an initial loss of 1 mm and a linear reservoir. The time step of the calculations amounts to 10 minutes.

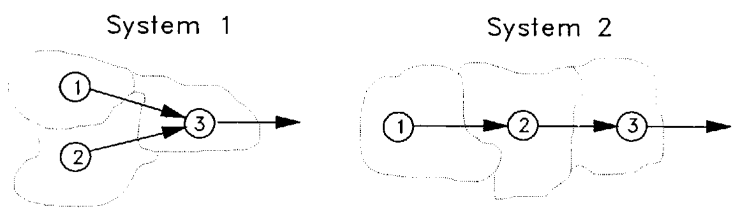


*Figure 6.1. Scheme of two fictitious systems.*

*Table 6.1. System characteristics*

| | System configuration 1 (parallel) | | | System configuration 2 (serial) | | |
|---|---|---|---|---|---|---|
| | Node 1 | Node 2 | Node 3 | Node 1 | Node 2 | Node 3 |
| Area [ha] | 100 | 100 | 100 | 100 | 100 | 100 |
| Storage [m3] | 7000 | 8000 | 5000 + ? * | 7000 | 8000 | 5000 + ? * |
| Storage [mm] | 7 | 8 | 5 + ? * | 7 | 8 | 5 + ? * |
| Dis.Cap. [m3/h] | 700 | 600 | 2100 + ? ** | 700 | 1300 | 2100 + ? ** |
| Overcap. [mm/h] | 0.7 | 0.6 | 0.8 + ? ** | 0.7 | 0.6 | 0.8 + ? ** |

* : the storage cap. of Node 3 is enlarged to 6000, 7000, 8000 and 9000 m3 (= 6, 7, 8, 9 mm)

** : the discharge capacity of Node 3 is enlarged to 2200, 2300, 2400, and 2500 m3/h (over-capacity = 0.9, 1.0, 1.1, 1.2, 1.3 mm/h)

Three aspects are investigated:

1. By implementing systems control, it may be possible to reduce the systems capacities required and hence to reduce construction cost. In section 6.2 the possible gains for the two systems are quantified in terms of the amount of storage and/or discharge capacity that would be required in a local controlled system to achieve the same performance as an optimal controlled system.
2. Section 6.3 deals with the influence of the control horizon and possible errors in predicted inflow on the operation strategy.
3. Finally, in section 6.4, it is investigated to what extent pollution loads can be minimized by incorporating the current pollution concentrations in the decision making (using Eq. 4.23), assuming these concentrations are known.

(It is stressed that all cases presented in this chapter are pure theoretical.)

## 6.2 Required system capacity

We consider a (fictitious) system with a capacity problem. The upstream sections of the system (Node 1 & 2) are sized 'properly', according to the Dutch standards. The main problems are to be expected at the downstream section, at Node 3, where the storage capacity (related to the connected area) is relatively small. In practice, a situation like this could for example occur when an existing system has been expanded in time due to urban developments. As discussed in Chapter 2, the conventional solution to these kinds of problems is to increase the storage and possibly the discharge capacity of the system. According to the present standards, a plausible solution to this problem would be to implement a storage settlement tank at Node 3.

The aim of this analysis is to quantify the benefits that may be expected from applying real time control, compared to the conventional solutions. For this purpose, the effects of increasing the storage and discharge capacity of Node 3 are investigated. The systems performance is evaluated on the basis of the calculated total overflow volumes of the 53 events (= 1 year).

As indicated in Table 6.1, both systems are simulated for 10 different cases. In 5 cases, the storage capacity of Node 3 is increased by 1000 $m^3$ (= 1 mm) for each successive case, using the initial maximum discharge capacity (= 2100 $m^3/h$). In the other 5 cases, the storage capacity is kept at its initial level (= 5000 $m^3$) and the discharge capacity is increased by 100 $m^3/h$ (= 0.1 mm/hr) for each successive case. All cases are simulated for three different operation strategies:

1. local control, using a fixed stage-discharge relationship, which reads
   if filling degree at Node $i$ > 10% then $Q_i = 1/3\ Q_{max}$;
   if filling degree at Node $i$ > 30% then $Q_i = 2/3\ Q_{max}$;
   if filling degree at Node $i$ > 50% then $Q_i = Q_{max}$.
2. local control, using a fixed stage-discharge relationship, which reads
   if filling degree at Node $i$ > 5% then $Q_i = 1/3\ Q_{max}$;
   if filling degree at Node $i$ > 10% then $Q_i = 2/3\ Q_{max}$;
   if filling degree at Node $i$ > 15% then $Q_i = Q_{max}$.
3. optimal control, using a control horizon of 1 hour (= 6 time steps). The basic unit costs are chosen so that the use of storage is maximized at all 3 sub-systems, meaning that overflows are not allowed to occur unless all storage has been used (see Sections 4.5 & 5.3.2).

Assuming the system is controlled by pumps, the first strategy may be regarded as more realistic than the second one, as the maximum pump capacity is generally not yet activated at a filling degree of 15%. Strategy 2 is, however, a better solution to discharge as much water as possible (with local control). For a fair comparison of the results of local and optimal control, the results obtained by the second strategy should be used because in applying optimal control it is (also) assumed that the full discharge capacity is available from the beginning of the rain event.

The calculated total (yearly) overflow volumes for each case are presented in Fig. 6.2, 6.3, 6.4 and 6.5.

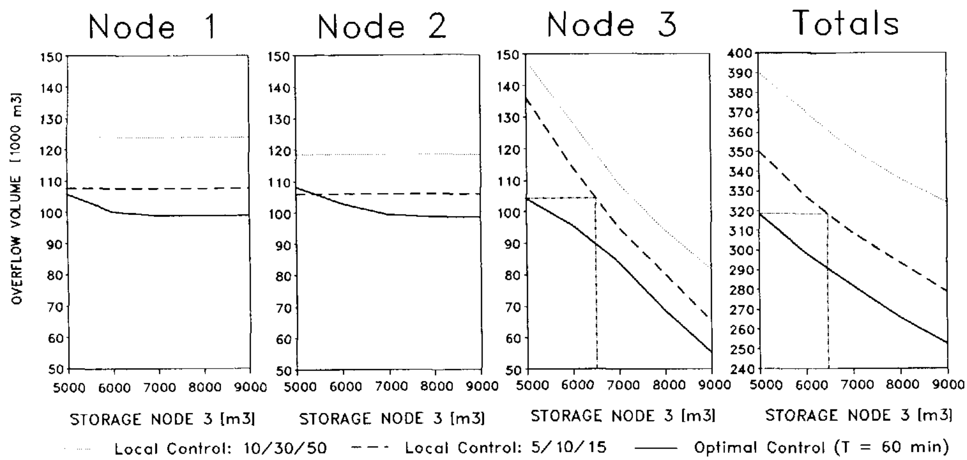


*Figure 6.2 Overflow System 1: effects of increasing the storage capacity at Node 3*

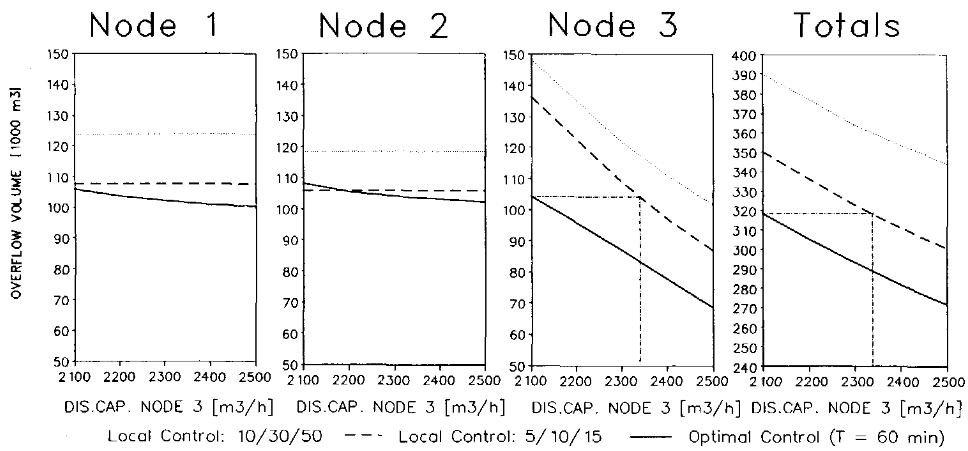


*Figure 6.3 Overflow System 1: effects of increasing the discharge capacity at Node 3*

At the starting point of the calculations (i.e. Sto.3 = 5000 $m^3$ & Dis.cap.3 = 2100 $m^3/h$), the overflow volumes at Node 1 and 2 with local control are (almost) equal to the volumes that are determined for the optimal controlled system. Obviously strategy 2 (= discharge as much as possible) is a suitable strategy for Nodes 1 and 2, to minimize overflows. This holds good for both systems.

Using local control the overflow volume of Node 3 is much greater than those of Nodes 1 & 2, whereas for the optimal controlled system the overflow volumes of all

3 nodes are at an equal level. Regarding the integral system it is apparently not a good solution to keep the discharges of Nodes 1 and 2 at their maximum. Applying a dynamic operation, in combination with the temporal variability of the inflow, results in a significant reduction of the overflow of Node 3 without increasing the overflow volumes at Nodes 1 and 2.

Increasing the storage or discharge capacity of Node 3 has obviously no effect on the upstream sections of the system when local control is applied. For the optimization model, however, it means that some extra capacity is added to the system that can be used to upgrade its performance. As a result, the overflow volumes of Node 1 and 2 can be reduced, although capacity is added only at Node 3.

As can be seen from Fig. 6.2, the storage capacity of Node 3 has to be augmented by (at least) 1500 $m^3$ (= 1.5 mm) to reduce the overflow volume of Node 3 to the level that is achieved by applying optimal control, without increasing the storage capacity. To reach this performance by augmenting the discharge capacity of Node 3, requires an increase of the over-capacity of (at least) 240 $m^3/h$ (= 0.24 mm/h).

It should be noted that increasing the discharge capacity is usually not considered to be a good solution in case Node 3 is discharging to a treatment plant as this would require an expansion of the treatment plant. To avoid such a problem in this theoretical example it may simply be assumed that Node 3 is connected to another node with sufficient capacity.

However, as explained in Section 2.5, increasing the capacity of the sewerage system may influence the performance of the treatment plant, meaning that the total effect of this 'solution', concerning pollution loads to the environment, could be nil or even become worse compared to the old situation. Therefore it may be suggested that whenever we change the sewerage system it should be investigated what effect may be expected at the treatment plant. It needs no argument that a dynamic operation is required to maintain optimum flow rates to the plant.

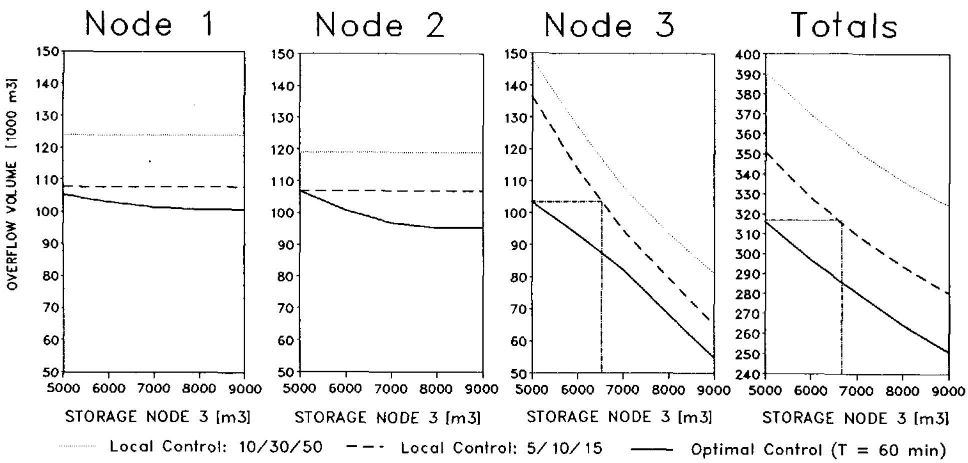


*Figure 6.4 Overflow System 2: effects of increasing the storage capacity at Node 3*

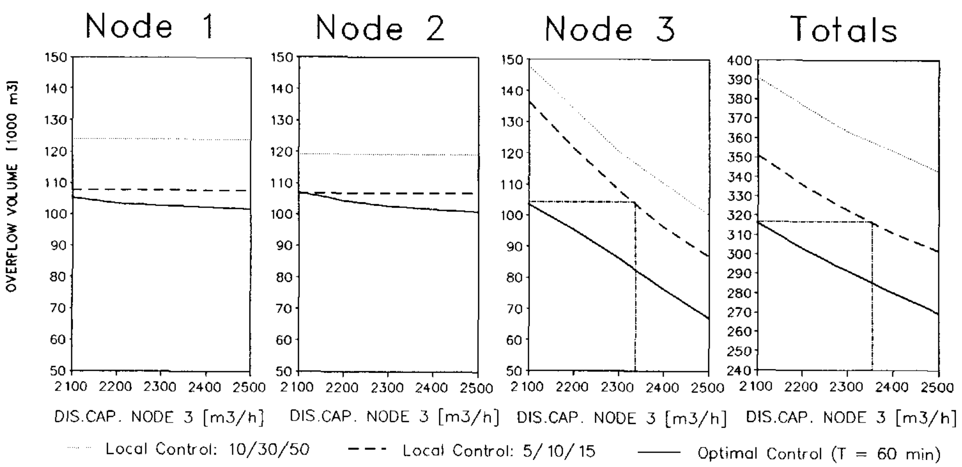


*Figure 6.5 Overflow System 2: effects of increasing the discharge capacity at Node 3*

Fig. 6.4 and 6.5 present the results of System 2. The difference with the results of System 1 is not significant. A small positive bias may be found for System 2, indicating that a systems configuration with nodes in series may offer more 'elements to play with' than a 'parallel' configuration. (Compared to System 1 the controllability of the flow in System 2 is theoretically greater due to the bigger discharge capacity of Node 2). The difference is however very small and therefore no general conclusions on this aspect can be drawn on the basis of these results.

## 6.3 Control horizon and forecast errors

In the next calculations we use the systems characteristics as indicated in Table 6.1, i.e. the storage capacity of Node 3 amounts to 5000 $m^3$ and the discharge capacity to 2100 $m^3/h$. The aim is to investigate the extent to which control decisions are affected by the control horizons and by errors in the predicted inflow.

Firstly, four cases with increasing control horizon are simulated ($T$= 30, 60, 120, 240 min). The inflow $I_i(t)$ for $t=1$ to $T$ is assumed to be known. A perfect forecast of the rainfall that is entering the system for a time horizon of 240 min is not a realistic option with the present state of technology, but for the problem under consideration it is interesting to see what could be gained if this information would be available. The results are presented in Fig. 6.6.

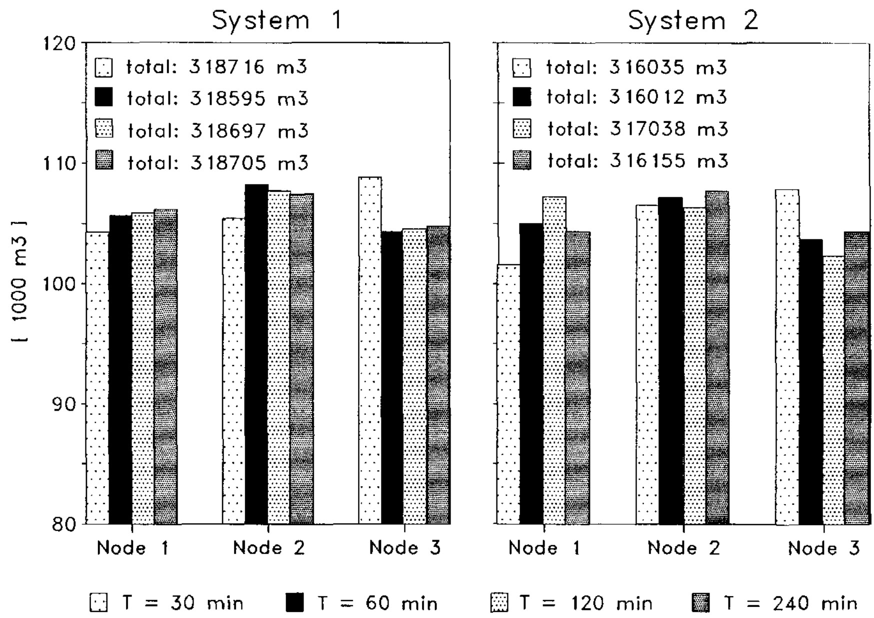


*Figure 6.6 Effects of different control horizons*

From this figure it can be seen that the difference between the total overflow volumes is negligible. Apparently, increasing the control horizon has in this case no significant influence on the totals, but only on the distribution of the overflow volumes over the 3 nodes.

In formulating the objective function, no distinction has been made about the place of overflow. As a consequence, the optimization model will keep the filling degree of the different nodes during the storm event, if possible, at an equal level. By increasing the control horizon, the model will succeed better in achieving this, as temporal variations of the inflow are foreseen at an earlier stage. Moreover it supplies a better estimate of the coefficient $\alpha_{i,2}$ (which denotes the rate of increase of the stored volume $V_i$ ($= \partial V_i/\partial t$), see Eq. 4.15). As a consequence, the difference in overflow volumes of the various nodes will be smaller with increasing control horizon.

As discussed in Chapter 3, the ability of the system to correct a perturbation of the system determines the extent to which one may benefit from predicting disturbances. In the case where the system is able to return to its desired state within acceptable time limits, then information on the current systems state is in principle sufficient to derive a suitable operation strategy. A perturbation is in this case defined as a difference in (relative) use of storage at the 3 nodes. As the discharge capacities of the 3 nodes are of the same magnitude as the variations of the system loading, it means that the ability of the system to correct a perturbation is rather big (and hence the applied control horizon has no significant influence).

The impact of an inaccurate inflow forecast is investigated for two cases, i.e. using a control horizon of 30 min ($T = 3$) and 240 min ($T = 24$). The errors that are introduced in the first case are listed in Table 6.2, denoting the predicted inflow as a percentage of the actual inflow.

*Table 6.2. Forecast errors*

| T= 3 | t= 1 | t= 2 | t= 3 |
|---|---|---|---|
| error(t) | - 20% | - 50% | - 100% |
| error(t) | - 10% | - 20% | - 30% |
| error(t) | + 10% | + 20% | + 30% |
| error(t) | + 20% | + 50% | + 100% |

In the second case the control horizon amounts to 240 min ($T = 24$). The effects of both an underestimate of 100% and an overestimate of 100% of the inflow are presented in Fig. 6.8.

Like the control horizon, an inaccurate inflow forecast hardly affects the total overflow volumes. The difference in systems performance in the various cases is to be found in the distribution of this volume over the nodes. As shown in Fig. 6.8, even if the predicted inflow during the control horizon amounts to zero or the inflow is assumed to be twice the actual inflow, the optimization model will produce an acceptable strategy.

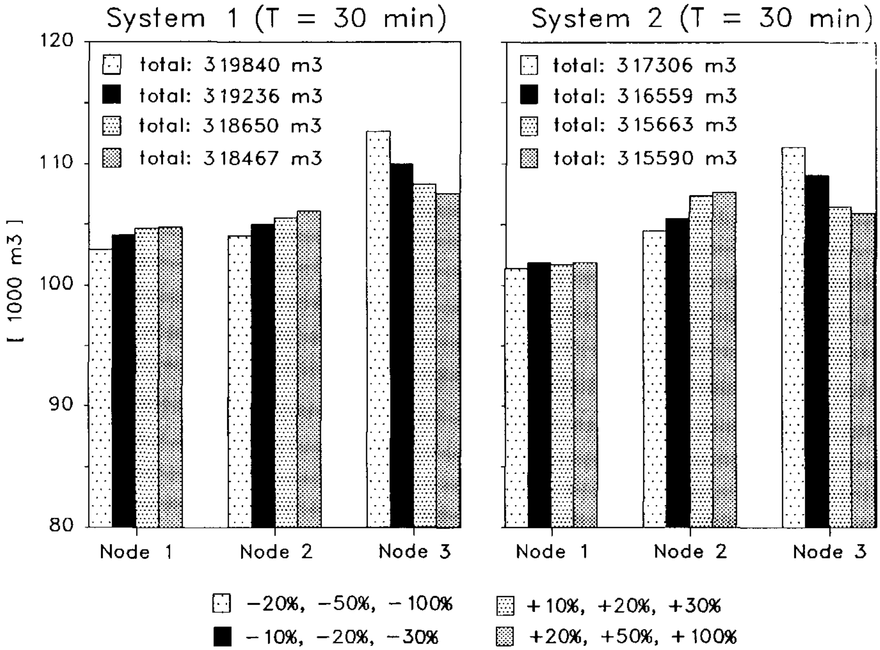


*Figure 6.7 Effects of prediction errors, using a control horizon of 30 min*

The tendency is the same in both cases. An underestimate of the predicted inflow has a positive effect on those parts of the system having sufficient capacity available (Node 1 & 2), and a negative effect on the overflow volume of Node 3 where the capacity is relatively limited. When the inflow is overestimated, it is just the other way round.

This behaviour of the decision model can be explained as follows. Using an overestimated inflow means that the value of the coefficient $\alpha_{i,2}$ (approximation of $\partial V_i/\partial t$, see Eq. 4.15), will be overestimated more for nodes having less storage capacity. As a consequence, the unit costs of storage of these nodes will be greater, compared to the unit costs of storage of other nodes having more storage capacity. Simply stated, when the inflow is overestimated the model becomes (automatically) more 'cautious' in use of storage at nodes that have relatively less capacity. (Note that with this in view the control horizon is also an important factor in the decision making). Conversely this implies that an underestimated inflow will augment the use of storage at these particular nodes.

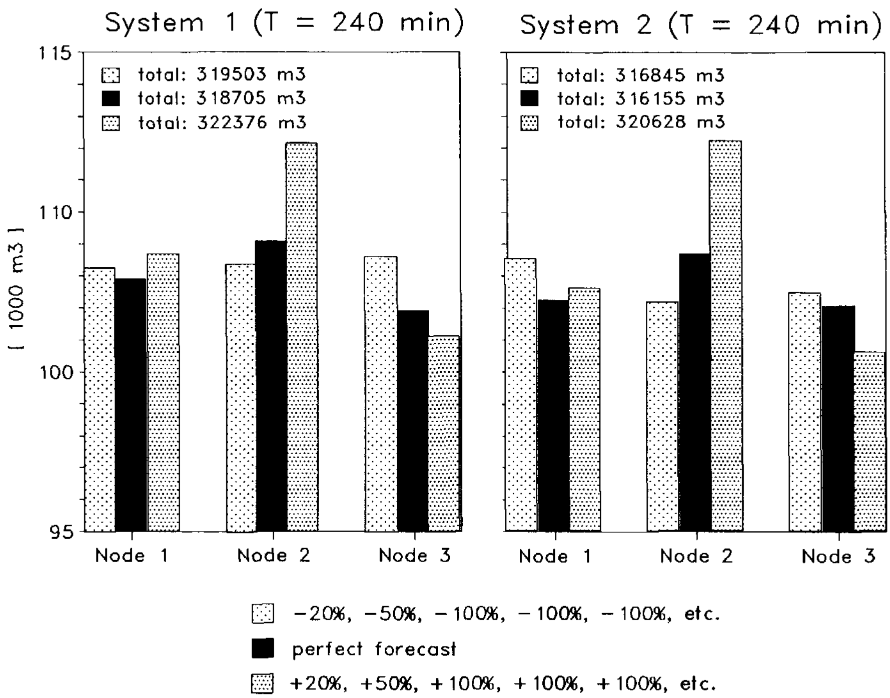


*Figure 6.8 Effects of prediction errors (control horizon = 240 min)*

In Chapter 7, an investigation into the topic will be discussed for a system of a realistic size, i.e. the urban drainage system of Rotterdam. In this system a distinction is made in the location of overflows, depending on the sensitivity of the receiving water.

## 6.4 Water pollution control

Besides flood protection, the general purpose of operating an urban drainage system is to minimize the impact of pollution loads on receiving waters. As explained in Section 5.3.3, the case studies presented in this report are confined to a quantitative analysis. However, since much effort is being put into the development of pollution transport models, it may be expected that, sooner or later, a model will become available that can be used to predict pollution loads with an acceptable degree of accuracy. The question is then, how to use this information for operational purposes?

Béron et al. (1988) state that *'it would be interesting to study ways to improve the control algorithm in order to investigate further whether local automatic control (the present way of operation) is in fact as effective as control based on pollution loads. Eventually, it might be demonstrated that an improved algorithm would significantly reduce the overall impact, and thus increase protection of the receiving stream.'* (The first step towards such an improved algorithm has in fact been discussed above).

The LOCUS model incorporates a simple pollution transport model, which is described in Section 5.3.3. Based on this model, it is investigated to what extent a strategy, which aims at minimization of pollution loads, differs from a 'quantitative' optimization aiming at optimal use of available storage. For the problem under consideration, it is thus assumed that the pollution model is correct. It is not said that the model predictions are accurate.

In Section 4.5.3 it was discussed how the unit cost of overflows may be modified to minimize the pollution loads on the environment. The proposed unit cost function reads

$$co_i^t = \beta_{i,1} + \beta_{i,2} C_i(t) \tag{5.1}$$

where $co_i^t$ is the unit cost of $O_i$; $C_i(t)$ is the pollution Concentration at Node $i$ at time $t$ [$m^3/\Delta t$]; $\beta_{i,1}$ is the basic cost (depending on the type and sensitivity of the receiving water body); and $\beta_{i,2}$ is a factor, which determines the 'weight' of $C_i(t)$ in determining the unit cost of overflows. If $\beta_{i,2}$ is equal to zero, the value of $C_i(t)$ has no effect on the operation strategy, meaning that the model minimize overflow volumes only.

In this analysis, only System 2 is used. The model parameters as used in the four (2x2) cases are given in Table 6.3. The pollution concentrations of the inflow are assumed to be constant. The inflow series (53 events) are the same as used above. The control horizon amounts to 60 min.

*Table 6.3 Four investigated cases*

| | | Node 1 | Node 2 | Node 3 |
|---|---|---|---|---|
| Case 1a | Pollution concentration (g/m3) | 0 | 1 | 0 |
| | Coefficient ß1 | 10000 | 10000 | 10000 |
| | Coefficient ß2 | 0 | 0 | 0 |
| Case 1b | Pollution concentration (g/m3) | 0 | 1 | 0 |
| | Coefficient ß1 | 10000 | 10000 | 10000 |
| | Coefficient ß2 | 10000 | 10000 | 10000 |
| Case 2a | Pollution concentration (g/m3) | 0.5 | 1 | 1.5 |
| | Coefficient ß1 | 10000 | 10000 | 10000 |
| | Coefficient ß2 | 0 | 0 | 0 |
| Case 2b | Pollution concentration (g/m3) | 0.5 | 1 | 1.5 |
| | Coefficient ß1 | 10000 | 10000 | 10000 |
| | Coefficient ß2 | 10000 | 10000 | 10000 |

In Cases 1a & 1b, the pollution concentrations of the inflow to Node 1 and 3 are equal to zero to facilitate an interpretation of the results (and to check whether the model behaviour is as expected). The results of the simulations are presented in Fig. 6.9

In Case 1a, the value of $\beta_2$ is set equal to zero, meaning that the model minimizes overflows only. Since all nodes have the same basic cost of storage ($\beta_1$) and the same inflow, the overflow volumes of the three nodes are of the same order.

In Case 1b, $\beta_2$ amounts to 10000, meaning that the unit costs of storage $co_i^t$ have a linear increase depending on the value of the current pollution concentration, as indicated by Eq. 5.1. The simulation results are as expected. The overflow volumes of Node 2 decrease, at the expense of an increase of the overflow volume of Node 1, as this node contains no pollution. As soon as a possible overflow of Node 2 can be foreseen (which means that the control horizon is an important parameter), the discharge of Node 1 to Node 2 is stopped. Since the polluted water of Node 2 is no longer diluted by the flow of Node 1, the concentration will augment. Although in Case 1b the overflow volume of Node 2 is reduced to 60% compared to the volume of Case 1b, the pollution loads are reduced to 'only' 70%. For the same reason a small increase in the pollution loads of Node 3 is found. In total the pollution load is reduced by 24%.

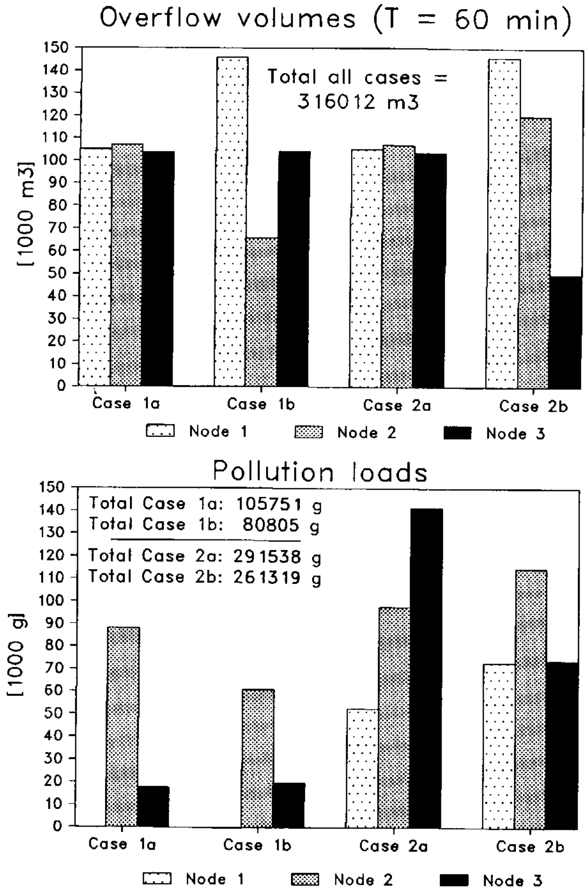


*Figure 6.9 Minimizing pollution loads*

In Cases 2a & 2b, the pollution concentrations are increased in downstream direction. In Case 2a $\beta_2$ is equal to zero, meaning that the optimization problem is identical to the one of Case 1. This is shown by the calculated overflow volumes. The related pollution loads exhibit the difference in pollution input of the three Nodes.

In Case 2b, the aim is to minimize the pollution loads. The coefficient $\beta_2$ has the same value for all Nodes, meaning that the pollution loads of the three Nodes are treated equally. As a result, the overflow volume is decreasing with increasing pollution concentration. This is clearly demonstrated by comparing the overflow volumes of Case 2b with the pollution loads of Case 2a.

Note that the total overflow volumes of all four cases are exactly the same. This is due to the fact that the objective function is formulated in a way that the model optimizes the use of storage under all conditions. As mentioned in Section 4.5.2, this can be ensured by applying basic unit costs of overflow ($= \beta_{i,1}$) which are greater than the maximum unit cost of storage ($=$ max $\kappa_i$).

The results may look promising, but they should be interpreted with care. It can be concluded that regarding water pollution control the operation strategy can further be refined, if a reliable pollution transport model were available. The pollution model used is certainly not accurate enough to be of practical value.

Furthermore in operating an urban drainage system it is often more important to direct overflows to the less sensitive receiving water than to minimize the total pollution output of the system. LOCUS includes several parameters to achieve this, without the use of a pollution transport model.

## 6.5 Concluding remarks

A simple example illustrates the use of LOCUS in analyzing the performance of an urban drainage system that is operated in real time. The potential of this improved systems control can be quantified by comparing the results of the same system that is locally controlled, which is at present the normal way of operation of most urban drainage systems. Obviously not all aspects that may be important have been considered. Some of them are dealt with in the next chapters.

In Chapter 1, it was mentioned that the performance of most urban drainage systems needs some upgrading. Based on the results of this case study it can be concluded that it is worth investigating the potential of real time control before constructing extra storage in the system. Moreover, when it has been decided to add some extra capacity to the system, a dynamic operation is still required to improve the performance of the entire system. Applying local control means that the systems performance is improved only locally, i.e. at the site where the capacity has been increased.

In general, increasing the control horizon may improve the decisions on the operation strategy as it enables the model to anticipate on spatial and temporal variations of the inflow. Besides, the estimates of the rates of increase of the stored volume (Eq. 4.15) at the various sub-systems are improved, and hence the decisions on the use of storage. The extent to which one may benefit from predicting disturbances depends the response time of the system and its ability to correct a perturbation (or the time required to return to its desired state).

The length of the control horizon has in this example no significant effect on the total overflow volume. The ability of the system to correct a perturbation (i.e. a difference in relative use of storage) is of the same magnitude as the variation of the system loading, and hence a proper decisions on how to operate the system may be based on the current systems state.

However, if the objective is not only to minimize the total overflow volume but also to direct overflows to the less sensitive receiving water then the control horizon may become an important factor. In this case the desired systems behaviour is obviously not to keep the use of storage as much as possible at a relatively equal level. To make a proper decision on the moment where to leave this concept we need information on future disturbances. However, since the time horizon for which inflow can be specified is usually limited the reaction time might be too short. Therefore to reduce the probability of an overflow at a sensitive place we should also restrict the use of storage at these places, even if no overflows can be foreseen within the control horizon. As discussed in Section 4.5.1, this can be expressed in the objective function by the ratio between the coefficients $\kappa_i$ which define the allowable differences in filling degrees of the different nodes.

Similar conclusions can be drawn concerning the required reliability of the predicted inflow. The effects of forecast errors on the total overflow volume are negligible. Inaccurate inflow predictions will however affect the decisions on the use of storage and hence the location of possible overflows. If the inflow is overestimated, then a positive effect may be expected at those places (nodes) in the system that have less capacity as compared to other places. If the actual inflow is underestimated then it is the other way round.

Regarding the sewerage system, water pollution control is generally not performed by directly controlling the water quality processes, but by means of water quantity control. The main objective is to minimize combined sewer overflows (differentiated in time and place, depending on the function and sensitivity of the receiving water body), while maintaining optimum flow rates to the treatment plant (depending on the operational conditions at the plant). This objectives can in principle be expressed by an objective function comprising quantity variables only (Section 4.5).

Deriving an operation strategy based on pollution loads requires obviously a model which can predict these loads. Suppose such a model were available then a possibility to minimize pollution loads of overflows is to increase the unit cost of overflows with increasing pollution concentrations. As a consequence pollution loads are reduced by decreasing the overflow volumes with increasing pollution concentrations.

The question is whether minimizing (total) pollution loads is in fact the main issue. In operating the system it is usually considered more important to prevent overflows and, if necessary, to direct overflows to the less sensitive streams, rather than minimizing the (total) pollution emissions by overflows. This is due to the fact that besides reliable pollution transport models, comprehensive ecological models to predict (or quantify) the impact of overflows on the receiving water quality are still lacking. As a result, a desired systems performance (and hence the objective function of the operational optimization problem) can be expressed in terms of water quantity variables only (flows, water levels).

# 7 Case study of Rotterdam: Control of overflow location and importance of inflow forecasting

## 7.1 Introduction

Last decade, the municipality of Rotterdam carried out a big project to rehabilitate and modernize its sewerage system. The project was induced by the need to reduce discharges of untreated waste water to the river Nieuwe Maas, by leading it to a treatment plant. The starting point in rehabilitating the system was that the overflows to the small receiving waters within the city (e.g., ponds, city canals) should be kept to a minimum. The maximum theoretical overflow frequency (i.e. according to the 'Kuipers method', Chapter 2), is set to 3-4 times per year.

This has partly been realized by constructing some extra storage in the system. To keep the amount of storage within acceptable limits, it has been decided to introduce controlled overflow structures (pumps and valves), which during heavy storms can be used to discharge a part of the sewage to the river Nieuwe Maas. The allowable frequency of controlled overflows is restricted by the national water authorities (being the water quality manager of the Nieuwe Maas) to about 15 times per year. Especially to operate these overflow structures the municipality has installed an advanced monitoring and remote control system.

The part of the Rotterdam system that has been modernized is divided into three major districts. The Eastern and Western District on the right bank of the river Nieuwe Maas and the Southern District on the left bank of the river. The Eastern and Southern Districts discharge to the treatment plant Dokhaven, and the Western District is connected to the plant Kralingseveer. Each district consists of several sub-catchments, where waste water and surplus storm water is collected and discharged to a main pumping station by means of gravity flow and a number of small (locally controlled) pumping stations. The available storage capacity at these sub-catchment varies between 5-11 mm. The main pumping stations are equipped with water level and flow meters and a Programmable Logic Controller (PLC) to maintain a pre-set flow, depending on the water level in the pump well. As the pumps are speed regulated, the discharges are in principle continuously controllable over their full range, i.e. 0-100% of their respective capacity.

For this case study, simulations have been performed of the Eastern and Southern Districts. As the results of the calculations of both districts lead to similar conclusions, it has been decided to include the simulation results of the Southern district only. A more extensive description of the case study is given by (Breur, 1992).

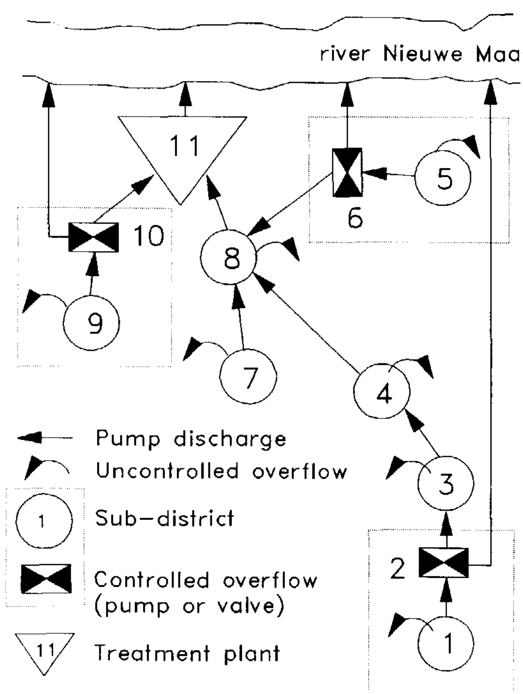


*Figure 7.1 Schematization of the Southern District*

### 7.2 Operational problem

The pumps are controlled by the local PLC. This process computer decides on the basis of pre-defined set points and the local water level the desired pump discharge. Process data (like water levels, flows, pump data) are send to the central control room at intervals of 20 seconds. The control room is manned 24 hours a day. Besides the actual process data, the operator has weather forecasts at his disposal, which are provided by the national meteorological institute KNMI. These forecasts comprise both radar images, that are send by modem, and facsimile messages giving the expected rainfall depth for the coming hours (including its probability).

*Table 7.1 System characteristics*

| Node | Area | Storage | Storage | DWF | Discharge cap. [m3/h] | |
|---|---|---|---|---|---|---|
| | [ha] | [m3] | [mm] | [m3/h] | To downstream node | Controlled Overflow |
| 1 | 166 | 15692 | 9.4 | 612 | 2760 | |
| 2 | - | - | - | - | 1260 | 1500 |
| 3 | 108 | 9931 | 9.1 | 900 | 2316 | |
| 4 | 46 | 4246 | 9.2 | 1110 | 4800 | |
| 5 | 232 | 15311 | 6.6 | 1272 | 3300 / 6000 | |
| 6 | - | - | - | - | 0 or 3300 | 6000 or 0 |
| 7 | 75 | 5198 | 7.0 | 300 | 2100 | |
| 8 | 76 | 3464 | 4.6 | 3000 | 9000 | |
| 9 | 178 | 20010 | 11.2 | 762 | 2190 | |
| 10 | - | - | - | - | 0 or 2190 | 2190 or 0 |
| 11 | - | - | - | - | 9540 | |

Note: The pumped overflow of Node 2 and the discharge of 2 to Node 3 may be operated at the same time, the valves at 6 and 10 direct the water either to the downstream node or to the river.

The pumping stations are locally controlled, but the set points of the pumps can be adjusted from the central control room. If necessary, the operator may overrule the local PLC and completely take over the control of pumps and valves (supervisory control). Until recently, interference of operators was restricted to extraordinary operational conditions, like extreme rainfall, technical trouble, disruptions due to maintenance work etc. However, since the operator has rainfall forecasts at his disposal, the possibilities of remote control of the flow regulators are used more often (Geerse, 1990), especially to reduce pumping rates during dry weather periods in which energy is more expensive (the electricity tariffs are not constant during the day). As waste water has to be temporarily stored this might increase the risk of overflows. To minimize this risk long term rainfall forecasts (in the order of several hours) are indispensable. Furthermore, if the operator knows that the rain has stopped, and no rain is to be expected in the coming hours, he may decide to stop the controlled overflows to the river although the (sub)system is still filled.

Summarized, it can be stated that the controllability of the urban drainage system of Rotterdam is extensive; the system is equipped with advanced measuring and control devices, the discharge rates of the pumping stations are continuously variable, flows may be directed to other districts by opening and closing of valves, and last but not least, the overflows are controlled by pumps, meaning that at any time the overflow may be operated to discharge water out of the system.

The key problem is the formulation of the operation strategy, by which the operational objectives are met in the best way. These objectives are (in order of priority of the municipality):

- to minimize urban surface flooding;
- to minimize overflows to receiving waters in the city (Nodes 1, 3, 4, 5, 7, 8, 9);
- to minimize overflows to the river Nieuwe Maas (Nodes 2, 6, 10);
- to minimize energy cost;
- to optimize the discharge rates to the treatment plant.

At present, the operator decides on the operation strategy on the basis of experience (heuristic control). As was mentioned in Section 4.1.1, the main shortcomings of this approach are that the decisions may be inconsistent and difficult to evaluate. Furthermore, the experience gained by trial-and-error will be lost when the operator leaves his job. To reduce these problems and to improve the systems performance, the municipality of Rotterdam has become interested in the development of a decision support model to assist the operator. If the model appears to function satisfactorily, it might ultimately also be used to execute the strategy, meaning that in the future the supervisory control system may eventually be replaced by a central automatic control system.

As a first step, a model study has been conducted with LOCUS (which will serve as a basis of the decision support model to be developed). The main aim of the case study is to quantify the potential of real time control, thereby making a distinction in the possible place of overflows. Furthermore, the accuracy and time horizon of inflow forecasts that are required to obtain an acceptable operation strategy are investigated.

## 7.3 Set up of the study

In an early stage of the development of LOCUS, a case study was carried out for the Western District of Rotterdam, (Beenen, 1991), (Nelen et al, 1991a). For this district, the overflow volumes of the locally controlled and optimally controlled systems have been compared for 15 storms. These storms are the 15 events with the greatest rain depth that were recorded in a period of 2.5 years. It was found that the overflow volume on city canals could be reduced by 14%, and the pumped overflows to the river Nieuwe Maas by 51%. The total reduction of overflow volume amounts to 36%. Since the solution algorithm of LOCUS (at that time) was based on the revised Simplex method, the number of events and the size of the system that could be handled by the model were limited. This problem has been tackled by replacing the revised Simplex Method by a much faster network flow algorithm (as was discussed in Section 4.1).

The new version of LOCUS can handle bigger systems and allows for the simulation of longer time series of rain events. The rain input for this case study was abstracted from the Lelystad records of the period 1970-1985 (Section 5.3.2). As selection criteria a minimum rain depth of 4 mm, and a maximum dry weather period of 12 hours were applied. This resulted in a series of 333 rain events, which were transformed into an inflow series, using an initial loss of 1 mm and a linear reservoir with a reservoir (time) constant of 15 minutes. The applied time step in the simulations amounts to 10 minutes.

The inflow series was simulated for a number of cases. First, the locally controlled system was simulated with the 'reference' model of LOCUS. The stage-discharge relationships, as programmed in the local PLC of the pumping stations, were derived from the sewer plan of the municipality of Rotterdam. Information for a comprehensive calibration of the model is lacking but the model results were discussed with the operators to verify whether the model behaviour is consistent with the behaviour of the actual system. (As mentioned earlier, since we are dealing with a comparative study, a calibration is not absolutely necessary.)

The results of the reference model are compared with the results of the optimization model for the different cases. In all cases, the objective function is formulated according to the above mentioned operational tasks. Preventing an overflow to a sensitive receiving water in the city (Nodes 1, 3, 4, 5, 7, 8 & 9) is considered more important than a (controlled) overflow to the river Nieuwe Maas (Nodes 2, 6 & 10). Obviously the total overflow volume of the system has to be minimized, meaning that an overflow should be prevented unless all storage capacity has been used.

## 7.4 Control horizon

The 333 events are simulated for three cases, with an increasing control horizon, i.e. $T=$ 10, 40 and 90 min (or $T=$ 1, 4, 9). Applying $T=$ 1 means that the optimization problem is solved on basis of the actual systems state and the inflow for the next time step. This is the shortest time horizon for which the operational optimization problem can be solved: based on the present systems state we make a decision on the flow for the next time step. The optimization model, as formulated in Chapter 4, does not allow a control horizon equal to 0. A horizon of 40 min may be considered feasible. Due to the delay in the runoff process this information can be obtained on the basis of actual rainfall data. To get a proper value of the areal rainfall, a dense network of rain gauges and radar images are needed. Reliable inflow forecasts for a time horizon of 90 min are with the present state of technology more difficult to obtain, but it is of course interesting to see what can be gained if this information were available.

The results of the simulations are presented in Fig. 7.2. Compared to the locally controlled system, a significant reduction of overflows can be achieved. The total overflow volume of the reference model amounts to $15.8 \cdot 10^6$ m$^3$ (during a period of 15 years), whereas the total overflow volume of the optimally controlled system is (in all cases) in the order of $5.4 \cdot 10^6$ m$^3$ (i.e. a reduction of 65%). This reduction is mainly due to a decrease in controlled overflows. The reduction of uncontrolled overflows to city canals is limited to about 0 - $0.7 \cdot 10^6$ m$^3$ (depending on the control horizon). This is due to the fact that the set points of the locally controlled system (the present situation) are chosen so that overflows to city canals are at a minimum. Apparently this is at present achieved at the expense of controlled overflows.

Regarding the overflow frequencies it can be concluded that the standards as mentioned in section 7.1 are met. The results of the reference model show that the overflow frequency to city waters is in the order of 2-5 times/yr and the pumped overflows operate about 15 times/yr. Using optimized control, the frequency and

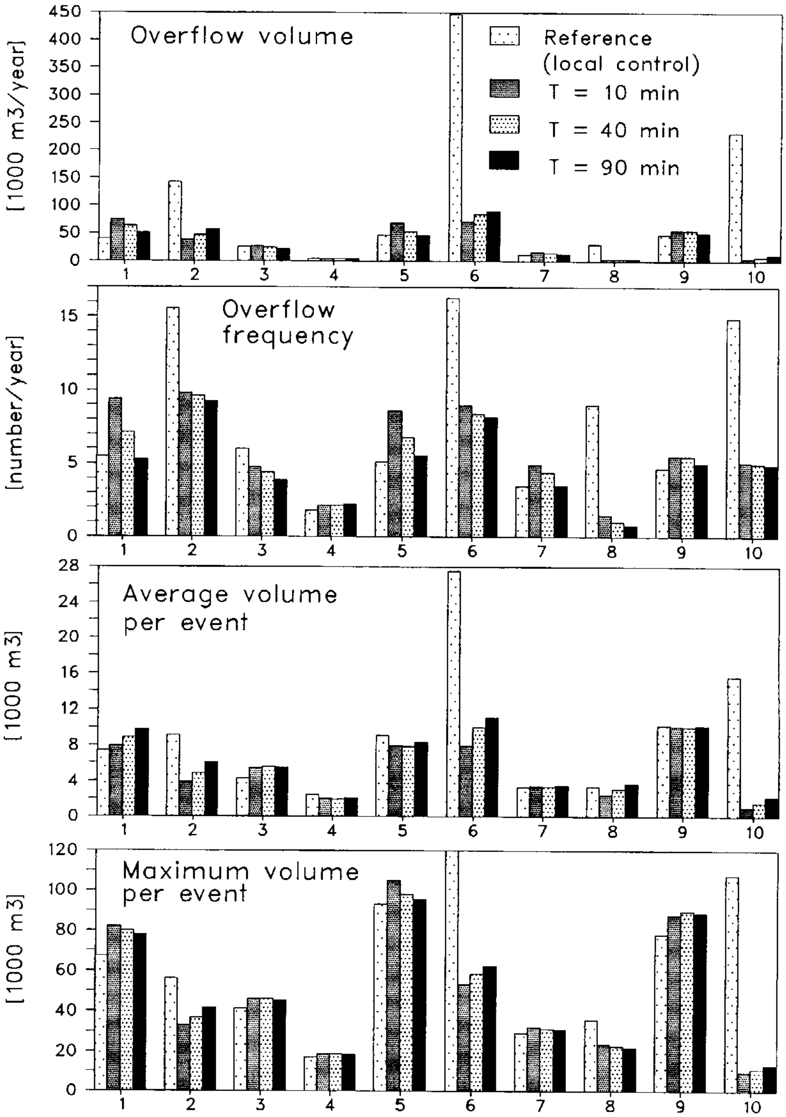


*Figure 7.2 Effects of increasing control horizon*

volume of controlled overflows can be decreased drastically. Depending on the control horizon this may result in a (small) increase of uncontrolled overflows. The differences in total overflow volumes are not much affected when increasing the control horizon ($5.40 \cdot 10^6$, $5.36 \cdot 10^6$, and $5.21 \cdot 10^6$ m3 in the respective cases). This indicates that in all cases the available storage capacity is used to its maximum. The difference is mainly to be found in the location of overflows.

When using a control horizon of 10 minutes, a small increase of overflows to city waters is found compared to the reference model (in the order of 5%). In this case, the pumped overflows are activated too late. The distribution of overflow volumes among the nodes alters when increasing the control horizon. The controlled overflows augment, whereas the uncontrolled overflows diminish (the total remains more or less the same).

Therefore, it can be concluded that for the Rotterdam system the main effect of increasing the control horizon is that the decisions on the location of overflows are improved (as they can be foreseen at an earlier stage). Concerning the total overflow volume the length of the control horizon is less important. This is illustrated by the left graph of Fig. 7.3.

It is noted that the risk of an overflow at a certain place can further be decreased by applying a greater value of the unit cost of storage (as explained in Section 3.5.2). In this case study, the same value of the coefficient $\kappa_i$ for all $i$ nodes was applied, indicating that the filling degree at all nodes was kept at a relatively equal level as much as possible. By applying a value $\kappa_j$ ($>\kappa_i$), the filling degree at node $j$ will be kept at a level equal to $\kappa_i/\kappa_j$ times the filling degree of node $i$. The storage capacity of node $j$ will therefore only be used to its maximum when a possible overflow at node $i$ can be predicted. An example of this can be found in (Breur, 1992).

## 7.5 Under- and overestimated inflow

The required accuracy of the inflow forecast is investigated by introducing prediction errors. Four cases, with a control horizon of 40 minutes ($T = 4$) were simulated for the inflow series of 333 events. The errors introduced are listed in Table 7.2.

*Table 7.2 Forecast errors in the four simulated cases*

| T = 4 | t=1 | t=2 | t=3 | t=4 |
|---|---|---|---|---|
| Case 1 (--) | - 50% | - 100% | - 100% | - 100% |
| Case 2 (-) | - 15% | - 30% | - 50% | - 70% |
| Case 3 (+) | + 15% | + 30% | + 50% | + 70% |
| Case 4 (++) | + 50% | + 100% | + 100% | + 100% |

The results of the four cases are presented in Fig. 7.4, together with the results of the case with no prediction errors and the reference model.

The tendency is similar to the effects of increasing the control horizon. The total overflow volumes are hardly affected by the inaccuracies of the predicted inflow. Even if no inflow is predicted (Case 1) or the predicted inflow is twice the actual inflow (Case 4), the operation strategy derived with LOCUS remains acceptable. Fig. 7.4 shows clearly that an overestimate of the inflow has a positive effect on those places where overflows are given the highest unit cost. Conversely, an underestimate of the inflow leads to less controlled overflows, at the expense of an increase of overflow at the more sensitive places.

To illustrate the effects of the length of the control horizon and inaccuracies in inflow predictions on the performance of the entire system, the totals of all cases are presented below.

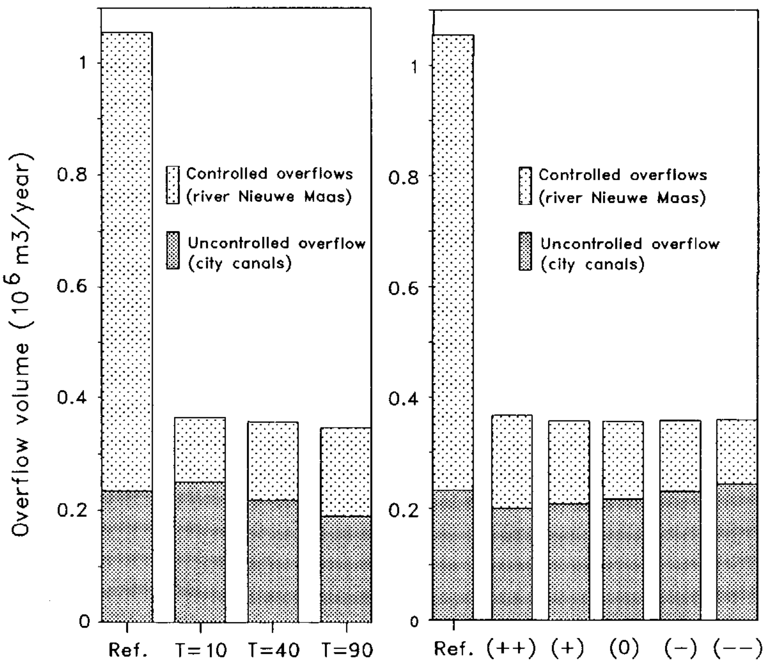


*Figure 7.3 Simulation results Southern District, totals*

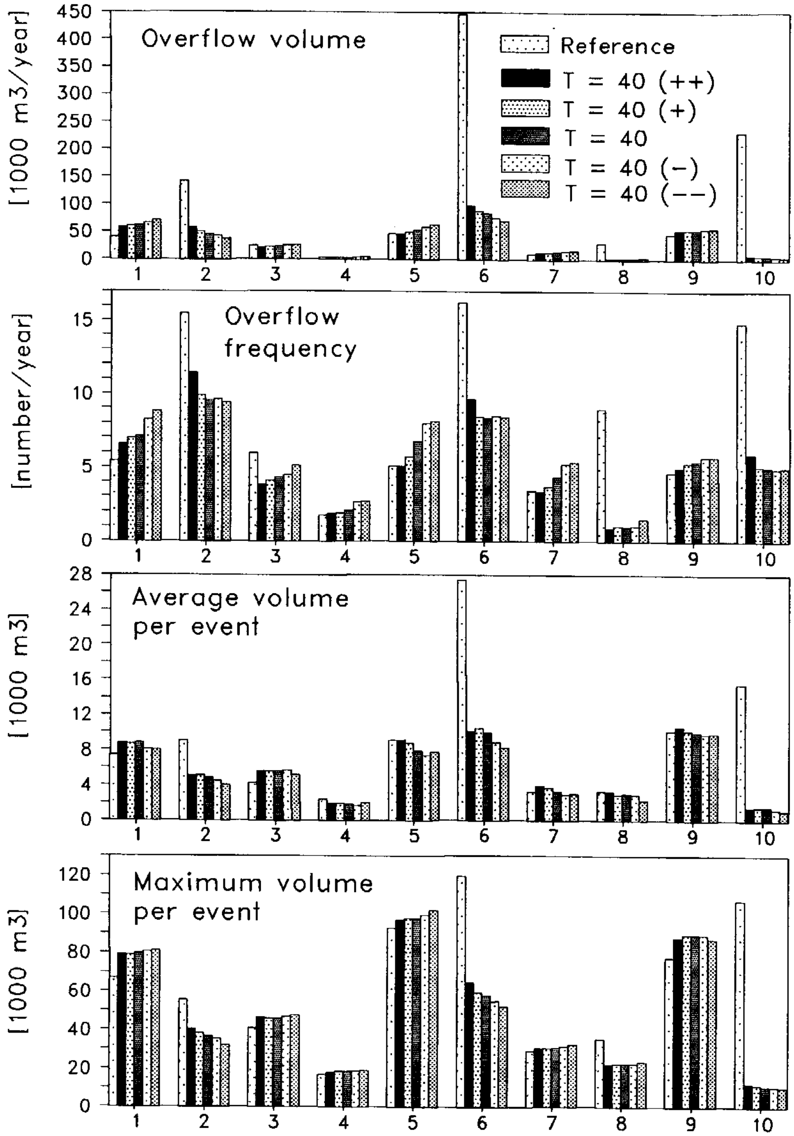


*Figure 7.4 Effects of prediction errors*

## 7.6 Concluding remarks

The case study has shown that in Rotterdam a considerable reduction of overflows can be achieved by applying a real time operation. In this report only the results of the Southern district are presented. As was mentioned in the introduction, the same calculations have been performed for the Western district, leading to similar conclusions.

Especially the controlled (pumped) overflows to the river Nieuwe Maas are at present operated too often and too long. The model calculations show that these overflow volumes can be reduced by about 70%, without affecting the overflow volumes to the more sensitive city canals.

Concerning minimizing the total overflow volume, it can be concluded that the optimization model is rather insensitive to both the length of the control horizon and possible errors in predicted inflow. The reasons for this were explained in Chapter 6. The installed pumping capacities in the drainage system of Rotterdam are relatively large, i.e. they are of the same magnitude as the variations of the system loading. This means that the system is able to correct a perturbation (more or less) in real time. It takes only a few time steps for the system to return to its desired state (expressed in use of storage at the various nodes). As a consequence the applied control horizon has no significant influence on the total overflow volumes. However, the more the operational objectives are differentiated in time and place, the more the time horizon and the accuracy of the predicted inflow become important.

Nevertheless, it has been shown that in this case LOCUS may derive a suitable operation strategy even if the inflow is not known with a great deal of confidence. In fact, the optimization problem can be formulated in such a way that this uncertainty is taken into account. If an accurate figure of the expected inflow is not available one has to ensure that the use of storage at the most sensitive places is kept at a lower level as compared to other nodes. This can in principle be achieved in two ways: by an overestimation of the inflow and by applying a greater value of the unit cost of storing water at a particular node.

It is noted that it is generally better to use a long control horizon (in the order of 1-2 hr) that is uncertain than a short horizon (e.g. 10-20 min) for which we have an accurate inflow prediction. This allows for a better estimate of the rate of increase of the stored volume (= the coefficient $\alpha_2$ in Eq. 4.15), which is used in determining the unit cost of storage ($cv_i^t$). Because all elements of the system are interrelated, the operational problem should at least be formulated for the longest flow time in the system (and preferably somewhat longer for the reason just mentioned). To control the risk of an overflow at a certain place, two parameters can be used. The filling and emptying behaviour of the system is determined by the unit costs of storage ($cv_i^t$). As soon as a possible overflow can be predicted, this behaviour will be influenced by the unit cost of overflow ($co_i^t$) (which is generally given a value much greater than the unit cost of storage). The determination of a proper set of unit costs requires a basic knowledge on how the problem is formulated and solved, and should not be a process of trial and error (Sections 4.5 & 5.3.2).

# 8 Case study of Damhus, Copenhagen: Impact of rainfall distribution

## 8.1 Introduction

In Chapters 6 & 7 emphasis was put on the parameters of the optimization model and their influence on the control decisions. In this chapter the factors that may contribute to the potential of real time control of an urban drainage system are investigated. As was mentioned, three contributing factors can be distinguished:

1. The input of the system (dry weather flow, rainfall) is distributed in time and space.
2. The system itself shows always a certain discrepancy in planned and actual behaviour due to schematizations in the design. Further, the available storage and discharge capacities of the different sub-sections of the system are as a rule not optimally attuned. As a result of urban developments, renovation, maintenance work, and other reasons, some sections of the drainage system will have relatively more storage or discharge capacity available compared to other sections.
3. The effects of the output of the system are of different temporal and spatial scale.

Attention is paid to the first two factors. The importance of incorporating rainfall distribution when using series of historical rainfall data as the basis for urban drainage design and operation has been investigated for a district of the city of Copenhagen, called Damhus. The main reason for choosing this study area is the availability of the required rain data, but it is interesting that plans are being developed to renovate the system and to implement a real time control system within the near future. For this catchment, the potential of real time control has been quantified, particularly with respect to the possibilities to minimize combined sewer overflows (CSO), and the extent to which rainfall distribution contributes to this.

## 8.2 Problem definition

There are few studies quantifying the effects of storm characteristics on runoff response, e.g., (Niemczynowicz, 1984), (Nouh, 1990), (Watts et al., 1991), (Andersen et al., 1991). Most studies conducted in this field are focusing on the effects of moving storms on the runoff hydrograph and difference in magnitude of the peak discharge. The main finding of these studies is that the peak discharge from a storm moving downstream exceeds that from a storm moving upstream, but that the degree of this 'maximal bias' varies greatly depending on the system characteristics. Therefore, in determining the catchment hydrograph, one may improve the results by incorporating storm movement parameters. However, in (Andersen et al., 1991) it is concluded that from a statistical point of view the difference between homogeneous and moving rain is not significant.

As was mentioned in Chapter 2, the governing factors in designing an urban storm drainage system are to be found in the statistical properties of the system output, rather than the effects of a single storm. A possibility to gain insight to these properties is to perform long term simulations (say, in the order of 10-40 years) using continuous rain data and to analyze the model results on their statistical properties. Generally, such simulations are based on recorded rain events of only one point measurement. Evidently it is preferred to use a rain series that is recorded in or nearby the studied catchment, but often this information is lacking. Moreover, incorporating the temporal and spatial distribution of rainfall requires data of a dense network of rain gauges (and possibly a weather radar), which is generally not available. Also a probabilistic model describing this phenomenon for a relatively small area and with a high degree of accuracy has not been derived yet (Witter, 1984), (IAWPRC, 1989).

Since detailed rainfall information is generally lacking, it is investigated here what errors are introduced by neglecting the heterogeneity of real storms by using the records of one rain gauge for the whole catchment, compared to the results obtained by using the information of a network of rain gauges. To benefit from the variability of the loading a flexible operation is required. The question arises to what extent rainfall distribution affects the probability of a peak flow or CSO, when the system is real time controlled, or what is the importance of the first mentioned contributing factor to the potential of real time control ?.

Although the used models, i.e. SAMBA and LOCUS, both incorporate a simple pollution transport model (based on an ideal mixed reservoir approach), it has been decided to compare the effects of the different rainfall distribution models on the simulated peak flow, and CSO. In designing an urban storm drainage system, these are generally considered important variables, which can be predicted with a higher degree of accuracy than pollution loads. A comparison based on a stochastic interpretation of pollution loads would be more suited concerning the objectives of the drainage system, but since the comparisons are made relatively, the variables involved are not considered significant for the conclusions.

## 8.3 Catchment and available data

The Damhus district covers an area of about 4000 ha, which is approximately 1/3 of the total area of Copenhagen. The schematization of the system, as applied in this study, is presented in Fig. 8.1. The system characteristics and the applied model parameters are summarized in Table 8.1. It is noted that no special efforts have been made to calibrate and verify the used models. Partly because necessary data is missing for a good calibration (an extensive measuring programme has started recently), but also because this is not necessary for the problem in consideration. All conclusions are drawn based on model comparisons, thereby using the same model parameters and inflow data for all cases.

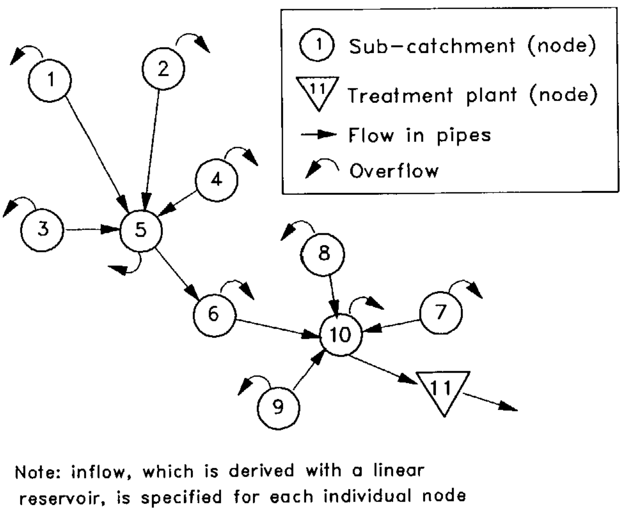


*Figure 8.1. Scheme of the Damhus District*

*Table 8.1. System characteristics*

| node | Area | Imperv. area | DWF | Storage Cap. | Dis. Cap. | Flow time in pipes | |
|---|---|---|---|---|---|---|---|
| | [ha] | [%] | [m3/s] | [m3] | [m3/s] | to | [min] |
| 1 | 266.1 | 64 | 0.233 | 8142 | 1.3 | 5 | 30 |
| 2 | 451.6 | 55 | 0.025 | 17094 | 0.4 | 5 | 30 |
| 3 | 241.6 | 37 | 0.120 | 3384 | 0.8 | 5 | 20 |
| 4 | 677.0 | 57 | 0.482 | 21701 | 6.0 | 5 | 10 |
| 5 | 541.4 | 44 | 0.593 | 15000 | 6.0 | 6 | 10 |
| 6 | 494.5 | 45 | 0.712 | 42699 | 7.7 | 10 | 5 |
| 7 | 285.0 | 58 | 0.165 | 22012 | 0.8 | 10 | 5 |
| 8 | 236.0 | 100 | 0.825 | 3357 | 1.6 | 10 | 5 |
| 9 | 494.4 | 38 | 0.307 | 11996 | 5.5 | 10 | 5 |
| 10 | - | - | - | 57802 | 1.8 | 11 | 0 |

In Copenhagen rainfall is measured continuously using a relatively dense network of 14 rain gauges, out of which 8 are situated inside or nearby the Damhus district. The data base of the period 1979-1989 contains 2878 rain events. To reduce the computational time, a selection has been made. Since the model results are compared mainly with respect to overflows, only the storms with a rain depth greater than 3 mm have been considered. Smaller storms are assumed not to be important with respect to peak flows and overflows. This criterion results in a series of 246 events.

Out of these 246 events, 17 storms have been selected to investigate the importance of a high spatial resolution of rainfall data. To get a broad view the storms are chosen so that 9 storms lead to an overflow at only a few overflow structures of the system, during 4 storms about 50% of the overflow structures are operating and 4 storms may be considered as extreme events during which the entire system is overflowing.

Three different rainfall distribution models are compared, both for an uncontrolled and a controlled system. The six approaches are summarized in Table 8.2. The study can be divided into two stages (a & b). First, a detailed analysis is conducted on the impact of the spatial resolution of the rainfall information on the model results, using the 17 selected storm events. Afterwards, simulations are made of all major events that have been measured in the period of 1979-1989 to investigate the effects of rainfall distribution on overflow volumes.

*Table 8.2. Set up of the study*

| | | Uncontrolled system (SAMBA / LOCUS-reference model) | | | Controlled system (LOCUS) | | |
|---|---|---|---|---|---|---|---|
| Approach nr. | | 1 | 2 | 3 | 4 | 5 | 6 |
| rainfall | | Homogeneous (1 r.g.) | Distributed (Kriging) | Distributed (8 r.g.) | Homogeneous (1 r.g.) | Distributed (Kriging) | Distributed (8 r.g.) |
| nr. of events | a. | 17 | 17 | 17 | 17 | 17 | |
| | b. | 246 | | 246 | 246 | | 246 |

Homogeneous rain (approach 1 & 4) is modelled by using the data of 1 rain gauge for all catchments. The rain gauge used to model the homogeneous rain is located in the centre of the Damhus district. A straightforward approach to take the rainfall distribution into account is to use simply the value of the measurement of the station that is located at the shortest distance to the specific inflow point of the drainage system (approach 3 & 6). This approach is further referred to as the 'nearest rain gauge' (NRG) approach. A possibility to obtain a better estimator of the rain depth at a certain location is to interpolate the observed values, e.g., by using the Kriging Method (approach 2 & 5).

Kriging is a spatial interpolation method which is specially developed to express the structural properties of a (natural) phenomenon which is temporally and spatially distributed and which shows a certain structure. The observations are considered to be stochastic variables independent of the place of measurement. The unknown value $Z(x_0)$, i.e. the rain depth at a certain point $x_0$, is a linear combination of the known values $Z(x_i)$, multiplied by a weighting factor $\lambda_i$.

In formula the Kriging method reads

$$Z(x_0) = \sum_{i=1}^{n} \lambda_i Z(x_i) \tag{8.1}$$

where

$$\sum_{i=1}^{n} \lambda_i = 1 \ \wedge \ 0 \leq \lambda_i \leq 1 \tag{8.2}$$

Kriging determines the weights $\lambda_i$ which provide the best estimator possible, i.e. whose estimation variance is at minimum. For this purpose, a so-called semi-variogram is derived from all observations. This graph describes the mean squared difference (or semi-variance) of two point measurements separated by a certain distance. Therefore, to establish a semi-variogram each pair of data points is to be considered. As the variance will vary in time such a variogram has to be established for each time interval of the hyetograph that is taken into account. (As this is quite laborious the method is mostly applied to describe the spatial distribution of total rain depths in estimating the areal rainfall, which requires one variogram only). In the analysis of the 17 selected events, a time interval of 15 minutes is applied. The rainfall depth at a certain place $Z(x_0)$ is derived by minimization of the estimation variance, using the method of the Lagrange Multipliers. For this purpose the GEOEAS model of the American meteorologic office was used. For a description of the mathematical background of Kriging and its use in hydro-sciences reference is made to (Delhomme, 1978).

### 8.4. Remarks on the used models

To begin with, the uncontrolled system has been simulated with both SAMBA and the LOCUS reference model (to have a comparison). SAMBA is part of the MOUSE package and specially developed for analysis and design of sewer systems on personal computers, based on time-series calculations (MOUSE, 1990). To minimize computation time the flow routing is carried out using the time-area method, describing both the runoff from surfaces and the flow in pipes. In order to simulate spatially distributed rain, a modified version of the SAMBA model was used. In this version, a rain gauge has to be specified for each inflow point of the system. It is noted that the commercial version of SAMBA does not (yet) incorporate this feature, but uses only one rain gauge for the entire system.

It is noted that SAMBA and the LOCUS 'reference model' differ in the way the surface runoff hydrograph is determined and in the methods of flow routing. In LOCUS, the inflow to each calculation point (node) has to be specified separately. A constant initial loss of 0.6 mm and a linear reservoir model are used for each sub-catchment to transform the rainfall data into inflow data. The flow routing in LOCUS proceeds from a lumped storage approach (Fig. 8.1), which is derived from the MOUSE catchment data. The results of both models concerning overflows are in the same order.

A special version of SAMBA, called SAMBA-CONTROL, can be used to simulate dynamic controlled systems on the basis of a predefined control scenario (Harremoës et al., 1989). In Chapter 4 it was concluded that for a systems analysis the use of mathematical optimization to derive the operation strategy is to be preferred as this approach provides maximum flexibility and consistency in decision making. Therefore the LOCUS model is used.

In the preceding chapters the impact of the control horizon has been investigated. It was shown that the optimization model is rather insensitive to the length of the control horizon and forecast errors. To determine a suitable control horizon for the problem under consideration, 3 runs have been made for 200 rain events using control horizons of respectively 30 minutes, 1 hour and 6 hours. The latter is not a realistic option, but can be considered as an approximation of the optimum. The results obtained with a forecast of 30 minutes are improved by expanding the control horizon to 1 hour. As was explained in Chapter 7, a longer horizon allows for a better estimation of the rate of increase of the stored volume (= the coefficient $\alpha_2$ in Eq. 4.15). Besides, the control horizon should at least be equal to the longest flow time in the system (and preferably somewhat longer for the reason just mentioned). A further improvement of the results obtained by using a forecast of 6 hours appears to be very small. Therefore, a control horizon of 1 hour has been applied to the calculations that are discussed below.

## 8.5 Selected events

As mentioned above, the 17 selected rain events have been simulated with both SAMBA and the LOCUS 'reference model'. Because the difference between the calculated overflows is not significant it has been decided to include the results of SAMBA only. These results are presented in Table 8.3.

To compare the different rainfall distribution models, the model results in the right columns of Table 8.3 are presented as index values, which are related to the results that are obtained by simulating the 'Kriging' data (= 1.00), assuming that this is the most realistic approach. The 3 different rain models are compared in terms of their impact on rain inflow, overflow volume, total outflow, maximum flow, and the emptying time of the system. The latter is the period of time during which 98% of the rain inflow is discharged.

Since only 17 events are analyzed, no general conclusions can be drawn on possible long term trends, but from Table 8.3 it can be concluded that rainfall distribution may have a considerable impact when investigating a single storm event. Neglecting this phenomenon by using the data of only one rain gauge for the whole catchment may lead to overestimates of the maximum flow, overflow and outflow of the system, due to the fact that rainfall peaks are assumed to occur at all places at the same time. However, underestimates may also occur, simply because the specific rainfall station, whose data is used to simulate the homogeneous rain, does not measure all rainfall peaks that might occur in the region.

*Table 8.3. Simulation results of 17 selected rain events (SAMBA)*

| | SAMBA (Uncontrolled) | | SAMBA: index values (Kriging = 1.00) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| event nr. | CSO [m3] | | overflow | | outflow | | max. flow | | rain inflow | | empty. time | | | |
| | 1 r.g. | Kriging | 1 r.g. | 8 r.g. | 1 r.g. | 8 r.g. | 1 r.g. | 8 r.g. | 1 r.g. | 8 r.g. | 1 r.g. | 8.r.g. | | |
| 1 | 2547 | 2434 | 1.05 | 0.78 | 0.83 | 1.14 | 3.52 | 1.22 | 0.83 | 0.97 | 0.85 | 1.25 | | |
| 2 | 4618 | 4794 | 0.96 | 1.04 | 1.00 | 1.01 | 1.35 | 1.04 | 1.00 | 1.00 | 1.00 | 1.01 | | |
| 3 | 4665 | 6339 | 0.74 | 0.98 | 0.85 | 0.98 | 1.02 | 1.15 | 0.83 | 0.97 | 0.83 | 0.98 | | |
| 4 | 13753 | 6975 | 1.97 | 1.06 | 1.29 | 1.05 | 5.81 | 1.22 | 1.25 | 1.07 | 1.36 | 1.04 | | |
| 5 | 5937 | 7740 | 0.77 | 1.00 | 1.02 | 1.01 | 0.84 | 1.10 | 0.98 | 1.05 | 1.03 | 1.01 | | |
| 6 | 2214 | 12718 | 0.17 | 0.93 | 0.64 | 1.11 | 0.35 | 0.82 | 0.58 | 0.92 | 0.64 | 1.23 | | |
| 7 | 7577 | 13844 | 0.55 | 1.02 | 0.48 | 1.00 | 1.21 | 1.16 | 0.68 | 0.96 | 0.40 | 1.02 | | |
| 8 | 22136 | 15992 | 1.38 | 1.14 | 1.14 | 1.11 | 2.00 | 0.95 | 1.03 | 1.04 | 1.22 | 1.14 | | |
| 9 | 57276 | 19942 | 2.87 | 1.29 | 1.92 | 1.15 | 3.64 | 1.15 | 1.71 | 1.11 | 2.26 | 1.22 | | |
| 10 | 37288 | 24809 | 1.50 | 1.15 | 1.04 | 1.02 | 1.39 | 1.08 | 1.17 | 1.06 | 1.03 | 1.01 | | |
| 11 | 38504 | 29177 | 1.32 | 1.22 | 1.24 | 1.02 | 1.05 | 1.02 | 1.13 | 1.07 | 1.36 | 1.02 | | |
| 12 | 78832 | 36616 | 2.15 | 1.17 | 1.47 | 1.15 | 2.68 | 1.24 | 1.50 | 1.08 | 1.62 | 1.21 | | |
| 13 | 83215 | 50689 | 1.64 | 1.23 | 0.95 | 0.98 | 2.11 | 0.89 | 1.18 | 1.04 | 0.94 | 0.99 | | |
| 14 | 248717 | 83092 | 2.99 | 1.53 | 0.86 | 0.95 | 2.79 | 1.02 | 1.97 | 1.21 | 0.79 | 0.95 | | |
| 15 | 204901 | 151885 | 1.35 | 1.02 | 1.06 | 1.11 | 3.15 | 1.09 | 1.19 | 1.00 | 1.11 | 1.20 | | |
| 16 | 418204 | 309402 | 1.35 | 1.04 | 0.98 | 1.00 | 0.86 | 1.39 | 1.27 | 1.04 | 1.02 | 1.06 | | |
| 17 | 661114 | 603482 | 1.10 | 1.00 | 1.00 | 0.98 | 1.18 | 1.09 | 1.09 | 0.99 | 0.98 | 0.99 | | |
| total | 1891498 | 1379930 | | | | | | | | | | | | |
| average | | | 1.40 | 1.09 | 1.05 | 1.04 | 2.06 | 1.10 | 1.14 | 1.03 | 1.08 | 1.08 | | |
| std.dev. | | | 0.73 | 0.16 | 0.31 | 0.07 | 1.36 | 0.13 | 0.34 | 0.07 | 0.40 | 0.10 | | |

In the case of a one point measurement (1 r.g.), the total inflow of the investigated 17 events show a range of 58%-197%, compared to the 'Kriging' approach. The average overestimate amounts to 14%. More significant is the difference between the calculated overflow volumes and maximum flows at a structure. The calculated overflow volumes with homogeneous rain range from 17%-287%, with an average overestimate of 40%. The average maximum flow is even twice the average value, which is calculated with the 'Kriging' data. The relatively high values of the standard deviations indicate big differences between the individual events.

The main difference between the distribution models is obviously found when comparing the results of approach 1 and 2, i.e. a one point measurement versus the spatially interpolated data of a network of 8 rain gauges. The difference between the results obtained with the straightforward NRG approach (8 r.g.) and the more detailed 'Kriging' approach, appears to be relatively small. In Table 8.3, the average values are close to 1, with a standard deviation in the order of 10%. Nevertheless it can be noticed that the NRG approach tends to small overestimates of the systems output.

The required resolution of rainfall information depends on the nature of the project. For a time-series simulation, the straightforward NRG approach may be sufficient, whereas in analyzing individual events the highest possible degree of rainfall information will often be required. For practical reasons and because in this case the NRG approach appears to supply a reasonable view on the impact of rainfall distribution (due to the density of the rain gauge network), it has been decided to use this approach for the time-series calculations.

## 8.6 Time series calculations

The series of 246 events covers a period of almost 11 years. The uncontrolled system, with homogeneous and distributed rain (case 1b & 3b of Table 8.2) are simulated with both the SAMBA model and the LOCUS 'reference model'. The calculated overflows are in the same order. To have a fair comparison, the curves of Fig. 8.2 and 8.4, representing the CSO volumes of the uncontrolled system are derived on basis of the results of the reference model. Since the reference model and the optimization model are based on an identical system description, the difference between the curves of the uncontrolled and controlled system is due only to the effects of real time control. Out of these 246 events, 135 events lead to an overflow when the system is uncontrolled. The controlled system counts 130 overflows. Concerning the overflow frequency, real time control appears to have in this case only a small positive effect.

The calculated CSO volumes of the uncontrolled and controlled system are presented in Fig. 8.2 and 8.3 respectively. The results of the calculations with distributed rain are placed in ascending order (being the most realistic approach). The figures show clearly that large overestimates as well as large underestimates may occur, when simulating time series based on a one point measurement. This is a quite obvious result, which can be explained by the stochastic nature of rain events. When using the data of one rain gauge not all rain peaks that occur in the catchment are included, leading to a possible underestimation of the actual rain volume. On the other hand, when a high rain intensity is measured at this specific rain station it is unlikely that this intensity occurs at all places in the catchment. Assuming that the peak rainfall occurs at all places at the same time may lead to large overestimates of maximum flow, overflow and outflow of the system.

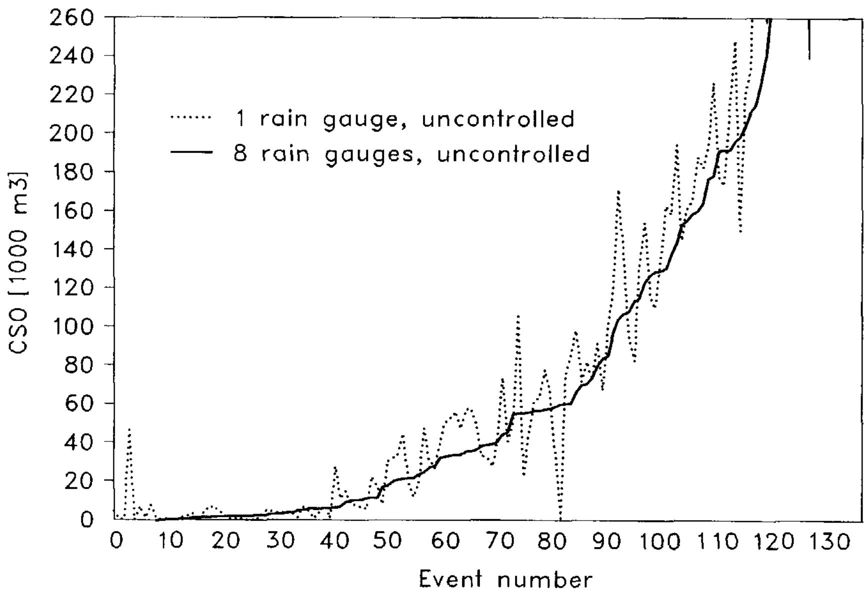


*Figure 8.2. Overflow volumes of the uncontrolled system*

From Fig. 8.2 it can be seen that overestimates happen somewhat more frequent. The total overflow volume of the uncontrolled system that is calculated on the basis of the data of one rain gauge is about 10% greater than the value calculated using the data of all 8 rain gauges. For the controlled system this difference amounts to 5%. A first thought could have been that the number of under- and overestimates would be more or less the same as the number of measured peak rainfalls will be in the same order at all rain gauges. However, we have to consider that in case of homogeneous rain (1 r.g.) the peak rainfall is assumed to occur at all places at the same time, which increases the probability of an overflow.

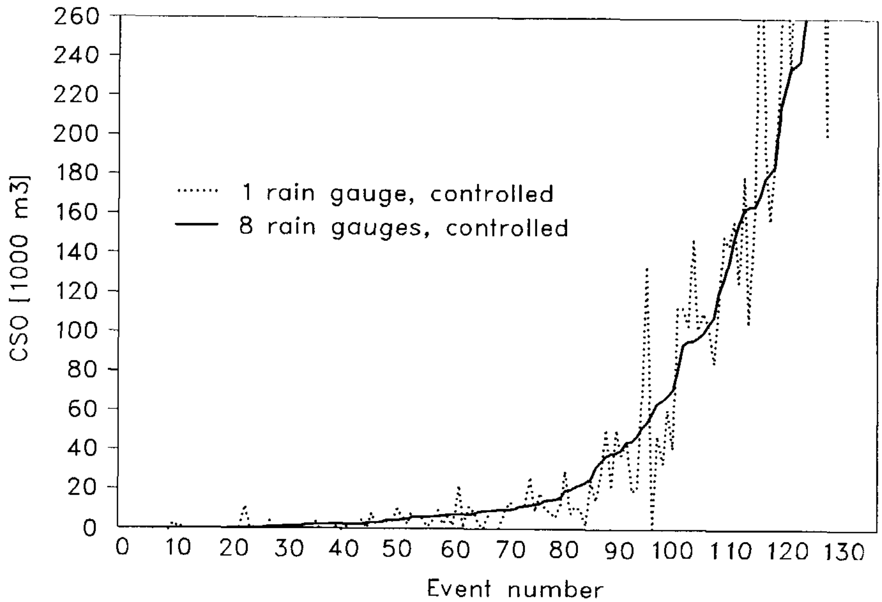


*Figure 8.3. Overflow volumes of the controlled system*

It can be concluded that the use of data of one rain gauge for the whole catchment (homogeneous rainfall) may lead to great errors when investigating a single event. To get insight into the probability of a certain CSO volume, all results have been sorted in ascending order. The curves that best fit to the sorted results have been derived by a multiple regression analysis. Fig. 8.4 represents the distribution (or probability) functions of the CSO volumes in the four investigated cases. Note that the difference with Fig. 8.2 & 8.3 is that in Fig. 8.4 two points of different curves meeting a vertical line do not necessarily represent the calculated CSO volume of the same event.

The CSO distribution functions for 1 and 8 rain gauges are very much alike. As was mentioned above, the probability of an overestimate is somewhat greater when using the data of one rain gauge, but this is not significant. This conclusion holds good for both the uncontrolled and the controlled system.

The potential of real time control to reduce CSO is represented in Fig. 8.4 by the grey shaded area that is enclosed between the curves of the uncontrolled and controlled system. This potential is only due to a better use of the available system capacities. Apparently, from a statistical point of view rainfall distribution does NOT contribute to this.

The possibilities to reduce CSO by means of an improved operation are restricted to about 85% of the rain events that lead to an overflow. The remaining 15% are the extreme events, that may occur about 2-3 times per year, during which all available storage will be used, no matter what operation strategy is applied. The overflow volumes during such extreme events may reach values in the order of $5 \cdot 10^5$ $m^3$ and are therefore not included in the figures.

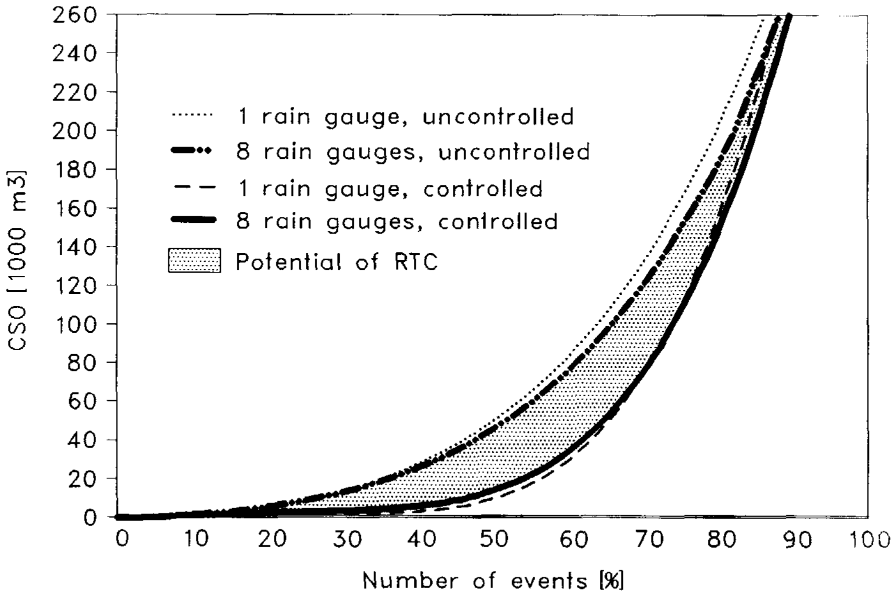


*Figure 8.4. Distribution functions of CSO volumes*

It is noted that these severe rain storms (with a return period in the order of 1-10 years) are generally used as a design load when investigating the hydraulic performance of the system. Flooding problems are usually not solved by improving the use of storage, but by employing a system with sufficient discharge capacity. Furthermore it should be considered that a possible failure of the control system should not result in surface flooding. Therefore, even an uncontrolled system should be able to discharge its design load. With respect to this, application of a dynamic operation may be useful to control the risk of flooding at vulnerable places in the system.

## 8.7 Concluding remarks

The simulations have shown that system variables may be largely over- and underestimated when analyzing the systems performance on the basis of data of one rain gauge. Therefore, if the objective is to predict the performance of a drainage system for a single rain event, (e.g., to determine the peak flow), the temporal and spatial distribution of rainfall is not to be neglected. In such a case the highest possible degree of spatial information on the rainfall may be desirable.

It is noted that the effects of rainfall distribution is one of the aspects that make a good calibration of a computer model generally difficult. When the results of a model simulation are compared with measured data, to determine the model parameters that give least deviation from these measurements, this aspect is often neglected (as detailed information on the actual rainfall is generally lacking).

From a statistical point of view, the effects of rainfall distribution are not significant when evaluating the system concerning overflows. The general approach is to simulate time series of measured rain events, and to analyze the model results on their statistical properties. For the Damhus district (an urban catchment of 4000 ha) time series calculations have been performed using the data of one rain gauge for the whole catchment (homogeneous rain) and using the data of a network of 8 rain gauges (distributed rain). Although great differences are found between the results of individual events, both approaches lead to an almost similar distribution function of CSO volumes. The use of the data of one rain gauge results in small overestimates.

The simulations are conducted for both an uncontrolled and a controlled system, leading to a similar conclusion, namely that from a statistical point of view the effects of rainfall distribution are not significant. For the Damhus district this means that the potential of real time control to reduce CSO can be determined on the basis of a time series simulation of historic rain events that have been measured by one station. In other words, concerning the possibilities of minimizing CSO by means of real time control, the main contributing factor is to be found in the system itself. For the Damhus district the reduction of CSO is mainly achieved by a better use of the available system capacities. Over a long term, the effects of rainfall distribution do NOT contribute to this.

It should not be concluded that rainfall distribution can be neglected when investigating the potential of real time control concerning other aspects. The use of data of one rain gauge will lead to an overestimation of the total peak inflow (as it is assumed that the peak rainfall occurs at the same time at all places), and consequently the peak discharge rates to the treatment plant will be overestimated. The general aim of real time control is to improve the use of available system capacities for all storms that the system is exposed to. This means that not only overflows are to be minimized. Other important operational objectives (that are to be met at the same time) are, among others, to direct overflows to less sensitive receiving waters and to maintain optimal flow rates to the treatment plant. For example, due to rainfall distribution it might be possible that peak flows can be reduced without affecting CSO frequency and volumes. An example of this will be discussed in the next chapter.

# 9 Case study of Westfriesland: Reduction of peak flows to the treatment plant

## 9.1 Introduction

The Wervershoof treatment plant serves a complex system of combined sewer systems and a few separate systems of several villages in the region of Westfriesland, in the province of North Holland (Fig. 9.1). The catchment area is approximately 250 $km^2$. In 1986, the plant had a biological capacity of 130,000 inhabitant equivalents and a hydraulic capacity of 3600 $m^3/h$. The characteristics of the main sub-catchments are presented in Table 9.1.

Soon after the plant was put into operation in 1980, it appeared that its hydraulic capacity would become insufficient within a few years, due to urban development. The hydraulic capacity of a treatment plant is conventionally set equal to the sum of the installed pump capacities of the connected sewer systems. This approach would ultimately lead to a required treatment capacity of 5800 m3/h. An alternative solution was found by limiting the total discharge of the pumping stations to the treatment plant, by implementing a real time control system.

Bakker et al. (1984) demonstrated that the peak flow to the treatment plant could be reduced, without seriously affecting the overflow frequencies of the connected sewer systems, by making use of the spatial variability of rainfall and by optimizing the use of the available storage in the combined sewer systems. At first, this meant that an expansion of the plant could be postponed for several years. Furthermore, when an expansion of the plant would become inevitable due to urban development, the ultimate capacity could be reduced considerably, compared to the capacity that would have been required according to the traditional standards.

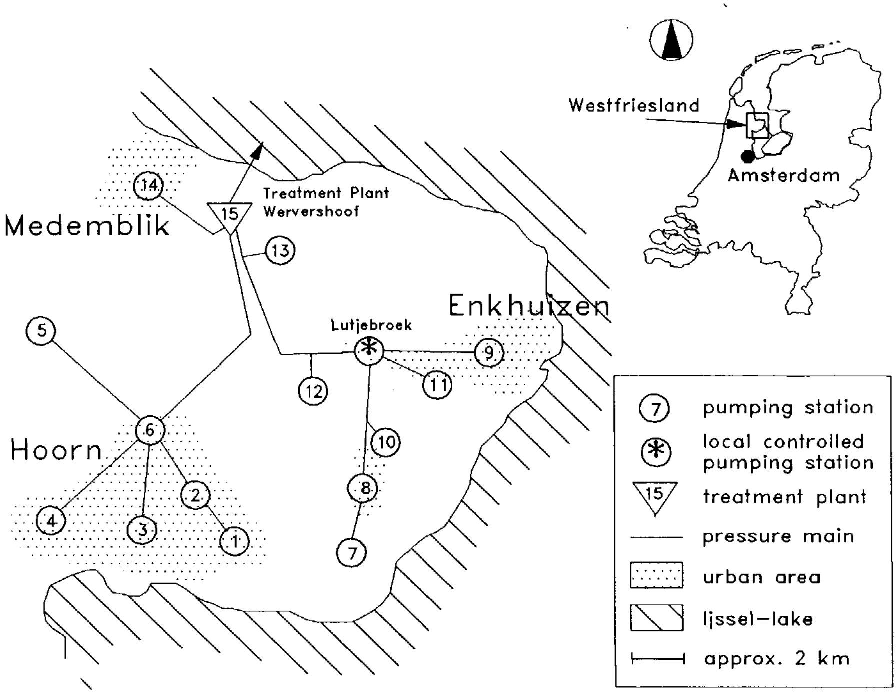


*Figure 9.1 Overview of the main pumping stations and pressure mains*

To realize this solution a real time control system was necessary for the main pumping stations of the sewage collection and transport system. The main constraint for a successful implementation of the control system, as formulated by the water board 'Uitwaterende Suizen', was that the Dutch design criteria concerning the overflow frequency (as discussed in section 2.2.2) were to be met, despite the reduced 'over-capacity'.

The control system has been in uninterrupted operation since May 1983. The impact of the control system has been investigated by the author and has been reported in (Nelen, 1988a), (Nelen, 1988b). Some results of this evaluation study are summarized below. In addition to this evaluation study, the performance of the present control scenario is compared with LOCUS.

*Table 9.1 Characteristics of the sub-catchments (1986)*

| Node | Name | Area [ha] | DWF [m3/h] | Storage capacity [m3] | Storage capacity [mm] | Pumping capacity [m3/h] | Down. node | Flow time [10 min] |
|---|---|---|---|---|---|---|---|---|
| 1 | Koewijzend | 5 | 20 | 500 | 10.0 | 160 | 2 | 1 |
| 2 | Krijterslaan | 9 | 100 | 800 | 8.9 | 330 | 6 | 1 |
| 3 | Oosterpoort | 90 | 250 | 9000 | 10.0 | 1236 | 6 | 1 |
| 4 | Grote Waal | 20 | 110 | 1850 | 9.2 | 180 | 6 | 1 |
| 5 | Wognum Sijbekarspel Nibbixwoud | 18 | 120 | 1770 | 9.8 | 480 | 6 | 1 |
| 6 | Hoorn | 10 | 300 | 2000 | (20.0) | 2200 | 15 | 1 |
| 7 | Venhuizen Torenweg B. Zijpweg | 12 | 72 | 1200 | 10.0 | 200 | 8 | 1 |
| 8 | Stedebroec-Z | 28 | 150 | 2500 | 8.9 | 1000 | 15 * | 2 |
| 9 | Enkhuizen | 52 | 170 | 5000 | 9.6 | 800 | 15 * | 2 |
| 10 | Stedebroec-N | 15 | 50 | 1300 | 8.6 | 275 | 15 * | 2 |
| 11 | P.Gielenstr. Raadhuislaan | 13 | 50 | 1290 | 9.9 | 170 | 15 * | 2 |
| 12 | Hoogkarspel | 15 | 90 | 1450 | 9.9 | 220 | 15 | 1 |
| 13 | Wervershoof Onderdijk | 20 | 125 | 1700 | 8.5 | 280 | 15 | 1 |
| 14 | Medemblik | 26 | 150 | 2000 | 7.7 | 500 | 15 | 1 |
| ** | Uncontrolled flow to the treatment plant (small districts not included in the control system) | | | | | 350 | | |
| 15 | Treatment Plant Wervershoof | | | | | | | |

* = via pumping station Lutjebroek (which is locally controlled)
** = these small districts are not included in Figure 9.1

## 9.2 Present control scenario

This section explains the basic principles of the control scenario as applied in Westfriesland. A detailed description is included in (Nelen, 1988a). The system is set up as a fully automatic and centralized control system. From a total of 125 pumping stations, 27 have been selected to be part of the telemetry network, out of which 24 are centrally controlled. (In the simulations, only the 20 major sub-catchments are considered). The relatively small and most upstream situated pumping stations, as well as the pumping stations of separated sewer systems, are left out of the central control system. These stations are operated automatically, on the basis of the local water level.

Further, one major pumping station, Lutjebroek (Fig. 9.1), is not included in the central control system as it only serves to pump through the sewage to the treatment plant. If the upstream pumps are turned off, Lutjebroek will automatically follow.

The main function of the central control system is to 'switch free' or to 'block' the pumps in the pumping stations. The control system does not interfere in the regime of the pumps itself (e.g., by changing set points). The pumps are turned on and off by the local controller on the basis of the water level in the pump well. However, before pumps can be activated, the local controller needs 'permission' of the central control system. For example, although the water might have reached a level by which the pump should be operated at full capacity (according to the local set point), the central control system may decide to 'block' this level of operation and to set 'free' only a lower level of operation, in order to create some extra 'space' for pumping stations with a higher priority.

During dry weather periods the only action of the control system is to determine the total discharge to the treatment plant. In this case, all the pumping stations are set 'free'. In rain periods, when the total discharge of the pumping stations reaches a value which is (almost) equal to the hydraulic capacity of the treatment plant the control system switches from its so called 'shortened program' to its 'main program' and starts controlling the pumps.

The water level in the pump well is considered to be a sufficiently accurate indication of the current loading (filling degree) of the sub-system, as the area is completely flat. The main principles of the control scenario are as follows.

1. Based on the measured water levels at the pumping stations a 'relative water level' or 'priority' is calculated, i.e. the actual water level multiplied with some adjustable weighting factors, which are determined on the basis of the sensitivity of the receiving water and the area of the district.
2. The pumping stations are ranked according to their priority.
3. An appropriate combination of discharge capacities (up to the hydraulic capacity of the treatment plant) is determined on the basis of the 'priority-list' and the current operation of the pumping stations, taking into account some (adjustable) control parameters, such as a maximum switching frequency of pumps, a waiting time before a pump can switch to another level of operation etc..
4. Pumps (discharge capacities) are set 'free' on the basis of the derived combination.

5. Before the control commands are executed, a number of hardware constraints are checked.

This process is repeated with intervals of 5 minutes. The moment that all the pumping stations are set 'free', the control system returns to its 'shortened program'.

## 9.3 Results of the evaluation study

### *9.3.1 Recorded data*

The impact of the control system has been investigated on the basis of the recorded pump data and so called 'high water situations' of a period of 3 years. The main results of this investigation are discussed below.

The records are clear regarding the maximum discharge to the treatment plant: the hydraulic capacity of the plant, which is about 70% of the total pumping capacity of the contributing pumping stations, was not exceeded during this period, except for some peak-discharges of very short duration (in the order of 15 minutes). These peaks are due to the fact that the pumping station Lutjebroek is not included in the control system. When the upstream pumping stations of Lutjebroek are 'blocked' it takes some time before the locally controlled pumps of Lutjebroek are turned off. Meanwhile the pumping stations in Hoorn and/or Medemblik may operate at their maximum capacity. As a result the total discharge may exceed the treatment plant capacity for short periods.

The most relevant data concerning the combined sewer overflows (CSO) are the recorded 'high water situations'. 'High water' is defined as a situation where the water level at the pumping station exceeds the 90% storage level. The overflow threshold is theoretically at the 100% level. Therefore, not all recorded 'high water situations' refer to a CSO. In the evaluation study, the duration of a 'high water situation' was used to indicate whether an overflow did occur or not. Obviously, this approach provides only a rough estimate of the overflow frequency. To determine the actual overflow frequency requires measurements at the site of the overflow structures, which is practically not feasible due to the great number of overflow structures (> 100).

Based on the assumption that a 'high water situation' with a duration of 30 minutes or more is a reasonable indication for a CSO, it could be concluded that the standard of the maximum overflow frequency of 7 per year was met for most sub-systems, despite the reduced 'over-capacity'. At a few sub-systems this standard was exceeded. However, it could be shown that the pumps of these sub-systems were hardly ever 'blocked' by the control system. Therefore, the relatively high number of 'high water situations' was in these cases more an indication of a limited amount of available storage than malfunctioning of the control system. As the central control system hardly interferes in the pumping regime of these particular sub-systems, their overflow frequencies would also exceed the standard without the central control system.

Each sub-system has its own particular rules and problems. The control scenario incorporates several control parameters that can be used for improving the operation of the sewage transport system. Optimization of these parameters appeared to be a continuous process as the drainage conditions vary in time. A proper use of the control system therefore requires experienced operators, although the system is fully automatic. The evaluation study has learned that a proper modification of the control parameters at the right time and to the right value is not as easy as it might look at a first glance.

### *9.3.2 Model study*

The information derived from the recorded data was not sufficient to quantify the effects of the control system, not only because the available information was too limited, but also because a reference was lacking. Therefore, a rainfall-runoff model has been developed (using spreadsheet software) which describes the drainage system of Westfriesland, including the real time control system. Like LOCUS, the model is based on a lumped storage approach and it allows the simulation of the system with and without the central control system. In the case that the pumps are locally controlled it may occur that all pumping stations operate at full capacity at the same time, meaning that the hydraulic capacity of the treatment plant has to be equal to the sum of the installed pumping capacities, i.e. 5800 $m^3$/hr.

At first, the characteristics of the sewer systems were determined on the basis of the sewer plans as formulated by the municipalities. Some data on available storage appeared to be incorrect. This could be concluded on the basis of the recorded 'high water situations'. (A side benefit of the control system is that it supplies useful information to the water board on the actual state of the sewer system). Although the available measurement data were too limited to enable a comprehensive calibration of the model, some information could be derived to make some verifications on the used model parameters. The applied parameters are given in Table 9.1.

The rain input to the model was derived from the Lelystad series (Section 5.3) of the period 1968-1980. From these series, 128 storms with a minimum rain depth of 5 mm, incorporating a maximum dry weather period of 8 hr, have been selected for the simulations. The rainfall data have been transformed into inflow data, by applying a constant initial loss equal to 1 mm and a linear reservoir, using a reservoir (time) constant of 30 min.

In Chapter 8, it was concluded that concerning the probability of overflows the effects of rainfall distribution are not significant when simulating long time series. Regarding minimizing overflows, the main contributing factor to the potential of real time operation is to be found in the differences between the available capacities of the sub-systems. When reducing the maximum flow to the treatment plant, an accurate estimate of the areal rainfall may become more important. Unfortunately, in Westfriesland only daily records of 3 rainfall stations are available. Continuous rainfall data of a network of rain gauges that could be used for the simulations are lacking.

In the evaluation study, a simple rainfall distribution model has been applied, using a (small) linear decrease of the rainfall depth related to the distance of the measuring point. This point has been chosen at the most critical place in the system, i.e in Hoorn. The rainfall depth in Enkhuizen and Medemblik was reduced by about 10%. The spatial distribution was assumed to be small in order not to overestimate the effects of rainfall distribution. Furthermore it can be shown that most events allow for a reduction of the flow to the treatment plant even if the rainfall is assumed to be homogeneous. By a better attuning of the systems capacities and by using the temporal variability of the system loading this can be realized .

Based on a comparison between the simulation results of the uncontrolled and controlled system, it could be concluded that the maximum flow to the Wervershoof treatment plant could be reduced to about 70% of the sum of the installed pumping capacities (= 4100 $m^3/h$), without increasing the CSO frequency. In Fig. 9.2, the difference between the thick reference line (= local control) and the dotted line, representing the results with the control scenario and a treatment plant capacity (TP) of 4100 $m^3/h$, is very small (especially in the range 0%-85%). Due to the reduced 'over-capacity', it was found that the overflow volumes will increase during severe storms, with a return period of approximately 0.3-1 year.

Reducing the maximum discharge to 3600 $m^3/h$ ( = the treatment plant capacity at the time of the evaluation study), has no serious effect on the CSO frequency (being the main constraint of the water quality manager when the control system was implemented), but CSO volumes appear to increase significantly. Therefore, in 1989 it was decided to expand the Wervershoof treatment plant. For practical reasons, the plant has not been expanded to a capacity of 4100 $m^3/h$ (as was indicated by the evaluation study), but to 4400 $m^3/h$, which is about 75% of the capacity that would be required according to the traditional approach.

## 9.4 Comparison with LOCUS

In Section 4.1 some general drawbacks of a control scenario are mentioned. It was concluded that a scenario may have advantages in practical control applications, but (theoretically) it does not guarantee optimum systems performance for all possible loadings. Therefore, in addition to the evaluation study, the system has been simulated with the LOCUS model. It should be noted that this case study is purely theoretical. The aim is only to determine the extent to which the control scenario approximates the optimum strategy.

It should be noted that the results of LOCUS are positively influenced by the fact that the model assumes that full pumping capacity is available for each possible filling degree (i.e. from the start of the rain event), whereas the control scenario activates the pumps according to their local set points. In practice, the 'rain weather pumps' are usually not activated before a certain water level in the system has been reached. Furthermore, the pumping stations in Westfriesland can only be operated at 3 (or less) different capacity levels, whereas LOCUS assumes that the pumps can be operated continuously within the full range (0-100%) of their capacity. The results should therefore be interpreted with care.

The derived distribution functions of the calculated overflows for the different ways of operation are presented in Fig. 9.2. Note that when applying a treatment plant (TP) capacity of 3600 $m^3/h$, the results of LOCUS approach the results of the control scenario for the most severe storms (number of events > 85%). Apparently, during these storms the operation strategy is of no importance.

Due to the assumptions made in the simulations with LOCUS, Fig. 9.2 may only be interpreted as an indication of the potential improvement of the results of the control scenario, which is mainly to be found in the range of 0-70% of the rainfall events (which lead to an overflow of the system). The difference between the results of the scenario and LOCUS is, however, not only due to the reasons that are mentioned above.

An important aspect that may influence the results of the scenario in a negative way is that the control scenario is based on the concept that central control is only required from the moment that an overloading of the treatment plant can be expected. At this moment the 'main program' of the central computer is activated. Before that time, the control scenario does not interfere in the local control of the pumping stations. As explained above, during these periods the control system checks only whether the treatment plant capacity is not exceeded, using its 'shortened program'. As a result the control scenario may react too late. At the moment the 'main program' is activated, it may occur that some parts of the system are filled to an extent that an overflow cannot be prevented, while elsewhere in the system some storage capacity is still available. Because the pumping capacity of the drainage system is small as compared to the storage capacity of the system, the ability of the system to correct uneven use of storage within the system is limited. Therefore, optimum use of storage can only be achieved when systems control is applied from the beginning of the storm. (This conclusion holds good for most Dutch urban drainage systems.)

For the sewerage system of Westfriesland it can be concluded that the maximum discharge to the Wervershoof treatment plant can be reduced. At the same time, a reduction of CSO may be feasible. The conventional approach setting the required hydraulic capacity of the plant equal to the sum of maximum pumping capacities of the connected sewer systems appears to be an inefficient solution.

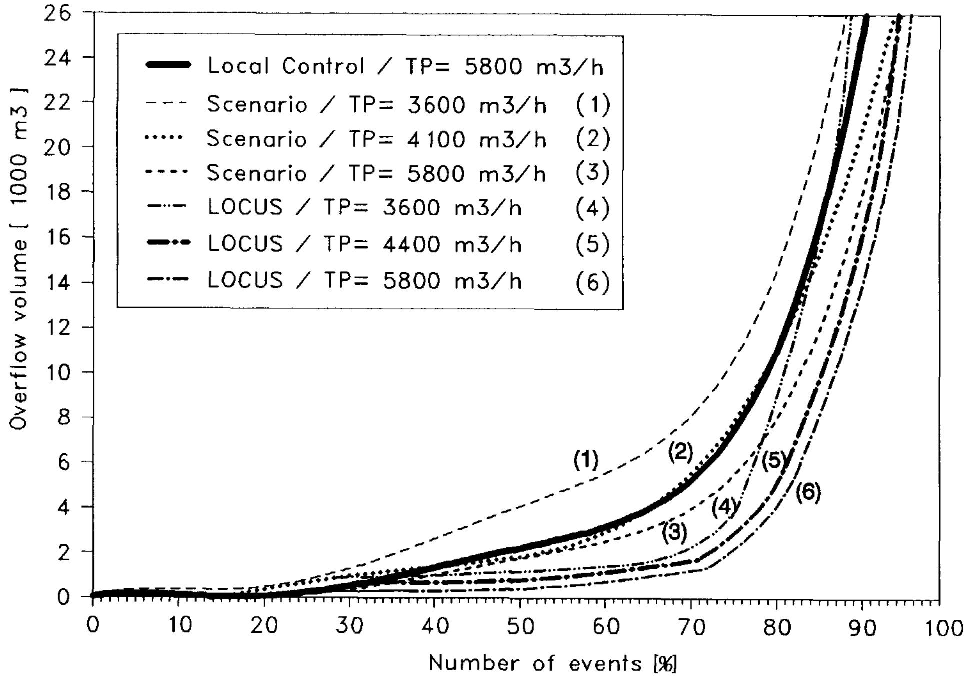


*Figure 9.2 Distribution functions of overflow volumes for different operation strategies (period 1968-1980)*

## 9.5 Concluding remarks

In (Nelen, 1988a) it was demonstrated that a reduction of the hydraulic capacity of the treatment plant Wervershoof is feasible. Due to rainfall distribution and an improved use of storage this reduction can be achieved without an increase of the overflow frequency of the system. This has been shown by an analysis of recorded pump data of a period over three years and by a model study of the system. Due to the reduced discharge capacity, the overflow volumes may augment during heavy storms.

Although the Wervershoof control system functions satisfactory (concerning the constraint of the allowable overflow frequency), an important lesson can be learned from a comparison between the control scenario and the decision model of LOCUS. Presently, the central control system is activated when an overloading of the treatment plant can be expected. This means that the water levels at several pumping stations have reached a value, by which they should be operated at full capacity according to the local set point. At this moment, it might be too late to achieve full use of storage of all sub-systems. To accomplish this, (central) systems control is required from the start of the rain event as the controllability of the system is limited. Therefore it may be suggested that to upgrade the performance of the present control system, the local set points of the 'rain weather pumps' should be set at the lowest possible level to activate the 'main' control program as soon as possible.

The control parameters that are used in the control scenario to determine the 'weight' of the input variable in determining the priority list of the pumping stations, should be modified with changing conditions of the drainage system. To determine the proper values of the control parameters requires experienced operators who have sufficient knowledge of the decision process. Due to the complexity of the problem, this appears to be rather difficult in practice.

As opposed to a rule based scenario, mathematical optimization may have the advantages of being more flexible and easier in use. In formulating the operational objectives, LOCUS requires only one set of unit costs, which remains valid for all operational conditions, even if the system would change. For example, if the system capacity is expanded or impervious area is increased, there is no need to modify the objective function of the optimization model, whereas the control parameters of the scenario may need modification. Furthermore it is the author's belief that giving a 'weight' (a unit cost) to an overflow at a certain place may be easier (and more understandable) for an operator than giving proper values to a number of control parameters, which are used to calculate the priority of a pump.

The disadvantage of an optimization model is that hardware constraints are difficult to incorporate in the optimization routine, meaning that the derived optimal strategy needs to be 'translated' to an executable strategy that approximates the optimum strategy. Concerning the latter a rule based scenario may be preferred. Therefore in practice it may be worth investigating the possibility of combining the strong points of both methods.

# 10 Summary, discussion and conclusions

## 10.1 Control of urban drainage systems

This study deals with the control of (Dutch) urban drainage systems. The conventional solution to urban drainage problems employs a system with a certain storage, transport and treatment capacity. The present standards on the basis of which these capacities are determined are outlined in Chapter 2. The limited efficiency of this solution is due to the lack in flexibility in operating the urban drainage system under dynamic loading.

Flow regulators in Dutch urban drainage systems (e.g., pumps, valves) are in most cases locally controlled, meaning that they maintain a pre-set flow related to the water level at the regulator site. Local control leads by definition to an uneven use of the available systems capacities because

- the input of the system (surface runoff, DWF) is distributed in time and space;
- the available systems capacities are as a rule not homogeneously distributed over the system (some sub-systems have more capacity available than other sub-systems);
- the effects of the system output on the environment are of different temporal and spatial scale.

Therefore, for an effective use of the systems capacities the set points of the flow regulators should be modified in real time, i.e. on the basis of currently measured process data throughout the system. The concept of real time control is to optimize the systems performance by taking account of the above mentioned aspects. The general aim is to minimize basement flooding and combined sewer overflows (CSO) to receiving waters, while maintaining optimum flow rates to the treatment plant depending on its current operational state. If overflows cannot be prevented, they should be directed as much as possible to the less sensitive receiving water.

Since every urban storm drainage system has its own typical features and the potential to improve the systems performance strongly depends on local conditions, no general rules can be formulated to quantify the potential of real time control. For this purpose a control simulation model is needed, on the basis of which the performance of the controlled system can be assessed in a consistent way for each particular system. Such a model has been developed in this project. An important requirement in developing the model was that it allows for time series calculations to gain insight into the statistical properties of the output variables. The simulation results of a single event have no value as not only the system, but also the rain event that is simulated are unique

A key problem in controlling a system is the formulation of the operation strategy, which describes the desired systems state as a function of time and place, or the time sequence of the set points of all flow regulators in the system (Chapter 3). In a general sense, to determine an optimal strategy means that the operation 'costs' have to be minimized, given a certain perturbation of the system, subject to the constraint that the strategy has to be feasible.

Conversely this implies that, to be able to solve the operational problem, information is required on the operational objectives (i.e. the 'cost'), the current systems state (i.e. the perturbation) and the physics of the system (i.e. the constraints of the problem). The operation 'costs' are determined by formulating a desired systems behaviour, which describes the operational objectives and how deviations from these objectives are assessed. Therefore it is necessary to define first the system bounds and what is understood by a perturbation of the system. The desired systems behaviour can be formulated in terms of performance criteria, which indicate the allowable limits of the systems state variables, or an objective function, which expresses how a deviation of a state variable from its desired value is evaluated. Only the latter provides a definition of the optimum solution.

There are several ways to solve the operational optimization problem, (Chapter 4). The applicable methods known from Operations Research may be divided into three broad categories, namely heuristic methods (i.e. based on experience), rule based scenarios (decision trees), and mathematical optimization. The latter implies that the objective function, composed of 'unit cost functions' of the systems variables, is minimized subject to a set of constraints, which describe the system. This approach has been used in developing the simulation model as it provides most flexibility and consistency in decision making as compared to the other two methods.

In formulating an objective function, the basic principle is to apply unit cost functions of the system state variables (storage, discharge, overflow, etc.) in which the unit costs are depending on the current value of the particular variable. It can be demonstrated that applying constant unit costs may lead to unsatisfactory results (Section 4.3). For example, to control the use of storage in the various sub-systems it is necessary to increase the unit costs of storage with increasing filling degree. The unit costs of overflows are determined on the basis of the function and sensitivity of the receiving water. As overflows are to be prevented as much as possible their unit costs should obviously be given a greater value than the costs of storage and transport. On the basis of such considerations it is possible to formulate an objective function, which is valid for all operational conditions, and which is independent of the system and its loading that is simulated, (Section 4.5).

The resulting optimization problem is a non-linear programming problem, which is summarized in Fig. 4.5. To solve the problem we can replace the non-linear problem by a succession of linear programming problems. This means that at each time step of the simulated inflow hydrograph the optimization problem is re-formulated using the results of the preceding time step. Main advantage of this approach is that it allows the use of a powerful network flow algorithm (Chapter 5). Besides it provides a possibility of using variable bounds of the systems state variables, which may be used to improve the flow routing in the model. For example, the (maximum) flow along an arc (branch) can be related to the current state at the upstream and downstream node (Section 4.6).

The optimization model has been incorporated in a newly developed modelling package, called LOCUS (Chapter 5), which is an acronym of 'Local versus Optimal Control of Urban drainage Systems'. The name denotes that besides optimal controlled systems, local controlled systems can be simulated as well (i.e. the present way of operation of most urban drainage systems). The latter has been included in LOCUS to serve as a reference. As the reference model and the optimization model are based on an identical system description, the difference between the results of both models is due only to the way the system is operated and hence the effects of optimal control can be quantified by comparing the results. LOCUS consists of several sub-programs to create the input-files required to run the models and to facilitate post processing of the model results. These sub-programs can be operated from a menu.

In the second part of the report (Chapter 6-9), four case studies conducted with the LOCUS package are presented. The main results are discussed below.

## 10.2 Case studies

### *10.2.1 Performance of the model*

The optimization model requires an inflow prediction for a certain time horizon, which is called the control horizon. The smallest horizon for which the problem can be solved is one time step, meaning that the decision on the set points is based on the current systems state ($t = 0$) and the predicted inflow in the next time step ($t = 1$). Obviously, the control horizon and the accuracy of the inflow forecast will influence the control decisions.

In general, the extent to which one may benefit from predicting disturbances depends the response time of the system and its ability to correct a perturbation (or the time required to return to its desired state). Increasing the control horizon may improve the decisions on the operation strategy as it enables the model to anticipate on spatial and temporal variations of the inflow. However, it can be demonstrated that when the objective is to minimize the total overflow volume only, the length of the control horizon has no significant influence. In this case the objective is to keep the use of storage at the various sub-systems at a relatively equal level (related to the storage capacity of the sub-system) as this guarantees a minimum risk of CSO. The ability of the system to correct a perturbation (i.e. a difference in relative use of storage) is of the same magnitude as the variation of the system loading. This means that the system is able to return to its desired state within one or only a few time steps and hence a proper decision on how to operate the system may be made on the basis of the current systems state only (without using rainfall forecasts).

The control horizon becomes more important when we have to decide on where to overflow. In this case the desired systems behaviour is obviously not to keep the use of storage throughout the system as much as possible at a relatively equal level. Since the time horizon for which inflow can be specified is usually limited the reaction time might be too short. Therefore to reduce the probability of an overflow to a sensitive water we should not only increase the control horizon but also restrict the use of storage at these places, even if no overflows can be foreseen within the control horizon. The desired difference in the use of storage at the various sites within the system is expressed in the objective function by the unit cost function of storage (Section 4.5.1).

Similar conclusions can be drawn concerning the required reliability of the predicted inflow. The effects of forecast errors on the total overflow volume are negligible. Inaccurate inflow predictions will, however, affect the decisions on the use of storage and hence the location of possible overflows. If the inflow is overestimated, then a positive effect may be expected at those places (nodes) in the system that have less capacity as compared with other places. If the actual inflow is underestimated then it is the other way round. Simply stated, the optimization model becomes (automatically) more 'cautious' at the sensitive sections of the system, when it expects more inflow, thereby increasing the risk of overflows at the other sections.

The effects of the control horizon and forecast errors have been illustrated for a small fictitious system (Chapter 6) and the sewerage system of Rotterdam (Chapter 7).

Finally it is noted that the decisions on the operation strategy could be improved concerning pollution loads, if an accurate pollution model were available. Since the actual pollution concentrations are very difficult to predict or to measure, this conclusion is not (yet) of practical value. The question is further whether minimizing (total) pollution loads is in fact the main issue. The present knowledge on the ecological impact of CSO discharges may be sufficient to make a classification of the receiving waters, but information on temporal and spatial effects of overflows on the receiving water quality is still limited. Therefore it is usually more important to prevent overflows and, if necessary, to direct overflows to the less sensitive waters, rather than minimizing the total pollution emissions by overflows.

### *10.2.2 Recommendations to engineering applications*

The case studies conducted with LOCUS allow for some general conclusions. To begin with, it has been demonstrated that real time control may lead to a significant reduction of CSO. Besides an increased protection of the receiving water, this could also mean that construction cost may be saved (Chapter 6). Therefore it may be suggested that before extra capacity is added to the system it should be investigated to what extent real time control may contribute to abate urban drainage problems. In terms of money, the saving in construction cost will often be much greater than the cost of installing a control system !

This conclusion is valid not only concerning the required storage and transport capacity, but also concerning the capacity of the treatment plant. Due to the temporal and spatial distribution of rainfall and a better use of storage in the sewer system it may be possible to reduce the peak flow to the treatment plant, without increasing CSO. This possibility is certainly not restricted to regional treatment plants, that serve a number of villages, like in the presented case of Westfriesland (Chapter 9). In smaller systems, it is also worth investigating the effects of real time control, to strike the best balance between storage, discharge and treatment capacity in the system that is required to achieve optimum systems performance. Although a potential saving will differ for every system, it can safely be stated that the conventional approach setting the required hydraulic capacity of the treatment plant equal to the sum of maximum pumping capacities of the connected sewer systems is an inefficient solution.

The possibility to improve the systems performance by means of real time control is in most cases restricted to the lowest 80-90% of the rain events that lead to an overflow. During the remaining 10-20% of the rain events, i.e. the severe storms with a return period of about 0.3-1 year and longer, full system capacity will be used, no matter what operation strategy is being applied. As a consequence, real time control will generally not contribute in a reduction of basement flooding, although a dynamic operation may be useful in controlling the place of floods. Besides, it should be considered that it is not accepted that a possible failure of the control system results in flooding of the catchment. The hydraulic capacity of the system should therefore be sufficient to discharge its design load, independently of the applied operation strategy.

The investigation of the Damhus district in Copenhagen (Chapter 8) is focused on the effects of spatially distributed rainfall. The simulations have shown that system variables may be largely over- and underestimated when analyzing the systems performance on the basis of data of one rain gauge. Therefore, if the objective is to predict the performance of a drainage system for a single rain event, (e.g., to determine the peak flow), the temporal and spatial distribution of rainfall is not to be neglected. In such a case the highest possible degree of spatial information on the rainfall may be desirable. However, it appears that from a statistical point of view, the effects of rainfall distribution are not significant concerning the probability of a CSO. This holds good for both an uncontrolled and a controlled system, which indicates that the main contributing factor to the potential of real time control is to be found in the system itself (i.e. the distribution of available capacities within the system) and the temporal variability of the system input.

## 10.3 Topics for further research

There is no end to the development of a model. Regarding LOCUS, the following modifications and extensions are suggested. In the first place it would be interesting to investigate whether the flow routing of the model can be improved, as at present the model fails in simulating the effects of hydrodynamic phenomena like backwater and surcharge. The concept of variable bounds of the flow as discussed in Section 4.6 has not been worked out in detail, but theoretically this concept allows the use of any equation of motion with which the flow between two nodes in the system can be calculated explicitly out of the values of the system state variables at the former time step.

To improve the decisions on the operation strategy, and to improve the applicability of the model in the design concept as discussed below, an improvement and extension of the water quality modules of the model (Section 5.3.3) is desirable. The pollution transport module could be modified to describe the process of sedimentation and re-suspension of settled material. A treatment plant module as proposed by (Harremoës, 1989) and discussed in Section 2.5, has been developed. The applicability of this module should be tested and if necessary, the model should be modified. After the modelling of the water quality processes in the system have been upgraded, it would be worth extending the model by adding a receiving water module, that can be used to predict the effects of urban discharges to the receiving water quality.

Until now, LOCUS has been used only to simulate an urban drainage system during storm conditions. However, benefits of an improved operation may also be expected during dry weather periods, such as a reduction of energy cost and improved treatment plant performance. This operational optimization problem can be formulated as a mathematical problem too. Recently an investigation has started into the topic.

In practice, LOCUS may be used on-line to derive the optimal set points of the flow regulators, or the 'optimal path' of the system. It can be expected that the model will provide a suitable operation strategy in on-line applications, given the model is provided with a proper estimate of the current systems state. Besides, one should be aware that the constraints of the problem may vary in time due to possible temporal modifications of the system (e.g, during maintenance work).
Therefore it may be suggested that the optimization model should be used in practice in combination with an expert system (Section 4.1.1). The purpose of this expert system is to diagnose the current systems state (using currently monitored process data), to formulate the optimization problem and to interpret the model results. Note that in formulating the optimization problem not only the constraints vary in time, but the objective function as well (e.g., the objectives will differ during rain weather situations and dry weather situations). The development of such a general 'decision model' is the topic of a research, that was started in November 1991 at the Delft University of Technology, in continuation of this research. This decision model will be tested and implemented in Rotterdam.

The optimization model does not generate the required adjustments of the flow regulators to achieve minimum deviation from these set points. This is a task of the controller. It is noted that a desired systems performance can be achieved in different ways, meaning that decisions can be made at different levels (Section 3.1.2). For example, by applying advanced (multi-variable) controllers the system could be made self-regulating. This means that the system returns to its desired state after a disturbance without outside manipulations, or without changing set points (Schuurmans, 1991). As the desired state of an urban drainage system will vary in time, depending on the current drainage conditions, a decision maker (operator or model) will remain indispensable to achieve optimum performance, but using advanced controllers will generally mean that the task of this decision maker can be reduced. The other option is to use simpler controllers (e.g., on/off, PID) with variable set points that are determined in real time by a decision model. As decisions are made at a 'higher' level, this type of operation is usually more flexible.

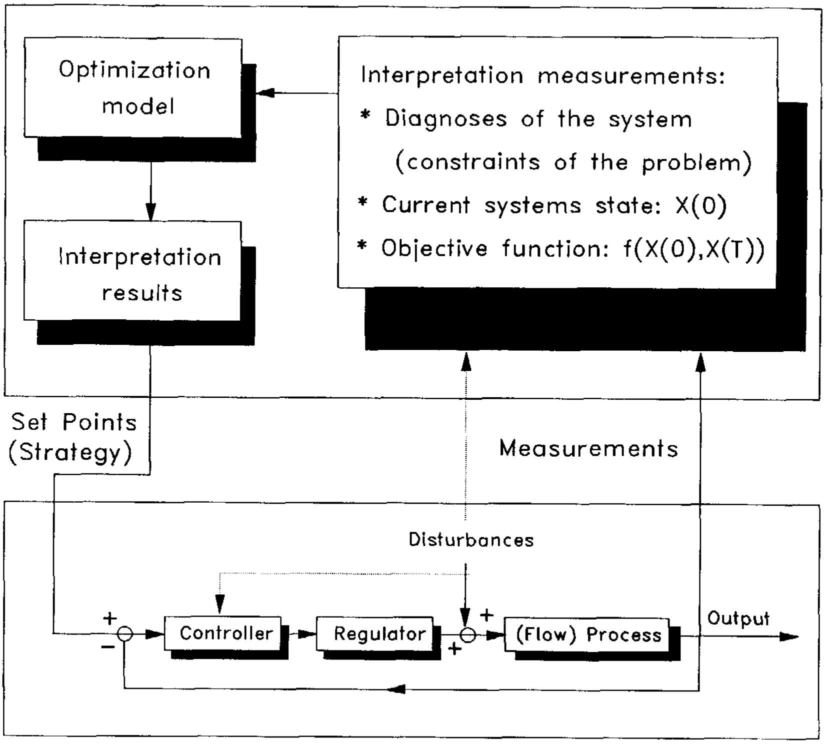


*Figure 10.1 Scheme of a decision model*

## 10.4 The design problem

The model has been developed primarily to be used for systems analyses. It is important to address the operational problem already in the design or rehabilitation phase of the system, as this particular phase provides most flexibility in changing the sizes of the system components and in choosing the appropriate type and capacity of the flow regulators.

The operational optimization problem is obviously not the same as the problem of determining the optimum design, but it may be clear that both problems are closely related. The cases studies presented in this report have clearly demonstrated that the effects of an improved operation are not to be neglected in choosing the appropriate system dimensions. These effects can be assessed with the LOCUS model.

In Fig. 10.2 at the end of this chapter, a possible scheme of a design process is presented showing some components that are usually being neglected in the present design methods. Obviously many modifications could be added without completing the figure. For example, the potential of source controls in reducing the volume and/or the flux of surface runoff that is entering the storm water system is certainly not to be neglected. The figure has been included mainly to illustrate the role of real time control (and that of the model developed), in the design process.

Among the various aspects of urban storm drainage technology which do require further scrutiny, the water quality processes are least understood. Further research is needed to develop reliable models which are able to describe the pollution transport in the system and the processes at the treatment plant under variable loading. Furthermore, comprehensive ecological research related to the impact of urban discharges on the receiving water quality is required to specify water quality standards to each receiving water body.

The limited information regarding the water quality variables should, however, not mean that the present water quality models are useless. Cornell (1972) stressed a significant point in any reliability analysis: '*It is important to engineering applications that we avoid the tendency to model only those probabilistic aspects that we think to know how to analyze. It is far better to have an approximate model of the whole problem than an exact model of only a portion of it*'.

In designing an urban drainage system, a distinction can be made between hydraulic criteria and water quality criteria. The first are generally related to the risk of basement flooding. The hydraulic design aims at determining the systems capacities that are required to handle severe storms with a certain return period (say in the order of 10 years) without surcharge throughout the entire system. The available hydrodynamic flow models are useful tools which facilitate a detailed analysis of the systems behaviour.

The search for an efficient design that meets the required limitation of pollution loads aims at striking a proper balance between the storage, discharge and treatment capacity. Concerning this problem the use of time series calculations is the most suited approach as it allows for a probabilistic interpretation of the project. When using a design storm, such interpretation is only being made concerning the rainfall, whereas calculations of a series of rain events take account of the different processes that might influence the statistical properties of the systems variables. If possible, the spatial distribution of the rainfall should be taken into account.

These time series calculations can be performed by LOCUS. The approach as outlined by Fig. 10.2 is an iterative procedure, meaning that the system is modified until an acceptable (or least cost) solution is found whereby the system outputs are kept within the stated limits. An important feature of LOCUS is that the objective function remains valid when the system parameters are altered. This means that several runs can be made for different system capacities using the same set of basic unit costs.

## 10.5 Administrative aspects

It can be concluded that real time control is a realistic and economically feasible alternative to the 'static' operation of most urban drainage systems today. However, to make full use of this innovative technique requires a number of administrative problems to be solved. The present regulations and funding arrangements appear to be one of the main obstacles to a successful implementation of real time operation of urban drainage systems. To begin with the use of fixed design standards that prescribe 'static' solutions, such as an amount of storage that has to be available, or a treatment plant capacity that has to be equal to the peak discharge of the sewer system, should be discouraged.

Besides the existence of separate administrations within one system may cause problems. The municipality might like to reduce the necessary amount of in-line storage, while the water quality manager might like to reduce the peak flows to the treatment plant. Within the concept of integral water management, the ultimate aim is the same, namely to improve the systems performance and to minimize operation and construction costs of the urban drainage system. This means that the sewer manager should take account of the environmental impacts of sewer discharges, which 'officially' is not his responsibility. At the same time the water quality manager should be aware of the problems of the municipality, which are obviously not only directed to water resources management and which are to be solved with limited financial resources. As long as the different administrators can not agree on how to share the cost and benefits of a real time control project, inefficient solutions to urban drainage problems will be unavoidable. However, it is most likely that the change in mind towards environmental protection, the necessity of economizing the administrative budgets and the (general) tendency of integral water management, will speed up the needed developments concerning the existing regulations and funding arrangements.

## 10.6 Concluding remarks

Referring to the objectives of this study, it can be stated that a model has been developed which can be used to assess the performance of an urban drainage system that is controlled in real time. The case studies conducted with the LOCUS model have demonstrated the applicability of the model and especially the potential of real time control. Obviously, real time control does not solve all urban drainage problems, but it has been shown that the performance of urban drainage systems can considerably be improved, leading to better environmental protection and possible savings of construction and operation costs.

In practice, only a few applications of real time control of urban drainage systems can be found. To the authors knowledge none of them uses mathematical optimization to derive the operation strategy. The above mentioned application of LOCUS to the Rotterdam system will probably be the first one.

Last years, many urban drainage systems were equipped with an automated monitoring and (remote) control system to facilitate daily operation. As yet, most water managers are interested in data acquisition and monitoring systems for planning and maintenance purposes. Remote (manual) control is usually restricted to extraordinairy situations, but practice has shown that an increased insight into the (actual) dynamic behaviour of the system will stimulate active control of the system under all circumstances. The author hopes that this study may contribute in developing an efficient control concept for these systems.

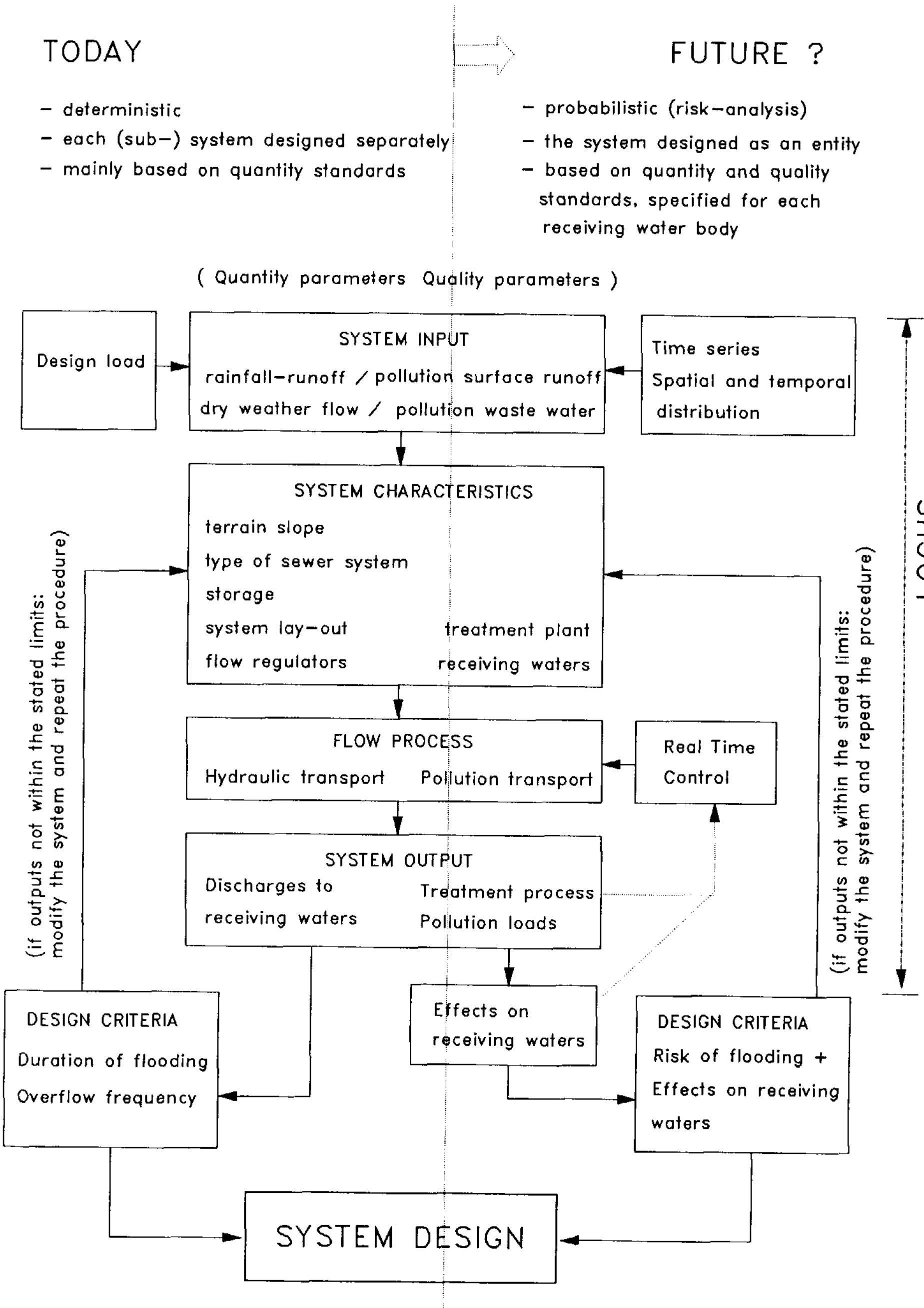


*Figure 10.2 Towards a new design approach*

# 11 Samenvatting, discussie en conclusies

## 11.1 Sturing van stedelijke afvoer systemen

Deze studie handelt over sturing van (Nederlandse) stedelijke afvoersystemen. De conventionele oplossing voor de afvoer van neerslag in stedelijk gebied gebruikt een systeem met een zekere bergings-, transport- en zuiveringscapaciteit. De huidige methoden gebaseerd waarop deze capaciteiten worden bepaald zijn samengevat in Hoofdstuk 2. De beperkte efficiëntie van deze oplossing is het gevolg van het gebrek aan flexibiliteit in de besturing van het afvoersysteem onder dynamische belasting.

Regelkunstwerken in afvoersystemen (pompen, kleppen) worden in de meeste gevallen lokaal gestuurd, gebaseerd op vooraf vastgestelde set points (gewenste waarden) die zijn gerelateerd aan het waterpeil bij het kunstwerk. Dit leidt per definitie tot een ongelijkmatig gebruik van de aanwezige systeemcapaciteiten omdat

- de 'input' van het systeem (neerslag-inloop, DWA) spreiding vertoont in tijd en ruimte;
- de beschikbare systeemcapaciteiten nooit homogeen verdeeld zijn over het systeem (op sommige plaatsen is relatief meer capaciteit beschikbaar dan elders in het systeem);
- de effecten van de systeem 'output' op de omgeving veranderlijk zijn in tijd en plaats.

Voor een effectief gebruik van de systeemcapaciteiten zullen de set points van de regelkunstwerken moeten worden aangepast in 'real time', gebaseerd op actuele meetwaarden van procesvariabelen van het gehele systeem. Het doel van 'real time' sturing is het systeem gedrag te optimaliseren door rekening te houden met bovengenoemde aspecten. Door sturing wordt beoogd 'water-op-straat' en overstortingen te minimaliseren, en tegelijkertijd de afvoer naar de rwzi af te stemmen op de capaciteit en de actuele condities op de rwzi. Verder zullen overstortingen, indien onvermijdelijk, zoveel mogelijk moeten worden gedirigeerd naar de minst kwetsbare wateren.

Omdat elk watersysteem specifieke kenmerken heeft en de mogelijkheden om het systeemgedrag te verbeteren in hoge mate afhankelijk zijn van lokale omstandigheden, is het onmogelijk om algemeen geldende regels te formuleren waarmee de effecten van 'real time' sturing zouden zijn te kwantificeren. Hiervoor is een sturings-simulatie model nodig, op basis waarvan deze effecten kunnen worden vastgesteld op een consistente wijze voor ieder specifiek systeem. Een dergelijk model is ontwikkeld in dit onderzoek. Bij de ontwikkeling van het model was een belangrijke eis dat het model geschikt moest zijn voor het doorrekenen van tijdreeksen om aldoende inzicht te krijgen in de statistische eigenschappen van de 'output' variabelen.

Een kernprobleem bij sturing is de formulering van de operationele strategie, die het gewenste systeemgedrag beschrijft als een functie van tijd en plaats, ofwel de tijdreeks van set points van alle regelkunstwerken in het systeem (Hoofdstuk 3). De bepaling van de optimale strategie betekent in algemene zin dat de operationele 'kosten' moeten worden geminimaliseerd, gegeven een bepaalde verstoring van het systeem, onder de randvoorwaarde dat de strategie uitvoerbaar moet zijn. Er is derhalve informatie nodig over de operationele doelen (de 'kosten'), de actuele systeemtoestand (de verstoring) en de fysica van het systeem (de randvoorwaarden van het probleem).

De 'kosten' worden bepaald door het formuleren van een gewenst systeemgedrag, die de operationele doelen beschrijft en de wijze waarop een afwijking van deze doelen wordt beoordeeld. Het is daarom belangrijk om eerst de systeemgrenzen te definiëren en hetgeen wordt verstaan onder een systeemverstoring. Het gewenste systeemgedrag kan worden omschreven met behulp van criteria, die de toelaatbare grenzen van de toestandsvariabelen aangeven, of door een doelfunctie, die beschrijft hoe een afwijking van een toestandsvariabele van zijn gewenste waarde wordt geëvalueerd. Alleen in het laatste geval is de optimale oplossing gedefinieerd.

Het operationele optimalisatie probleem kan op verschillende manieren worden opgelost (Hoofdstuk 4). De toepasbare methoden, bekend uit de Operations Research, kunnen worden gegroepeerd in 3 categorieën, namelijk heuristische methoden (i.e. gebaseerd op ervaring), scenario's gebaseerd op vooraf vastgestelde regels (beslissingsbomen), en mathematische optimalisatie methoden. De laatstgenoemde impliceert dat de doelfunctie, bestaande uit 'eenheidskostenfuncties' van de toestandsvariabelen, wordt geminimaliseerd onderhevig aan een set randvoorwaarden, die het systeem beschrijven. Deze methode is gebruikt bij de ontwikkeling van het model omdat dit de meest flexibele en consistente methodiek is, vergeleken met de twee andere genoemde methoden.

Bij het formuleren van een geschikte doelfunctie moeten de 'eenheidskosten' van de systeemvariabelen worden gerelateerd aan de actuele waarde van de betreffende variabele. In deze studie is aangetoond dat het gebruik van constante eenheidskosten tot onbevredigende resultaten leidt (Par. 4.3). Optimalisatie van het gebruik van berging vereist dat de eenheidskosten worden gerelateerd aan de actuele vullingsgraad van het systeem. De eenheidskosten van overstortingen worden bepaald afhankelijk van de locatie van de overstort en de functie en kwetsbaarheid van het ontvangende water. Daar overstortingen zoveel mogelijk dienen te worden voorkomen moeten deze 'kosten' een grotere waarde gegeven worden dan de 'eenheidskosten' van berging en transport. Gebaseerd op dergelijke beschouwingen is het mogelijk een doelfunctie te formuleren, die geldig is voor alle omstandigheden en die onafhankelijk is van het systeem en de belasting die worden gesimuleerd (Par. 4.5).

Het resulterende optimalisatie probleem is een niet-lineair programmerings probleem, dat is samengevat in Fig. 4.5. Een effectieve manier om dit probleem op te lossen is het te benaderen door een successie van lineaire optimalisatie problemen. Dit betekent dat voor elke tijdstap van de simulatie het optimalisatie probleem opnieuw wordt geformuleerd, gebruik makend van de resultaten van de voorafgaande tijdstap. Een belangrijk voordeel van deze aanpak is dat de sub-problemen kunnen worden opgelost met behulp van krachtige netwerk algoritmen (Hoofdstuk 5). Daarnaast biedt deze methode de mogelijkheid om de grenzen van de toestandsvariabelen (de capaciteitsvoorwaarden) te variëren in de tijd, hetgeen gebruikt kan worden om het stromingsmodel te verbeteren (Par. 4.6).

Het optimalisatie model is geprogrammeerd in een nieuw ontwikkeld modellen pakket, genaamd LOCUS (Hoofdstuk 5), hetgeen een acroniem is voor 'Local versus Optimal Control of Urban drainage Systems'. Zoals de naam aangeeft kunnen met LOCUS, naast optimaal gestuurde systemen, tevens lokaal gestuurde systemen worden gesimuleerd (de huidige wijze van sturing van de meeste systemen). De laatste optie kan dienen als referentie model. Daar beide modellen zijn gebaseerd op een identieke systeembeschrijving zijn de verschillen tussen de uitkomsten alleen toe te schrijven aan de wijze waarop het systeem wordt gestuurd. Hierdoor de potentiële mogelijkheden van (geoptimaliseerde) sturing eenvoudig kunnen worden gekwantificeerd.

## 11.2 Case studies

In de Hoofdstukken 6-9 worden vier case-studies besproken. De belangrijkste resultaten worden hieronder samengevat.

### *11.2.1 Gedrag van het model*

Het optimalisatie model vereist een invoervoorspelling voor een zekere tijdshorizon. De minimale voorspelling is één tijdstap, hetgeen betekent dat de beslissing over de te volgen strategie is gebaseerd op de huidige systeemtoestand ($t = 0$) en de voorspelde invoer in de komende tijdstap ($t = 1$). Het spreekt voor zich dat de voorspellingshorizon en de nauwkeurigheid van de voorspelling de beslissingen zullen beïnvloeden.

In het algemeen is het nut van een langere voorspellingshorizon afhankelijk van de response tijd en de regelmacht van het systeem (ofwel de benodigde tijd alvorens het systeem haar gewenste toestand bereikt). Een langere horizon kan tot betere beslissingen omtrent de operationele strategie leiden, omdat het model in staat is te anticiperen op tijdelijke en ruimtelijke variaties van de systeem input. Echter, er is aangetoond dat wanneer de operationele doelstelling is beperkt tot het minimaliseren van de totale overstort hoeveelheid, de voorspellingshorizon geen significante rol speelt. In dit geval is het doel feitelijk om het gebruik van berging in de te onderscheiden sub-systemen gedurende de bui op een relatief gelijk niveau te houden (gerelateerd aan de beschikbare berging). Dit garandeert immers een optimaal gebruik van berging en een minimum risico voor overstortingen. De regelmacht van het systeem om een verstoring (ofwel een verschil in relatief gebruik van berging) te corrigeren is meestal van dezelfde orde als de variaties die optreden in de systeembelasting. Dit betekent dat het systeem in principe binnen één of enkele tijdstappen teruggebracht kan worden naar de gewenste toestand. Een goede strategie kan in dit geval derhalve worden afgeleid op basis van enkel de actuele systeemtoestand (zonder neerslagvoorspelling).

De voorspellingshorizon wordt belangrijker wanneer een beslissing moet worden genomen over de locatie van een eventuele overstorting. De mogelijkheden voor een betrouwbare neerslagvoorspelling zijn echter beperkt. Wanneer onderscheid gemaakt wordt in de locatie van overstortingen is toepassing van een langere voorspellingshorizon (in de orde van 2 uur) veelal niet voldoende. Om overstortingen op kwetsbare wateren te minimaliseren zal tevens het gebruik van berging op deze plaatsen moeten worden beperkt, zelfs als geen overstortingen kunnen worden voorzien binnen de sturingshorizon. Het gewenste verschil in het gebruik van berging kan worden uitgedrukt in de doelfunctie (Par. 4.5.1).

Een vergelijkbare tendens wordt gevonden met betrekking tot de effecten van voorspellingsfouten. Een onnauwkeurige voorspelling heeft een verwaarloosbare invloed op de totale overstortingshoeveelheid. Een overschatting van de neerslag-inloop heeft echter een negatieve invloed op sub-systemen met relatief veel bergingscapaciteit en een positief effect op plaatsen met beperkte capaciteit. Met een onderschatting is het andersom. Simpel gesteld wordt het model (automatisch) 'voorzichtiger' op plaatsen in het systeem met geringe capaciteit wanneer de inloop verwacht. Dit verhoogt het risico van overstortingen op de overige plaatsen.

Tot slot wordt opgemerkt dat de beslissing t.a.v. de operationele strategie zou kunnen worden verbeterd met betrekking tot de vuiluitworp, indien een betrouwbaar vuiltransport model beschikbaar zou zijn. Omdat betrouwbare methoden voor voorspellingen en/of (on-line) metingen van vuilconcentraties nog niet beschikbaar zijn, is deze conclusie (nog) niet van praktische waarde. Daarbij moet de vraag gesteld worden of het minimaliseren van de (totale) vuiluitworp feitelijk het belangrijkste operationele doel is. De huidige kennis over de effecten van overstortingen op het aquatisch ecosysteem mag misschien toereikend zijn voor een classificatie van ontvangende wateren, maar de beschikbare informatie over deze effecten in tijd en ruimte is nog zeer beperkt. Voor het sturingsprobleem is het daarom belangrijker om overstortingen zoveel mogelijk te voorkomen en te dirigeren naar de minst kwetsbare watergangen, dan om de vuiluitworp te minimaliseren.

### *11.2.2 Aanbevelingen voor de praktijk*

In deze studie is aangetoond dat door 'real time' sturing een aanmerkelijke reductie van overstortingen is te bewerkstelligen. Naast een verbeterd waterkwaliteitsbeheer zou dit ook kunnen betekenen dat besparingen mogelijk zijn op de benodigde systeem-capaciteiten (Hoofdstuk 6). Aanbevolen wordt om, voordat besloten wordt tot systeem-uitbreidingen, de potentiële mogelijkheden van sturing te onderzoeken. De besparingen die hiermee kunnen worden bereikt zullen veelal veel groter zijn dan de kosten van een besturingssysteem !

Dit geldt niet alleen voor de benodigde bergings- en transportcapaciteit, maar ook voor de capaciteit van de rwzi. Door de variabiliteit van neerslag in tijd en ruimte en een verbeterd gebruik van de aanwezige berging in de aangesloten rioolstelsels is het veelal mogelijk de piekafvoer naar de rwzi te reduceren, zonder (of met slechts een geringe) toename van overstortingen. Deze mogelijkheid is zeker niet beperkt tot regionale afvalwatersystemen, zoals in de besproken studie van Westfriesland (Hoofdstuk 9). Ook voor kleinere systemen is het waard om de mogelijkheden van sturing te onderzoeken, om de meest efficiënte balans te vinden tussen de te installeren bergings-, afvoer- en zuiveringscapacietit. Ofschoon de voordelen voor ieder systeem anders zullen zijn kan gerust worden gesteld dat de conventionele benadering, waarbij de benodigde capaciteit van de rwzi gelijk wordt gesteld aan de som van de maximale afvoercapaciteiten van de aangesloten stelsels, een inefficiënte oplossing is.

De mogelijkheden tot een verbeterd gebruik van de systeemcapaciteiten zijn beperkt tot ongeveer de laagste 80-90% van de buien die tot een overstorting leiden. Gedurende de resterende 10-20% van de buien, ofwel de zware buien met een herhalingsperiode van ca. 0.3-1 jaar en langer, is zodanig dat het gehele systeem gevuld raakt, onafhankelijk van de operationele strategie. Real time sturing zal in het algemeen niet bijdragen tot een reductie van 'water-op-straat', echter het kan zinvol zijn om de lokatie van een overbelasting van het systeem te sturen. Verder moet worden bedacht dat een mogelijk falen van het besturingssysteem niet mag leiden tot ongewenste overstromingen. De hydraulische capaciteit van het systeem moet derhalve voldoende zijn om de ontwerpbelasting te verwerken, zelfs in de ongestuurde situatie.

De studie van het Damhus district in Kopenhagen heeft aangetoond dat grote verschillen in modeluitkomsten voorkomen wanneer het systeem wordt geanalyseerd voor een homogene neerslag en een bui met ruimtelijke spreiding. Voor een analyse van een individuele gebeurtenis (bijv. bij de bepaling van de piek afvoer, bij modelcalibratie) wordt dan ook aangeraden om de hoogst mogelijke graad van neerslag informatie te gebruiken. Echter, vanuit statistisch oogpunt zijn de effecten van neerslagspreiding, met betrekking tot overstortingen, van minder belang. Dit geldt voor zowel een lokaal als een optimaal gestuurd systeem. Er kan worden geconcludeerd dat de belangrijkste bijdragende factoren tot de potentie van sturing moeten worden gezocht in de tijdelijke variabiliteit van de neerslag en de verdeling van de aanwezige systeemcapaciteiten.

## 11.3 Onderwerpen voor verder onderzoek

Er is geen einde aan de ontwikkeling van een model. Voor LOCUS worden o.m. de volgende aanpassingen en uitbreidingen voorgesteld. Allereerst is het interessant te bekijken of het stromingsmodel kan worden verbeterd. Het model is momenteel niet in staat om opstuwing en andere hydrodynamische effecten te simuleren. Een mogelijke verbetering van de hydrodynamica is te bewerkstelligen door toepassing van variabele randvoorwaarden voor de stroming tussen twee knopen (Par. 4.6). Deze randvoorwaarden volgen dan uit de berekende waterstanden op de voorgaande tijdstap (een expliciete rekenmethode). Theoretisch stelt dit concept geen enkele beperking aan de toe te passen stromingsvergelijking.

Om de beslissingen omtrent de operationele strategie te verbeteren met betrekking tot de vuiluitworp, verdient het aanbeveling om de waterkwaliteitsmodules in LOCUS te verbeteren en uit te breiden. Het vuiltransport model, dat nu is gebaseerd op het concept van een ideaal gemengd vat, zou kunnen worden uitgebreid met een sedimentatie en re-suspensie term. Verder is een zuiveringsmodel gewenst, die het zuiveringsrendement beschrijft als functie van de belasting. Nadat deze modules zijn ontwikkeld en getest zou de mogelijkheid kunnen worden bekeken om het pakket uit te breiden met een ontvangend water model, waarmee de tijdsafhankelijke effecten van vuilemissies kunnen worden afgeschat.

Deze studie is met name gericht op het operationele probleem gedurende neerslag perioden. Echter, ook gedurende droogweerperioden kan sturing belangrijke voordelen opleveren, bijvoorbeeld met betrekking tot de energiekosten en de belasting (rendement) van de rwzi. Dit operationele probleem kan ook worden geformuleerd als een mathematisch optimalisatie probleem, dat kan worden opgelost met de technieken van Operations Research. Recent is een onderzoek naar dit probleem gestart. Het ontwikkelde model zal uiteindelijk in het LOCUS pakket worden ingebouwd.

In de praktijk zou LOCUS on-line gebruikt kunnen worden om de optimale set points van de regelkunstwerken, ofwel 'het optimale pad' voor het systeem, te bepalen. Er kan worden verwacht dat het model een geschikte strategie genereert, zolang het wordt voorzien van een juiste schatting van de huidige systeemtoestand (afgeleid uit actuele metingen). Daarnaast zal rekening gehouden moeten worden met het feit dat de fysische randvoorwaarden van het optimalisatie probleem kunnen variëren. Deze wijzigingen kunnen permanent (systeemuitbreidingen) of van tijdelijke aard zijn (bijv. onderhoudswerkzaamheden, een pomp die niet functioneert). Opgemerkt wordt dat het optimalisatie model niet de benodigde acties voor de regelkunstwerken genereert die nodig zijn om het 'optimal pad' ook zo goed mogelijk te volgen (het 'tracking' probleem). Dit is de taak van de regelaar (controller).

Voor toepassing in de praktijk wordt, gezien het bovenstaande, aanbevolen om het optimalisatiemodel te gebruiken in combinatie met een expert-systeem (Fig. 10.1). Taak van het expert-systeem is een diagnose te maken van de actuele toestand van het systeem (interpretatie van metingen), de invoerfiles voor het optimalisatie model te genereren, en de uitkomsten van het optimalisatie model te verwerken tot acties voor de regelkunstwerken. De ontwikkeling van zo'n beslissingsmodel is onderwerp van een studie, die najaar 1991 is aangevangen op de TU Delft. Dit beslissingsmodel zal worden getest en geïmplementeerd in Rotterdam.

## 11.4 Het ontwerp probleem

Het model is in de eerste plaats ontwikkeld ten behoeve van systeem analyses. Het is belangrijk om het operationele probleem reeds te beschouwen in de ontwerp of rehabilitatie fase van het systeem (en niet slechts als het systeem reeds is gebouwd), omdat deze fase de meeste flexibiliteit biedt in de keuze van systeem dimensies en het type regelkunstwerken. Het operationele probleem is niet hetzelfde als het probleem van het optimale ontwerp, dat is gericht op minimalisatie van constructie en operationele kosten, onderhevig aan een set ontwerp criteria. Echter, beide problemen kunnen niet los van elkaar worden gezien. De case studies hebben aangetoond dat de mogelijkheden van sturing niet mogen worden verwaarloosd in het proces van het systeemontwerp.

In Fig. 10.2 is schematisch een mogelijke ontwerpprocedure weergegeven. Het bevat een aantal componenten die hedentendage nog niet in het ontwerp worden betrokken. Het heeft niet de pretentie een compleet overzicht te geven. Zo zullen bijvoorbeeld de mogelijkheden van 'source controls' in de toekomst ook een belangrijke rol gaan spelen. De figuur is met name bedoeld om de rol van sturing (en dat van het ontwikkelde model) te illustreren.

Van de verscheidene aspecten van de stedelijke waterbeheersing die nader onderzoek behoeven is met name de kennis omtrent de waterkwaliteitsprocessen onvoldoende. Verder onderzoek is nodig voor de ontwikkeling van betrouwbare vuilemissie modellen die in staat zijn om de processen in het rioleringssysteem en op de rwzi (onder variabele belasting) voldoende nauwkeurig te beschrijven. Bovendien is verder onderzoek nodig naar de ecologie van het ontvangende water en de effecten hierop van afvoeren van afvalwatersystemen, met als doel om kwaliteitscriteria te specificeren voor ieder ontvangend water.

De beperkte informatie met betrekking tot de waterkwaliteit betekent niet dat de huidige modellen zinloos zouden zijn. Cornell (1972) noemde een belangrijk punt voor elke betrouwbaarheidsanalyse: *'In de ingenieurs praktijk is het belangrijk dat we de tendens vermijden om enkel die probabilistische aspecten te modelleren waarvan we denken te weten hoe deze te analyseren. Het is een stuk beter om een benadering van het gehele probleem te hebben dan een exact model van slechts een onderdeel hiervan.'*

In het ontwerp kan een onderscheid worden gemaakt tussen hydraulische criteria en waterkwaliteits criteria. De eerstgenoemde hebben met name betrekking op het risico van 'water-op-straat'. Het hydraulisch ontwerp kan worden gebaseerd op één of enkele ontwerpbui(en) met een zekere herhalingsperiode (bijv. in de orde van 10 jaar). De beschikbare hydrodynamische modellen kunnen worden gebruikt voor een gedetailleerde analyse van het hydrodynamische gedrag van het systeem.

Het bepalen van een efficiënt systeem ontwerp dat voldoet aan de eisen m.b.t. de toelaatbare vuiluitworp is in feite gericht op het vinden van een geschikte balans tussen de te installeren bergings-, afvoer- en zuiveringscapaciteit. Voor dit probleem is het gebruik van tijdreeksanalyses de meest geschikte methode voor een probabilistische interpretatie van het project. Wanneer een ontwerpbui wordt gebruikt beperkt deze interpretatie zich tot de neerslag, terwijl door simulaties van reeksen van buien rekening gehouden kan worden met de verschillende processen die de statistische eigenschappen van de systeemvariabelen zouden kunnen beïnvloeden.

Deze tijdreeksanalyses kunnen worden uitgevoerd met LOCUS. De aanpak zoals geïllustreerd door Fig. 10.2 is een iteratieve procedure, waarbij het systeem wordt aangepast totdat een acceptabele (en minst dure) oplossing is gevonden, waarbij de output variabelen binnen de aangegeven grenzen vallen. Een belangrijk kenmerk van

LOCUS is dat de doelfunctie geldig blijft wanneer de systeem parameters worden gewijzigd. Dit betekent dat met het model verscheidene 'runs' kunnen worden gemaakt voor verschillende systeemcapaciteiten, gebruik makende van dezelfde set 'eenheidskosten'.

## 11.5 Bestuurlijke aspecten

Er kan worden geconcludeerd dat real time sturing een realistisch en economisch haalbaar alternatief is voor de 'statische' besturing van de meeste afvalwatersystemen. Echter, om de voordelen van deze innovatieve techniek te kunnen benutten zullen een aantal bestuurlijke problemen moeten worden opgelost.

De huidige normering en financiële regelingen blijken één van de belangrijkste obstakels te vormen voor de implementatie van real time sturing. Allereerst wordt het gebruik van 'statische' normen, zoals een hoeveelheid berging die aanwezig moet zijn of de rwzi capaciteit die vastgesteld wordt door sommatie van de afvoercapaciteiten van de toeleverende stelsels, ten zeerste afgeraden, omdat zij veelal een goed gebruik van sturing of een andere innovatieve techniek in de weg staan. Bij normering zal eerder gedacht moeten worden in dynamische termen, zoals een gewenst systeemgedrag (dat wordt geverifieerd aan de hand van metingen).

Daarnaast kan het bestaan van verschillende beheerders binnen één systeem tot problemen leiden bij de introductie van real time sturing. Zo zal de gemeente in de regel geïnteresseerd zijn in de mogelijkheden tot reductie van de benodigde berging, terwijl de waterkwaliteitsbeheerder de piekafvoer naar de rwzi zou willen reduceren, door zo goed mogelijk gebruik te maken van de aanwezige berging in de rioolstelsels. Binnen het concept van het Integraal Waterbeheer is het uiteindelijke doel natuurlijk hetzelfde, namelijk een zo goed mogelijk functioneren van het afvalwatersysteem tegen minimale kosten.

Kortom, de waterbeheerders zullen de handen ineen moeten slaan en over hun beheersgrenzen moeten kijken. Zolang dit niet gebeurd zullen inefficiënte oplossingen onvermijdelijk blijven. De veranderde houding ten aanzien van milieuproblemen, welke opgelost moeten worden met een beperkt budget, zal hopelijk de benodigde veranderingen van de huidige regelingen versnellen.

## 11.6 Ter afsluiting

Refererend aan de doelstelling van deze studie kan worden gesteld dat een computer model is ontwikkeld dat kan worden gebruikt om de mogelijkheden en effecten te bepalen van een real time gestuurd afvalwatersysteem. Verschillende case studies die met het model zijn uitgevoerd hebben de toepassing ervan gedemonstreerd, en vooral de potentie van deze innovatieve techniek. Het spreekt voor zich dat een verbeterd operationeel beheer niet de oplossing vormt voor alle problemen, maar het is aangetoond dat het functioneren van het afvalwatersysteem aanmerkelijk kan worden verbeterd. Voordelen zijn een verbeterd milieubeheer, maar ook mogelijke besparingen op investeringen en operationele kosten.

Het aantal praktijkvoorbeelden van real time sturing in het stedelijk waterbeheer is vooralsnog beperkt. Geen van de auteur bekende voorbeelden in binnen- en buitenland maakt gebruik van mathematische optimalisatie voor het afleiden van de operationele strategie. Waarschijnlijk zal het bovengenoemde project in Rotterdam de eerste applicatie op dit gebied zijn.

De laatste jaren heeft een toenemend aantal waterbeheerders besloten tot de installatie van een geautomatiseerd informatiesysteem. Bovendien is veelal de mogelijkheid aanwezig om regelkunstwerken op afstand te besturen, zodat snel kan worden ingegrepen in buitengewone bedrijfsituaties. Hoewel data acquisitie, signalering en alarmering nu nog de belangrijkste drijfveren zijn voor de aanschaf van dergelijke systemen, heeft de praktijk reeds aangetoond dat de behoefte om actief het systeem te gaan sturen zal toenemen naarmate men meer inzicht krijgt in het (werkelijke) dynamische gedrag van het systeem. De auteur hoopt dat deze studie zal bijdragen in de ontwikkeling van een geschikt besturingsconcept voor deze systemen.

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Post academic courses at the Delft University of Technology

- course: Riolering en oppervlaktewater (Sewer system and surface waters), september 1989

  contribution: 'Besturing van rioleringssystemen' (Control of sewer systems)
- course: Korte termijn sturing van waterkwaliteit, (Control of water quality), juni 1991

  contribution: 'Sturing in het stedelijk waterbeheer', (Control in urban water management)
- course: 2e generatie riolering, (Second generation sewerage systems) sept. 91

  contribution: 'Sturing van afvalwatersystemen' (Control of waste water systems)

Agricultural University of Wageningen

- course: Capita selecta Integrated Water Management, 2 november 1988
- contribution: 'Real Time Control van rioleringssystemen'.

International Institute for Hydraulics and Environmental Engineering (IHE), Delft

- course: Short Internat. Course on Urban Drainage in Developping Countries, Nov. 1988,
- contributions: 'Control of urban drainage systems' + 'The use of electronic spreadsheets'

(Note: this course was repeated at IHE, in November 1989, and at The University of Essen, Germany, in november 1990)

Intensive short course on real time control of urban drainage systems, organized by the IAWPRC Task Group on RTCUDS, Berghotel, Amersfoort, 26-29 June 1989.

- contribution: 'National Aspects of Real Time Control'

TRITON course on Integrated Urban Storm Drainage, organized by the Technical University of Denmark.

- contribution: catchment and rainfall data: the effects of rainfall distribution

# List of Symbols

| Symbol | Definition | Dimension |
|---|---|---|
| $a_{ij}$ | coefficient | - |
| $A$ | area | $L^2$ |
| $b_j$ | coefficient (upper capacity constraint) | - |
| $c_{ij}$ | unit cost | - |
| $cv$ | unit cost of V | - |
| $cq$ | unit cost of Q | - |
| $co$ | unit cost of O | - |
| $C$ | Chézy resistance coefficient | $L^{1/2}\ T^{-1}$ |
| $C$ | runoff coefficient | - |
| $C$ | pollution concentration | $M\ L^3$ |
| $D$ | diameter | L |
| $F$ | objective function | - |
| $g$ | gravitational acceleration | $L\ T^{-2}$ |
| $i$ | rain intensity | $L\ T^{-1}$ |
| $I$ | gradient, friction loss per unit lenght | - |
| $I$ | inflow | $L^3\ T^{-1}$ |
| $k_m$ | Manning resistance coefficient | $L^{1/3}\ T^{-1}$ |
| $L$ | lenght of a conduit | L |
| $n$ | number of Nodes | - |
| $O$ | outflow, overflow | $L^3\ T^{-1}$ |
| $Q$ | flow rate, discharge | $L^3\ T^{-1}$ |
| $R$ | hydraulic radius | L |
| $Re$ | number of Reynolds | - |
| $S$ | stored volume | $L^3$ |
| $t$ | time (step) | T |
| $t_c$ | time of concentration | T |

| Symbol | Definition | Dimension |
|---|---|---|
| $T$ | control (forecast) horizon | T |
| $\underline{u}$ | control variable | |
| $v$ | mean flow velocity | $L\,T^{-1}$ |
| $V$ | stored volume | $L^3\,T^{-1}$ |
| $\underline{x}$ | vector depicting the systems state | |
| $x_{ij}$ | systems state variable | |
| $z$ | friction loss | L |
| | | |
| $\alpha_1$ | filling degree (Eq. 4.15) | - |
| $\alpha_2$ | rate of increase of $V$ (approx. of $\delta V/\delta t$, Eq. 4.15) | - |
| $\beta_1$ | basic unit cost of $O$ | - |
| $\beta_2$ | weighing factor (Eq. 4.17) | - |
| $\lambda$ | weighing factor (Kriging, Eq. 8.1) | - |
| $\kappa$ | maximum of $cq$ (Eq. 4.15) | - |

*subscripts*

| | |
|---|---|
| $i$ | concerning Node $i$ |
| $min$ | minimum, lower bound of the variable |
| $max$ | maximum, upper bound of the variable |

*superscripts*

| | |
|---|---|
| $t$ | at time step t (variable may vary in time) |

# List of figures and tables

# Appendices

# A. Network Algorithms

Operations Research comprises a great variety of optimization problems and related solution techniques. A special group is formed by network models, which are developed to solve specially structured Linear Programming problems. A network is a graph consisting of nodes and arcs. Typical problems of network analysis are finding the shortest path between any two nodes, the shortest tree connecting all nodes with a common source, finding the maximum flow through a capacitated network and network flow (transhipment) problems. The algorithms developed for these problems are not restricted to one type of problem (as the name may suggest). For example, by interpreting the cost coefficients of a linear objective function of a Linear Programming problem as the length of arcs of a network, a shortest path algorithm can be used to solve the problem.

This appendix provides some basic principles of network analysis to make the basic idea of the solution technique as applied in LOCUS transparent. The major part of this appendix is quoted from (Wagner, 1975). For a detailed description of the mathematical backgrounds of network analysis, reference is made to various handbooks on (applied) Operations Research, such as (Müller-Merbach, 1971), (Wagner, 1975) and (Hillier et al., 1980).

## A.1 Maximum flow through a capacitated network

In the development of sophisticated techniques to solve network flow models, analysis of the maximum flow problem is of paramount importance. This problem can be formulated as: Given a network with arc capacities, where Node 0 is the source of all flow and Node P is the sink. What is the maximum amount of flow that can be routed from source to sink ?

First, a simple case is considered, where all arc capacities are equal to 1. Once it is understood how to obtain a solution to this problem it is easy to understand the minor modification required to solve the general case with arbitrary arc capacities.

The solution of the maximum flow problem with unit capacities can be found as follows. Begin with any feasible flow. The steps in the technique either determine that the flow is maximal or discover another solution with increased flow:

*Step 1*: Starting at Node 0, put (+) on each arc (0,*j*) without flow and *label* Node *j* with a check mark (√). Label Node 0 with the mark (⋀)

*Step 2*: Consider any Node *j* that is labelled (√). Put (+) on every flowless outward arc (*j*,*k*) *if* Node *k* is not labelled, and label Node *k* with (√). Then put (-) on every inward arc (*k*,*j*) with flow *if* Node *k* is not labelled, and label Node *k* with (√). Finally, cross the check on Node *j* (⋀) to indicate that the node also has been *scanned.*

*Step 3*: Continue with the operation in Step 2 until the sink (Node P) is labelled, or all labelled nodes have been scanned. A breakthrough occurs as soon as Node P is labelled, because a flow-augmenting path has been discovered from Node 0 to Node P. Such a path can be found by tracing back from the sink a sequence of arcs that have a (+) or a (-). Add a unit of flow on each (+) arc and remove the flow from each (-) arc in this sequence. Return to Step 1 erasing all the previous labels (√,⋀) and signs (+,-). If Node P remains unlabelled at the termination of Step 2, then the optimal solution is found.

The method is carried out for the example of Fig. A.1. The initial routing has two units of flow. An improved solution is found through the following steps (Fig. A.2):

1. Scan Node 0: put (+) on arcs (0,4) & (0,5) and label Node 4, 5 and 0 with (√).
2. Scan Node 5: put (-) on arc (3,5) and label Node 3 with (√). Cross the check (⋀) for Node 5;
3. Scan Node 3: put (-) on arc (1,3) and label Node 1 with (√). Cross the check (⋀) for Node 3;
4. Scan Node 1: put (-) on arc (1,2) and label Node 2 with (√). Cross the check (⋀) for Node 1;
5. Scan Node 2: put (-) on arc (2,6) and label Node 6 with (√): A flow-augmenting path has been found.

The revised solution is given in Fig. A.3. This flow can be shown as maximal by repeating the steps of the algorithm. The sequence of nodes scanned will be Nodes 0, 4 and 5. It will not be possible to label any other node.

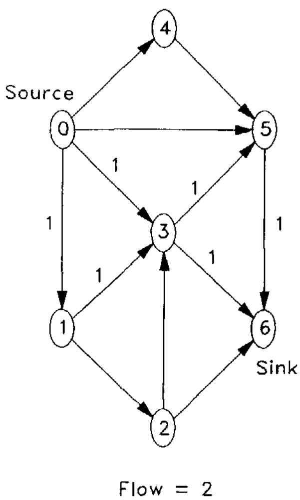



*Figure A.1*

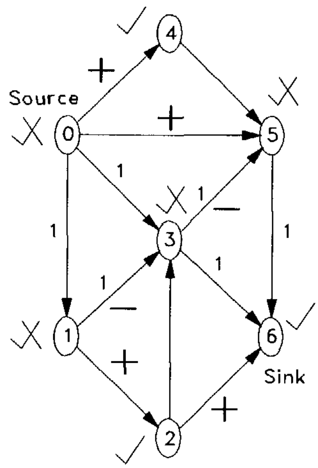



*Figure A.2*

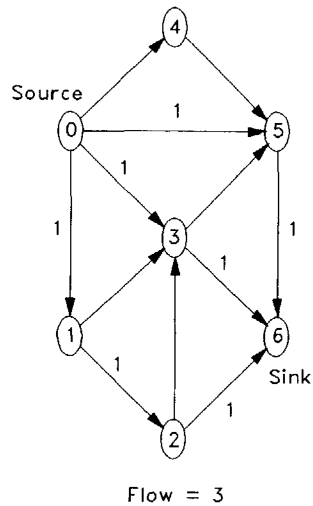



*Figure A.3*

To remove the restriction of unit arc capacities, the algorithm is modified in two respects:

- Put (+) on every outward arc with less then capacity flow in the labelling process.
- When a flow-augmenting path has been found, route as much flow as possible on the path, taking into account the *unused* capacity on each (+) arc and the current flow on each (-) arc.

The next example illustrates these modifications. Consider the network of Fig. A.4. The boxed numbers on each arc represent the arc's capacity. The other numbers represent a trial feasible flow. To initiate the procedure, the same labelling and scanning steps as employed in Fig. A.2 are followed. This yields the result shown in Fig. A.5 and the same flow-augmenting path as in Fig. A.2. Examining this path learns that the flow can be increased along this path by 2 units, thereby causing arcs (0,5) and (2,6) to be at their capacity levels. The revised solution is given in Fig. A.6. By repeating the steps of the algorithm it can be determined that this solution is optimal. The sequence of nodes scanned will be Nodes 0, 4, 5, 3, 1, and 2; from Node 2 it will not be possible to label Node 6.

At each iteration, *Step 3* results either in an increased flow or in termination. Therefore the algorithm is finite, since the maximum possible flow is bounded. It can be proved that when the algorithm terminates, the flow is equal to the minimum cut

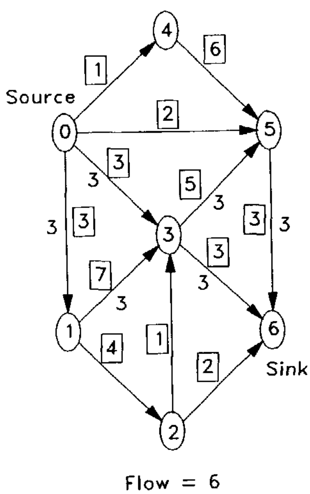



*Figure A.4*

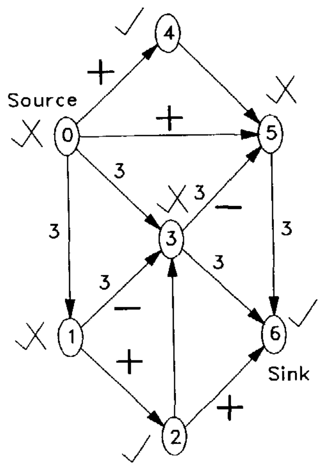



*Figure A.5*

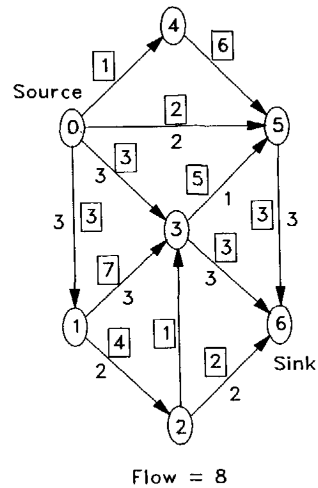



*Figure A.6*

capacity (a cut is a partition of the nodes in two classes), which is, by definition, the maximum flow $F$ in the network structure (2) through (5).

## A.2 A transportation model

Consider a transportation model of the form

$$\textit{minimize} \sum_{i=1}^{m} \sum_{j=1}^{n} c_{ij} x_{ij} \tag{A.1}$$

subject to

$$\sum_{j=1}^{n} x_{ij} = S_i; \textit{ for } i=1,2,\ldots,m \tag{A.2}$$

$$\sum_{i=1}^{m} x_{ij} = D_j; \textit{ for } j=1,2,\ldots,n \tag{A.3}$$

$$x_{ij} \geq 0; \textit{ for all } i \textit{ and } j \tag{A.4}$$

where all the $S_i$ (Supply) and $D_j$ (Demand) are positive integers, and $\Sigma\, S_i = \Sigma\, D_j$.

Two problems are called dual if they are structured as follows

*primary problem:*

$$\text{maximize} \sum_{j=1}^{n} c_j x_j$$

$$\sum_{j=1}^{n} a_{ij} x_j \leq b_i;\ \text{for } i = 1,2,\ldots,m$$

$$x_j \geq 0;\ \text{for } j = 1,2,\ldots,n$$

*dual problem:*

$$\text{minimize} \sum_{i=1}^{m} b_i y_i \qquad \text{(A.5)}$$

$$\sum_{i=1}^{m} a_{ij} y_i \geq c_j;\ \text{for } j = 1,2,\ldots,n \qquad \text{(A.6)}$$

$$y_i \geq 0;\ \text{for } i = 1,2,\ldots,m \qquad \text{(A.7)}$$

When two problems are dual then

- the dual problem of the dual problem is the primary problem, and
- the optimal solutions of both problems are equal (if they exist).

It can be shown that the feasibility restrictions for the associated dual linear program of (A.1) are

$$\text{relative cost} = c_{ij} - v_i - w_i \geq 0;\ \text{for all } i \text{ and } j \qquad \text{(A.8)}$$

where $v_i$ is the dual variable corresponding to the *i*th constraint in Eq. (A.2) and $w_j$ is the dual variable corresponding to *j*th constraint in Eq. (A.3).

The solution method maintains dual variables that are feasible in (A.8) at every iteration. Given a set of trial dual variables ($v_i$, $w_j$), the approach employs the maximum flow algorithm to seek a feasible solution to the transportation model. The technique consists of two steps:

*Step 1*: Given constants $v_i$ ($i = 1,2,..,n$) and $w_j$ ($j = 1,2,..,m$) yielding nonnegative relative cost. (Note that when all $c_{ij} \geq 0$ this step could be initiated by letting all $v_i = w_j = 0$) Determine whether a feasible solution exists using only routes with relative cost equal to 0. If so, stop the algorithm, since the solution is optimal; otherwise go to Step 2.

*Step 2*: Revise $v_i$ and $w_j$ such that at least one new route has relative cost equal to 0. Return to Step 1.

At termination, the final solution is optimal because the associated relative cost (A.8), which must be nonnegative for any feasible solution to the transportation model, equals to 0.

The following example illustrates the approach and amplifies the details of Step 2. Consider the transportation problem given in Table A.1. The table shows trial values of $v_i$ and $w_j$ and the corresponding relative costs.

*Table A.1 Transportation model and relative costs*

| | k1 | | k2 | | k3 | | k4 | | Vi | Supply |
|---|---|---|---|---|---|---|---|---|---|---|
| | cost | rel.cost | cost | rel.cost | cost | rel.cost | cost | rel.cost | | |
| r1 | 2 | 0 | 10 | 0 | 15 | 3 | 0 | 0 | 0 | 4 |
| r2 | 10 | 0 | 18 | 0 | 20 | 0 | 9 | 1 | 8 | 15 |
| r3 | 15 | 3 | 24 | 4 | 26 | 4 | 10 | 0 | 10 | 2 |
| r4 | 12 | 2 | 25 | 7 | 27 | 7 | 8 | 0 | 8 | 7 |
| Wj | | 2 | | 10 | | 12 | | 0 | | |
| Demand | 9 | | 11 | | 5 | | 3 | | | |

The maximum flow algorithm is employed in *Step 1* to find whether there exists a feasible solution using only routes with 0 relative cost in Table A.1. A network flow diagram comprised only the routes with relative cost equal to 0 in Table A.1 is constructed in Fig. A.7. All arcs out of the source node have capacity $S_i$, and the arcs into the sink node have capacity $D_j$. A trial flow of 22 units is also indicated in Fig. A.7; a feasible solution to the transportation problem requires a total flow of 28 units $(= \Sigma S_i = \Sigma D_j)$.

Since the trial flow is not feasible the maximum flow algorithm is carried out in *Step 1* to determine whether additional flow can be routed through the network. This process labels Nodes $r_3$, $r_4$, and $k_4$, and does not result in a flow augmenting path, as illustrated in Fig. A.8. Therefore the method proceeds with *Step 2.*

Since the flow in Fig. A.8 is maximal, but less then the required 28 units, it is necessary to introduce at least one new arc $(r_i, k_j)$. Given the nature of the flow algorithm it is reasonable to restrict attention to those arcs so that Node $r_i$ is scanned and Node $k_j$ is unlabelled. The addition of such an arc will permit the scanning of at least one more Node $k_j$.

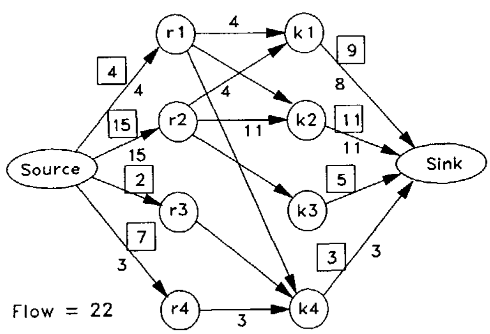



*Figure A.7*

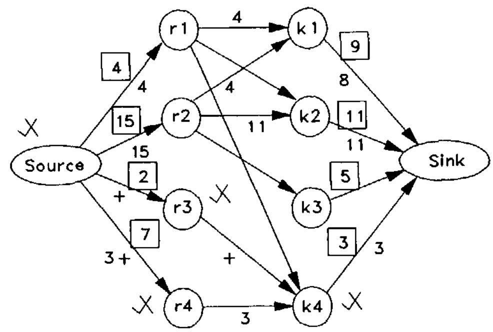



*Figure A.8*

In revising the $v_i$ and $w_j$ one should be careful not to destroy the restriction (A.8) for routes having flow, as well as for those marked with (+) in the scanning process. A rule that preserves the restriction (A.8) for all $i$ and $j$ is

(1) Add $\theta$ to $v_i$ if Node $r_i$ is scanned,

(2) Subtract $\theta$ from $w_j$ if Node $k_j$ is scanned,

where $\theta$= the smallest relative cost for arcs between every scanned Node $r_i$ and unlabelled Node $k_j$.

In Fig. A.8, Nodes $r_3$ and $r_4$ are scanned, and Nodes $k_1$, $k_2$ and $k_3$ are unlabelled. Examining in Table A.1 the entries at the intersection of Rows 3 and 4 and Columns 1, 2 and 3 yields:

$\theta$ = minimum(3,4,4,2,7,7) = 2. The revised $v_i$ and $w_j$ are shown in Table A.2 along with the new relative costs. Arc $(r_4,k_1)$ has now relative cost equal to 0, but arc $(r_1,k_4)$ has a positive relative cost. The associated network appears in Fig. A.9.

*Table A.2 Revised relative costs*

| | k1 | | k2 | | k3 | | k4 | | Vi | Supply |
|---|---|---|---|---|---|---|---|---|---|---|
| | cost | rel.cost | cost | rel.cost | cost | rel.cost | cost | rel.cost | | |
| r1 | 2 | 0 | 10 | 0 | 15 | 3 | 0 | 2 | 0 | 4 |
| r2 | 10 | 0 | 18 | 0 | 20 | 0 | 9 | 3 | 8 | 15 |
| r3 | 15 | 1 | 24 | 2 | 26 | 2 | 10 | 0 | 12 | 2 |
| r4 | 12 | 0 | 25 | 5 | 27 | 5 | 8 | 0 | 10 | 7 |
| Wj | | 2 | | 10 | | 12 | | -2 | | |
| Demand | 9 | | 11 | | 5 | | 3 | | | |

The maximum flow algorithm resumes and, in the process, labels Node $k_1$ and the sink. Given the unused capacity on the arc ($k_1$, sink), one additional unit of flow can be routed on this path. The resulting solution is shown in Fig. A.10. Since the total flow does not yet equal to 28, the maximum flow algorithm is applied again and summarized in Fig. A.10. The unused capacity from the source to Node $r_4$ limits the additional flow to 3 for the flow-augmenting path. The resulting solution is shown in Fig. A.11.

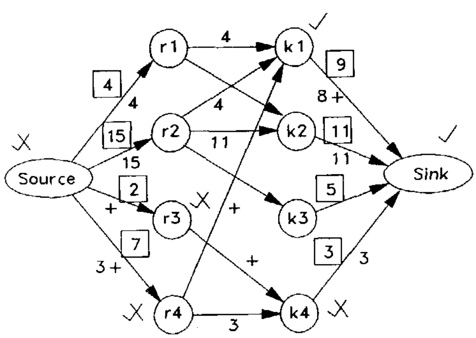



*Figure A.9*

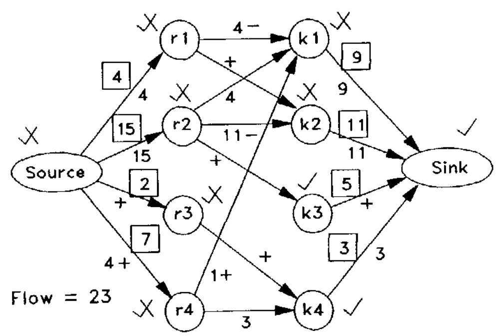



*Figure A.10*

Since the total flow now is still only 26, another attempt is made to increase the flow, as illustrated in Fig. A.11. The flow augmenting path allows 2 units of additional flow, thereby causing the total flow to be 28 units. The final optimal solution is shown in Fig. A.12.

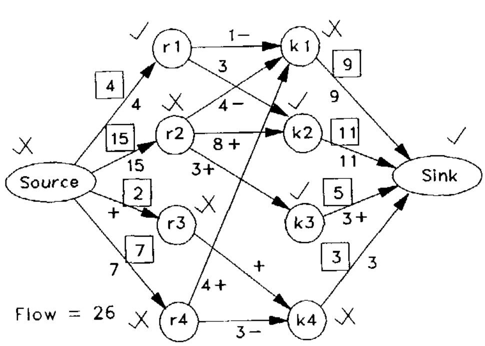



*Figure A.11*

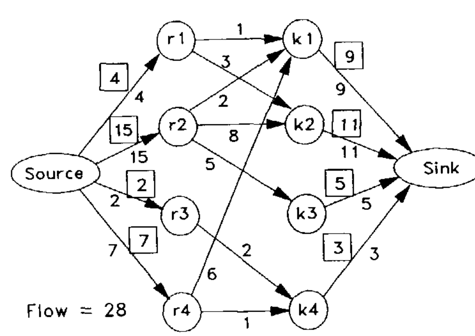



*Figure A.12*

## A.3 Concluding remarks

The maximum flow technique can also be imbedded in an efficient optimization algorithm to solve more complex network models. This algorithm, called the 'out-of-kilter algorithm, does not require that the network model should be converted into a transportation problem format. As was mentioned, the latter is explained above to make clear the basic concept of the approach and to illustrate how the maximum flow model is of value. The out-of-kilter algorithm is well suited to solving transhipment models and network models with arc capacities. The general problem is

$$\text{minimize} \sum_{i=1}^{N} \sum_{j=1}^{N} c_{ij} x_{ij} \tag{A.9}$$

subject to

$$\sum_{i=1}^{N} x_{kj} - \sum_{j=1}^{N} x_{ik} = 0 \; ; \textit{for } k = 1,2,\ldots,N \tag{A.10}$$

$$(r_{ij} \leq x_{ij} \leq u_{ij}) \; ; \textit{for all } i \textit{ and } j. \tag{A.11}$$

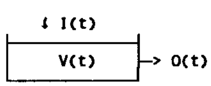
where $N$ is the number of nodes, $r_{ij}$ the lower bounds and $u_{ij}$ the upper bounds of the (integer) variable $x_{ij}$. Because the right-hand sides of Eq. A.10 are equal to 0, the problem is called a circulation model. Note the similarity of the problem as described by Fig. 4.5. To see how to convert a transportation problem into a circulation problem and for a detailed description of the out-of-kilter algorithm, reference is made to a.o. (Wagner, 1975).

# B. Dynamic Programming: a simple example

In section 4.2, it was stated that applying the principle of Dynamic Programming (DP) to the determination of control strategies means that each decision at a certain time must be optimal, independent of decisions at former time steps. This leads to the following process:

- Start at the end of n control decisions. Determine for each possible situation at this time step the optimum strategy and store them.
- Next, time step n-1 is being considered. Again determine the optimum startegy for each possible situation at this time step, but now taking into account the strategy as determined at $t=n$, and store them.
- Repeat this process, until $t=1$. As at this time step the state of the system is known (initial state), it is possible to determine the optimum strategy for all time steps 1,..,n.

As an example, an operation strategy is derived for the outflow of a reservoir. This example is quoted from (Schilling, 1986)




The operational objective is to keep the outflow ($O_t$) as constant as possible (say 20). Small deviations from the optimum outflow are considered less important than bigger deviations. This can be reached by using a quadratic objective function:

$$\text{minimize } Z = \sum^{T} (O_t - 20)^2$$

In this example, the following constraints are applied:

$$V_{t+1} = V_t - O_t + I_t$$
$$0 \leq V_t \leq V_{max} \; (= 15)$$
$$0 \leq O_t \leq O_{max} \; (= 40)$$

$V_0 = 10$ (initial state) and $I_1 = 20$ ; $I_2 = 37.5$ ; $I_3 = 10$ (inflow)

The problem can be rewritten as:

$$\text{minimize} \;\; Z = [\, (O_1-20)^2 + (O_2-20)^2 + (O_3-20)^2 \,]$$

subject to the constraints:

$$V_2 = V_1 - O_1 + 20$$
$$V_3 = V_2 - O_2 + 37.5$$
$$V_4 = V_3 - O_3 + 10$$

$$0 \le V_t \le 15 \quad ; t = 1,2,3,4$$
$$0 \le O_t \le 40 \quad ; t = 1,2,3$$

To solve this problem using Dynamic Programming, the optimum strategy has first to be found at the last time step $t=3$. Here, the problem is:

$$\text{minimize} \;\; Z_3 = (O_3 - 20)^2$$

subject to

$$0 \le V_4 = V_3 - O_3 + 10 \le 15$$
$$0 \le O_3 \le 40$$

The solution of this problem is found by: $\partial Z_3/\partial O_3 = 0$
resulting in $2(O_3 - 20) = 0$ ,or $O_3 = 20$

When this solution is feasible, it will be the optimum one for this time step. Therefore the constraint is checked:

$$0 \le V_4 = V_3 - O_3 + 10 \; (= V_3 - 10) \le 15$$

For $V_3 \ge 10$, the constraint is fulfilled, meaning that $O_3 = 20$ is a feasible solution. However, if
$V_3 < 10$, the constraint is only met, when $O_3 = V_3 + 10$.

Summarizing, the optimal strategy at $t=3$ is:

- $O_3 = 20$, when $V_3 \ge 10$ ; with $Z_3 = 0$
- $O_3 = V_3 + 10$, when $V_3 < 10$ ; with $Z_3 = (V_3 - 10)^2$

At t=2 the problem is:

$$\text{minimize } Z_2 = Z_3 + (O_2 - 20)^2$$

subject to

$$0 \leq V_3 = V_2 - O_2 + 37.5 \leq 15$$
$$0 \leq O_2 \leq 40$$

Due to the results at t=3, two possibilities have to be considered, namely $V_3 < 10$ and $V_3 \geq 10$.

First, the case $V_3 < 10$ will be investigated, with $Z_3 = (V_3 - 10)^2$. Using $V_3 = V_2 - O_2 + 37.5$, the problem can be rewritten as:

$$\text{minimize } Z_2 = [\,(V_2 - O_2 + 27.5)^2 + (O_2 - 20)^2\,]$$

$$\partial Z_2/\partial O_2 = 0, \text{ results in } O_2 = 0.5\,V_2 + 23.75$$

meaning that $V_3 = 0.5\,V_2 + 13.75$, which is in contradiction with the first assumption that $V_3 < 10$. Therefore, this case does not have to be further considered.

Investigating the case $V_3 \geq 10$ is simple as here counts $Z_3 = 0$, meaning that the problem is:

$$\text{minimize } Z_2 = (O_2 - 20)^2\text{, resulting in } O_2 = 20$$

To verify the feasibility of this solution, the constraints have to checked:

$$V_3 = V_2 - O_2 + 37.5 \leq 15$$

Using $O_2=20$ would mean that the maximum storage capacity (=15) will be exceeded. Therefore $O_2$ must be given the smallest value that guarantees that the constraint of maximum storage capacity will not be violated:

$$O_2 = V_2 + 22.5\text{, with } Z_2 = (V_2 + 2.5)^2$$

Finally, the first time step is considered:

$$\text{minimize } Z = Z_1 = Z_2 + (O_1 - 20)^2$$

subject to the constraints:

$$0 \le V_2 = V_1 - O_1 + 20 \le 15$$
$$0 \le O_1 \le 40$$

Filling in $V_2 = V_1 - O_1 + 20$ in $Z_2$ yields:

$$\text{minimize } Z = [ (V_1 - O_1 + 20 + 2.5)^2 + (O_1 - 20)^2 ]$$

$$\partial Z_1/\partial O_1 = 0, \text{ gives } O_1 = 0.5 V_1 + 21.25$$

Checking with the constraint yields:

$$V_2 = V_1 - 0.5 V_1 + 21.25 - 20 = 0.5 V_1 - 1.25$$

For $V_1 \ge 2.5$ the non-negativity constraint is always fulfilled. To satisfy the non-negativity constraint for $V_1 < 2.5$ the following equation holds:

$$0 \le V_2 = V_1 - O_1 + 20, \text{ which yields } O_1 = V_1 + 20$$

Summarizing, at $t=1$ the following strategy has to be applied:

- $O_1 = 0.5 V_1 + 21.25$, for $V_1 \ge 2.5$ ; $Z = 2 (0.5 V_1 + 1.25)^2$
- $O_1 = V_1 + 20$, for V1 < 2.5 ; $Z = V_1^2 + 6.25$

The optimal strategy (for the given inflow) as a function of the initial state (here: $V_0 = 10$) is given in the following table:

| time | stored volume (start) | strategy | stored volume (end) | outflow | cost |
|---|---|---|---|---|---|
| 1 | $10 \le V_1 < 2.5$<br>$2.5 \le V_1 < 15$ | $V_1 + 20$<br>$0.5 V_1 + 21.25$ | 10.0 | 26.25 | 39.0625 |
| 2 | $0 \le V_2 < 15$ | $V_2 + 22.5$ | 3.75 | 26.25 | 39.0625 |
| 3 | $0 \le V_3 < 10$<br>$10 \le V_3 < 15$ | $V_3 + 10$<br>20 | 15.0 | 20 | 0.0 |
| 4 | | | 5.0 | | |

# C. Results of simulations

## C.1 Chapter 5: Case study of an artificial catchment

* : the storage capacity of Node 3 is enlarged to 6000, 7000, 8000 and 9000 m3 (= 6, 7, 8, 9 mm)

** : the discharge capacity of Node 3 is enlarged to 2200, 2300, 2400, and 2500 m3/h (over-capacity = 0.9, 1.0, 1.1, 1.2, 1.3 mm/h)

| OVERFLOW VOLUMES (53 EVENTS > 5 MM, incl. DWP < 10 HRS ; year: 1981) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SYSTEM CONFIGURATION 1<br>1 –<br>\|– 3 –><br>2 – | | | | | | | | | | | | |
| DIS. 3 = 2100 | local control: 10/30/50 | | | | local control: 5/10/15 | | | | optimal control (FH = 60 min) | | | |
| STO. 3 | 1 | 2 | 3 | TOTAL | 1 | 2 | 3 | TOTAL | 1 | 2 | 3 | TOTAL |
| 5000 | 123875 | 118401 | 148051 | 390327 | 107767 | 106093 | 136412 | 350272 | 105917 | 108293 | 104385 | 318595 |
| 6000 | 123875 | 118401 | 128326 | 370602 | 107767 | 106093 | 114056 | 327916 | 100237 | 103004 | 95801 | 299042 |
| 7000 | 123875 | 118401 | 108747 | 351023 | 107767 | 106093 | 95037 | 308897 | 98977 | 99397 | 84147 | 282521 |
| 8000 | 123875 | 118401 | 94079 | 336355 | 107767 | 106093 | 80299 | 294159 | 98977 | 98799 | 68745 | 266521 |
| 9000 | 123875 | 118401 | 81686 | 323962 | 107767 | 106093 | 65400 | 279260 | 98977 | 98799 | 55195 | 252971 |
| STO. 3 = 5000 | | | | | | | | | | | | |
| DIS. 3 | 1 | 2 | 3 | TOTAL | 1 | 2 | 3 | TOTAL | 1 | 2 | 3 | TOTAL |
| 2100 | 123875 | 118401 | 148051 | 390327 | 107767 | 106093 | 136412 | 350272 | 105917 | 108293 | 104385 | 318595 |
| 2200 | 123875 | 118401 | 134946 | 377222 | 107767 | 106093 | 122436 | 336296 | 103829 | 105565 | 95779 | 305173 |
| 2300 | 123875 | 118401 | 121571 | 363847 | 107767 | 106093 | 109002 | 322862 | 102252 | 104139 | 87018 | 293409 |
| 2400 | 123875 | 118401 | 111131 | 353407 | 107767 | 106093 | 97121 | 310981 | 101043 | 103231 | 77728 | 282002 |
| 2500 | 123875 | 118401 | 101616 | 343892 | 107767 | 106093 | 86882 | 300742 | 100514 | 102367 | 68731 | 271612 |
| SYSTEM CONFIGURATION 2<br>1 — 2 — 3 —> | | | | | | | | | | | | |
| DIS. 3 = 2100 | local control: 10/30/50 | | | | local control: 5/10/15 | | | | optimal control (FH = 60 min) | | | |
| STO. 3 | 1 | 2 | 3 | TOTAL | 1 | 2 | 3 | TOTAL | 1 | 2 | 3 | TOTAL |
| 5000 | 123875 | 118969 | 147973 | 390817 | 107767 | 106862 | 136479 | 351108 | 105065 | 107253 | 103694 | 316012 |
| 6000 | 123875 | 118969 | 126986 | 369830 | 107767 | 106862 | 114082 | 328711 | 103217 | 100752 | 93451 | 297420 |
| 7000 | 123875 | 118969 | 108427 | 351271 | 107767 | 106862 | 94897 | 309526 | 101458 | 96812 | 82447 | 280717 |
| 8000 | 123875 | 118969 | 93573 | 336417 | 107767 | 106862 | 79769 | 294398 | 100873 | 95336 | 68508 | 264717 |
| 9000 | 123875 | 118969 | 81265 | 324109 | 107767 | 106862 | 65562 | 280191 | 100873 | 95336 | 54959 | 251168 |
| STO. 3 = 5000 | | | | | | | | | | | | |
| DIS. 3 | 1 | 2 | 3 | TOTAL | 1 | 2 | 3 | TOTAL | 1 | 2 | 3 | TOTAL |
| 2100 | 123875 | 118969 | 147973 | 390817 | 107767 | 106862 | 136479 | 351108 | 105065 | 107253 | 103694 | 316012 |
| 2200 | 123875 | 118969 | 134177 | 377021 | 107767 | 106862 | 121649 | 336278 | 103430 | 104306 | 95332 | 303068 |
| 2300 | 123875 | 118969 | 120375 | 363219 | 107767 | 106862 | 108094 | 322723 | 102748 | 102627 | 86166 | 291541 |
| 2400 | 123875 | 118969 | 110519 | 353363 | 107767 | 106862 | 96488 | 311117 | 102238 | 101638 | 76474 | 280350 |
| 2500 | 123875 | 118969 | 100099 | 342943 | 107767 | 106862 | 86836 | 301465 | 101802 | 100947 | 67101 | 269850 |

| | System configuration 1 (parallel) | | | System configuration 2 (serial) | | |
|---|---|---|---|---|---|---|
| | Node 1 | Node 2 | Node 3 | Node 1 | Node 2 | Node 3 |
| Area [ha] | 100 | 100 | 100 | 100 | 100 | 100 |
| Storage [m3] | 7000 | 8000 | 5000 | 7000 | 8000 | 5000 |
| Storage [mm] | 7 | 8 | 5 | 7 | 8 | 5 |
| Dis.Cap. [m3/h] | 700 | 600 | 2100 | 700 | 1300 | 2100 |
| Overcap. [mm/h] | 0.7 | 0.6 | 0.8 | 0.7 | 0.6 | 0.8 |

| OVERFLOW VOLUMES (53 EVENTS > 5 MM, incl. DWP < 10 HRS ; year: 1981) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| SYSTEM CONFIGURATION 1<br>1 --<br>]-- 3 --><br>2 -- | | | | | | | | | | |
| | Perfect forecast | | | | Forecast error (FH = 3) | | | | Error (FH = 24) | |
| | FH = 3 | FH = 6 | FH = 12 | FH = 24 | (+) | (-) | (++) | (--) | (--) | (++) |
| Node 1 | 104326 | 105917 | 105924 | 105957 | 104728 | 104162 | 104779 | 102966 | 106045 | 107446 |
| Node 2 | 105481 | 108293 | 108154 | 108102 | 105564 | 105058 | 106130 | 104141 | 106231 | 112183 |
| Node 3 | 108909 | 104385 | 104619 | 104646 | 108358 | 110016 | 107558 | 112733 | 107227 | 102747 |
| Total | 318716 | 318595 | 318697 | 318705 | 318650 | 319236 | 318467 | 319840 | 319503 | 322376 |
| SYSTEM CONFIGURATION 2<br>1 --- 2 --- 3 ---> | | | | | | | | | | |
| | Perfect forecast | | | | Forecast error (FH = 3) | | | | Error (FH = 24) | |
| | FH = 3 | FH = 6 | FH = 12 | FH = 24 | (+) | (-) | (++) | (--) | (--) | (++) |
| Node 1 | 101581 | 105065 | 107872 | 104334 | 101784 | 101896 | 101931 | 101445 | 107129 | 107064 |
| Node 2 | 106567 | 107253 | 106475 | 107653 | 107385 | 105540 | 107692 | 104493 | 104463 | 112090 |
| Node 3 | 107887 | 103694 | 102691 | 104168 | 106494 | 109123 | 105967 | 111368 | 105253 | 101474 |
| Total | 316035 | 316012 | 317038 | 316155 | 315663 | 316559 | 315590 | 317306 | 316845 | 320628 |

$\Delta$ t = 10 minutes
(--) = -20%, -50%, -100%, -100%, etc.
(-) = -10%, -20%, -30%
(+) = +10%, +20%, +30%
(++) = +20%, +50%, +100%, +100%, etc.

### C.2 Chapter 6: Case study of Rotterdam

| Results Case study Rotterdam, 333 rain events (period 1970-1985) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Reference Model (local control)** | | | | | | | | | | |
| Node | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Total [m3] | 609586 | 2132080 | 383743 | 66256 | 706079 | 6709800 | 171591 | 453535 | 718148 | 3480640 |
| Number of overflows | 82 | 233 | 90 | 27 | 77 | 244 | 52 | 135 | 70 | 223 |
| Maximum volume [m3] | 67257 | 55970 | 41262 | 17175 | 93109 | 119900 | 29213 | 35735 | 77876 | 107310 |
| Average volume [m3] | 1831 | 6403 | 1152 | 199 | 2120 | 20150 | 515 | 1362 | 2157 | 10452 |
| Average volume * [m3] | 7434 | 9151 | 4264 | 2454 | 9170 | 27499 | 3300 | 3360 | 10259 | 15608 |
| Total overflow [m3] | 15836370 | | Controlled overflow [m3] | | | | 12322520 | | | |
| Nr. of rains with overfl. | 315 | | Uncontrolled overflow [m3] | | | | 3513850 | | | |
| **Optimized Control, FH = 40 min** | | | | | | | | | | |
| Total [m3] | 953691 | 702453 | 368260 | 61785 | 805647 | 1262733 | 221082 | 46244 | 826079 | 115071 |
| Number of overflows | 107 | 144 | 66 | 32 | 102 | 126 | 66 | 15 | 82 | 75 |
| Maximum volume [m3] | 80082 | 36835 | 46030 | 18614 | 98175 | 58290 | 31218 | 22613 | 89659 | 11110 |
| Average volume [m3] | 2864 | 2109 | 1106 | 186 | 2419 | 3792 | 664 | 139 | 2481 | 346 |
| Average volume * [m3] | 8913 | 4878 | 5580 | 1931 | 7899 | 10022 | 3350 | 3083 | 10074 | 1534 |
| Total overflow [m3] | 5363045 | | Controlled overflow [m3] | | | | 2080257 | | | |
| Nr. of rains with overfl. | 157 | | Uncontrolled overflow [m3] | | | | 3282788 | | | |
| **Optimized control, FH = 90 min** | | | | | | | | | | |
| Total [m3] | 771651 | 850059 | 321748 | 69386 | 693508 | 1355388 | 185299 | 44628 | 761989 | 162779 |
| Number of overflows | 79 | 139 | 58 | 33 | 83 | 122 | 53 | 12 | 75 | 73 |
| Maximum volume [m3] | 77856 | 41774 | 45283 | 18484 | 95603 | 62164 | 30778 | 22126 | 88630 | 13162 |
| Average volume [m3] | 2317 | 2553 | 966 | 208 | 2083 | 4070 | 556 | 134 | 2288 | 489 |
| Average volume * [m3] | 9768 | 6116 | 5547 | 2103 | 8356 | 11110 | 3496 | 3719 | 10160 | 2230 |
| Total overflow [m3] | 5216435 | | Controlled overflow [m3] | | | | 2368226 | | | |
| Nr. of rains with overfl. | 150 | | Uncontrolled overflow [m3] | | | | 2848209 | | | |
| **Optimized Control, FH = 40 min, Forecast Error = 1.15; 1.30; 1.50; 1.70 (+)** | | | | | | | | | | |
| Total [m3] | 918467 | 768225 | 345958 | 57594 | 757739 | 1338945 | 210366 | 47141 | 819034 | 117588 |
| Number of overflows | 105 | 149 | 62 | 29 | 86 | 127 | 56 | 16 | 80 | 76 |
| Maximum volume [m3] | 79127 | 38420 | 46087 | 18623 | 98109 | 59932 | 30985 | 22553 | 89655 | 11916 |
| Average volume [m3] | 2758 | 2307 | 1039 | 173 | 2275 | 4021 | 632 | 142 | 2460 | 353 |
| Average volume * [m3] | 8747 | 5156 | 5580 | 1986 | 8811 | 10543 | 3757 | 2946 | 10238 | 1547 |
| Total overflow [m3] | 5381057 | | Controlled overflow [m3] | | | | 2224758 | | | |
| Nr. of rains with overfl. | 157 | | Uncontrolled overflow [m3] | | | | 3156299 | | | |
| **Optimized Control, FH = 40 min, Forecast Error = 0.85; 0.70; 0.50; 0.30 (-)** | | | | | | | | | | |
| Total [m3] | 1006368 | 653851 | 389397 | 69176 | 888077 | 1143326 | 234837 | 47546 | 849634 | 98228 |
| Number of overflows | 124 | 145 | 68 | 40 | 120 | 128 | 79 | 16 | 86 | 74 |
| Maximum volume [m3] | 80837 | 35595 | 47114 | 19076 | 99831 | 54969 | 31733 | 22670 | 89228 | 10846 |
| Average volume [m3] | 3022 | 1964 | 1169 | 208 | 2667 | 3433 | 705 | 143 | 2551 | 295 |
| Average volume * [m3] | 8116 | 4509 | 5726 | 1729 | 7401 | 8932 | 2973 | 2972 | 9879 | 1327 |
| Total overflow [m3] | 5380440 | | Controlled overflow [m3] | | | | 1895405 | | | |
| Nr. of rains with overfl. | 155 | | Uncontrolled overflow [m3] | | | | 3485035 | | | |

| Optimized Control, FH = 40 min, Forecast Error = 1.5; 2.0; 2.0; 2.0 (++) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Total [m3] | 876797 | 875647 | 326133 | 54936 | 709112 | 1475358 | 201876 | 46247 | 803408 | 134125 |
| Number of overflows | 99 | 172 | 58 | 28 | 77 | 145 | 51 | 14 | 75 | 89 |
| Maximum volume [m3] | 79267 | 40692 | 46856 | 18330 | 97237 | 65026 | 31001 | 22410 | 87917 | 12368 |
| Average volume [m3] | 2633 | 2630 | 979 | 165 | 2129 | 4431 | 606 | 139 | 2413 | 403 |
| Average volume * [m3] | 8857 | 5091 | 5623 | 1962 | 9209 | 10175 | 3958 | 3303 | 10712 | 1507 |
| Total overflow [m3] | 5503639 | | Controlled overflow [m3] | | | 2485130 | | | | |
| Nr. of rains with overfl. | 174 | | Uncontrolled overflow [m3] | | | 3018509 | | | | |
| **Optimized Control, FH = 40 min, Forecast Error = 0.5; 0.0; 0.0; 0.0 (--)** | | | | | | | | | | |
| Total [m3] | 1078246 | 574654 | 406078 | 84396 | 956320 | 1060974 | 247518 | 53641 | 855701 | 84791 |
| Number of overflows | 133 | 142 | 78 | 41 | 122 | 127 | 81 | 23 | 86 | 75 |
| Maximum volume [m3] | 81419 | 32462 | 47946 | 19384 | 102309 | 52119 | 32828 | 23716 | 87180 | 10452 |
| Average volume [m3] | 3238 | 1726 | 1219 | 253 | 2872 | 3186 | 743 | 161 | 2570 | 255 |
| Average volume * [m3] | 8107 | 4047 | 5206 | 2058 | 7839 | 8354 | 3056 | 2332 | 9950 | 1131 |
| Total overflow [m3] | 5402319 | | Controlled overflow [m3] | | | 1720419 | | | | |
| Nr. of rains with overfl. | 154 | | Uncontrolled overflow [m3] | | | 3681900 | | | | |
| **Optimized Control, FH = 10 min** | | | | | | | | | | |
| Total [m3] | 1119333 | 572525 | 390182 | 64952 | 1032541 | 1074706 | 252081 | 53602 | 831949 | 76070 |
| Number of overflows | 141 | 147 | 71 | 32 | 129 | 135 | 74 | 22 | 82 | 76 |
| Maximum volume [m3] | 82075 | 32816 | 46103 | 18673 | 105160 | 53225 | 32157 | 23555 | 87673 | 9499 |
| Average volume [m3] | 3361 | 1719 | 1172 | 195 | 3101 | 3227 | 757 | 161 | 2498 | 228 |
| Average volume * [m3] | 7939 | 3895 | 5496 | 2030 | 8004 | 7961 | 3407 | 2436 | 10146 | 1001 |
| Total overflow [m3] | 5467941 | | Controlled overflow [m3] | | | 1723301 | | | | |
| Nr. of rains with overfl. | 158 | | Unontrolled overflow [m3] | | | 3744640 | | | | |

## C.4 Chapter 8: Case study of Westfriesland

**HOMOGENEOUS RAIN - LOCAL CONTROL - CAP. TREATMENT PLANT = 5800 M3/HR**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 6707 | 46943 | 407539 | 260372 | 41241 | 513380 | 53452 | 61084 | 224369 | 48395 | 67473 | 17651 | 167326 | 153844 | 2069776 |
| Max | 1860 | 3685 | 37283 | 10773 | 6609 | 13031 | 4670 | 10435 | 21255 | 5718 | 5414 | 3695 | 9329 | 11110 | 144867 |
| Num | 31 | 105 | 98 | 204 | 56 | 238 | 99 | 45 | 102 | 79 | 111 | 34 | 171 | 141 | 108 |
| Avg | 216 | 447 | 4159 | 1276 | 736 | 2157 | 540 | 1357 | 2200 | 613 | 608 | 519 | 979 | 1091 | 1207 |

**HOMOGENEOUS RAIN - OPTIMIZED CONTROL (FH = 30 MIN) - CAP. TREATMENT PLANT = 3600 M3/HR**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 7254 | 32371 | 341837 | 201075 | 44581 | 406567 | 45220 | 86477 | 226721 | 60049 | 65100 | 32962 | 126377 | 122894 | 1799485 |
| Max | 1831 | 3934 | 39572 | 10695 | 7486 | 26033 | 5312 | 12529 | 24361 | 6892 | 6240 | 4631 | 10056 | 12411 | 171983 |
| Num | 41 | 62 | 73 | 152 | 63 | 78 | 73 | 68 | 87 | 82 | 91 | 71 | 108 | 97 | 82 |
| Avg | 177 | 522 | 4683 | 1323 | 708 | 5212 | 619 | 1272 | 2606 | 732 | 715 | 464 | 1170 | 1267 | 1534 |

**HOMOGENEOUS RAIN - OPTIMIZED CONTROL (FH = 30 MIN) - CAP. TREATMENT PLANT = 4400 M3/HR**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 6236 | 24709 | 253560 | 194103 | 32728 | 151840 | 33235 | 52633 | 151559 | 41422 | 43994 | 19262 | 99562 | 90023 | 1194866 |
| Max | 1820 | 3728 | 36663 | 10665 | 7034 | 14184 | 4823 | 11455 | 21952 | 6290 | 5585 | 4032 | 9239 | 11360 | 148830 |
| Num | 30 | 49 | 53 | 148 | 42 | 45 | 51 | 46 | 52 | 54 | 57 | 40 | 89 | 66 | 59 |
| Avg | 208 | 504 | 4784 | 1312 | 779 | 3374 | 652 | 1144 | 2915 | 767 | 772 | 482 | 1119 | 1364 | 1441 |

**HOMOGENEOUS RAIN - OPTIMIZED CONTROL (FH = 60 MIN) - CAP. TREATMENT PLANT = 4400 M3/HR**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 7199 | 27462 | 271331 | 193428 | 33968 | 147951 | 36653 | 49328 | 143302 | 39013 | 41721 | 18874 | 98049 | 85887 | 1194166 |
| Max | 1858 | 3787 | 37629 | 10665 | 7094 | 13975 | 5021 | 11189 | 21566 | 6188 | 5490 | 3965 | 9141 | 11096 | 148664 |
| Num | 42 | 54 | 58 | 145 | 46 | 50 | 53 | 41 | 48 | 50 | 55 | 38 | 88 | 62 | 59 |
| Avg | 171 | 509 | 4678 | 1334 | 738 | 2959 | 692 | 1203 | 2985 | 780 | 759 | 497 | 1114 | 1385 | 1415 |

**HOMOGENEOUS RAIN - OPTIMIZED CONTROL (FH = 30 MIN) - CAP. TREATMENT PLANT = 5800 M3/HR**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 5971 | 23021 | 233152 | 193215 | 29980 | 49196 | 29398 | 34500 | 112098 | 29932 | 38770 | 9634 | 95777 | 72280 | 956924 |
| Max | 1820 | 3610 | 35916 | 10665 | 6896 | 5466 | 4559 | 9588 | 18484 | 5390 | 5294 | 3047 | 9047 | 9911 | 129693 |
| Num | 30 | 43 | 47 | 145 | 34 | 34 | 43 | 20 | 37 | 34 | 50 | 12 | 88 | 56 | 48 |
| Avg | 199 | 535 | 4961 | 1333 | 882 | 1447 | 684 | 1725 | 3030 | 880 | 775 | 803 | 1088 | 1291 | 1402 |

**RAINFALL REDUCTION (DIST.MODEL 1) - LOCAL CONTROL - CAP. TREATMENT PLANT = 5800 M3/H**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 6707 | 46943 | 407539 | 260372 | 25963 | 485622 | 53452 | 61084 | 224369 | 48395 | 67473 | 17651 | 116932 | 98095 | 1920597 |
| Max | 1860 | 3685 | 37283 | 10773 | 5671 | 11571 | 4670 | 10435 | 21255 | 5718 | 5414 | 3695 | 7970 | 9163 | 139163 |
| Num | 31 | 105 | 98 | 204 | 36 | 240 | 99 | 45 | 102 | 79 | 111 | 34 | 138 | 107 | 102 |
| Avg | 216 | 447 | 4159 | 1276 | 721 | 2023 | 540 | 1357 | 2200 | 613 | 608 | 519 | 847 | 917 | 1175 |

**RAINFALL REDUCTION (DIST. MODEL 1) - OPTIMIZED CONTROL (FH = 30 MIN) - CAP. T. PL. = 4400 M3/H**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 6129 | 24416 | 249155 | 193993 | 22384 | 138539 | 32587 | 51499 | 147048 | 40376 | 43189 | 18433 | 68126 | 61984 | 1097858 |
| Max | 1820 | 3719 | 36757 | 10665 | 6012 | 13814 | 4757 | 11248 | 21645 | 6197 | 5571 | 4021 | 7885 | 9506 | 143617 |
| Num | 30 | 48 | 52 | 146 | 35 | 43 | 49 | 43 | 50 | 52 | 55 | 38 | 64 | 52 | 54 |
| Avg | 204 | 509 | 4791 | 1329 | 640 | 3222 | 665 | 1198 | 2941 | 776 | 785 | 485 | 1064 | 1192 | 1414 |

**RAINFALL REDUCTION (DIST. MODEL 2) - LOCAL CONTROL - CAP. TREATMENT PLANT = 5800 M3/H**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 2835 | 27951 | 267405 | 189552 | 25963 | 388783 | 37681 | 60193 | 224369 | 48395 | 67473 | 17651 | 167326 | 153844 | 1679421 |
| Max | 1410 | 3124 | 32242 | 9366 | 5671 | 10135 | 4009 | 10402 | 21255 | 5718 | 5414 | 3695 | 9329 | 11110 | 132880 |
| Num | 15 | 71 | 75 | 173 | 36 | 207 | 80 | 45 | 102 | 79 | 111 | 34 | 171 | 141 | 96 |
| Avg | 189 | 394 | 3565 | 1096 | 721 | 1878 | 471 | 1338 | 2200 | 613 | 608 | 519 | 979 | 1091 | 1119 |

**RAINFALL REDUCTION (DIST. MODEL 2) - OPTIMIZED CONTROL (FH = 30 MIN) - CAP. T.PL. = 4400 M3/H**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tot | 2422 | 11686 | 142518 | 102116 | 16554 | 82443 | 19576 | 40638 | 122564 | 34123 | 35927 | 13331 | 79458 | 73654 | 777010 |
| Max | 1132 | 2651 | 27117 | 7250 | 4961 | 13240 | 3650 | 9481 | 18176 | 5302 | 4563 | 2924 | 7259 | 9461 | 117167 |
| Num | 16 | 25 | 35 | 90 | 24 | 29 | 34 | 34 | 46 | 44 | 50 | 26 | 75 | 58 | 42 |
| Avg | 151 | 467 | 4072 | 1135 | 690 | 2843 | 576 | 1195 | 2664 | 776 | 719 | 513 | 1059 | 1270 | 1295 |

Tot= total [m3]; Max= maximum overflow volume [m3]; Num= Number of overflows; Avg= average overflow volume [m3]