OPTIMAL MANAGEMENT AND OPERATIONAL CONTROL OF URBAN SEWER SYSTEMS

by

Upaka Sanjeewa Rathnayake

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Department of Civil and Environmental Engineering
University of Strathclyde
Glasgow, UK

DECLARATION

Signed:

Date:

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DEDICATION

This thesis is dedicated to all the good people in this world.

ABSTRACT

Combined sewer networks control, like many other real world problems, is usually identified with competing and conflicting objectives. Decision makers have a great need of selecting the best possible control strategy in minimizing the combined sewer overflows (CSOs) when controlling the sewer networks. However, this control strategy should be cost effective to produce a feasible control approach in real world. Cost effectiveness has become significantly important in present economic recession.

Over the past decades, people have witnessed the control strategies based on minimization of CSOs. However, it is now, not only to minimize CSOs, but also to minimize the impact to the natural water from these CSOs. Therefore, this research explores the development of a holistic framework that is used for the multi-objective optimization of urban wastewater systems, considering flows and water quality in combined sewers and the cost of wastewater treatment.

Pollution levels of several water quality parameters in dry weather flows and stormwater runoff are considered. Pollutographs for several water quality parameters are generated for the stormwater runoff. Temporal and spatial variations of the stormwater runoff are incorporated using these pollutographs for different land-uses. Furthermore, pollutographs are developed for different storm conditions, including single, two consecutive and migrating storms.

Evolutionary algorithms are extensively used in solving the developed multi-objective optimization approach. Formulations for two different optimization approaches, one for the snapshot optimization and the other one for the dynamic optimization are developed. Simulation results from a full hydraulic model, including water quality routing are used in the optimization. The performance of the multi-objective optimization models are tested on a simple interceptor sewer system for several storm conditions. The proposed optimization approach for snapshot optimization gives the optimal CSO control settings where a single set of static control settings is used throughout the considered time period. However, the

proposed optimization approach for dynamic optimization is capable of producing control strategies over the full duration of storm period.

Furthermore, results for a number of alternative formulations in constraint handling for the developed multi-objective optimization approach are compared. They produce interesting findings. Overall, the constraint handling formulations developed outside the genetic (NSGA II) algorithm provides better control in combined sewer networks. In addition, the results of the multi-objective optimization demonstrate the benefits of the usage of optimization approach and its potential to establish the key properties of a range of control strategies through an analysis of the various trade-offs involved. Solutions from the dynamic optimization approach highlight the usage of the real-time control in combined sewer systems. Given that the technology is there to measure water quality and flow rates, collect data and send feedbacks to the sewer system through central processing unit and the usage of high performance computers, the developed optimization model is capable of handing the present society's concerns in combined sewer systems. The model is capable of controlling the existing sewer networks according to the receiving water regulations and the fund availability of the wastewater treatment plants. However, further research is required to apply the developed multi-objective optimization approach in real-time control of urban sewer systems.

LIST OF PUBLICATIONS ARISING FROM THIS WORK

- Rathnayake, U., and Tanyimboh, T.T. (2012) "Multi-objective optimization of urban wastewater systems." *HIC 2012 - 10th International Conference on Hydroinformatics*, Hamburg, Germany.

- Rathnayake, U., and Tanyimboh, T.T. (2012) "Integrated Optimal Control of Urban Sewer Systems." *IWA WCE 2012 – World Congress on Water, Climate and Energy,* Dublin, Ireland.

- Rathnayake, U., and Tanyimboh, T.T. (2012) "Optimal control of combined sewer systems using SWMM 5.0." *Transactions on The Built Environment,* 122, pp 87 – 96. (Urban Water 2012 – 1st International Conference on the Design, Construction, Maintenance, Monitoring and Control of Urban Water Systems, New Forest, United Kingdom, 25th – 27th April 2012)

THESIS CONTRIBUTION

'Optimal management and operational control of urban sewer systems' presents a novel control strategy to the existing combined sewer systems. This control strategy is capable of minimizing the environmental concerns due to combined sewer overflows (CSOs) and the economic concerns at the wastewater treatment plants. The developed control strategy handles a multi-objective optimization problem, based on the receiving water quality of CSOs and the treatment cost of the wastewater treatment plant. Given that the technology is there to measure the water quality and flow rates in real-time and the usage of high performance computers to reduce the simulation times, the solutions of the developed multi-objective optimization module demonstrate the applicability of the controlling strategy for the real-time control. Therefore, this thesis contributes in identifying potential solutions to an appealing problem (CSOs and their bad impact) in most of the countries in the world to-date. However, further research and the support from other governing bodies in real-time control are needed in applying the developed optimal control strategies in real world.

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NOTATIONS

α_n - Crowded-comparison operator in NSGA II

ΔVol - The net volume flows through the node over the time step

β – Space factor in simulated binary crossover in NSGA II

r **-** Rate of generation or depletion of pollutant inside the reactor (kg/m^3, mg/l)

θ_H **-** Hydraulic retention time (day, s)

$\Re$ – A random number

A - Flow cross sectional area in the conduit

A_1 - Cross sectional area at upstream end of the conduit (m^2)

A_2 - Cross sectional area at downstream end of the conduit (m^2)

A_C - Surface area of the CSO chamber

A_{store} - Surface area of node

$\bar{A}$ - Average flow cross sectional area in conduit (m^2)

$\sum A_s$ - Surface area contributed by the conduits connected to the node

BOD - Five-day biochemical oxygen demand

c_1 & c_2 – Children solutions are the simulated binary crossover in NSGA II

COD - Chemical oxygen demand

C_{BOD} – Concentrations five-day biochemical oxygen demand

C_{COD} – Concentrations chemical oxygen demand

C_{in}**-** Inflow concentration of the pollutant to the reactor (kg/m^3, mg/l)

C_{NOX} - Concentrations nitrates and nitrites

C_{out} **-** Outflow concentration of the pollutant to the reactor ($kg/m^3, mg/l$)

C_{PS} - Cost of operation and maintenance of a primary sedimentation tank

C_T - Total cost of treatment (€/year)

C_{TKN} - Concentrations total Kjeldahl nitrogen

C_{TSS} – Concentrations total suspended solids

d – Depth of water in subcatchment

d_p – Maximum depression storage

$F1$– First objective function

$F2$– Second objective function

F_1 - Combined population at 1st generation in NSGA II

g - Gravitational acceleration

h_L - Local energy loss per unit length of conduit

$h_{i,t}$ – Water level in i^{th} sewer chamber at time t

h_s - Water level of the storage tank

H - Hydraulic head / Elevation head plus any possible pressure head

H_i – Spill level of i^{th} sewer chamber

H_1 - Head at upstream node of conduit (m)

H_2 - Head at downstream node of conduit (m)

$I_{i,t}$ – Catchment inflow to node i at time t

k - 1.49 for US units and 1.0 for metric units

k_0 **-** Rate constant (day^{-1}, s^{-1})

K - Local loss coefficient at location x

K_i - Loss coefficient at location i along the conduit

L - Conduit length (m)

m **-** Mass flow (kg, g, mg)

n - Manning roughness coefficient

n_d – Distribution index in NSGA II

n_m - Mutation index

n_0 - Number of interceptor nodes or CSO chamber points

N - Population of size in NSGA II

NOX - Nitrates/nitrites

$O_{i,t}$ – Combined sewer over flow discharge at node i at time t

p_1 & p_2 – Parent solutions in simulated binary crossover in NSGA II

$P_{i,t}$ - Pollution load to the receiving water from the i^{th} CSO chamber at time t

P_m - Mutation probability

P_0 – Initial parent population in NSGA II

P_{t+1} – Subsequent patent population in NSGA II

q - Sewer discharge through the conduit

$q_{i,t}$ – Through flow in interceptor sewer at node i at time t

$q_{max,i}$ - Maximum flow rate at i^{th} conduit

q_s - Flow to the storage tank from CSO chamber

q_T - Amount of treated wastewater (m^3/day)

$\sum q$ - Net inflow to the node

$Q_e(t)$ – Flow rate in EQI

$Q_{i,t}$ – Flow from i^{th} sewer chamber to interceptor node i at time t

Q_0 - Offspring population at 1st generation in NSGA II

Q_{PS} - Wastewater volume flow rate in the primary sedimentation tank

Q_R **-** Flow rate in and out from the reactor (m^3/day, m^3/s)

Q_s – Surface runoff from subcatchment

Q_{t+1} – Subsequent offspring in NSGA II

R- Hydraulic radius of the flow cross section (m)

$\overline{R}$ - Average hydraulic radius in conduit (m)

S_F - Friction slope of pipe

t - Time

t_f– Final time step for EQI calculations

t_0 – Initial time step for EQI calculations

T - Last time step of the hydraulic simulation

TKN - Total Kjeldahl nitrogen

TSS - Total suspended solids

x - Longitudinal distance along the conduit

V- Flow velocity; q/A (m/s)

$\overline{V}$ - Average velocity in conduit (m/s)

V_i - Flow velocity at location i along the conduit (m/s)

V_R **-** Volume of the CSTR reactor (m^3, l)

Y_N - New value of the decision variable after mutation

Y_0 - Original value of the decision variable before mutation

CHAPTER 1

INTRODUCTION

1.1 Background

Stress on natural water bodies is increasing day by day with ongoing urbanization. Providing clean water according to the increasing demand is not an easy task. In addition, collection and safe transportation of wastewater to the treatment facilities are challenging tasks for most of the cities. Even though, there is enormous number of migrants to the urban areas, there is little space for infrastructure developments in wastewater collection and treatment systems.

Urbanization strongly incorporates with increasing stormwater runoff and deterioration of water quality. Newly developed lands increase impermeability of the ground and lead to increase the runoff. In addition, global warming has changed the usual climatic patterns and induced some strange climatic changes (Short et al, 2012). Short et al. (2012) have discussed the adaptation of urban water systems in a changing climate in detail. The uncertainties associated with climate change and their potential impacts on water delivery systems have been presented. Therefore, it is evident that the climate change can influence the water supply and sewer systems. In some parts of the world, this change has increased the precipitation and that leads to increase the runoff. These runoffs usually carry number of pollution sources, including nutrients, oil, grease, pesticides and many others. Drainage networks are designed to capture and transport these runoffs to the treatment plants.

Urban drainages can be categorized into two types, combined and separated. Separated systems have two different networks to carry stormwater and wastewater separately to the treatment plants. However, except some cities and in a country like Australia, all others have the combined systems. These combined systems gather and transport rainwater runoff, domestic wastewater with sewage, and industrial wastewater in the same pipe/drainage.

Combined sewer systems have capacity limitations and have not designed to carry stormwater runoff. During the dry weather periods these combined sewer networks are the passage to gather and transport domestic and industrial wastewater to the treatment plant. However, during the rainy seasons, additional stormwater enters to the combined system. Because of the capacity limitations, a significant amount of wastewater overflows during the storms. These overflows are combined sewer overflows (CSOs) and finally reach the natural water bodies without treatment. Therefore, they make significant environmental concerns. In addition, as it is stated above, urbanization plays an important role for CSOs. Especially urbanization without proper planning leads to have changes in land-use and then to increase the CSOs in most of the cities in the developing world (Kannapiran et al., 2008).

These CSOs are a major pollutant supplier to natural water bodies. However, many countries, including the United States of America and European Union countries, have strengthened the water quality regulations for CSO discharges during the last decade. In the United States, Environmental Protection Agency (US EPA) has implemented a long-term control plan for larger cities, by reducing to a maximum of four overflows per annum and a minimum treatment percentage of 85% from the total combined sewer volume before it reaches the receiving waters (Pleau et al., 2005). Even though, there are enough regulations in most of the countries, significant number of CSOs can be seen in the rainy season.

Figure 1.1 CSO on Platt Avenue in West Haven, Connecticut, April 23, 2006 (Zurcher, 2006)

Figure 1.2 CSO at San Francisco beach (Tehdely, 2011)

Figures 1.1 and 1.2 are couple of photographic examples of CSOs. It is well understood that the quality of receiving water is in danger due to these sudden CSOs and the following photographic example presents the impact of CSOs to the aquatic life.

Figure 1.3 Damage to the aquatic wildlife in river Thames due to CSOs (Utility Week, 2011)

According to "Utility Week News" on June 8th, 2011, a total of 500,000 tones of storm sewerage were released into the river Thames, following a heavy rainfall. This resulted in death of a large number of fish and other aquatic wildlife. This was due to the lowered dissolved oxygen level in the river (Utility Week News, 2001). This is a fine example to illustrate the impacts of CSOs.

Most of the developed countries have given up of constructing new combined sewer systems, but still have the constructed combined sewers in their cities. High construction cost and disruptions to the inhabitants have made difficulties in making these combined sewer systems separated. Therefore, designers, planners and authorities have been forced to search some alternative methods to deal the problems in combined sewer systems (Darsono and Labade, 2007). Finding a sustainable solution is still a challenging task. However, the optimal control of existing combined sewer systems is a potential solution and some researchers have shown the importance of optimal control of sewer systems over the past decade. Controlling the sewer network to minimize the CSO volume and number of CSOs was a discussed optimization strategy. However, concerns over the receiving water qualities and the pollution load to receiving water from CSOs have developed interests in considering a water quality based approach to control the existing combined sewer systems.

Therefore, the ultimate goal of a well planned, operated and maintained combined sewer system is to drain wastewater in a sanitized way during dry, wet and storm weather periods. Avoiding flooding from drains during the storm weather conditions and in addition, minimizing capital and maintenance costs are the further goals (Schutze et al., 2002a; Kannapiran et al., 2008).

Therefore, this study develops a novel optimal controlling strategy to the existing combined sewer systems considering the water quality of both combined sewer system and the receiving water bodies (usually the rivers) and the cost of wastewater treatment at the treatment plant. In addition, suggestions for further research are presented in detail.

1.2 Aims and Objectives of the Research

The primary aim of this research is to develop a novel optimal control strategy to the existing combined sewer networks in minimizing the environmental concerns due to the CSOs. However, it is understood that there is little literature on the optimal control on combined sewer systems respect to the growing environmental concerns. Identifying the objectives of the optimization process is itself a primary objective. Initial literature review on the PhD thesis was carried out in identifying these objectives. The developed control strategy should be able to assist the sewer controllers to have the optimal control settings according to the fund availability and the environmental regulations. Several objectives have been identified in achieving the above stated aim and they are

1 To identify the best objective functions among the others to develop the multi-objective optimization approach in controlling existing combined sewer networks.
2 Then to minimize or (if possible) to avoid combined sewer overflows in urban wastewater systems in storm weather periods

3 At the same time to minimize the pollution load to the receiving water from CSOs. In other words, to ensure or to increase the water quality standards in the wastewater discharge
4 Minimize the cost of operation and maintenance in wastewater treatment. This is a competing objective to the 2nd and the 3rd objectives stated above. However, ‘cost’ is one of the important objectives in most of the optimization problems.
5 To optimize the integration of above mentioned 2nd, 3rd and 4th objectives to develop the multi-objective optimization approach in optimal control of existing combined sewer networks.

The above stated 2nd, 3rd and 4th objectives were achieved by developing and solving a novel multi-objective optimization approach considering the pollution load to the receiving water from CSOs and the cost of wastewater treatment.

1.3 Scope of the research

Sewer (combined or separated) networks and wastewater treatment plant are the main components of an urban wastewater system. Optimal control of both sewer network and wastewater treatment plant is difficult. This is mainly because of the non-linearity of both systems (Shi and Qiao, 2010). Hydraulics and water quality of a sewer network are usually governed by the partial differential equations and therefore, difficult to solve for the solutions. Governing equations in wastewater treatment plants never second to the sewer systems. Therefore, it is understood that the optimal control of the urban wastewater systems is one of the difficult optimization problems, due to the computational and modeling difficulties. Therefore, this thesis work involves only the combined sewer network. Optimal control strategies based on multi-objective optimization on combined sewer networks are presented. However, when introducing the multi-objective optimization approach, some of the aspects of wastewater treatment plant and the receiving water were incorporated. In other words, minimizing pollution load from CSOs incorporates the receiving water qualities and minimizing wastewater treatment cost incorporates some of the economic aspects of the wastewater treatment plants..

1.4 Structure of the thesis

This thesis consists of 8 chapters and several appendices. Following the Introduction chapter which includes the background, aims and objectives and scope of the research presented earlier, the thesis is arranged as follows.

Chapter 2 presents the hydraulic and water quality analysis of the urban drainages. The governing equations and solution techniques for hydraulic simulations and the water quality routing are presented in detail. In addition, types of simulations and the capabilities of the Storm Water Management Model, SWMM 5.0 are discussed in this chapter.

Chapter 3 gives a detailed review of multi-objective optimization. Various solution techniques available in finding optimal solutions to multi-objective optimization approaches are presented. At the end of the chapter a detailed review of genetic algorithms and Non Sorted Genetic Algorithm, NSGA II is presented.

Chapter 4 gives a detailed literature review of controlling of urban drainage systems. It starts with design considerations of a simplified sewer system. Then, the chapter proceeds to the control aspects. This includes the previous work in optimal control of combined sewer systems and their notable outcomes to the research world and importance of water quality in urban sewer systems. The chapter concludes with two important identifications for the research, water quality at sewers and receiving water and the cost of wastewater treatment.

Chapter 5 describes pollution levels in the storm water runoff and wastewater. Different land-uses in catchments and their wash off concentrations for storm water are presented in detail. Pollutographs for single storm, migrating upstream and downstream storms and two consecutive storms are generated. Furthermore, the effect of the first flush phenomenon is presented in generating the pollutographs for different pollutants.

Chapter 6 develops the multi-objective optimization approach in controlling the combined sewer systems. Development of objective functions, various constraints and constraint handling techniques are discussed in detail. Results from a case study are presented in obtaining optimal solutions for snapshot optimization approach and compared for two different constraint handling approaches.

Chapter 7 further extends Chapter 6 in obtaining optimal solution for the dynamic optimization approach over the entire storm period. Results from several storm conditions, including single storm, migrating storms and two consecutive storms are presented and again compared for the two constraints handling approaches.

Chapter 8 summarizes the main findings. In addition, suggestions for future research are presented. Furthermore, the feasibility of the developed multi-objective optimization approach in real-time control is discussed.

Several appendices are presented at the end of the thesis, including the publications that arose from this research and some useful results of the three major chapters of the thesis, i.e. Chapter 5, 6 and 7.

CHAPTER 2

HYDRAULIC AND WATER QUALITY ANALYSES OF URBAN DRAINAGE

2.1 Introduction

Hydraulic simulation models play an important role in designing and controlling urban drainage networks. These computer models have drastically reduced the simulation times compared to manual calculations. United States Environmental Protection Agency's Stormwater Management model, SWMM is such a hydraulic and water quality routing model for urban drainages. This chapter presents the governing equations for hydraulic and water quality of urban drainage systems. In addition, a detailed description of the SWMM simulation model, including capabilities of it and types of simulations available in SWMM are presented.

2.2 Hydraulics of urban drainage systems

Following subsections provide the governing equations for the flow inside the sewers and the corresponding solution techniques.

2.2.1 Governing equations for flow routing

Gradually varied unsteady flow inside sewers is hydraulically routed using the mass conservation and momentum equations. This set of governing equations in one dimensional mode is the one dimensional Saint Venant flow equations. Equation 2.1 gives the continuity equation for the flow along an individual conduit.

$$\frac{\partial A}{\partial t}+\frac{\partial q}{\partial x}=0 \tag{2.1}$$

where,

A - flow cross sectional area

q - sewer discharge through the conduit

t - time

x - longitudinal distance along the conduit

Equation (2.2) is the momentum equation for the flow along an individual conduit.

$$\frac{\partial q}{\partial t}+\frac{\partial\left(q^2/A\right)}{\partial x}+gA\frac{\partial H}{\partial x}+gAS_F+gAh_L=0 \tag{2.2}$$

where,

g - gravitational acceleration

H - hydraulic head / Elevation head plus any possible pressure head

h_L - local energy loss per unit length of conduit

S_F - friction slope of pipe

2.2.2 Solution technique of the flow governing equations

The following subsection gives a detailed explanation to the solution techniques of the governing equations given in the preceding subsection. The friction slope (S_F) in Equation (2.2) is expressed using the Manning equation and shown in the Equation (2.3).

$$S_F=\frac{n^2V\left|V\right|}{k^2R^{4/3}} \tag{2.3}$$

where,

k - 1.49 for US units and 1.0 for metric units

n - Manning roughness coefficient

R- hydraulic radius of the flow cross section

V- flow velocity

The local energy loss (h_L) in Equation (2.2) can be expressed as a velocity head and shown in the Equation (2.4).

$$h_L = \frac{KV^2}{2gL} \tag{2.4}$$

Where,

K - local loss coefficient at location x

L - conduit length

The initial conditions for H and q at time t=0 and the boundary conditions at x=0 and x=L for all times t=t should be needed to solve the above-mentioned Equations (2.1) and (2.2) over a single conduit. An additional equation as shown in Equation (2.5) is introduced at the junction nodes for the continuity. It is assumed herein that a continuous water surface is maintained (without any jumps) between the water elevation at the node and in the conduits that enter and leave the node. The change in hydraulic head (H) at a particular node with respect to time is expressed as follows.

$$\frac{\partial H}{\partial t} = \frac{\sum q}{A_{store} + \sum A_s} \tag{2.5}$$

Where,

A_{store} - surface area of node

$\sum A_s$ - surface area contributed by the conduits connected to the node

$\sum q$ - net inflow to the node

Equations (2.1), (2.2) and (2.5) are solved using the finite difference technique to compute the flow in conduits and head at nodes at time t+Δt as the functions of known values at time *t*. Therefore, the flow in conduits can be expressed as shown in Equation (2.6).

$$q_{t+\Delta t} = \frac{q_t + \Delta q_{gravity} + \Delta q_{inertial}}{1 + \Delta q_{friction} + \Delta q_{losses}} \tag{2.6}$$

Δq terms in Equation (2.6) can be further expanded as shown in the following Equations (2.7) to (2.10).

$$\Delta q_{gravity} = \frac{g\bar{A}\left(H_1 - H_2\right)\Delta t}{L} \tag{2.7}$$

$$\Delta q_{inertial} = 2\bar{V}\left(\bar{A} - A_1\right) + \frac{\bar{V}^2\left(A_2 - A_1\right)\Delta t}{L} \tag{2.8}$$

$$\Delta q_{friction} = \frac{gn^2\left|\bar{V}\right|\Delta t}{k^2\bar{R}^{4/3}} \tag{2.9}$$

$$\Delta q_{losses} = \frac{\sum_i K_i \left|V\right|_i \Delta t}{2L} \tag{2.10}$$

where,

$\bar{A}$ - average flow cross sectional area in conduit

A_1 - cross sectional area at upstream end of the conduit

A_2 - cross sectional area at downstream end of the conduit

H_1 - head at upstream node of conduit

H_2 - head at downstream node of conduit

K_i - loss coefficient at location i along the conduit

$\bar{R}$ - average hydraulic radius in conduit

$\bar{V}$ - average velocity in conduit

V_i - flow velocity at location i along the conduit

Head at nodes / junctions for the successive time step can be obtained from the solution of the Equation (2.5) and shown in the Equation (2.11)

$$H_{t+\Delta t} = H_t + \frac{\Delta Vol}{\left(A_{store} + \sum A_s\right)_{t+\Delta t}} \tag{2.11}$$

where ΔVol is the net volume flows through the node over the time step. This can be further expressed as shown in the following equation.

$$\Delta Vol = 0.5\left|\left(\sum q\right)_t + \left(\sum q\right)_{t+\Delta t}\right|\Delta t \tag{2.12}$$

Equations (2.6) and (2.11) are solved using a method of successive approximations to obtain the flow rates and flow heads.

2.3 Water quality routing in urban drainage systems

Sewer / stormwater conduits are assumed to be behaving as continuously stirred tank reactor models (CSTR) for water quality modelling. Integration of conservation of mass equation is used to calculate the concentration of each water quality parameter leaving the particular conduit, at the end of the time step. The water quality routing principle described above is the same with the storage unit nodes such as tanks. However, the nodes without volumes use simply the mixture of all water quality concentrations, which enter the particular node. Figure 2.1 shows a simplified CSTR model.

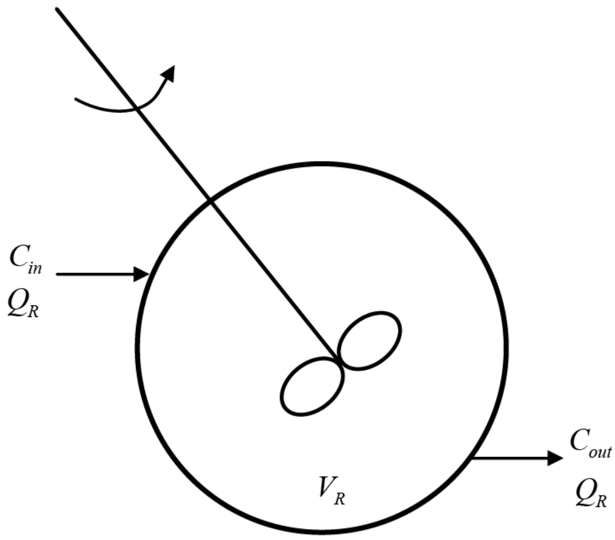

Figure 2.1 Simplified CSTR model

The governing equation for the CSTR models can be written as shown in the Equation (2.13)

$$\frac{dm}{dt} = Q_R\left(C_{in} - C_{out}\right) + \int_{V_R} r dv \tag{2.13}$$

where,

m - mass flow

Q_R - flow rate in and out from the reactor

C_{in}- inflow concentration of the pollutant to the reactor

C_{out} - outflow concentration of the pollutant to the reactor

r - rate of generation or depletion of pollutant inside the reactor

V_R - volume of the CSTR reactor

The reactor is in its steady state and the Equation (2.13) is solved inside SWMM 5.0 to obtain the concentration at the end of the reactor as shown in the following equations.

$$\frac{dm}{dt} = 0 \tag{2.14}$$

Therefore, the Equation (2.13) can be written in the following form.

$$\frac{Q_R\left(C_{in}-C_{out}\right)}{Q_R}+\frac{1}{Q_R}rV_R=0 \tag{2.15}$$

$$C_{in}-C_{out}+\frac{1}{Q_R}rV_R=0 \tag{2.16}$$

Further simplifications reduce the Equation (2.16) to the following equations.

$$C_{in}-C_{out}+r\theta_H=0; \qquad \theta_H=\frac{V_R}{Q_R} \tag{2.17}$$

where,

θ_H - hydraulic retention time

Assuming the 1st order reaction,

$$C_{in}-C_{out}-kC_{out}\theta_H=0; \qquad r=-k_0C_{out} \tag{2.18}$$

where,

k_0 - rate constant

After all these simple calculations, the final solution shown in Equation (2.19) can be obtained.

$$\frac{C_{out}}{C_{in}}=\frac{1}{1+k_0\theta_H} \tag{2.19}$$

2.4 Strom Water Management Model, SWMM 5.0

SWMM is a powerful hydraulic and water quality simulation model, which is capable of simulating stormwater runoff and routing processes, including water quality routing. The first version of SWMM was developed in 1971 by Water Supply and Water Resources Division National Risk Management Research Laboratory, United States Environmental Protection Agency. The latest version of SWMM is version 5 and it is the 5.0.022 amendment. This has a graphical user interface, where the user can graphically input the stormwater or sewer network data, edit the study area, insert various input data, run the hydrological, hydraulic and water quality simulations and finally examine the results in various formats. Program files, source codes and executables of SWMM simulation model can be freely downloaded from the US EPA website (http://www.epa.gov/nrmrl/wswrd/wq/models/swmm/).

SWMM is widely used all over the world for simulating rainfall and runoff for urban areas, including the water quality aspects for a single or long time events (Cambez et al. (2008), Fu et al. (2010), Shinma and Reis (2011), Vojinovic et al. (2008) and Zhang (2009)). Runoff and routing are two major components of SWMM 5.0. Runoff component collects precipitation and generate runoff with pollution loads through all the sub catchments. Routing component transports the generated runoff and pollutant load through pipes, channels and all other flow controlling devices.

The hydrological modeling includes time varying rainfall analysis, evaporation and infiltrations, snow melting and non-linear reservoir routing. The hydraulic modeling includes dynamic wave routing and kinematic wave routing of flow through open or close conduits, application of flow controllers and more importantly modeling any size of network. The water quality modeling includes pollutant build-up, pollutant wash-off including user defined treatment facilities.

Because of all of these capabilities, SWMM is a well-known simulation model for controlling and designing stormwater networks and combined sewers.

2.5 Hydraulic, water quality and hydrological formulations of SWMM 5.0

Governing equations given in the section 2.2 and 2.3 present the hydraulic and water quality formulations of SWMM 5.0 respectively. Therefore, this section presents the hydrological formulations.

2.5.1 Surface runoff from catchments

Figure 2.2 illustrates the conceptual view of surface runoff. Subcatchments are treated as non-linear reservoirs. Inflows to the subcatchments are either from precipitations or upper subcathments. However, there are several ways of outflows from the catchments, including evaporation, infiltration and surface runoff. Surface runoff (Q_s) from a particular subcatchment occurs when the depth of water (d) exceeds the maximum depression storage (d_p). Therefore, the surface runoff is treated as an overflow. This overflow is formulated using the Manning's equation.

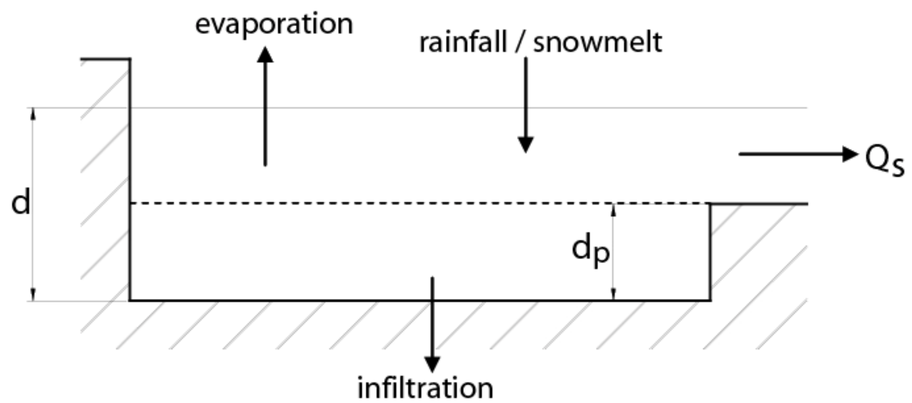

Figure 2.2 Conceptual view of surface runoff (Rossman, 2009)

Infiltration from the subcatchments can be calculated in three methods. Horton's method (Mays, 2005), assumes the infiltration decreases exponentially from a maximum rate to a minimum rate over the course of rainfall event. Users have to input the maximum and minimum rates of infiltration, the decay coefficient that describes the decreasing rate over the time and the time taken to dry the fully saturated soil. Green-Ampt method (Mays, 2005) assumes a sharp wetting front existing in the soil which separate soil with some initial moisture content below the saturated soil. Initial moisture deficit of the soil, hydraulic conductivity and the suction head at wetting front are the user input parameters. The last method is Curve number method (USDA-SCS, 1985). This method uses the “Soil Conservation Service”, (SCS) curve number to determine the surface runoff from a subcatchment. It assumes the total infiltration capacity is found from the curve number. Input parameters of this method are the curve number and the time taken to dry the fully saturated soil.

2.5.2 Pollutant build-up and wash-off

Pollutant build-up in a particular catchment is defined as the mass of pollutant per unit area of the catchment or mass per unit curb length (Rossman, 2009). This can be calculated in three different ways in SWMM 5.0. Defining the pollutant build-up as a power function is the first method. This assumes that the pollutant build-up accumulates proportionally to the some power of time, until it reached a maximum limit. Build-up according to an exponential curve is the second method. Saturation function method is the third and the last method. It assumes that the build-up begins at a linear rate and continuously declines with time until a saturation level is reached.

Pollutant wash-off from a catchment starts with the storm event. Pollutant wash-off varies with the land use of the catchment. It is measured by mass per hour. Wash-off can be formulated in three methods in SWMM 5.0. Exponential wash-off is the first method. Wash-off load is assumed to be proportional to a power of surface runoff and the remaining amount of build-up. Rating curve wash-off is the second method and it assumes that the rate of wash-off (mass per second) is proportional to a power of surface runoff. Event mean concentration is a special case of the rating curve method. However, this gives the wash-off pollutant concentration in mass per litre.

2.6 Types of simulations available in SWMM 5.0

2.6.1 Steady flow routing

This is the simplest possible flow routing method in SWMM 5.0. The flow is assumed to be steady and uniform within the each computational time step. Therefore, the upstream flow is directly transferred to the downstream conduit without any delay. The governing equations behind this steady flow routing method is simply the continuity equation and the Manning's equation. However, this method cannot be used to find the channel storage, backwater effects, losses during entrance and exit and in pressurized flows.

2.6.2 Kinematic flow routing

Continuity equation and a simplified form of momentum equation are used in this flow routing method. The maximum flow, which a conduit can bear, is the full normal flow value. When there is an excess flow, it is either lost from the system or ponded at the inlet node and then release to the system later as capacity available. Unlike the steady flow routing, kinematic flow routing allows flow and area to vary in both spatially and temporally inside the conduit. This leads to have delayed outflows. However, this method cannot be accounted for backwater effects, losses and pressurized flows.

2.6.3 Dynamic wave routing

This flow routing method solves the above stated (subsection 2.2.1) complete one-dimensional Saint Venant flow equations. Therefore, this method produces more accurate flow results. Since this method solve full momentum equations, backwater effects, losses at entrance and exits and pressurized flows can be handled. Any general network layout containing several downstream divisions can be handled using dynamic wave routing. However, in common, all these three routing methods use Manning's equation to relate the flow rate to flow depth and friction slope.

More information about the SWMM 5.0 simulation model can be found in Rossman (2009).

2.7 Summary and conclusions

This chapter has presented the flow and water quality formulations in urban drainages and their solution techniques. In addition, a detailed explanation of SWMM 5.0 has presented.

EPA SWMM 5.0 is powerful simulation model, which can be used in designing and controlling of urban stormwater / sewer networks. The model is capable of solving one-dimensional Saint Venant equations and simultaneously proceeding the hydrological and water quality analysis. Therefore, SWMM 5.0 is comprehensively used as the hydraulic and water quality simulator in chapters 6 and 7.

CHAPTER 3

OVERVIEW OF MULTI-OBJECTIVE OPTIMIZATION

3.1 Introduction

Chapter 3 gives a detailed overview of the multi-objective optimization. Various solution techniques, including classical methods and usage of evolutionary algorithms are presented.

The following example provides an overview of the effect of multi-objectives problem in real world problems. Civil Engineering Construction Company has number of design and construction considerations when it comes to design and construct a combined sewer network. Minimizing design and construction cost, minimizing pumping stations in the designed sewer network, maximizing safety when construction and in operation, maximizing system performance after construction and maximizing durability of the system are some of the considerations. However, the authorities, like municipal councils, would like to have a sewer network, which has minimum maintenance cost, minimum operation cost and minimum overflows during storm events. However, satisfying all these in one construction is impossible as some of these objectives conflict each other. Therefore, depending on the gravity of the objectives, the construction company has to come with several designs. Discussions among all-important parties would finally lead to a better solution among the proposed designs. Therefore, this practical example shows the importance of multi-objectives in real world problems.

3.2 Multi objective optimization

A generic single objective optimization problem can be mathematically formulated as follows

$$Min \ \ (orMax) \ f(x) \tag{3.1}$$

$$x \in S \tag{3.2}$$

where *f* is a scalar function and *S* is a set of constraints.

S can be defined as

$$S = \{g(x) = 0, h(x) \geq 0; x \in R\} \tag{3.3}$$

g(x) and *h(x)* are two functions of *x* and *R* is real numbers.

However, a multi-objective optimization problem can be mathematically formulated as follows

$$Min \ \ (orMax) \ \left[f_1(x), f_2(x), ..., f_n(x) \right] \tag{3.4}$$

$$x \in S \tag{3.5}$$

where $n > 1$ and *S* is the set of constraints as described in Equation (3.3).

Single objective optimization problems have only global one optimal solution, whereas multi-objective optimization problems have a set of optimal solutions. These set of optimal solutions are Pareto optimal solutions. A solution vector $x*$ ($x^* \in S$) is said to be Pareto optimal for a multi-objective optimization problem, if all other solution vectors x ($x \in S$) give higher values for at least to one of the objective functions f_i (i = 1, 2, …, n) or have the same value for all the objective functions. This definition is defined based on a minimizing multi-objective optimization problem. Formally, this definition can be presented as follows:

- A point x^* can be introduced as a weak Pareto optimum or a weak efficient solution for the multi-objective optimization problem, if and only if there is no solution x ($x \in S$) such that $f_i(x) < f_i(x^*)$ for all i, (i = 1, 2, …, n).

- A point x^* can be introduced as a strict Pareto optimum or a strict efficient solution for the multi-objective optimization problem, if and only if there is no solution x ($x \in S$) such that $f_i(x) \leq f_i(x^*)$ for all i, (i = 1, 2, …, n), with at least one strict inequality.

The graphical interpretation of these optimal solutions can be seen in the Figure 3.1. The shape of the Pareto optimal curve shows the trade-off between the considered two objectives of the multi-objective optimization problem. Solutions *A* and *B* are non-inferior or non-dominated solutions.

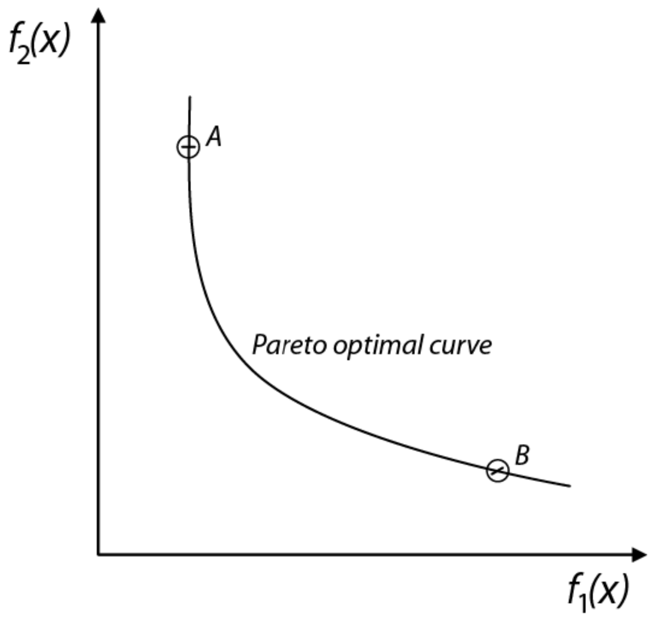

Figure 3.1 Example of a Pareto optimal curve

Figure 3.2 shows some examples of weak and strict Pareto optima. Solutions *A* and *E* are the weak Pareto optima whereas the solutions *B*, *C* and *D* are the strict Pareto optima. In other words, solutions *B* and *D* dominate the solutions *A* and *E* respectively.

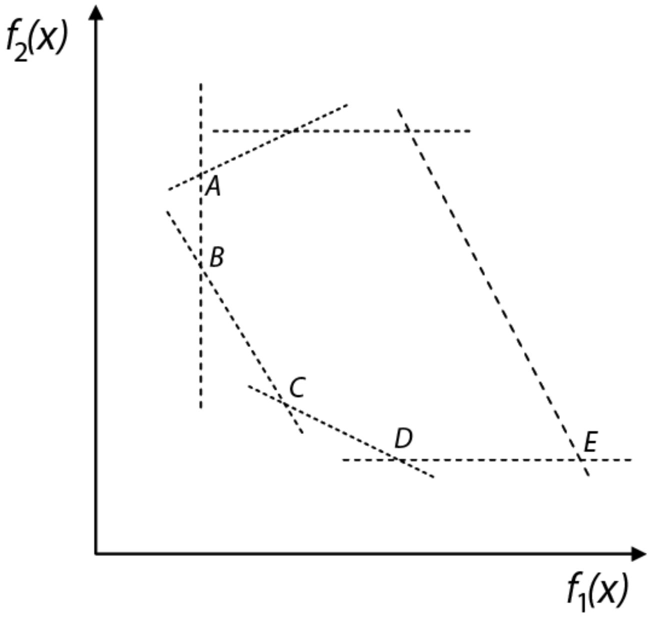

Figure 3.2 Example of weak and strict optima

3.3 Solution techniques to multi-objective optimization problems

Obtaining Pareto optimal curves is not straightforward and cannot be computed efficiently. Even though solutions can be obtained in theoretically, the computational difficulties have made the researchers to use the approximation methods (Caramia and Dell´Olmo, 2008). Following subsections give the various solution techniques to the multi-objective optimization problems.

3.3.1 Scalarization technique

Converting the multi-objective optimization problem into a single objective optimization problem and then solving it as a single objective optimization problem is the scalarization technique. Conversion can be done using some weighting factors among the multiple objectives. Each objective has to be given a weighting factor based on the priority of the objective and then to optimize the weighted sum of the objectives. The generic form of the multi-objective optimization problem given in Equations (3.4) & (3.5) is used to describe the scalarization technique.

$$Min \ \left[f_1(x), f_2(x), ..., f_n(x) \right] \tag{3.6}$$

$$x \in S \tag{3.7}$$

This minimizing multi-objective optimization problem is converted into a single objective optimization problem as shown in Equations (3.8) to (3.11).

$$Min \sum_{i=1}^{n} \gamma_i f_i(x) \tag{3.8}$$

$$\sum_{i=1}^{n} \gamma_i = 1 \tag{3.9}$$

$$\gamma_i > 0, \quad i = 1, 2, ..., n \tag{3.10}$$

$$x \in S \tag{3.11}$$

Where γ_i are the weighting factors.

However, obtaining accurate weighting factors for objectives is challenging (Caramia and Dell´Olmo, 2008). This is because of the complexity of the objective functions considered in optimization problems.

3.3.2 ε-constraint method

ε-constraint method is another way of solving the multi-objective optimization problems. In this method, all objectives are treated as constraints, but one as the objective function of the problem. These objective functions are constrained to be less than or equal to a given target value. Chankong and Haimes in 1983 have proposed this technique (Caramia and Dell´Olmo, 2008). Mathematically this method can be interpreted as given in the following equations (3.12) to (3.14).

$f_j(x)$ is kept as the objective function and the other objective functions are constrained.

$$Min\ f_j(x) \tag{3.12}$$

$$f_i(x) \leq \varepsilon_i \qquad \forall_i \in \{1, 2, ...n\} \setminus \{j\} \tag{3.13}$$

$$x \in S \tag{3.14}$$

Where ε_i is the target value.

3.3.3 Multi-level programming

Multi-level programming is another method of obtaining a single optimal solution in the entire Pareto surface. This method works according to the objective hierarchy. First, the minimizers of the most important objective function are found. Then, the minimizers of the second important objective function are found. This process continues until the last objective function. However, this method can only be used, if the hierarchical order among objectives is known and meaningful. More information about above discussed solution techniques can be found in Caramia and Dell´Olmo, (2008).

However, above stated all three methods find a single optimal solution to the multi-objective optimization problem. They can be handy if the trade-offs among the objectives are not so important to the users. However, having the trade-off between objectives is important in most of the real world problems.

3.3.4 Evolutionary computing

Evolutionary algorithms (EAs) have become a popular technique for solving optimization problems, when they are complex and difficult to solve using classical optimization solution techniques. EAs usually do not require a lot of information about the problem. In addition, they are relatively easy to implement. Furthermore, EAs produce robust solutions in less time, compared to the classical solution methods (Sbalzarini et al., 2000). EA in multi objective optimization gives a set of optimal solutions widely known as the Pareto optimal solutions to the optimization problem and that is a big advantage in solution techniques (Deb et al., 2002).

This method considers the integrity of the objectives and therefore, it is not essential to consider the relative importance among objectives. In other words, all objectives are treated together to obtain the optimal solutions in a single run. All the potential solutions are categorized as dominated and non-dominated ones and the solutions to the problem are the non-dominated ones (Deb et al., 2002).

Genetic algorithms (GAs) are under the category of evolutionary algorithms and simply search and optimization techniques used to find the exact or approximate solutions. Genetic algorithms come from a biological background, which involves mutation, selection and crossover. GAs give efficient and better solutions for the large scale optimization problems than classical methods (Zitzler and Thiele, 1998). Initially a set of candidate solutions is generated. In consecutive generations, solutions are selected from the individuals according to the ranks, which are based on the fitness of the solutions. Elitism can speed up the performance of the genetic algorithms significantly (Deb et al., 2002). It is the process of selecting better individuals. Elitism is important, since it allows solutions to get better over the time. In addition, GAs use two operators, crossover and mutation, to create a child population from the initial parent population. This procedure progresses until the algorithm reaches the maximum defined number of generations.

Important steps of GAs in finding optimal solutions to multi-objective optimization problems are described in the following sections.

Initialization

Individual solutions are randomly generated as the initial step to make the initial population. The population size may be few hundreds and usually it is defined by the user.

Selection

In each generation, a segment from the existing population is selected to generate the child population. This selection is basically based on the fitness of each individual. Literature shows two possible methods of selection. Roulette wheel selection method by Goldberg (1985) uses the probability equals to the normalized fitness values. Tournament selection method by Hancock (1994) involves running several tournaments among randomly chosen individuals. The winner of each tournament is selected.

Reproduction

Genetic operators, crossover and mutation are used to reproduce the next generation of solutions. Two parent solutions (chromosomes) are combined together to produce new solutions called offspring. As it is stated above, selection of parents are given the preference towards the fitness, therefore, the offspring is expected to be better (Konak et al., 2006). Mutation maintains the diversity from one generation of population to the next. The ultimate objective of these two operators is to find a better / converging set of solutions to the multi-objective optimization problem. Graphical interpretation of crossover and mutation operators based on binary coding can be seen from the Figures 3.3 and 3.4.

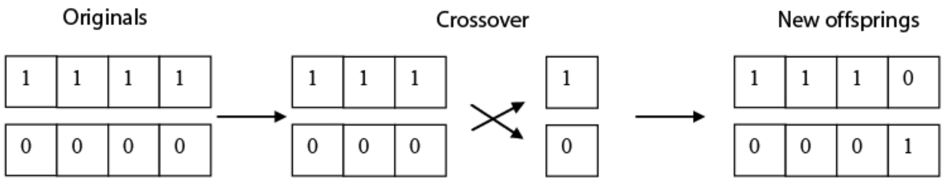

Figure 3.3 Process of crossover

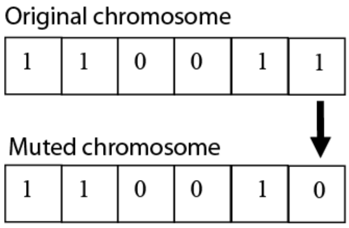

Figure 3.4 Process of mutation

Termination

Termination of the above stated processes is performed, whenever they reached the termination criteria. This might be reaching the maximum number of generations, finding a solution, user definition or a combination of above.

The operation of the basic GA is illustrated in Figure 3.5.

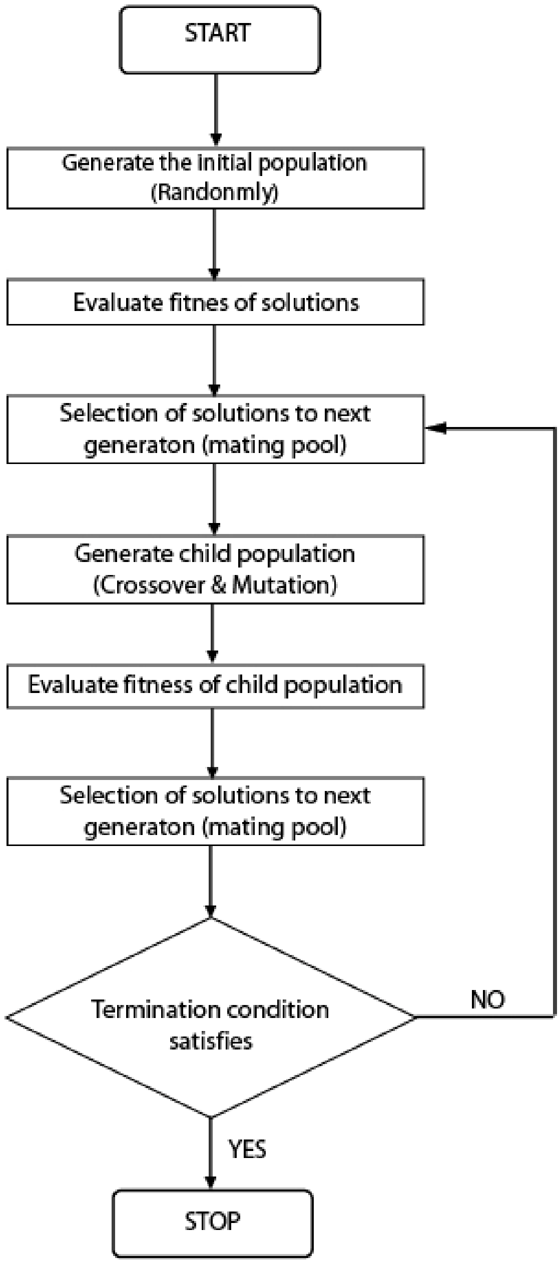

Figure 3.5 Flowchart of the basic GA operation

Though GAs have many advantages in solving difficult optimization problems, they are weak problem solving methods. If the problem is solvable with any other direct method, GAs are not the best methods to use. In addition, they usually require human supervision to get successful solutions. GAs have to be calibrated for each problem. The static parameters like mutation rate and population size have to be optimized for each problem (Choy et al., 1997-1998). Furthermore, GAs can converge to a local optimum and produce unsuccessful solutions. This is mainly because of the lack of diversity of the population. However, as it is already stated above, GAs are very useful with complex optimization problems.

Vector Evaluated Genetic Algorithm (VEGA) (Schaffer, 1985), Niched Pareto Genetic Algorithm (NPGA) (Horn et al., 1994), Weight-based Genetic Algorithm (WBGA) (Hajela and Lin, 1992), Strength Pareto Evolutionary Algorithm (SPEA) (Zitsler and Thiele, 1999) and Fast Non-dominated Sorting Genetic Algorithm (NSGA-II) (Deb et al., 2002) are few examples of GAs to solve optimization problems. Konak et al. (2006) have given a detailed explanation of these GAs. VEGA the first genetic algorithm was straightforward in implantation. However, tends to converge to the extreme end of the each objective. In addition, this does not have diversity and elitism mechanisms. NPGA uses a simple tournament selection process. However, it has problems related to niche size parameter (Konak et al., 2006). On the other hand, WBGA is a simple extension of single objective genetic algorithm. However, this has difficulties in nonconvex objective function space (Konak et al., 2006). SEPA is an advanced GA and has elitism mechanism. It is efficient and well tested. Nevertheless, it has a complex clustering algorithm. NSGA II by Deb et. al., is a ranking based non domination sorting algorithm. This GA has been widely used in many disciplines and discussed in detail from the following subsection.

3.4 Non-dominated Sorting Genetic Algorithm II (NSGA II)

Non-dominated sorting genetic algorithm (NSGA) was initially introduced by Deb et al. in 1990s. However, over the years, there were some criticisms on the proposed NSGA. High computational complexity of non-dominated sorting, lack of elitism and problems in sharing parameter are these main criticisms.

NSGA sorting algorithm has a computational complexity of $O(MN^3)$, where M is the number of objective functions and the N is the population size. Therefore, this makes NSGA computational expensive for the larger population sized problems.

Research on GA has shown that the elitism plays an important role in the performance of the algorithm. Simply, it can speed up the performance of the algorithm. However, NSGA algorithm has the lack of elitism. Therefore, an algorithm consisting an elitism strategy was an appealing concern.

Concept of sharing was the main mechanism to ensure the diversity of the population in traditional GAs. However, there were some criticisms in defining this sharing parameter and it is problem specific. Therefore, the need to have a GA with parameter less diversity preservation mechanism was another appealing question.

Therefore, a new version (NSGA II) of NSGA algorithm, understanding the above stated criticisms and answering those, in addition, which can handle multi-objectives in optimization, was introduced in 2002. This new version has advantages in elitism, computational simplicity and so on (Deb et al., 2002). NSGA II has been widely used over the past years by many researchers in various disciplines. Deb et al. (2002) has done a detailed comparison of NSGA II over the other GAs for several test problems. This research indicates that the NSGA II is powerful and robust in finding optimal solutions.

3.4.1 Step- by- step procedure for NSGA II

Step-by-step procedure is listed below and the detailed explanations can be found in Deb et al. (2002).

1. First, a random parent population of P_0 (size of N) is generated and the population is sorted according to the non-domination. Each solution is given a rank or fitness based on the non-domination level. Offspring population (Q_0) of same size as P_0, is generated using the binary tournament selection, recombination and mutation.
2. Then, P_0 and Q_0 are combined together to form a new population of size $2\times N$ and this population is sorted again according to the non-domination. The solutions belong to the best non-dominated set is F_1. Subsequently in descending order are F_2, F_3, … and so on.

3. If F_1 less than N ($F_1 < N$) all members in F_1 is considered for the next population generation (P_{t+1}). The other members are selected from the subsequent non-dominated sets starting from F_2. This procedure is continued until no more sets can be accommodated.
4. To choose exactly N number of population members from the last non-dominated front (F_l), a crowded-comparison operator (α_n) is used. This crowded-comparison operator in descending order is used to fill the best solutions from F_l.
5. Next the new offspring (Q_{t+1}) in size N is created by performing selection, crossover and mutation to the population P_{t+1}.
6. This procedure, in finding solutions to the multi-objective optimization problem, continues for a user defined number of iterations.

Figure 3.6 is from Deb et al. (2002) and explains this procedure graphically.

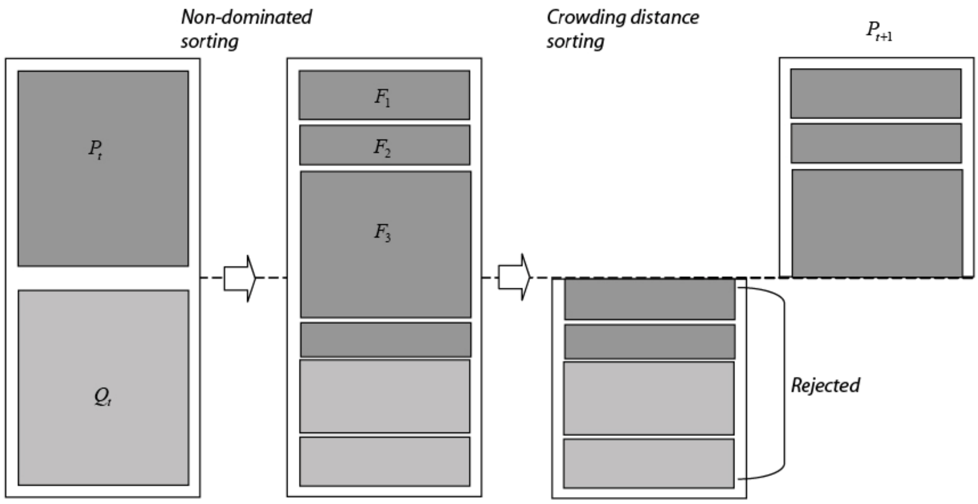

Figure 3.6 NSGA II procedure

3.4.2 Real coded crossover in NSGA II

NSGA II algorithm can be worked with both real coded variables and binary coded variables. Crossover and mutation operators in real coded and binary coded NSGA II are different. Simulated binary crossover operator (SBX) is used in real coded NSGA II whereas, single point crossover in binary coded NSGA II (Deb and Agrawal, 1995). The simulated binary crossover uses a probability distribution around two parents to create two children solutions. A detailed explanation of simulated binary crossover operator can be found in Deb and Agrawal (1995). However, the following formulations give a brief overview of this SBX.

Let us consider two parents selected to go through the crossover are p_1 and p_2 and their children after the crossover are c_1 and c_2. A space factor, β is defined as shown in Equation (3.15).

$$\beta = \left| \frac{c_1 - c_2}{p_1 - p_1} \right| \tag{3.15}$$

A probability distribution function is then defined according to the crossover distribution index (n_d) and given by the Equation (3.16).

$$P(\beta) = \begin{cases} 0.5(n_d + 1)\beta^{n_d} & \beta \leq 1 \\ 0.5(n_d + 1)\dfrac{1}{\beta^{n_d + 2}} & otherwise \end{cases} \tag{3.16}$$

Figure 3.7 from Deb and Agrawal (1995) gives a graphical interpretation of this probability function. Differences of various probability distributions according to the crossover distribution index can be seen from this figure.

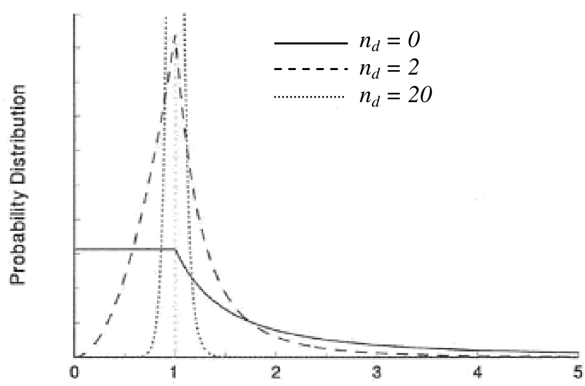

Figure 3.7 Illustration of probability distributions used in simulated binary crossover operator (Deb and Agrawal, 1995)

Usage of large n_d values has a high probability of creating solutions near the parent, whereas small n_d values has a high probability of creating distant solutions.

Depending on the upper and lower bounds of the variables the creation of children are different. In case of no upper and lower bound of the variables, a random number ($\Re$) is generated and that $\Re$ is used to find β' according to the Equation (3.17).

$$\int_0^{\beta'} P(\beta)\,d\beta = \Re \tag{3.17}$$

The children created from this β' are given in the Equation (3.18).

$$\begin{aligned} c_1 &= 0.5\left[(p_1+p_2)-\beta'|p_2-p_1|\right] \\ c_2 &= 0.5\left[(p_1+p_2)+\beta'|p_2-p_1|\right] \end{aligned} \tag{3.18}$$

In case of defined upper and lower bound of the variables, a modified probability distribution is used. Let us assume the lower and upper bounds of the variables to be x_i^L and x_i^U respectively and mathematically given in the Equation (3.19).

$$x_i^L \le x_i \le x_i^U \tag{3.19}$$

Then, the modified cumulative probabilities are given in the Equation (3.20).

$$\begin{aligned} P_1^1 &= \int_0^{\beta^L} P(\beta)\,d\beta \qquad \text{where } \beta^L = \frac{p_1+p_2-2x_i^L}{|p_2-p_1|} \\ P_2^1 &= \int_0^{\beta^U} P(\beta)\,d\beta \qquad \text{where } \beta^U = \frac{2x_i^U-p_1-p_2}{|p_2-p_1|} \end{aligned} \tag{3.20}$$

Next, new space factors β_1^1 and β_2^1 are found according to the Equation (3.21). These space factors are used in defining the two children solutions.

$$\beta_1^1 = \frac{P(\beta)}{P_1^1} \tag{3.21}$$

$$\beta_2^1 = \frac{P(\beta)}{P_2^1}$$

Finally, two children solutions are generated according to Equation (3.22).

$$\begin{aligned} c_1 &= 0.5\left[(p_1 + p_2) - \beta_1^1 \left|p_2 - p_1\right|\right] \\ c_2 &= 0.5\left[(p_1 + p_2) + \beta_2^1 \left|p_2 - p_1\right|\right] \end{aligned} \tag{3.22}$$

3.4.3 Real coded mutation in NSGA II

The polynomial mutation operator creates a new value for the decision variable, which is near the vicinity of the original value using a probability distribution. Following mathematical formulation gives an overview of polynomial mutation factor used in real coded NSGA II.

Following terminology is used in illustrating the polynomial mutation process in NSGA II.

n_m - Mutation index
P_m - Mutation probability
$\Re$ - Random number
$\Re_1, \Re_2, \Re_3, \Re_4, \Re_5, \Re_6$ & $\Re_7$ - Real numbers used in calculations
Y_0 - Original value of the decision variable
Y_L - Lower limit of the decision variable
Y_N - New value of the decision variable
Y_U - Upper limit of the decision variable

A random number ($\Re$) is generated and it is assumed here that the generated random number is less than the mutation probability ($\Re < P_m$).

A real number ($\Re_1$) is calculated according to the following conditional Equation (3.23).

$$\Re_1 = \begin{cases} \Re_2; & Y_o > Y_L, (Y_o - Y_L) < (Y_U - Y_o) \\ \Re_3; & Y_o > Y_L, (Y_o - Y_L) > (Y_U - Y_o) \end{cases} \tag{3.23}$$

where, $\Re_2$ and $\Re_3$ can be obtained from the Equations (3.24) & (3.25)

$$\Re_2 = \frac{Y_o - Y_L}{Y_U - Y_L} \tag{3.24}$$

$$\Re_3 = \frac{Y_U - Y_o}{Y_U - Y_L} \tag{3.25}$$

$\Re_4$ another real number is calculated using the mutation index and shown in the Equation (3.26).

$$\Re_4 = \frac{1.0}{n_m + 1.0} \tag{3.26}$$

$\Re_5$, another real number is calculated as shown in the Equation (3.27).

$$\Re_5 = 1 - \Re_1 \tag{3.27}$$

Depending on the value of $\Re$, a new real number ($\Re_6$) is calculated and this is given by the Equation (3.28).

$$\Re_6 = \begin{cases} (2\times\Re)+(1-(2\times\Re))\times{\Re_5}^{n_m+1}; & \Re\leq 0.5 \\ (2\times(1-\Re))+(2\times(\Re-0.5))\times{\Re_5}^{n_m+1}; & \Re>0.5 \end{cases} \tag{3.28}$$

In addition, $\Re_7$, another new real number is calculated in the two conditions shown in the Equation (3.29).

$$\Re_7 = \begin{cases} \left(\Re_6^{\Re_4}\right) - 1; & \Re \le 0.5 \\ 1 - \left(\Re_6^{\Re_4}\right); & \Re > 0.5 \end{cases} \tag{3.29}$$

Finally the Y_N is calculated according to the Equation (3.30).

$$Y_N = Y_o + \left(\Re_7 \times \left(Y_U - Y_L\right)\right) \tag{3.30}$$

There can be two extreme values for Y_N, when $Y_N < Y_L$ and when $Y_N > Y_U$. In such case NSGA II uses the following Y_N values for the successive generation.

$$Y_N = \begin{cases} Y_L: & Y_N < Y_L \\ Y_U; & Y_N > Y_U \end{cases} \tag{3.31}$$

A numerical example with $\Re$=0.09, P_m=0.1, n_m=20, Y_0 = 1.2, Y_U = 1.4 and Y_L = 0 gives $\Re_1 = 0.1429$, $\Re_4 = 0.04762$, $\Re_5 = 0.8571$, $\Re_6 = 0.2122$, $\Re_7 = -0.07116$ and $Y_N = 1.10037$.

The distribution indices use in Simulated binary crossover and Polynomial mutation in NSGA II define the shape of the probability distribution for the Simulated binary crossover and Polynomial mutation respectively (Deb and Agrawal, 1995).

3.4.4 Constraint handling in NSGA II

NSGA II proposes a simple constraint handling technique called tournament constraint handling approach. It uses the binary tournament selection, where two potential solutions are picked at random from the population and the better solution is selected. These two prospective solutions can be either feasible or infeasible based on the constraints. This can lead to three situations as follows:

1. Both solutions are feasible;
2. One is feasible and the other is not; and
3. Both are infeasible.

Solution 1 is deemed to be better than Solution 2 if one of the following conditions is true:

1. Solution 1 is feasible and Solution 2 is infeasible;
2. Both solutions are feasible and Solution 1 dominates Solution 2; and
3. Both are infeasible, but Solution 1 has a lower overall constraint violation

This approach does not require applying a penalty to handle the constraints. This is an advantage of the proposed approach in NSGA II. However, as given in condition 1, the infeasible solution is ignored with a feasible solution. Even though the solution is just infeasible, it is ignored in this constraint handling approach. However, this just infeasible solution may be better than the feasible solution in the real world practice. Therefore, the constraint handling approach, which is used in NSGA II, has both merits and weaknesses. More details of this constraint handling approach can be found in Deb et al. (2002).

3.5 Summary and conclusions

This chapter summarizes the multi-objective optimization and various solution techniques. Evolutionary algorithms show a better approach in solving multi-objective optimization problems. Instead of a single solution, a set of optimal solutions can be found using the evolutionary algorithms. NSGA II is a powerful and robust genetic algorithm. This is widely used by many researchers in various disciplines, including water resource management problems. NSGA II has been used to solve the developed multi-objective optimization problem and explained detail in coming chapters 6 & 7.

CHAPTER 4

CONTROL OF URBAN SEWER SYSTEMS

4.1 Introduction

Urban sewer systems consist of underground pipe network, treatment plants, pumping stations, storage tanks and manholes. In addition, various hydraulic components, such as orifices, weirs, sewer chambers and etc. can be found in a typical sewer system. Lateral sewers connect residences or industries to the sewer line. The branch sewers are larger in diameters than the lateral and connect the laterals to the interceptor sewer.

This chapter mainly discusses the controlling aspects of sewer systems. In addition, the usage of genetic algorithms in controlling aspects is reviewed in detail. Furthermore, a brief introduction on "designing urban sewer systems" is given in 4.2 section.

4.2 Designing urban sewer systems

Brief descriptions of simplified sewer design criteria are presented in the following subsections. However, for more information readers can refer a detailed sewer design manual. The following designing information is obtained from the UNDP – World Bank Water & Sanitation program manual by Bakalian et al. (1994).

4.2.1 Layout design

The main objective of the layout design is to reach all the point sources without much excavation and with small pipes if possible. The idea behind of this objective is to reduce the large pumping stations, in the sewer system. Therefore, a serious consideration is given for separating the sewer lines into smaller systems, even though, network layout optimization is important. Furthermore, these sewer lines are laid along the sidewalks to reduce as much as the excavation and restoration costs.

4.2.2 Design period

Trunk sewers and interceptor sewers are usually designed to a projected peak flow expected during a 25 to 50 year time, considering the current population and population growth in the area. The idea behind of such long design period is to consider the economic growth of that particular area. However, the design period is totally dependent on the available capital cost and projected maintenance cost. Furthermore, the uncertainty of the land use change in future years is a challenging task. The shorter design periods minimize the initial capital requirements. Therefore, phase by phase construction has some advantages. The errors in predictions can be minimized, especially when it comes to land use change.

4.2.3 Design flow

Wastewater flows are usually less than the clean water supply flows. Water is lost from various methods, before it enters the sewer pipes. Gardening is one such example. To determine the exact design flow, it is important to consider the diurnal effects of water usage and the climatic and weather patterns of the areas. Design flow is usually a peak factor to the returned flow. This factor depends on the above stated specifications and in addition, the rate of urbanization and industrialization.

4.2.4 Minimum diameter

A minimum diameter for the sewer lines is defined to avoid the clogging by large particles. For example, in the United States, the house lines are kept at a minimum diameter of 150 mm, whereas the minimum of 200 mm for street sewers. However, there may be some exceptions.

4.2.5 Maintaining self cleansing

Conventional sewers are designed to a minimum of 0.6 m/s of a velocity to maintain the self cleansing. However, the simplified sewer design is somewhat different from the conventional systems and designed to maintain a 0.1 kg/m^2 of boundary shear stress. This stress is sufficient to re-suspend 1 mm sand particles.

4.2.6 Depth of sewers

Sewer network start points should closer to the surface. However, there should be a top cover to avoid any structural damages. In simplified sewer systems, usual practice is to have a minimum depth of 0.65 m below sidewalks, 0.95 – 1.5 m below residential streets and 2.5 m below the heavily travelled streets.

4.2.7 Detention tanks / Storage tanks

Detention tanks are designed to provide addition storage facilities to the sewer systems. Sewer volumes can be stored in these tanks during the peak flow times or storm events. Therefore, the flow load to the downstream wastewater treatment plant is reduced. In addition, CSOs volumes can be reduced. Therefore, having storage or detention tanks are an additional advantage to sewer system. However, the stored flow volumes are released to the sewer system in non-peak times.

These detention tanks play the role of sedimentation tank during the detention period. Heavy solid particles deposit to the bottom of the tank. This is an additional advantage of the storage tank. Therefore, there should be a procedure for periodic cleaning. However, the deposition is totally dependent on the detention time.

A detention tank can be designed to locate either on-line or off-line in the sewer system. On-line tank is a part of the CSO chamber. They fill when the inflow exceeds the maximum allowable through flow. Discharge from these tanks usually controlled by a throttle at the downstream end. However, off-line storage tanks are physically separated tanks from the main system. Off-line storage tanks are usually designed when the land area at CSO chambers are important. Flow is diverted to these tanks in storm conditions.

Both on-line and off-line storage tanks have advantages and disadvantages. Controlling an on-line storage tank is easier than the off-line storage tank. Pumping is not required for on-line storage tanks, whereas it is an essential task for the off-line storage tanks. Flow from either one of CSO chamber to off-line storage tank or vice versa can be gravity fed. However, the other way has to be by pumps. On-line tanks have to bigger in volume to cope the excessive flows. However, the sewer lines which connect CSO chambers and off-line storage tanks have their own capacity and add additional storages. This will lead designers to reduce the volumes of the off-line storage tanks. Usually detention tanks are covered. This is because of the sanitation and safety standards.

There are three basic types of tanks, tank chambers, tank sewers and tank shafts. Tank chambers are simply tanks made by reinforced concrete. Tanks sewers are oversized sewer lines designed to store flow volumes. However, tank chambers can store larger volumes compared to tank sewers in a small area. But when the area is much important tank shafts are designed to store the flow volumes. More details of these storage tanks can be found in UPM (1998).

4.3 Controlling aspects of sewer systems

It was already mentioned in the Chapter 1, the final goal of a well planned, operated and maintained sewer system is to collect and transport the wastewater to the treatment plant in a sanitized way in dry, wet and storm weather periods (Schutze, et al., 2002a&b and Kannapiran, et al., 2008). However, CSOs can be seen in most of the cities all around the world in storm weather periods. Controlling the sewer systems plays an important role in minimizing these CSOs and their impacts to the environment. Real time control, RTC was on the discussion table for many years and in some cities, these RTC strategies are already implemented. Following couple of subsections (4.3.1 & 4.3.2) discuss the various aspects of RTC.

4.3.1 Real time control in urban wastewater systems

Real time control monitors the process variables and at the same time operates the flow controllers using the feedback from the monitoring. Various hardware components are used in RTC. Sensors to monitor and examine the process, Actuators to influence the process and controllers to regulate actuators to reach the minimum deviations of the controlled process are the main components. Rain gauges, water level gauges, flow gauges and quality gauges are the most common sensors used in RTC of urban wastewater systems. Pumps, gates, weirs, valves, chemical dosing devices and aeration devices are the most common actuators. PID (proportional-integral-derivative) controllers and PLC (programmable logic controllers) are some examples to the controllers used in RTC (Schutze et al., 2004)

The control algorithm in RTC can either be an off-line approach or an on-line approach. A pre-defined control algorithm is used in off-line approach, for example "if then and else" rules. However, an on-line approach chooses the best possible control action from some of multiple control actions, at each and every control time step using the optimization techniques. This will require more details of the systems of concern regularly (Schutze et al., 2001). Having a surveillance system for the urban wastewater system is an advantage. However, this requires high-tech instruments as well as some capital to establish such a system. On the other hand, the surveillance system helps the maintenance department to identify the sudden breakdowns in the system and to receive the feedback of the performance of the RTC system (Carstensen et al., 1996; Schilling et al., 1996).

4.3.2 Different approaches in real time control in urban wastewater systems

Three RTC approaches for combined sewer systems can be found in the literature, including Volume based RTC (Duchesne et al., 2004), Pollution based RTC (Weinreich et al., 1997), and Water quality and emission based RTC (Petruck et al., 1998). Volume based RTC uses the better sewer storage strategies and transport strategies in the sewer system. In other words, this approach tries to maximize the storage facilities in the sewer system to minimize the CSOs (Duchesne et al., 2004). However, there is little or no water quality details in this approach.

Pollution based RTC uses the flow volume and the water quality measurements. The main objective of this approach is to drain as much as polluted wastewater to the wastewater treatment plant. However, this approach does not require the receiving water qualities (Petruck et al., 1998). Weinreich et al. (1997) has covered the basics on this approach and the main objective of that research was to minimize the pollutant load to the receiving waters through combined sewer overflows. However, it was a linear optimization strategy. Total Phosphorus and the total Ammonia-Nitrogen were considered as the pollutants. Lacour and Schutze have shown the importance of pollution based RTC control of sewer systems respect to the turbidity measurements (Lacour and Schutze, 2010). However, according to their conclusions, the study should be validated for a larger sewer system and further research is needed to the RTC algorithm.

The main objective of the water quality and emission based RTC is to reduce critical values of water quality parameter in the receiving water. Water quality and volumetric measurements in sewer system and receiving waters are used to control the combined sewer system (Petruck et al., 1998; Vanrolleghem et al., 2004).

4.4 Multi-Objective Optimization in urban sewer systems

Many real world designing and controlling problems have several objectives and sometimes these objectives conflict each other. In most of the cases, cost is one of the important objectives. Getting a trade-off between conflicting objectives is not that straightforward. Multi-objective optimization provides a better approach in dealing with several objectives simultaneously. This has been discussed in detail in Chapter 3. Following subsections briefly discuss the usage of multi-objective optimization in urban sewer systems.

4.4.1 Genetic algorithms in urban drainage modelling

Genetic algorithms are widely used to solve difficult multi-objective optimization problems in the real world. Genetic algorithms have been used extensively in the field of water resource management. These include, designing water distribution networks, groundwater monitoring, management of water resources and rainfall runoff calibration problems and on urban drainage modeling (Rauch and Harremoes, 1999b).

4.4.2 Examples of single objective optimization problems in urban wastewater systems

Single objective optimization has used to obtain the optimal solution to multi-objective optimization problems in urban wastewater systems. This is because of the complexity of the problems in urban wastewater systems. Two objective functions, minimizing combined sewer overflow volume and maximizing the mean dissolved oxygen level at river, were used to define two single objective optimization problems by Rauch and Harremoes (1999b) in their work on urban wastewater systems.

Schutze et al. (2002a) have identified several objectives in controlling urban wastewater systems including concentrations of dissolve oxygen and ammonia.

However, only one objective was selected in the optimization process and this was because of the computational difficulties.

A good example showing of conversion of a multi-objective optimization problem in sewer systems can be found in Cembrano et al. (2004). Single objective optimization problem was implemented on optimal control of urban drainage systems using the Scalarization technique. Combined sewer overflows, flow rates in sewers and volumes of real reservoirs were integrated using weights.

Work by Darsono et al. (2007) gives another example of usage of single objective optimization in urban wastewater systems. Minimizing the occurrence and the magnitude of the CSOs and maximizing the through flows to the wastewater treatment plant are the objective functions that they have used in their work. However, they have introduced a weight over the later objective to treat the problem as a single objective optimization problem.

As it was already stated in the chapter 3, the main disadvantage of single objective optimization is that, there is only one optimal solution to the problem, instead a set of optimal solutions.

4.4.3 Optimal control of combined sewer systems using in-line storages

Optimal control of urban water system is to identify the best control strategies with respect to different objectives. In addition, better management and operational practices of urban drainage systems ensure the better health and sanitation of human, aquatic environmental improvements, pollution load reduction at receiving waters, reducing the number of flooding and etc. It is well known that not all of these objectives can be fulfilled at the same time since some of these conflict each other. Therefore, the objectives of the potential studies can be minimizing the volume CSOs, minimizing the frequency of CSOs, maintaining the water quality standards at the receiving water as well as at the treatment plant and minimizing the cost at treatment. The goal is to satisfy the most of these and at the same time, keep the conflicting objectives at a fair situation (Fu et al., 2007; Rauch and Harremoes, 1999b).

However, many researchers have used volumetric measures in controlling urban sewer system. They have tried to minimize the CSOs by introducing some in-line storage facilities, storage tanks and storage pipes along the urban wastewater system. Work by Beraud et al. (2010), Darsono et al. (2007) and Cembrano et al. (2004) are few examples for usage of volumetric methods. Furthermore, some researchers have used simplified hydraulic models in their work (Meirlaen et al., 2002), and this is because of the complexity of the problem.

4.5 Receiving water quality due to combined sewer overflows

4.5.1 Background of water quality in receiving water

Higher concentration levels of pollutants can be lead to have severe damage to the biological environment in the receiving water. Dissolve oxygen concentration of less than 4 mg/L for more than 10 minutes and unionized ammonia concentration of more than 0.1 mg/L for more than 5 minutes are two examples of critical levels of pollutants. Beyond these limits, the aquatic life is in a severe danger (Petruck et al., 1998). A very good example, which is illustrated in Chapter 1 shows the damage to the fish population in river Thames due to the CSOs.

However, most of the combined sewer systems allow CSOs during the storm weather periods. In the UK, allowable CSOs are given by the Equation (4.1) called, *Formula A*.

$$Formula\ A = DWF + 1360POP + 2IND \tag{4.1}$$

where *Formula A* is the CSO setting (L/day), *DWF* is the average dry weather flow including infiltration and industrial discharges (L/day), *POP* is the population considered and *IND* is the average industrial discharge (L/day). Settings should be there to prevent CSOs until the incoming flows exceed *Formula A*. The United States has its own control policy for the CSOs. Control policy can de found in US Environmental Protection Agency (US EPA, 2002). However, in brief they have introduced nine minimum controls as listed below (US EPA, 1995).

1. Proper operation and regular maintenance programs for the sewer system and the CSOs
2. Maximum use of the collection system for storage
3. Review and modification of pretreatment requirements to assure CSO impacts are minimized
4. Maximization of flow to the publicly owned treatment works for treatment
5. Prohibition of CSOs during dry weather
6. Control of solid and floatable materials in CSOs
7. Pollution prevention
8. Public notification to ensure that the public receives adequate notification of CSO occurrences and CSO impacts
9. Monitoring to effectively characterize CSO impacts and the efficacy of CSO controls

Most of the other developed countries have policies to minimize CSOs because of the bad impact on receiving water.

Research work by Fu et al. (2007 & 2010b) shows some encouraging conclusions related to receiving water quality. Multi-objective optimization problem, considers some aspects of receiving water quality in a combined sewer system, was solved using NSGA II. Dissolve oxygen levels and ammonia concentration were considered in developing the multi-objective optimization problem.

4.5.2 Relationship between combined sewer overflows and receiving water quality

Worldwide it is assumed that the total combined sewer overflow volume and the combined sewer overflow frequency are good indices for the pollution impact of the receiving waters. Simply many researchers have assumed by reducing the amount and frequency of CSOs reflects better water qualities at the receiving water (Darsono et al., 2007).

However, there is enough research to show a clear uncertainty in this assumption (Lau et al, 2002; Rauch and Harremoes, 1998&1999a). Lau et al., (2002) show that the overflow spill frequency or volume can be used as an indicator of receiving water quality, but with enough care. The dissolve oxygen (DO) and biochemical oxygen demand (BOD) concentrations have not shown a good relation against the spill frequency and spill volume. Up to some extent, these water quality indicators decreases with the spill frequency and spill volumes, but thereafter they are more likely to be constants according to Lau et al., (2002). Furthermore, another water quality parameter, ammonia concentration, has shown a complex relation with the spill frequency and the volume. In addition, Rauch et al. (1999a) have clearly explained and concluded that the reduction of CSO volumes does not have a link in increasing the DO concentration in receiving waters. These findings have led the researchers to make doubts in assuming the minimization of CSO frequency and volume, which guarantees the receiving water quality.

4.5.3 Effluent quality index (*EQI*)

Biochemical Oxygen Demand *(BOD)*, Chemical Oxygen Demand (*COD*), Total ammonia, dissolved oxygen, Total Suspended Solids (*TSS*) and some other combinations of these are the most common variables to measure the quality of the water (Lau et al., 2002). Furthermore, phosphates, pH, heavy metal are some other important water quality parameters.

Effluent quality index (*EQI*) is formulated to assess the pollution load in a water body as a single variable. Five important water quality parameters, total suspended solids (*TSS*), chemical oxygen demand (*COD*), five-day biochemical oxygen demand (*BOD*), total Kjeldahl nitrogen (*TKN*) and nitrates/nitrites (*NOX*) are accumulated together in forming this *EQI*.

EQI was used as a performance index (Copp, 2002a; Copp et al., 2002b) and a sensitivity index (Kim, et al., 2006; Lee et al., 2006; Kim et al., 2009) in literature. *EQI* was identified as a better index to express the quality of the wastewater and the pollution load to receiving water bodies by many researchers. Therefore, this index can be used in representing the damage to the receiving waters from the CSOs. Effluent quality index is described in Equation 4.2.

$$EQI = \frac{1}{1000\left(t_f - t_0\right)} \int_{t_0}^{t_f} \left(2C_{TSS} + C_{COD} + 2C_{BOD} + 20C_{NOX} + 20C_{TKN}\right) Q_e\left(t\right) dt \qquad (4.2)$$

where $Q_e(t)$, t_f, and t_0 are the flow rate, final and initial time respectively. C_{TSS}, C_{COD}, C_{NOX}, C_{BOD} and C_{TKN} are the concentrations of total suspended solids, chemical oxygen demand, nitrates and nitrites, five-day biochemical oxygen demand and total Kjeldahl nitrogen, respectively. The numerical values in front of these concentrations represent the weighting factors. These weighting factors are used to denote the contribution of each water quality parameter (Mussati et al., 2002). These factors are based on the Flandes' effluent quality formula for calculating fines (Vanrolleghem et al., 1996).

However, Kim et al. (2009) have tried to incorporate total phosphorous into the effluent quality index. This modified EQI is shown in the Equation (4.3)

$$EQI = \frac{1}{1000\left(t_f - t_0\right)} \int_{t_0}^{t_f} \left(2C_{TSS} + C_{COD} + 2C_{BOD} + 20C_{NOX} + 20C_{TKN} + 100C_{TP}\right) Q_e\left(t\right) dt \qquad (4.3)$$

where C_{TP} are the concentrations of total phosphorous respectively.

4.6 Cost modelling in wastewater treatment processes

The total operating and maintenance cost of a wastewater treatment plant is related to global plant parameters, such as average flow rate and population in the catchment area (Gillot et al., 1999). In addition, quality of raw sewage and the required quality of effluent are couple of important considerations.

The treatment and operational costs are often assumed to be a percentage of the construction cost by the wastewater treatment plant designers (Uluatama, 1991). However, there are some attempts in describing the operational and maintenance costs of wastewater treatment plants respected to the treated wastewater volume flow rate. Treatment and operational costs have an "S-curve" increment-variation with the design flow rates (Friedler and Pisanty, 2006). Though there are some attempts in modeling the wastewater treatment cost as a soley function of treated wastewater volume, finding an universal cost function based on the volume flow rate is difficult.

Hernandez-Sancho and Sala-Garrido (2008) proposed a cost function to the wastewater treatment based on the treated wastewater volume flow rate. Their research was carried out for 339 wastewater treatment plants in Spain. These plants are varying in their sizes, below a treated volume of 50000 m^3/year to over 5000000 m^3/year. Table 4.1 provides the information of sample of wastewater treatment plants were considered.

Table 4.1 Sample description used for cost modelling (Hernandez-Sancho and Sala-Garrido, 2008)

Size (m3/year)	Number of plants	Total amount of water treated
Below 50,000	73	2,150,091
50,001 – 100,000	62	4,400,190
100,001 – 250,000	71	11,149,781
250,001 – 500,000	44	15,409,301
500,001 – 1,000,000	28	19,994,506
1,000,001 – 5,000,000	38	90,420,798
Over 5,000,000	23	322,855,057
Total sample	339	466,379,724

Despite the various treatment processes applied in different treatment plants, the average treatment cost considerably depends on the size of the treatment plant. For example the average cost of treatment for the smallest size (below 50,000 m^3/year) group shows 0.669 €/m^3 whereas the largest size (over 5,000,000 m^3/year) shows 0.085 €/m^3. These average cost based on the size or the treated wastewater volume in detail can be found in the Table 4.2.

Table 4.2 Breakdown of cost (€/m^3) for treated wastewater volume (Hernandez-Sancho and Sala-Garrido, 2008)

Size (m3/year)	Personnel	Energy	Maintenance	Waste	Other	Total
Below 50,000	0.369	0.095	0.069	0.024	0.112	0.669
50,001 – 100,000	0.229	0.067	0.042	0.021	0.059	0.418
100,001 – 250,000	0.139	0.053	0.028	0.021	0.034	0.275
250,001 – 500,000	0.108	0.047	0.025	0.021	0.022	0.223
500,001 – 1,000,000	0.094	0.041	0.018	0.023	0.026	0.202
1,000,001 – 5,000,000	0.064	0.038	0.013	0.022	0.019	0.156
Over 5,000,000	0.03	0.016	0.01	0.015	0.013	0.085
Total sample	0.048	0.024	0.013	0.018	0.016	0.119

Table 4.2 illustrates the various cost components in wastewater treatment. This table shows that the 'personal' and 'energy' cost components play important roles to with respected to the treated volume flow rate. Small plants show a higher personal and energy cost whereas the larger plants shows lower rates.

Based on the total cost of wastewater treatment against the treated wastewater volume an empirical cost function was proposed by proposed by Hernandez-Sancho and Sala-Garrido (2008) shown in the Equation (4.4).

$$C_T = 916.862 q_T^{0.659} \tag{4.4}$$

where C_T and q_T are the total cost of treatment (€/year) and amount of treated wastewater respectively. Personnel, energy, maintenance, waste and other costs are added together to present the total cost of treatment. Figure 4.1 shows the relationship between the cost and the treated wastewater volume according to Hernandez-Sancho and Sala-Garrido (2008).

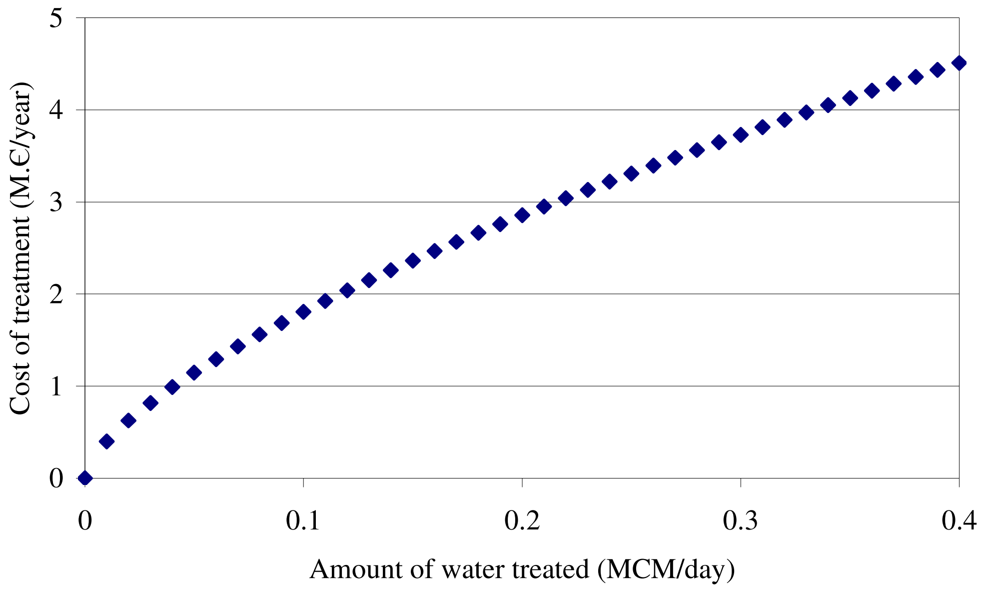

Figure 4.1 Relationship between the cost and treated volume

However, the proposed cost function by Hernandez-Sancho and Sala-Garrido (2008) has not been verified with other wastewater treatment plants across the world. Importantly, there should be correction factor at the treatment cost depending on the economic level (rate of inflation) of the countries. However, literature does not provide other detailed cost functions to the wastewater treatment plant respected to the treated wastewater volume flow rate. Even though, with various weaknesses, this cost function is used to generate one of the objectives of the multi-objective optimization approaches developed in Chapters 6 & 7.

Capacity determination of a wastewater treatment plant is based on the flow rate to the treatment plant from the dry weather flow (DWF). It is generally designed to occupy 6×DWF. However, the full treatment capacity of a particular treatment plant is further limited to 3×DWF. The excess flow is diverted to an equalization tank (Lima and Bachmann, 2002). Whenever the flow rate to the treatment plant is lower than the 3×DWF, the stored wastewater from the equalization tank is released to the treatment process.

Equalization tank acts the same role of a primary sedimentation tank. Therefore, the operational and maintenance cost of an equalization tank is assumed to be the same of a primary sedimentation tank. Wastewater treatment technologies: a general review gives the construction, annual operational and maintenance costs of unit processes as a function of treated wastewater volume (United Nations, 2003). These functions give the cost in United States Dollars (US$). The annual operational and maintenance cost of a primary sedimentation tank is proposed to have a linear relationship with the treated wastewater volume. The operation and maintenance costs of sludge pumps are included in this formulation. This cost expression is shown in the following equation.

$$C_{PS} = 1.69 * Q_{PS} + 11376 \tag{4.5}$$

where C_{PS} and Q_{PS} are the cost of operation and maintenance of a primary sedimentation tank and the wastewater volume flow rate (m^3/day) of this tank respectively.

4.7 Summary and conclusions

Multi-objective optimization has played an important role in controlling the urban wastewater systems. Real time control technologies have shown considerable improvements in reducing CSOs. However, the combination of multi-objective optimization and RTC technologies opens an optimal operational control to the exciting combined sewer systems. Water quality in the sewer systems and the receiving water and cost of wastewater treatment are two important parameters, which can be used in the optimization process.

CHAPTER 5

COMPOSITION OF WASTEWATER AND STORMWATER

5.1 Introduction

Chapter 5 describes the composition of wastewater and stormwater. Effects of the various land uses in generating different pollution levels in surface runoff are presented. Pollutographs are generated considering the temporal and spatial variations of the stormwater runoff for several storm conditions, including a single storm (from Thomas et al., 2000) migrating upstream storms, migrating downstream storm and two consecutive storms. This is the main original contribution of this chapter.

5.2 Composition of the wastewater (or DWF)

Wastewater contains a variety of organic and inorganic compounds. Carbohydrates, fats, oils, proteins, and wood are few examples of organic compounds in wastewater. Various toxic elements such as arsenic, chromium, cadmium, mercury, etc are few examples of inorganic compounds. The pollutants from CSOs are a potential threat to the aquatic life in the receiving water (Bay and Greenstein, 1996).

Composition of the wastewater varies from country to country and place to place. However, average concentrations of compounds in wastewater are presented in Metcalf and Eddy (1991). Concentration levels of five water quality parameters, including total suspended solids (TSS), chemical oxygen demand (COD), biochecmical oxygen demand (BOD), total Kjeldahl nitrogen (TKN) and nitrates&nitrites (NOX) are presented in the following table. These concentrations are used in the calculations of effluent quality indices (EQI) in the coming chapters.

Table 5.1 Pollutant composition of wastewater (Metcalf and Eddy, 1991)

Water quality constituent	Concentration level (mg/L)		
	Weak	Medium	Strong
TSS	100	220	350
COD	250	500	1000
BOD	110	220	400
TKN	12	25	50
NOX	20	40	85

As it is tabulated in Table 5.1, there are three average pollution levels listed in Metcalf and Eddy (1991).

5.3 Composition of stormwater runoff

5.3.1 Various land use patterns

Land use patterns are different from one catchment to another. Residential, industrial, commercial and rural are few examples of different land uses. These different land uses have different soil characteristics. For an example, one of the most important parameters to determine the amount of runoff from a catchment is the infiltration rate of the soil. Infiltration rates have significant differences from catchment to catchment. These differences can even be seen in the similar catchments, but with different soil covers. Osuji et al. (2010) have illustrated this in detail.

Not only the amount of runoff, but also the composition of the stormwater runoff is different from one land use to another. This is because of the different characteristics of the pollutant build-up and pollutant wash-off. For an example, it is obvious that an agricultural catchment should have more nitrogen (or its compounds) in the surface runoff.

5.3.2 First flush

Pollutants deposited on the catchments during the dry weather periods are dislocated and washed-off during the wet and storm weather periods. However, during the initial phase of runoff hydrograph, the concentrations of pollutant wash-off are significantly high. These higher concentration levels are notable for surface runoff after a dry period. The stormwater runoff contains high levels of pollutant loads is called the first flush.

First flush is a rapid change. The catchments are cleansed by the first flush. Therefore, with the progression of the time, the concentration level or pollutant load wash-off reduces significantly. More details on first flush phenomenon are explained in Maestre and Pitt (2005).

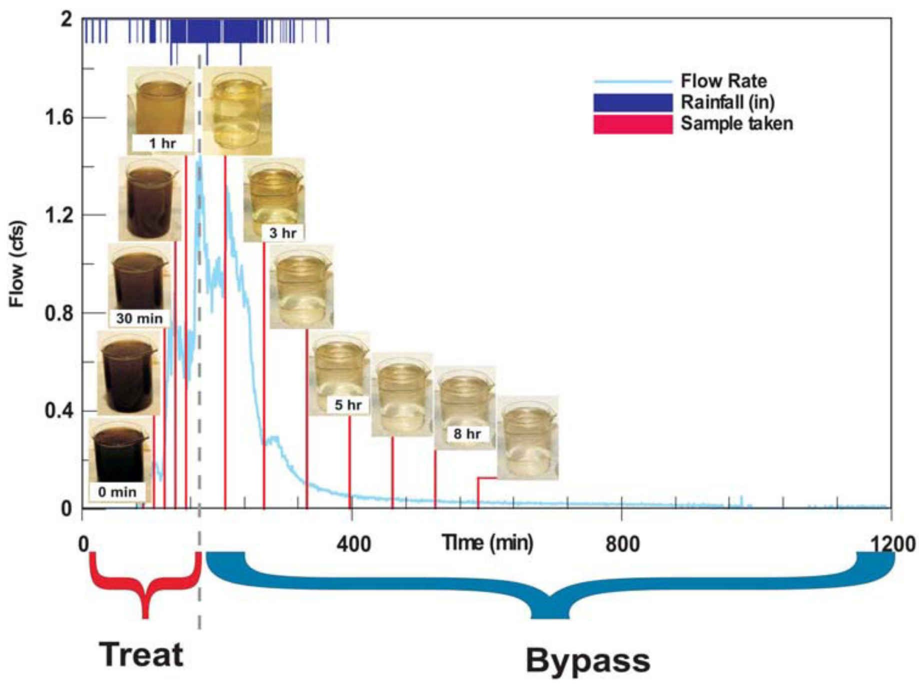

Figure 5.1 Water samples along the runoff hydrograph (Stenstorm and Kayhanian, 2005)

Figure 5.1 from Stenstorm and Kayhanian, (2005) is a very good example to illustrate the first flush phenomenon. Pollutant load of the runoff can be clearly visualized from the colour of the water samples taken at different times along the runoff hydrograph.

5.3.3 Pollution levels of land uses

Duncan (1999) has given a detailed overview about the composition of the stormwater runoff for different land uses. The outcome of his research work is based on a statistical review of 40-year data from literature. Table 5.2 presents the composition of stormwater runoff for several important land-uses.

Table 5.2 Pollutant composition of stormwater runoff (Duncan, 1999)

Land-use	TSS (mg/L)	COD (mg/L)	BOD (mg/L)	TKN (mg/L)	NOX (mg/L)
Residential	50 - 400	35 – 175	8.0 - 25	1.2 - 5.5	1.2 - 5.5
Industrial	45 - 500	70 – 410	7.0 - 25	1.2 - 4.2	1.2 - 4.2
Commercial	50 - 350	30 – 220	9.5 - 22	1.1 - 3.5	1.1 - 3.5
Agricultural	65 - 550	12 – 85	1.0 - 10	1.5 - 9.5	1.5 - 9.5
Mid urban	35 - 850	25 – 75	4.0 - 12	1.5 - 7.5	1.5 - 7.5

5.4 Generation of pollutographs for single storm condition

5.4.1 Single storm runoff hydrographs

Interceptor sewer system case study developed in Thomas (2000) was modified used in the Chapters 6 & 7. Therefore, the geometric details and the runoff hydrographs given by Thomas (2000) were incorporated in this thesis work. Runoff hydrographs illustrated in Thomas (2000) are used as single storm runoff hydrographs in this research study. These single storm runoff hydrographs are shown in the Figure 5.2. Rimrose, Strand Road, Millers Bridge, Bankhall Relief, Northern, Bankhall and Sandhills Lane are the seven catchments used to model the interceptor sewer system in Thomas (2000). These runoff hydrographs last for two and half hours as shown in the Figure 5.2.

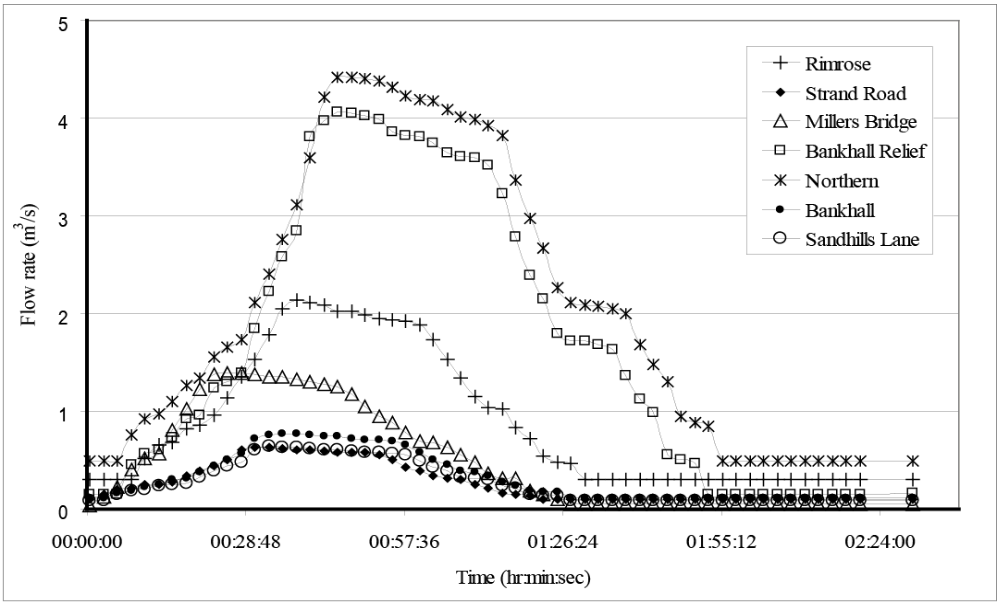

Figure 5.2 Stormwater runoff hydrograph for single storm condition (Thomas, 2000)

5.4.2 Land uses for catchments

Different land-uses were hypothetically assigned to the above seven catchments. Flow rates of the average DWFs were considered, when assigning these land-uses to the respective catchments. DWFs to these seven catchments are tabulated in Table 5.3.

Table 5.3 Dry weather flows for catchments (Thomas, 2000)

Catchments	Fixed inflow (m^3/s)	DWF (m^3/s)	Total (m^3/s)
Rimrose	1.24	0.3	1.54
Strand Road	0.25	0.09	0.34
Millers Bridge	0.97	0.04	1.01
Bankhall Relief	0.69	0.14	0.83
Northern	2.13	0.5	2.63
Bankhall	0.29	0.11	0.40
Sandhills Lane	0.31	0.09	0.40

It was assumed that higher DWF rates are conveyed to the sewer networks from residential land use. Therefore, Rimrose and Northern catchments were assigned as the residential areas. Furthermore, agricultural land use was assumed to convey the lowest DWFs. Therefore, Millers Bridge catchment was assigned as an agricultural area. These land use patterns and assigned catchments based on the DWF rates are described in Table 5.4

Table 5.4 Assumed land-use patterns of catchments

Catchment	Land-use pattern
Rimrose & Northern	Residential
Strand Road & Sandhills Lane	Commercial
Millers Bridge	Agricultural
Bankhall Relief	Industrial
Bankhall	Mid Urban

5.4.3 Single storm pollutographs

In addition to the different pollution compositions for different land uses (stated in section 5.3.3), the temporal variations of the water quality constituents in stormwater runoff are significant. Pollutographs represent these concentration variations with time. However, the shapes of the pollutographs of different water quality constituents are different from each other. These shapes were reviewed from the previous literature (Morris et al., 1998; Yusop et al., 2005; Li et al., 2007; Nazahiyah et al., 2007; Kim et al., 2007; Qin et al., 2010). Different pollutographs for TSS, COD, BOD, TKN and NOX were developed for every catchment. This is the main original contribution of this chapter. Figures 5.3 – 5.6 present few examples of developed pollutographs for the different land-uses. Rest of the pollutographs can be found in the Appendix A.

Figure 5.3 presents the generated TSS pollutograph and the stormwater runoff hydrograph for the catchment Rimrose for single storm condition. TSS shows a rapid peak during the first flush.

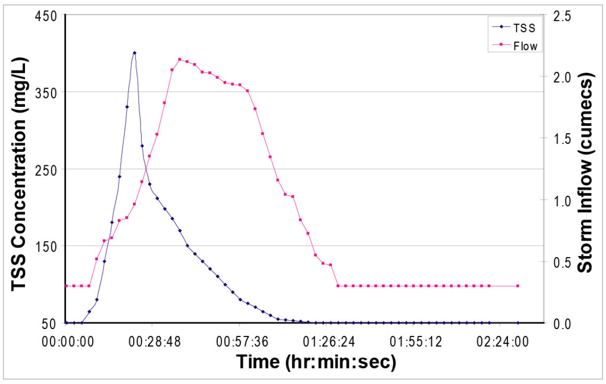

Figure 5.3 TSS pollutograph for Rimrose for single storm

Figure 5.4 presents the TKN pollutograph and the stormwater runoff hydrograph for the catchment Rimrose for the single storm condition. However, this shows a mild peak compared to the TSS pollutograph of Rimrose.

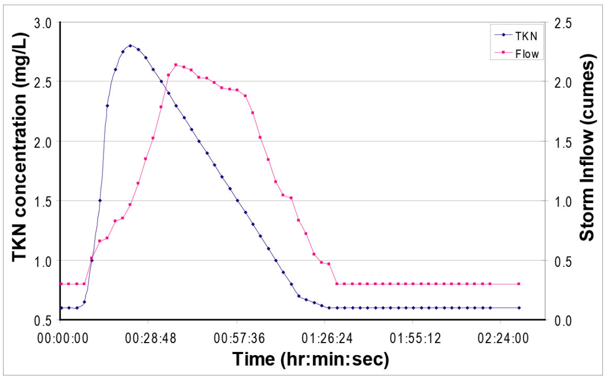

Figure 5.4 TKN pollutograph for Rimrose for single storm

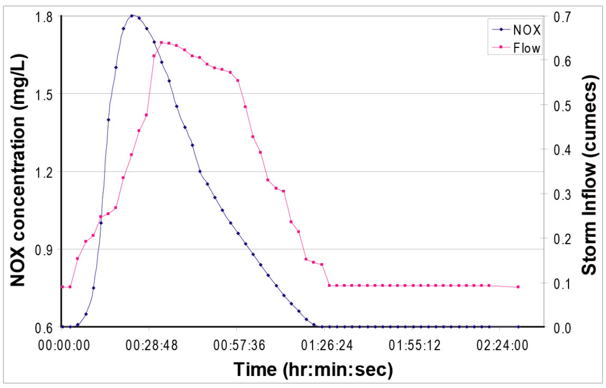

Figure 5.5 NOX pollutograph for Sandhills Lane for single storm

Figure 5.5 shows the NOX pollutograph and the corresponding stormwater runoff hydrographs for the catchments of Sandhills Lane. It has a commercial land-use. Furthermore, Figure 5.4 shows the TKN pollutograph for a residential land-use. However, the agricultural land-use catchment, Millersbridge should have more TKN and NOX concentrations. This is because of the fertilizers used in the agricultural lands. Figure 5.6 presents the NOX pollutograph for the agricultural land-use. It can be clearly seen that the peak concentrations of the NOX pollutograph is higher than that of in Figure 5.5.

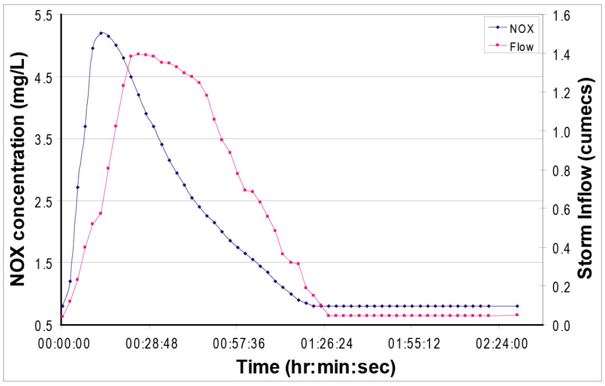

Figure 5.6 NOX pollutograph for Millers Bridge for single storm

Pollutographs shown above were generated to have the peaks before the corresponding peaks of the stormwater runoff. This is to satisfy the first flush phenomenon. As it is already stated above (subsection 5.3.2), during the first flush, stormwater runoff has higher pollution concentration levels compared to the remainder of the storm.

Concentrations at each 2 minutes and 30 seconds were fed to the pollutograph to produce smooth curves. Differences of the shapes of the pollutographs are clearly visualized from the above pollutographs. Especially the TSS pollutographs, show a sudden drop, whereas others show a mild drop after the first flush

5.5 Generation of pollutographs for two consecutive storm conditions

5.5.1 Two consecutive storm runoff hydrograph

Successive rainfall events are frequent. These successive rainfall events in a catchment produce runoff hydrographs with several peaks. Catchments are recharged during the first rainfall event. Even though this rainfall event is considerable event, corresponding runoff may not be considerable due to this recharge. However, in a consecutive rainfall event, the runoff is significant, even in a trivial rainfall event. Runoff is usually transferred to the combined sewer networks and therefore, the risk of having CSOs increase.

Two consecutive storms were considered to investigate the effects from successive rainfalls. It is herein assumed that the consecutive storms produce two peaks in the runoff hydrograph as shown in the Figure 5.7. Single storm runoff hydrographs given in Figure 5.2 were assumed to be repeated to produce the two consecutive storms. Even though the second storm event does not produce an identical storm runoff hydrograph similar to the first in real world, it is herein assumed that the storm runoff hydrographs are identical for the both storm events, solely because of the computational ease.

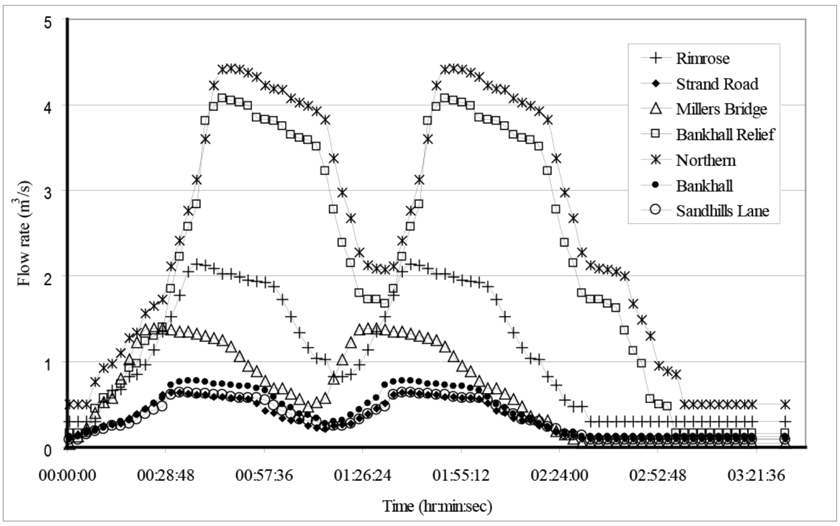

Figure 5.7 Stormwater runoff hydrograph for two consecutive storms

5.5.2 Two consecutive storm pollutographs

Not only successive rainfall events produce several peaks in the runoff hydrograph, but also they make impacts on the shapes of the pollutographs. After the first flush, a continuous depletion can be seen in the pollutographs for single storms. However, in consecutive storms there can be another rising limb in the pollutograph after the first flush. This happens usually before the second peak of the runoff hydrograph. Gamerith (2006) and Qin et al., (2010) have shown this difference in pollutograph in their research work. Some of the generated pollutographs based on this concept are shown in the Figures 5.8 to 5.10. The full set of generated pollutographs for two consecutive storms can be found in the Appendix A.

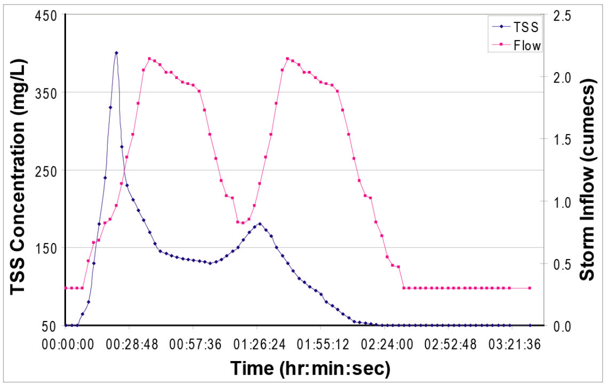

Figure 5.8 TSS pollutograph for Rimrose
for two consecutive storms

Figure 5.8 presents the TSS pollutograph for Rimrose and the stormwater runoff hydrograph for two consecutive storms. Similar to the single storm condition, this shows rapid initial peak. However, second peak of the pollutograph can be identified. This is because of the second peak of the storm hydrograph.

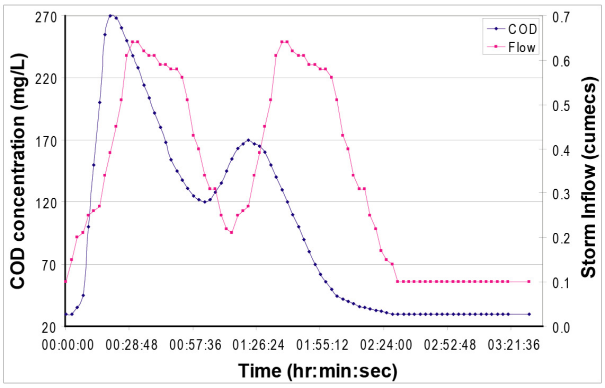

Figure 5.9 COD pollutograph for Strand Road for two consecutive storms

Figure 5.9 shows the COD pollutograph and the corresponding stormwater runoff hydrograph for the Strand Road catchment under the two consecutive storms. Second peak corresponds to the second peak at the stormwater runoff hydrograph can be identified.

Figure 5.10 illustrates the BOD pollutograph for the catchment of Millers Bridge and the corresponding stormwater runoff hydrograph. Similar to the above presented figures (5.8 and 5.9) a second peak in the pollutograph can be identified. However, the concentration of the second peak is lower than the first peak. First peak is from the first flush. It is obvious that the surface pollution levels of the catchment are lower after the first flush.

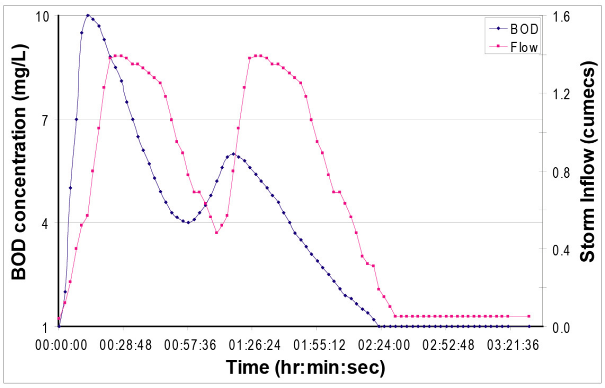

Figure 5.10 BOD pollutograph for Millers Bridge for two consecutive storms

5.6 Generation of pollutographs for migrating storm conditions

5.6.1 Migrating storm runoff hydrographs

Uniform storms are generally applied in most of the research of sewer systems (Vaes et al., 2002). However, in real world conditions this may not be applicable. Spatial distributions of the rainfall events, catchment characteristics (such as land use) lead to have non-uniform hydrographs. Therefore, migrating storms affect the runoff hydrograph and the magnitude of the peak discharge of the runoff hydrograph. The overall peak discharge of a downstream migrating storm exceeds that of an upstream migrating storm (Nelen et al., 1992; Vaes et al., 2002; Hochedlinger et al., 2006; Zawilski and Brzezińska, 2011). Importantly, spatial and temporal rainfall distribution should not be neglected, when considering the real time control of drainage system (Nelen et al., 1992).

Runoff hydrographs shown in the Figures 5.2 and 5.7 assumes the runoff for all catchments reach CSO chambers at the same time. Considering the migrating aspects of the storms, two different sets of runoff hydrographs were developed as shown in Figures 5.11 and 5.12. Figure 5.11 shows the migrating downstream runoff hydrograph whereas the 5.12 is for the migrating upstream storms. Distances between the sewer chambers of the corresponding catchments have used as a guideline to identify the reaching time of the hydrographs. Distances between the CSO chambers are given in the Table 5.5.

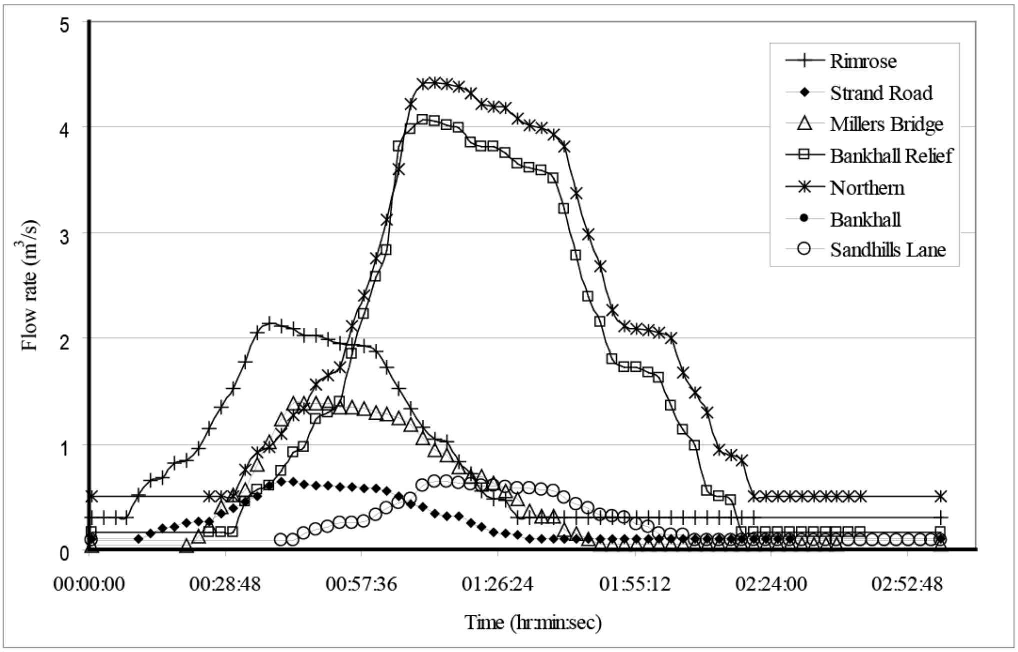

Figure 5.11 Stormwater runoff hydrograph for migrating downstream storms

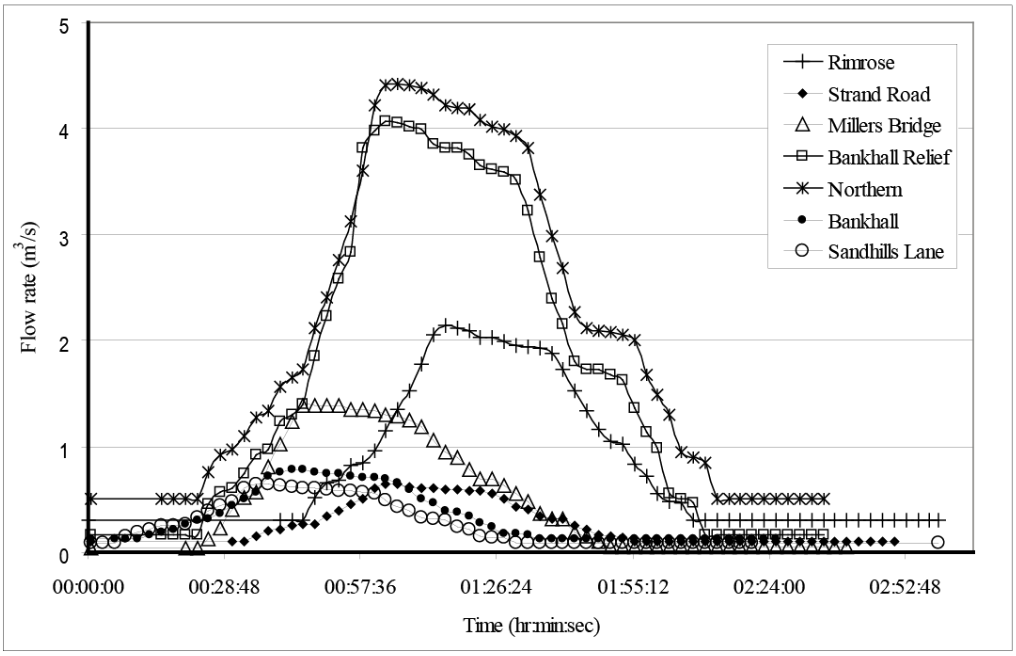

Figure 5.12 Stormwater runoff hydrograph for migrating upstream storms

Table 5.5 Distances between CSO chambers (Thomas, 2000)

CSO chambers	Distance (m)
Rimrose to Strand Road	895
Strand Road to Millers Bridge	740
Millers Bridge to Bankhall Relief	465
Bankhall Relief to Northern	19
Northern to Bankhall	710
Bankhall to Sandhills Lane	350
Sandhills Lane to Wastewater treatment plant	196

Time delays were introduced to generate the migrating runoff hydrographs. Longer distances between subsequent CSO chambers were given higher time delays between the corresponding runoff hydrographs. Introduction of time delays for migrating downstream storms starts at Strand Road, whereas for migrating upstream storms at Bankhall. Time delay information for migrating downstream and upstream storms is given in the Table 5.6.

Table 5.6 Time delays for migrating downstream and upstream storms

Catchment	Migrating downstream time delays (min)	Migrating upstream time delays (min)
Rimrose	0	40
Strand Road	10	30
Millers Bridge	20	20
Bankhall Relief	25	15
Northern	25	15
Bankhall	35	5
Sandhills Lane	40	0

Bankhall Relief and Northern catchments were assumed to have the same time delays since the corresponding CSO chambers are only 19 m away from each other.

5.6.2 Migrating storm pollutographs

Pollutographs were also introduced the same time delays as the corresponding runoff hydrographs. These time delay changes can be seen from the pollutographs shown in the Figures 5.13 to 5.16. The full set of generated pollutographs for migrating storms can be found in the Appendix A.

Migrating downstream storm pollutographs

Figures 5.13 and 5.14 show the BOD pollutographs and the corresponding stormwater runoffs for the Millers Bridge and Sandhills Lane catchments. The concentration levels are different in two figures due to the different land-uses. However, the migrating effects can be clearly visualized in the two figures. The peak of the BOD pollutograph of the Figure 5.13 is roughly at the 30 minutes of the storm. However, the peak of BOD pollutograph at Figure 5.14 has migrated roughly to the 1 hr time duration. This is according to the migration of the corresponding stormwater runoff hydrograph.

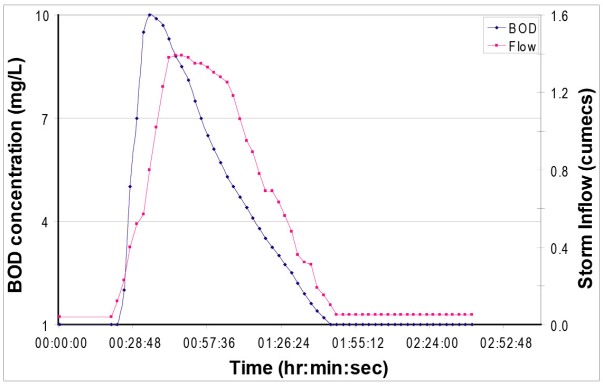

Figure 5.13 BOD pollutograph for Millers Bridge
for migrating downstream storms

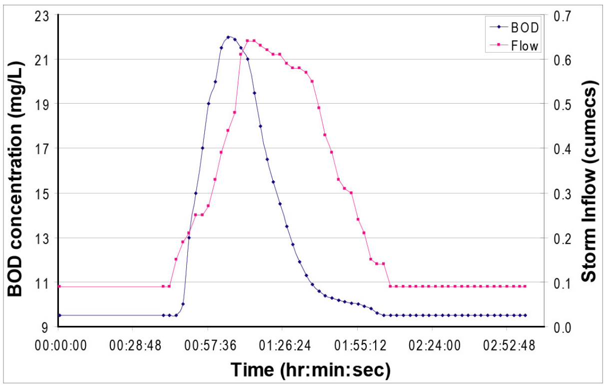

Figure 5.14 BOD pollutograph for Sandhills Lane
for migrating downstream storms

Migrating upstream storm pollutographs

Figures 5.15 and 5.16 show the COD pollutographs and the corresponding stormwater runoffs for the Rimrose and Bankhall catchments. The concentration levels are different in the two catchments due to the different land-uses. However, the migrating upstream effects can be clearly visualized in the two figures. The peak of the COD pollutograph of the Rimrose catchment is roughly at the 1 hr of the storm. However, the peak of COD pollutograph at Bankhall catchment has migrated roughly to the 30 minutes of time duration. This is according to the migration of the corresponding stormwater runoff hydrograph. It is noted herein that the Bankhall catchment is at the downstream of the combined sewer network.

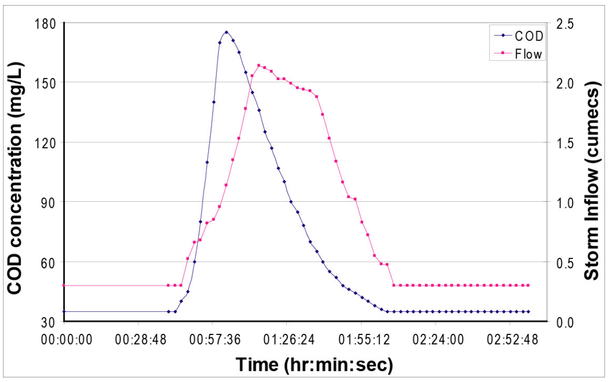

Figure 5.15 COD pollutograph for Rimrose
for migrating upstream storms

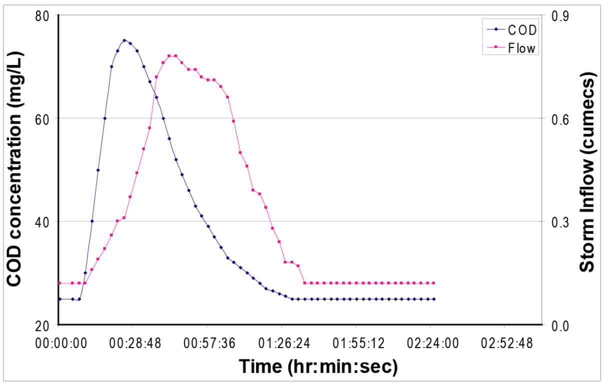

Figure 5.16 COD pollutograph for Bankhall for migrating upstream storms

5.7 Summary and conclusions

This chapter discusses the pollution levels in wastewater and stormwater. The spatial and temporal variations were introduced to the pollutographs. This is the novelty of the chapter. Different storm conditions, including single, two consecutive and migrating storms were considered in generating the pollutographs. They were generated according to the first flush concept. First flush in most of the catchments produces sediment wash-off and ultimately ends at the drainages. This leads to introduce problems in capacity reduction in drainages and cleansing the drainages. However, by considering the generated pollutographs above, it can be concluded herein that the pollutographs have a direct influence of the land use characteristics.

These generated runoff hydrographs and pollutographs are extensively used in the coming two chapters to compute pollution load from the hydraulic and water quality simulations.

CHAPTER 6

SNAPSHOT OPTIMIZATION

6.1 Introduction

This chapter develops the multi-objective optimization approach in controlling combined sewer system. Pollution load to the receiving water and the cost of wastewater treatment have incorporated in developing the optimization approach. This is the novelty of the Chapter 6. The ultimate objective of this chapter is to obtain a single set of optimal control settings irrespective of the duration of the storm; in other words, to obtain a singe set of control settings throughout the duration of the storm.

6.2 Pollution load evaluation

Effluent quality index (*EQI*) is formulated to calculate the pollution load in a water body as a single variable. Five important water quality parameters, total suspended solids (*TSS*), chemical oxygen demand (*COD*), five-day biochemical oxygen demand (*BOD*), total Kjeldahl nitrogen (*TKN*) and nitrates/nitrites (*NOX*) are accumulated together in forming this single measure. A detailed explanation of this *EQI* can be found in subsection 4.5.3 and Rathnayake and Tanyimboh (2012).

Equation (4.2) in subsection 4.5.3 gives the mathematical formulation of *EQI* and again expresses here in Equation (6.1).

$$EQI = \frac{1}{1000\left(t_f - t_0\right)} \int_{t_0}^{t_f} \left(2C_{TSS} + C_{COD} + 2C_{BOD} + 20C_{NOX} + 20C_{TKN}\right) Q_e\left(t\right) dt \qquad (6.1)$$

where $Q_e(t)$, t_f, and t_0 are the flow rate, final and initial time respectively. C_{TSS}, C_{COD}, C_{NOX}, C_{BOD} and C_{TKN} are the concentrations of total suspended solids, chemical oxygen demand, nitrates and nitrites, five-day biochemical oxygen demand and total Kjeldahl nitrogen, respectively.

6.3 Wastewater treatment cost

Available fund is always a critical factor in most of the new projects and maintenance of existing facilities. More often, it is the key factor, which decides the project or maintenance is going to be happened or not. This is the same with wastewater treatment plants. The funding availability for maintenance and operation of wastewater treatment plants is limited. Therefore, authorities always want to minimize the maintenance and treatment cost of treatment plants.

Wastewater treatment plants are generally designed and constructed to have an overall capacity of 6×DWF. However, the full treatment capacity is further limited to 3×DWF and the surplus flow is temporarily stored in equalization tanks. These equalization tanks have the same role of primary sedimentation tanks. In a case where the total flow is more than 6×DWF, the storm tanks fill completely and overflow to the nearby natural water. Based on various cost models from literature (described in subsection 4.5.4), a generic cost function was generated. The cost function is based on the treated water volume. The treatment cost (C_T) is described in the following Equation (6.2).

$$C_T = \begin{cases} A \times q_T^{\,0.659}, & q_T \leq 3 \times DWF \quad (6.2a) \\ B + \frac{2}{3}C, & 3 \times DWF \leq q_T \leq 6 \times DWF \quad (6.2b) \\ B + \frac{2}{3}D, & q_T \geq 6 \times DWF \quad (6.2c) \end{cases}$$

where,

$$A = 916.862 \times 86400^{0.659} \tag{6.3}$$

$$B = 916.862 \times (3 \times DWF)^{0.659} \tag{6.4}$$

$$C = 1.69 \times (q_T - 3 \times DWF) + 11376 \tag{6.5}$$

$$D = (1.69 \times 3 \times DWF) + 11376 \tag{6.6}$$

where q_T is the treated wastewater volume flow rate.

Total treatment cost, including personnel, energy, maintenance, waste and other costs, when the wastewater flow rate is less than or equal to 3×DWF is given by Hernandez-Sancho and Sala-Garrido (2008). This is explained in detailed in the section 4.6 at Chapter 4. However, the additional cost, including storage cost, should be included, when the flow rate is more than 3×DWF. Equations (6.2b) & (6.2c) give the wastewater treatment cost at the treatment plant and the operational and maintenance cost for storage tanks when the flow rate is in between 3×DWF and 6×DWF and when more than 6×DWF respectively. Numerical value 2/3 in Equations (6.2b) & (6.2c) is used as a typical conversion rate for € to US$.

6.4 Multi-objective optimization problem formulation

6.4.1 Objective functions

The optimization of many engineering problems engages multiple objectives, which are usually challenging each other. In controlling combined sewer networks, maintaining receiving water quality due to CSOs and wastewater treatment cost at downstream treatment plant are two such objectives. Therefore, the first objective function was formulated to minimize the pollution load to receiving water through the CSOs. *EQI*, which gives the pollution load, was used to formulate this objective function, which is given in Equation (6.7).

$$Minimize \;\; F1 = \sum_{t=1}^{T} \sum_{i=1}^{n_0} P_{i,t} \tag{6.7}$$

where T, t, n_0 and $P_{i,t}$ are the last time step of the hydraulic simulation, results reporting time step from hydraulic simulator, the number of interceptor nodes or CSO chamber points and the pollution load to the receiving water from the i^{th} CSO chamber at time t respectively. $P_{i,t}$ can be expressed as shown in the Equation (6.8).

$$P_{i,t} = EQI_{i,t} \tag{6.8}$$

where $EQI_{i,t}$ is the effluent quality index at node i at time t (shown in Equation (6.1)).

The second objective function was formulated to minimize the cost of wastewater treatment at the downstream treatment plant (shown in Equation (6.2)).

$$Minimize \;\; F2 = \sum_{t=1}^{T} C_{T,t} \tag{6.9}$$

where $C_{T,t}$ is the treatment cost at the wastewater treatment plant at time t.

6.4.2 Constraints

Schematics of a typical interceptor sewer and a CSO chamber are shown in the Figure 6.1. Inflows from catchments' DWF and stormwater runoffs (I_i) are introduced to CSO chambers.

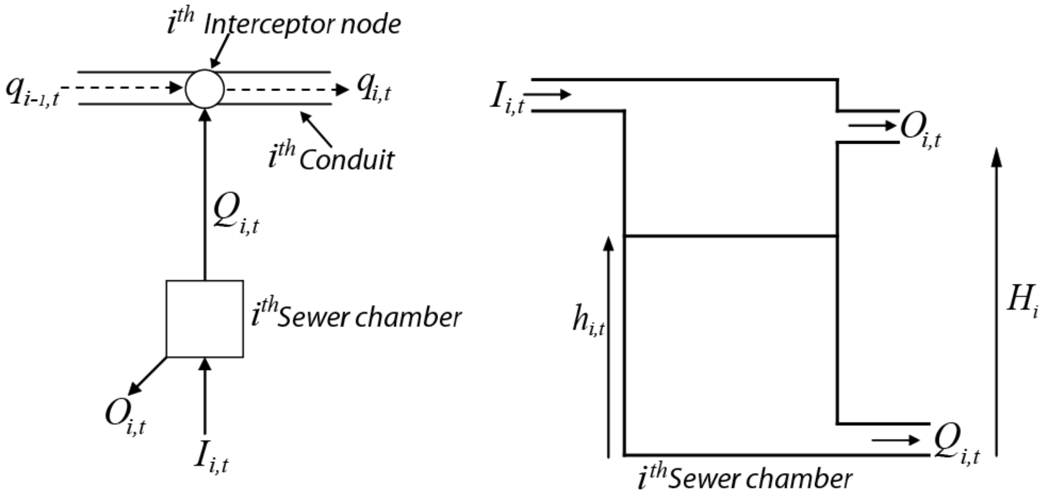

Figure 6.1 Schematic diagram of sewer chamber

where,

$h_{i,t}$ – Water level in i^{th} sewer chamber at time t

H_i – Spill level of i^{th} sewer chamber

$I_{i,t}$ – Catchment inflow to node i at time t

$q_{i,t}$ – Through flow in interceptor sewer at node i at time t

$Q_{i,t}$ – Flow from i^{th} sewer chamber to interceptor node i at time t

$O_{i,t}$ – Combined sewer over flow discharge at node i at time t

Considering the continuity of the i^{th} interceptor node, the Equation (6.10) can be obtained.

$$Q_{i,t} + q_{i-1,t} - q_{i,t} = 0 \tag{6.10}$$

Considering the continuity of the i^{th} sewer chamber, shown in Figure 6.1, the following two conditional Equations (6.11) & (6.12) can be obtained. When the water level in the sewer chamber ($h_{i,t}$) is less than the spill level of the chamber (H_i), Equation (6.11) governs the continuity, whereas Equation (6.12) is for when the water level of sewer chamber is greater than the spill level.

$$A_C \frac{\Delta h_{i,t}}{\Delta t} = I_{i,t} - Q_{i,t} \qquad ; h_{i,t} < H_i \tag{6.11}$$

$$A_C \frac{\Delta h_{i,t}}{\Delta t} = I_{i,t} - Q_{i,t} - O_{i,t}; \qquad h_{i,t} > H_i \tag{6.12}$$

where A_C is the surface area of the CSO chamber.

In addition to the continuity equations, flow rates through the conduits are limited to have defined maximum flow rates. These flow rate constraints in i^{th} conduit at time t are described in Equation (6.13)

$$0 \le q_{i,t} \le q_{\max,i} \tag{6.13}$$

where $q_{max,i}$ is the maximum flow rate at i^{th} conduit.

6.4.3 On-line storage tanks

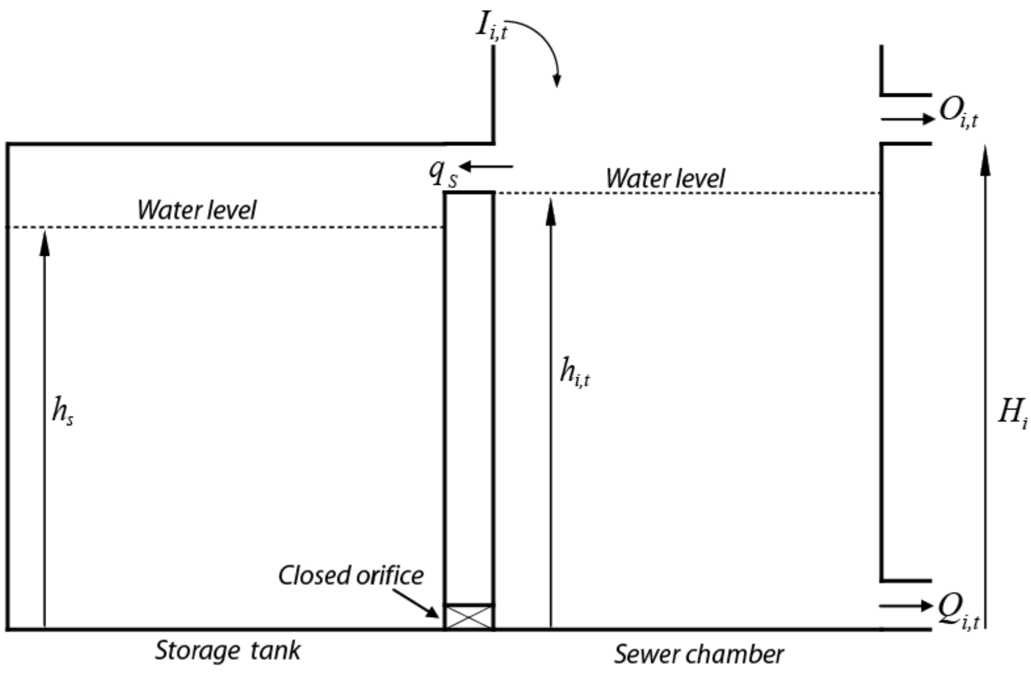

Figure 6.2 Schematic diagram of sewer chamber with on-line storage tank

Sewer chambers often have storage tanks. These can be either on-line or off-line storage tanks. A detailed explanation of these storage tanks is given in the Chapter 4 subsection 4.2.7. Figure 6.2 shows the schematic diagram of an on-line storage tank.

q_s and h_s are flow to the storage tank from CSO chamber and the water level of the storage tank respectively. When the water level of the sewer chamber ($h_{i,t}$) reaches the spill level of the chamber (*H*i), the storage tank starts filling. Flow to the storage tank (q_s) stops when the storage tank reaches its maximum capacity. This will then lead to CSOs through the corresponding CSO chamber.

6.5 Solutions to the multi-objective optimization approach

6.5.1 Solution algorithm for snapshot optimization

The hydraulic model, U.S. EPA SWMM 5.0 (Rossman, 2009) and the multi-objective optimization module, NSGA II (Deb et al., 2002) were coupled using "C" programming language. As it was stated earlier in Chapter 3 section 3.4, NSGA II has already been successfully applied to many practical optimization problems in various disciplines, including urban wastewater systems (Vojinovic et al., 2008; Fu et al. 2010; Shinma et al., 2011).

It is assumed here that wastewater flow from CSO chamber to the interceptor sewer is controlled using an orifice at the bottom of the CSO chamber. The orifice openings were initially generated randomly. Hence, the decision variables of the optimization approach, the flow rates through the interceptor sewer sections ($q_{i,t}$) were indirectly generated. Next, a full hydraulic simulation, including water quality routing was carried out using SWMM 5.0. The results obtained from the hydraulic simulation were used to calculate the pollution load, *F1* and the wastewater treatment cost, *F2*. Finally, the NSGA II optimization module was run to obtain the optimal solutions. The obtained optimal solutions are plotted as a Pareto optimal front.

Depending on the sewer network controller's aspirations, optimal solutions can be selected from the Pareto optimal front. Then, the optimal control settings (orifice opening) for the corresponding optimal solutions can be obtained. The algorithm which was used to find the solutions to the multi-objective optimization approach is shown in the Figure 6.3.

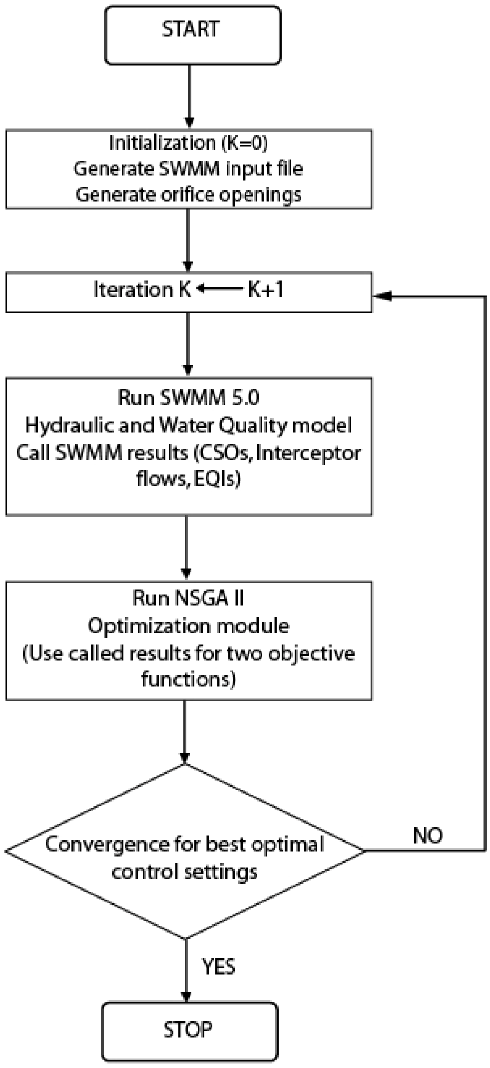

Figure 6.3 Solution algorithm for optimal control settings for snapshot optimization

6.5.2 Different constraint handling methods

Continuity equations shown in Equations (6.10) to (6.12) are automatically satisfied by the hydraulic model (SWMM 5.0). However, two alternative formulations were separately used in handling the flow constraints shown in the Equation (6.13).

NSGA II constraint handling approach

Formulations for Deb's tournament constraint handling approach (NSGA II approach) were developed in the first approach. These constraint-handling formulations were externally applied to the hydraulic model. They use the binary tournament selection, where two potential solutions are picked at random from the population and the better solution is selected. A detailed description of this constraint handling approach can be found in Chapter 3 subsection 3.4.4.

SWMM constraint handling approach

Controlling flows inside the sewer conduits using the hydraulic model is the second approach (SWMM 5.0 approach). The maximum flow rates allowed through conduits, shown in Equation (6.13), were formulated inside the hydraulic model. SWMM 5.0 conduit features in defining the maximum flow rates were used in formulating the maximum flow rates allowed through conduits.

6.6 Case study

Strengths and limitations are always in the case study research (Hodkinson, 2001). Case studies can help people understanding the complex inter-relationships. Importantly most of these case studies are grounded in lively reality. They can facilitate the exploration of the unexpected and unusual. In addition, multiple case studies can enable research to focus on the significance of the characteristics of the problem. Furthermore, in combining all above stated strengths lead to facilitate rich conceptual or theoretical developments of the problems in real world.

However, there are some limitations of the case study research approach. It is sometimes difficult to find all the required data of the considered case study. Therefore, assumptions are often in most of the case study approaches. In addition, it is sometimes difficult to explain the unexpected results from the case studies in theory. They just simply the results and further research is always necessary for the output of the case studies. Case studies are usually time-consuming. Sometimes they are costly, especially when it comes to collect the required data by the research group or the literature has little data for that particular case study. Therefore, there are merits and weakness of the case study approach.

6.6.1 Interceptor sewer system

It is difficult to find a fully illustrated case study from the literature to model the developed optimization strategy. There are some literature found case studies, however, lack of the availability of geometrical and hydraulic information lead to stick to one literature found case study interceptor sewer system.

The interceptor sewer system in Thomas (2000) was modified as presented in the following paragraphs. Longitudinal section of the simplified interceptor sewer is shown in Figure 6.4.

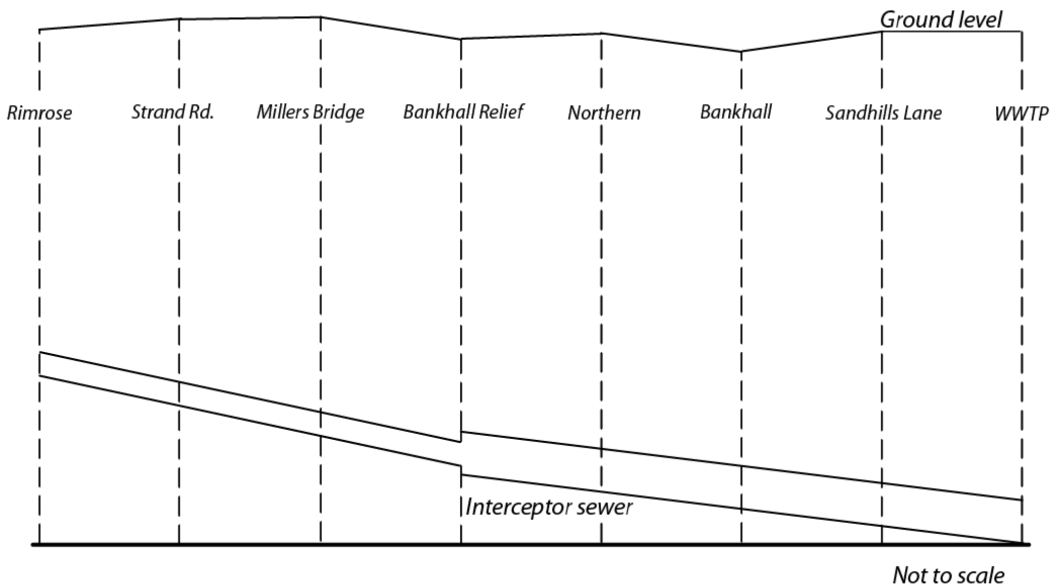

Figure 6.4 Longitudinal section of the interceptor sewer (Thomas, 2000)

It is a common practice to have storage tanks in the sewer systems. Therefore, in addition to the CSO chambers described in Thomas (2000), two additional on-line storage tanks (T8 and T9) were introduced to Strand Rd. (T2) and Northern (T5) CSO chambers respectively. Figure 6.5 gives a detailed graphical view of the modified interceptor sewer system.

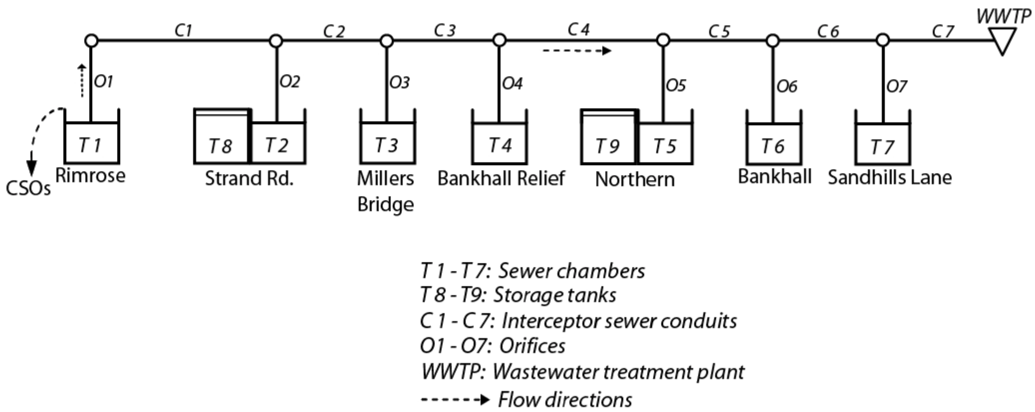

Figure 6.5 Modified interceptor sewer system

Tables 6.1 and 6.2 give the geometrical information of the interceptor sewer and the geometrical information of the CSO chambers and the DWFs.

Table 6.1 Geometrical information for interceptor and inflows (Thomas, 2000)

Interceptor point	Invert elevation (m)	Sewer diameter (m)	Length of sewers (m)
Rimrose (T1)	4.075	1.66	895
Strand Rd. (T2)	2.882	1.66	740
Millers Bridge (T3)	1.895	1.66	465
Bankhall Relief (T4)	1.275	2.44	19
Nothern (T5)	1.256	2.44	710
Bankhall (T6)	0.546	2.44	350
Sandhills Lane (T7)	0.196	2.44	196

Table 6.2 Geometrical information for CSO chambers and DWFs (Thomas, 2000)

Interceptor point	Chamber area (m^2)	Chamber height (m)	Orifice height (m)	Orifice width (m)	DWF (m^3/s)
T1	282.82	6.42	1.45	1.25	1.54
T2	136.03	7.91	0.625	1.70	0.34
T3	50.31	8.95	0.625	1.50	1.01
T4	169.78	9.04	0.625	2.08	0.83
T5	328.24	9.18	1.45	2.65	2.63
T6	167.06	9.47	0.625	1.80	0.4
T7	147.95	10.26	0.625	1.65	0.4
T8	136.03	7.91	NA	NA	NA
T9	328.24	9.18	NA	NA	NA

6.6.2 Application of multi-objective optimization for single storm condition

The developed multi-objective optimization model (section 6.4) was applied to the above stated simplified interceptor system. Maximum flow rates allowed through C1, C2 and C3 are 3.26 m^3/s and that of C4, C5, C6 and C7 are 7.72 m^3/s. Therefore, flow rates inside the conduits were kept below these maximum flow rates using the constraint handling approaches. T8 and T9 on-line storage tanks were controlled, such that no overflows occur from these storage tanks. This was performed using the control rules in SWMM 5.0.

Diurnal effects of the DWF were not considered. Therefore, average flow rates of DWFs (given in Table 6.2) were fed to the T1, T2, T3, T4, T5, T6, and T7 CSO chambers. Runoff hydrographs for a single storm given in Figure 5.2, subsection 5.4.1 were fed to the corresponding CSO chambers. These runoff hydrographs were for mild storm events. Therefore, they were multiplied by a factor of five to obtain high intensity storm profiles (Thomas, 2000).

Pollution levels of the DWFs are given in the Table 5.1, section 5.2. A medium range of pollution level (TSS – 220 mg/L, COD – 500 mg/L, BOD – 220 mg/L, TKN – 25 mg/L and NOX – 40 mg/L) was assumed for the DWFs. These pollution levels were fed with the DWFs to the corresponding CSO chambers. As it was already stated in the Chapter 5, subsection 5.4.2, Table 5.4, five different land-uses, including residential, industrial, commercial, agricultural and mid urban were assumed to the CSO chamber catchments. Generated pollutographs for TSS, COD, BOD, TKN and NOX were fed with the runoff hydrographs for the corresponding CSO chambers. More details on these generated pollutographs can be found in Chapter 5, subsection 5.4.3 and in the Appendix A.

A basic real-coded NSGA II program was used to perform the multi-objective optimization. The optimization process was carried out with a population of 100, 100 generations and a crossover probability of 1. The distribution indices for crossover and mutation operators were kept at 20. Different mutation probabilities were tried in different runs. The reason for selecting different mutation probabilities was to compare the performance of the mutation probabilities for this optimization problem. Polynomial mutation, described in Deb et al., (2002), was used for this optimization approach. The polynomial mutation operator creates a new value for the decision variable, which is near the vicinity of the original value using a probability distribution. Many optimization runs with different random seeds were conducted. This is to ensure that the developed GA produces consistence optimal results.

Routing time-step in SWMM 5.0 was kept at 30 seconds, and the snapshot optimization model was run for 15 minutes, 30 minutes, 1 hour and 2 hours and 30 minutes separately. Two separate runs were conducted for each time interval for NSGA II and SWMM constraints handling approaches. As it is already stated in the preceding paragraph different mutation probability were tested. However, these mutation probability tests were only performed to the 15 minutes interval. Based on the best performing mutation probability the next optimization runs were carried out.

6.7 Results and discussion

6.7.1 Optimization results for 15 minutes

Snapshot optimization for 15 minutes using NSGA II constraint handling approach

Figure 6.6 shows the Pareto optimal fronts archived from the snapshot optimization for different mutation probabilities. These Pareto optimal fronts were developed for the feasible solutions. Mutation probability, 0.14 (1/7 ≈ 0.14) corresponds to the reciprocal of the number of decision variables of the developed multi-objective optimization problem. It can be seen from the Figure 6.6 that the performance of mutation probability 0.6, over the entire population of solutions, is better than any of other optimal fronts. This suggests that the higher mutation probability is beneficial for this multi-objective optimization problem.

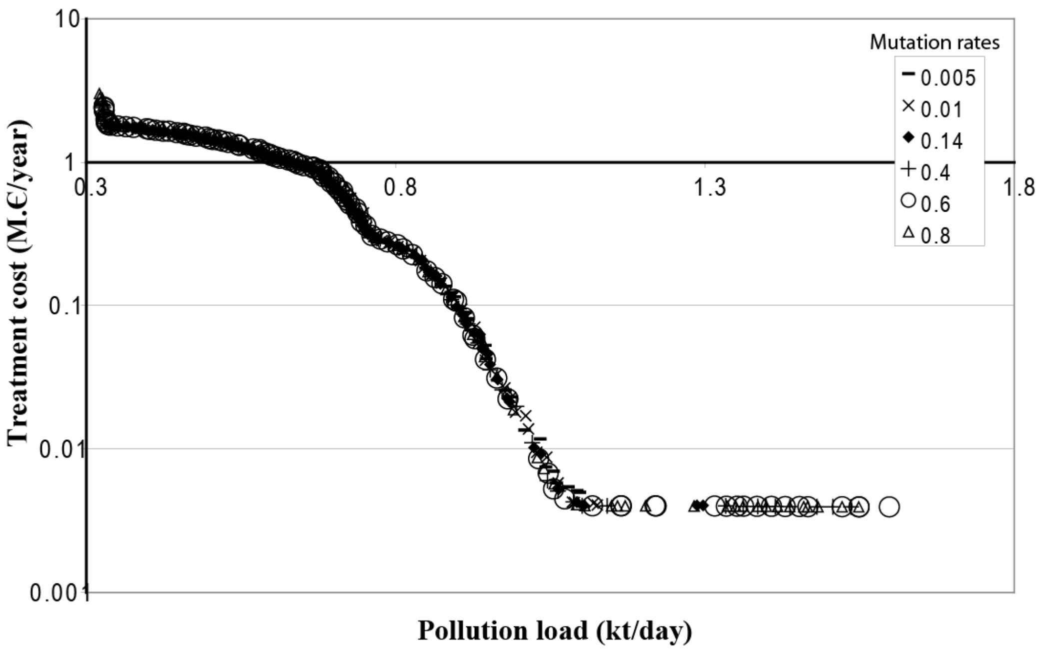

Figure 6.6 Pareto optimal fronts for different mutation rates for NSGA II approach for 15 minutes

Several mutation probabilities including, 0.2, 0.3, 0.5, and 0.7, which are not presented in the Figure 6.6, were tested. This is because they were not outperforming the mutation probability 0.6 and in addition, to keep the clarity of the Figure 6.6.

Figure 6.7 shows the Pareto optimal fronts obtained for different random seeds for the mutation rate of 0.6. All the Pareto optimal fronts are nearly the same. This suggests that the developed multi-objective optimization problem is robust and illustrates a consistent performance.

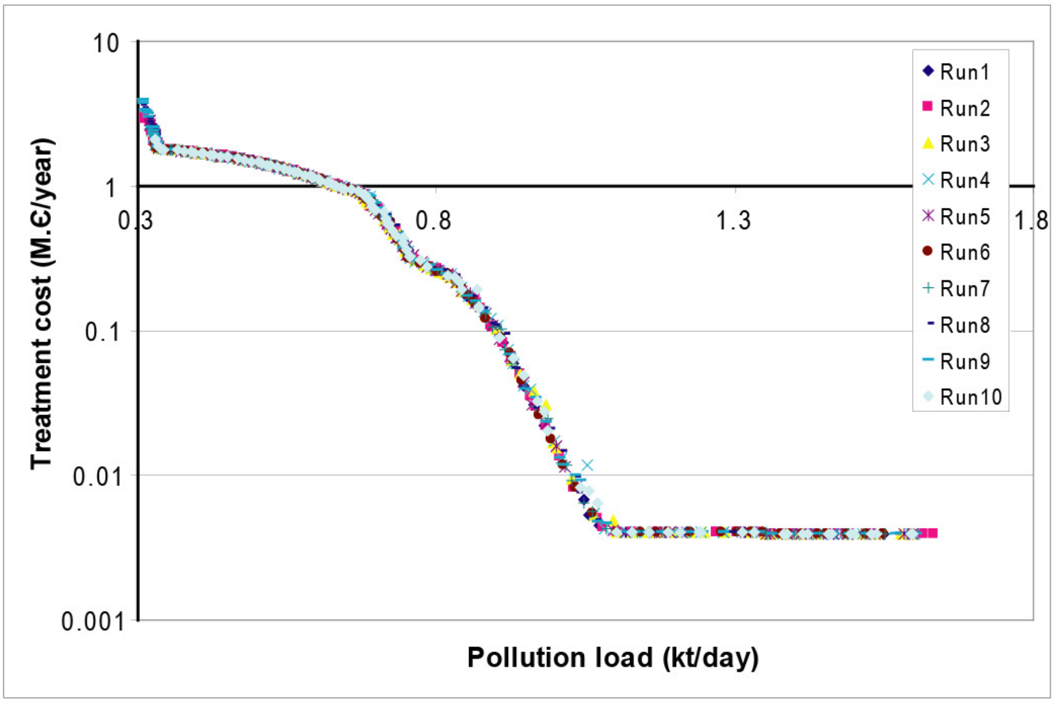

Figure 6.7 Pareto optimal fronts for different random seeds for NSGA II approach for 15 minutes

Figures 6.8 and 6.9 give the progress of the GA for the two objective functions. Minimum values of the two objectives in several generations show the convergence of the multi-objective optimization approach.

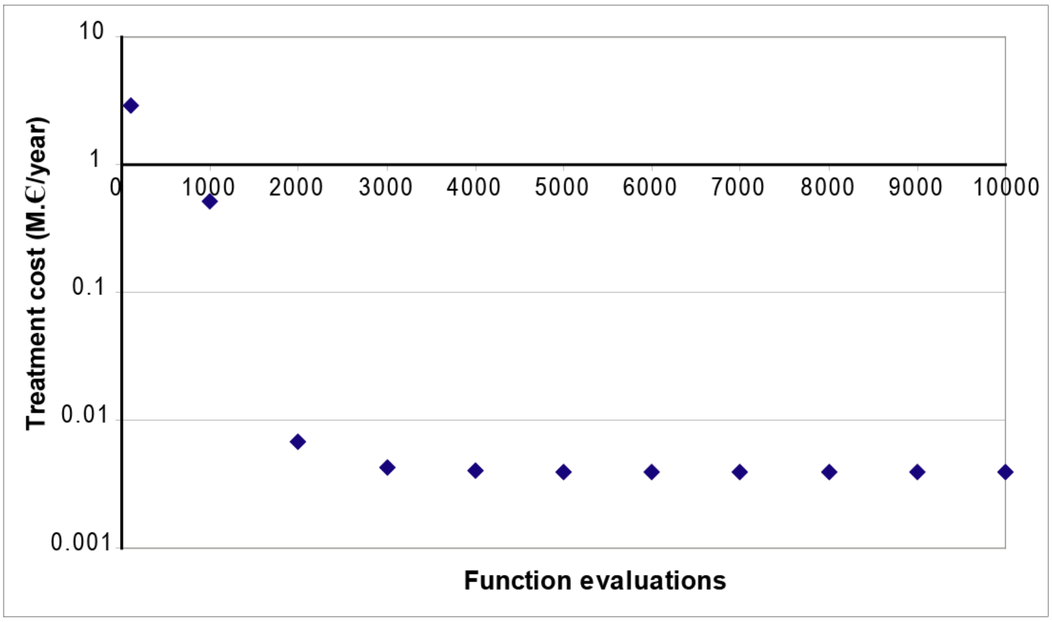

Figure 6.8 Function evaluations for minimum treatment cost for NSGA II approach for 15 minutes

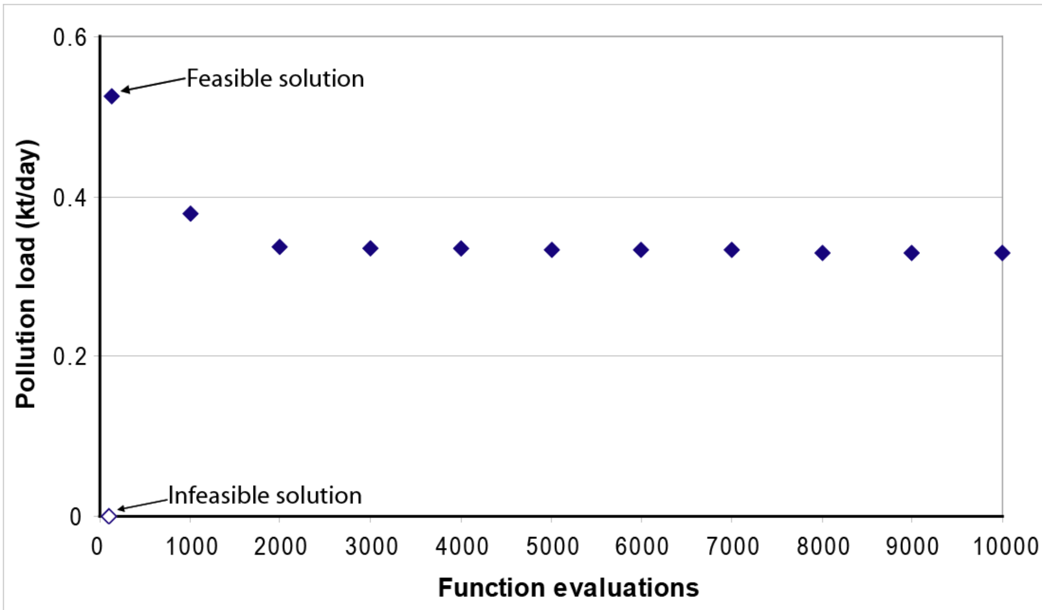

Figure 6.9 Function evaluations for minimum pollution load for NSGA II approach for 15 minutes

As it is expected in GAs, it can be clearly seen that a rapid convergence during the initial generation. However, during the later generations convergence rate keeps steady. The minimum pollution load at 1st generation shows a value of close to 0. However, further analysis has shown that this particular solution is infeasible, which has a constraint violation. Therefore, the solution labeled as "Feasible solution" in the 1st generation is minimum pollution load.

Snapshot optimization for 15 minutes using SWMM constraint handling approach

Figure 6.10 demonstrates the Pareto optimal fronts archived from the snapshot optimization for different mutation probabilities using the SWMM constraint handling approach. Mutation probability of 0.4 over the entire population of solutions gives the best Pareto optimal front. This again suggests that the higher mutation probability is beneficial for the developed multi-objective optimization problem. In addition to the mutation probabilities presented in Figure 6.10, few other mutation rates were tested. However, they have not shown the over performing Pareto optimal fronts than that of 0.4 mutation rate.

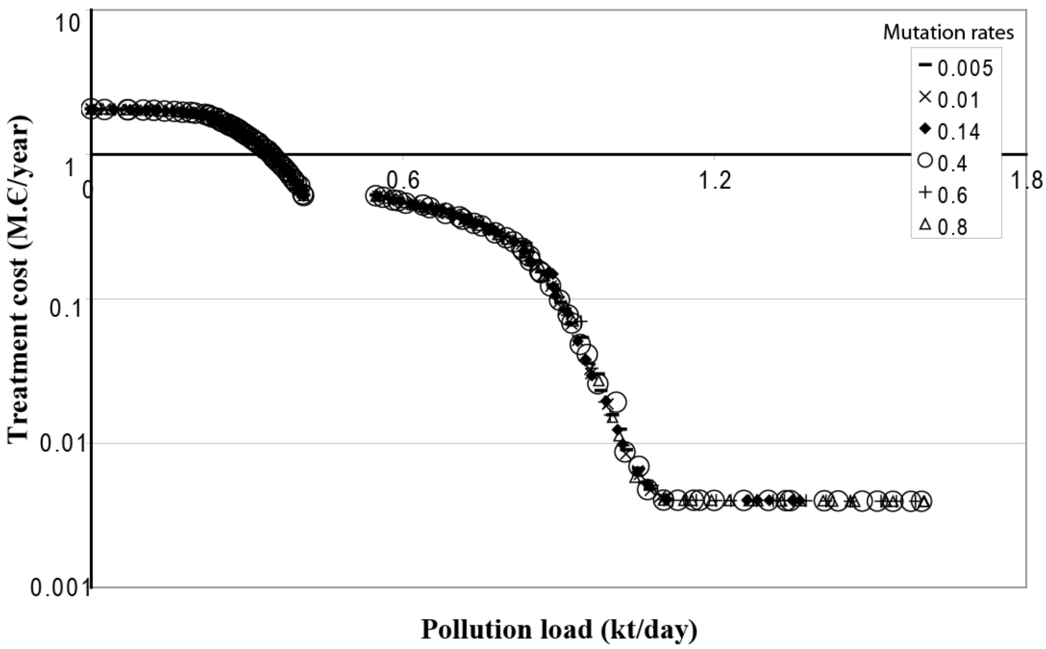

Figure 6.10 Pareto optimal fronts for different mutation rates for SWMM approach for 15 minutes

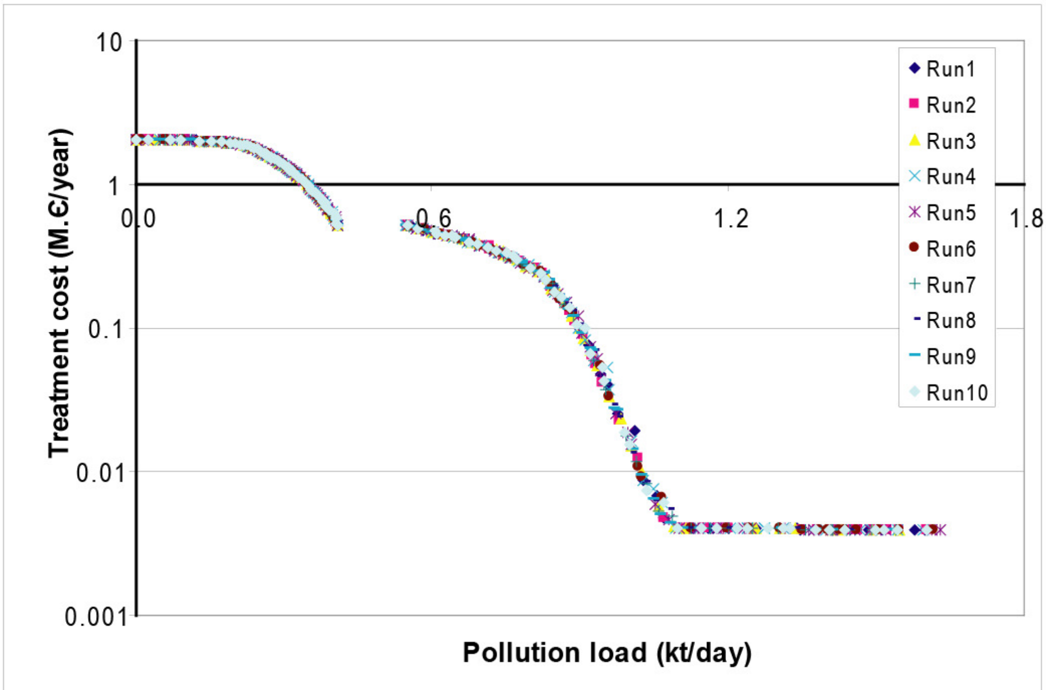

Figure 6.11 Pareto optimal fronts for different random seeds for SWMM approach for 15 minutes

Figure 6.11 shows the Pareto optimal fronts obtained for different random seeds for the mutation rate of 0.4. As they are in Figure 6.7, all the Pareto optimal fronts are nearly the same. This again suggests that the developed multi-objective optimization problem is robust and illustrates a consistent performance under the SWMM constraint handling approach.

Figures 6.12 and 6.13 illustrate the progress of the GA for the two objective functions under the SWMM constraint handling approach. Figure 6.12 shows the expected performance for a GA. However, the Figure 6.13 has the 0 minimum pollution load starting from the 1st generation. Therefore, the performance of the GA for the 2nd objective is not clear enough. However, Appendix B, which gives the function evaluations for the results of other snapshot optimization approaches, shows the rapid convergence rate.

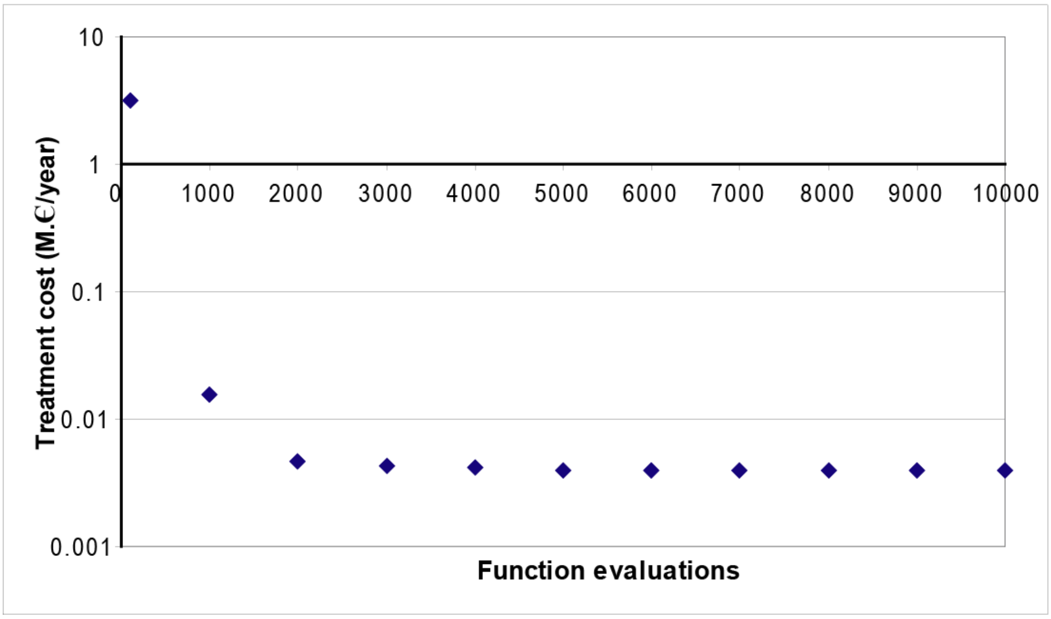

Figure 6.12 Function evaluations for minimum treatment cost for SWMM approach for 15 minutes

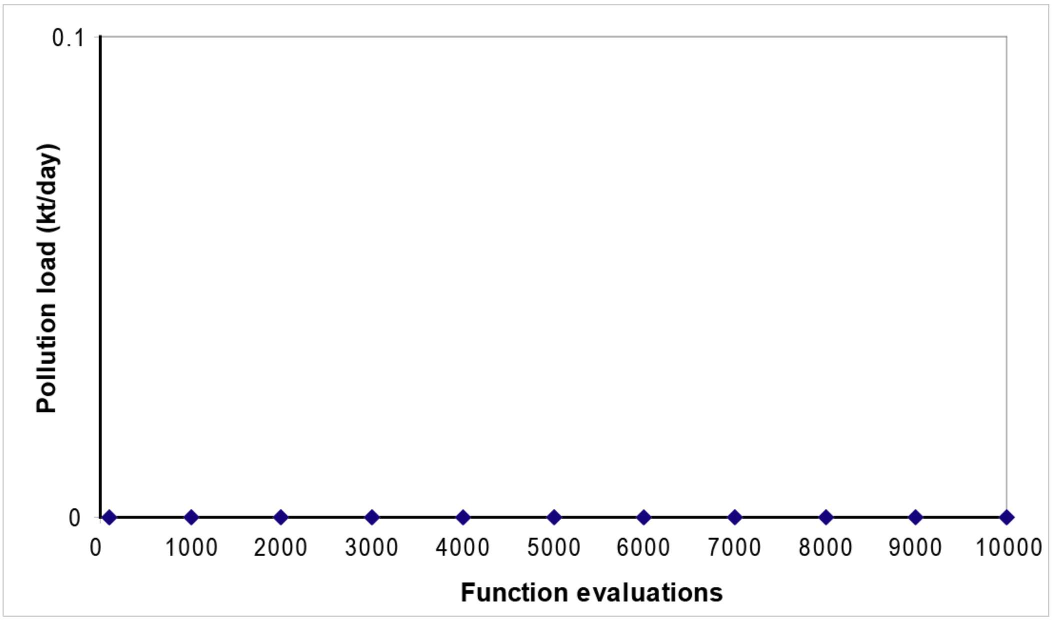

Figure 6.13 Function evaluations for minimum pollution load for SWMM approach for 15 minutes

Comparison of Pareto optimal fronts for two constraint handling approaches

The best Pareto optimal fronts achieved in two different constraint handling approaches can be seen from the Figure 6.14.

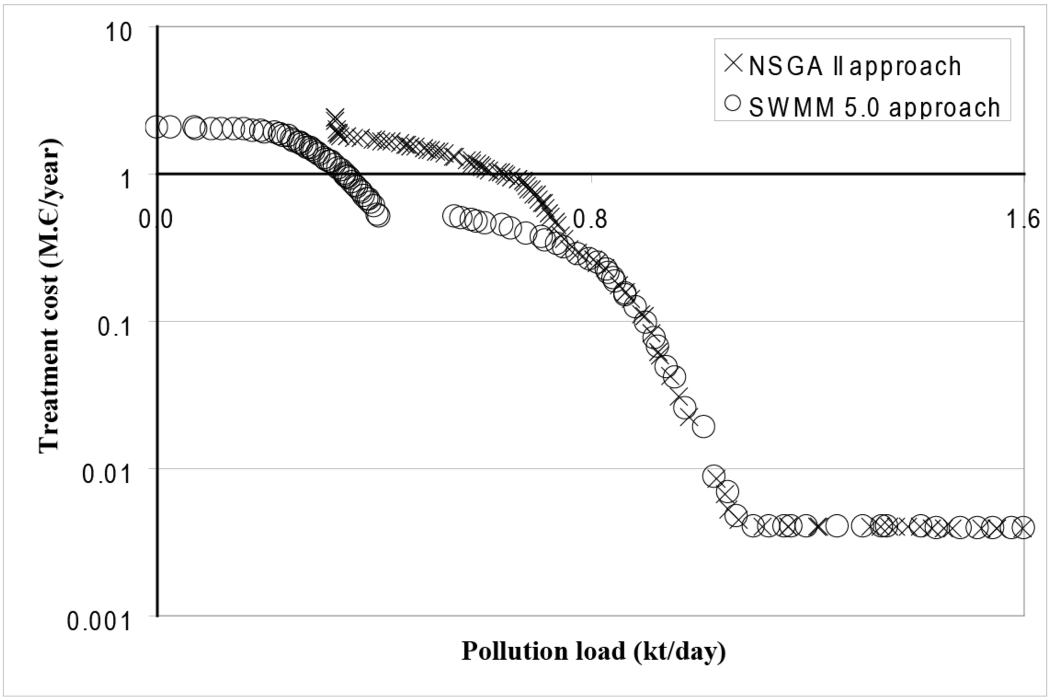

Figure 6.14 Best Pareto optimal front achieved for 15 minutes

During the second half of the pollution load axis (0.8 - 1.6 kt/day), these two Pareto optimal fronts virtually the same. However, during the first half of the pollution load axis (0 – 0.8 kt/day) they show a separation. In fact, the Pareto optimal front from SWMM constraint handling approach shows the minimum solutions than that of NSGA II constraint handling approach.

6.7.2 Hydraulic simulation results for 15 minutes

Hydraulic simulation results from NSGA II constraint handling approach

Solutions A_{T1} to H_{T1} (Figure 6.15), along the best Pareto optimal front for NSGA II constraint handling approach, were selected for further assessment. Table 6.3 gives the control settings (orifice openings) for these selected solutions, A_{T1} to H_{T1} during the time period of 15 minutes. Solution A_{T1} gives the minimum pollution load to receiving water from NSGA II constraint handling approach. In other words, it has the maximum wastewater treatment cost. However, solution H_{T1} gives the minimum wastewater treatment cost at downstream treatment plant and which has the maximum pollution load to the receiving water.

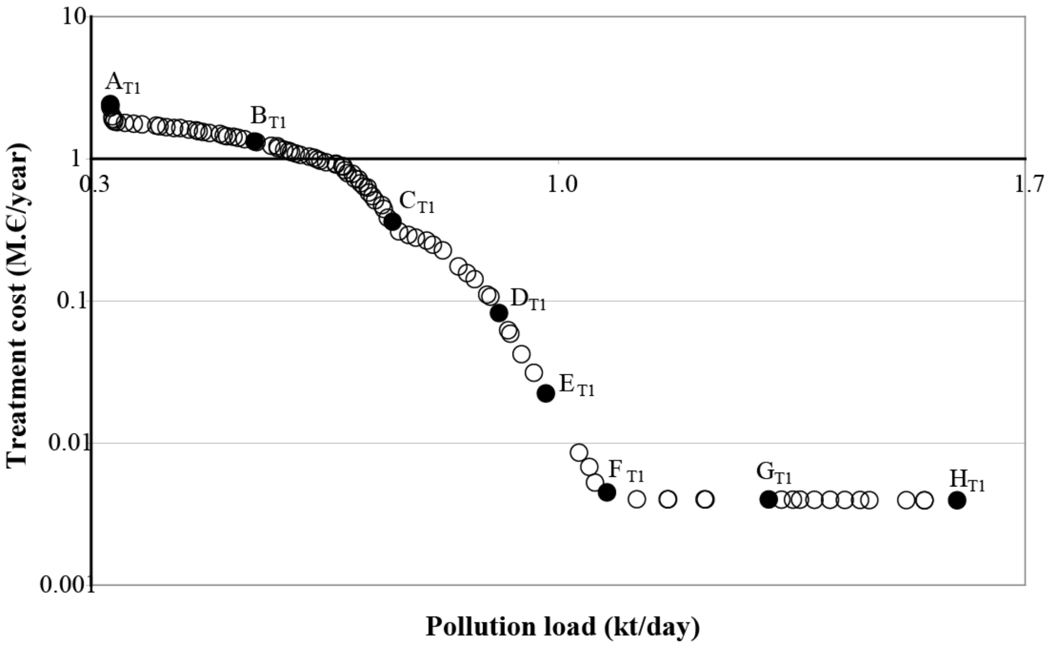

Figure 6.15 Selected solutions for T=15 minutes from NSGA II approach

Table 6.3 Orifice openings for selected solutions (A_{T1} – H_{T1})

Solution	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
A_{T1}	42.0	0	21.8	24.1	0	0	0
B_{T1}	42.8	0	8.7	13.7	0	0	0
C_{T1}	41.1	0	20.3	0.6	0	0	0
D_{T1}	41.8	0	11.2	0	0	0	0
E_{T1}	42.6	0	7.1	0	0	0	0
F_{T1}	43.8	0	1.0	0	0	0	0
G_{T1}	35.2	0	0	0	0	0	0
H_{T1}	5.0	0	0	0	0	0	0

Even though some of the orifice openings show 0 openings (orifices are closed), they are not numerically 0. For example, the exact numerical value of orifice opening for O6 at A_{T1} is 0.000152 m (0.152 mm). Therefore, this can be physically considered 0, in other words the orifice O6 is almost at the closed position.

Results from full hydraulic simulations for these selected solutions (A_{TI} – H_{T1}) are presented in the following tables (Tables 6.4 to 6.7).

Table 6.4 Flow rates through interceptor sewer sections at t = 15 minutes for selected solutions (A_{T1} – H_{T1})

Solution	Interceptor sewer flow rates (m^3/s)						
	C1	C2	C3	C4	C5	C6	C7
A_{T1}	2.74	1.65	3.25	5.85	4.89	3.02	1.80
B_{T1}	2.76	1.61	1.83	3.68	3.00	1.61	0.72
C_{T1}	2.72	1.62	3.21	3.16	2.28	0.65	0.10
D_{T1}	2.75	1.65	2.20	2.08	1.25	0.21	0.01
E_{T1}	2.77	1.59	1.65	1.48	0.73	0.07	0
F_{T1}	2.76	1.66	0.73	0.35	0.03	0	0
G_{T1}	2.44	1.41	0.41	0.11	0	0	0
H_{T1}	0.39	0.10	0.01	0	0	0	0

Table 6.4 gives the flow rates through the interceptor sewer sections at 15 minutes for solutions A_{T1} to H_{T1}. As it is already stated in subsection 6.6.2 the flow rates through sewer conduits were constrained to maximum flow rates. Maximum flow rates allowed through C1, C2 and C3 are 3.26 m^3/s and that of C4, C5, C6 and C7 are 7.72 m^3/s. It can be clearly seen in Table 6.4 that the flow rates through these conduits are less than to the maximum allowed flow rate for all the tabulated cases.

Table 6.5 Combined sewer overflow rates at CSO chambers at t = 15 minutes for selected solutions (A_{T1} – H_{T1})

Solution	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
A_{T1}	0	0	2.25	0	0	0	0
B_{T1}	0	0	3.72	0	0	0	0
C_{T1}	0	0	2.42	4.26	0	0	0
D_{T1}	0	0	3.44	4.36	0	0	0
E_{T1}	0	0	3.92	4.36	0	0	0
F_{T1}	0	0	4.54	4.37	0	0	0
G_{T1}	1.98	0	4.74	4.36	0	0	0
H_{T1}	4.49	0	4.74	4.36	0	0	0

Table 6.5 illustrates the CSO rates at CSO chambers at 15 minutes for solutions A_{T1} to H_{T1}. Solution A_{T1} that corresponds to the minimum pollution load to receiving water has smaller CSO rates than Solution H_{T1} that corresponds to the minimum wastewater treatment cost. However, it can be herein noted that the CSO chambers T2 and T5, which have on-line storage tanks, have no CSOs.

Table 6.6 Pollution loads at CSO chambers at t = 15 minutes for selected solutions (A_{T1} – H_{T1})

Solution	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
A_{T1}	0	0	0.329	0	0	0	0
B_{T1}	0	0	0.545	0	0	0	0
C_{T1}	0	0	0.353	0.399	0	0	0
D_{T1}	0	0	0.504	0.408	0	0	0
E_{T1}	0	0	0.573	0.408	0	0	0
F_{T1}	0	0	0.663	0.411	0	0	0
G_{T1}	0.214	0	0.691	0.410	0	0	0
H_{T1}	0.495	0	0.692	0.410	0	0	0

Table 6.6 shows the pollution load to the receiving water from CSO chambers for Solutions A_{T1} to H_{T1}. Solution A_{T1} has the minimum pollution load to the receiving water, whereas the Solution H_{T1} has the maximum pollution load.

Table 6.7 Wastewater depths at CSO chambers and storage tanks at t = 15 minutes for selected solutions (A_{T1} – H_{T1})

Solution	Wastewater depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
A_{T1}	5.26	6.26	8.22	5.53	8.17	7.17	7.63	1.75	7.42
B_{T1}	5.19	6.26	8.32	7.98	8.17	7.18	7.63	1.71	7.4
C_{T1}	5.34	6.26	8.23	8.45	8.17	7.18	7.63	1.76	7.42
D_{T1}	5.29	6.26	8.3	8.45	8.17	7.18	7.63	1.71	7.43
E_{T1}	5.22	6.26	8.33	8.45	8.17	7.18	7.63	1.71	7.34
F_{T1}	5.16	6.26	8.38	8.45	8.18	7.18	7.63	1.76	7.4
G_{T1}	5.66	6.26	8.39	8.45	8.18	7.18	7.62	1.78	7.24
H_{T1}	5.84	6.26	8.39	8.45	8.18	7.18	7.62	1.79	7.24

Table 6.7 exhibits the wastewater depths at CSO chambers and storage tanks for Solutions A_{T1} to H_{T1}. It can be seen in Table 6.7 that the storage tanks (T8 and T9) store wastewater. This prevents CSOs at T2 and T5 CSO chambers. However, since these two storage tanks were controlled, such that no CSOs occur. Therefore, the wastewater depths of these two storage tanks were kept lower than their geometric heights.

Hydraulic simulation results from SWMM constraint handling approach

Solutions S_{T1} to Z_{T1} (Figure 6.16), along the best Pareto optimal front for SWMM constraint handling approach, were selected for further assessment. All these solutions presented in the Figure 6.16 are feasible solutions. Solution S_{T1} gives the minimum pollution load to the receiving water, whereas the solution Z_{T1} gives the minimum wastewater treatment cost at downstream treatment plant.

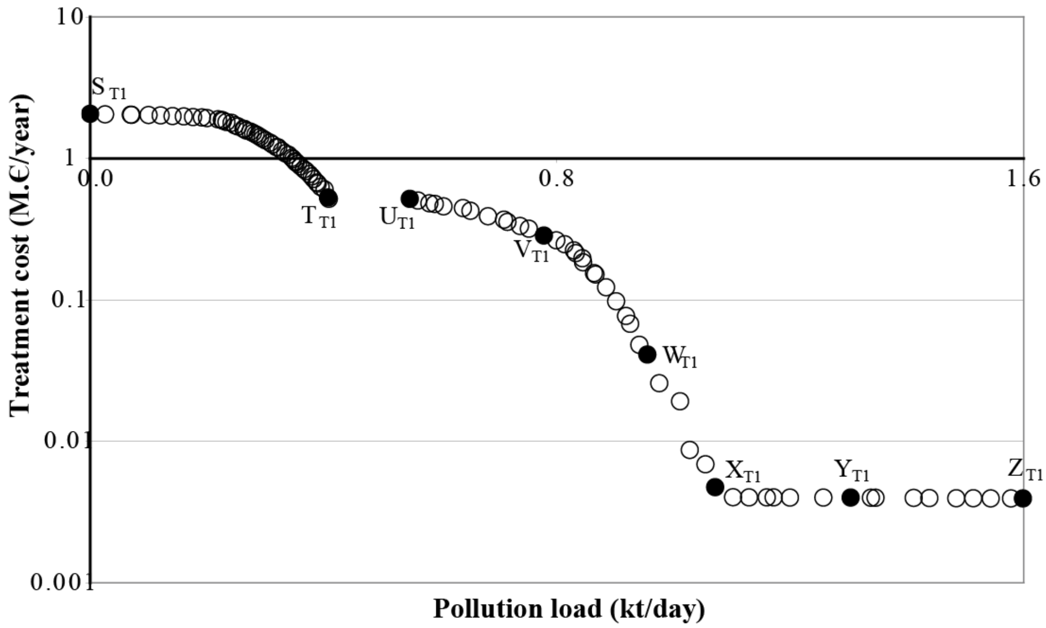

Figure 6.16 Selected solutions for T=15 minutes from SWMM approach

Table 6.8 presents the control settings (orifice openings) for these selected solutions, (S_{T1} to Z_{T1}), during the time period of 0 to 15 minutes. Similar to Table 6.3, orifice openings 0 in Table 6.8 are not numerically zero.

Table 6.8 Orifice openings for selected solutions ($S_{T1} – Z_{T1}$)

Solution	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
S_{T1}	41.9	0	35.7	14.0	0	0	0
T_{T1}	40.5	0	35.5	0	0	0	0
U_{T1}	40.8	0	33.9	0	0	0	0
V_{T1}	40.8	0	19.3	0	0	0	0
W_{T1}	47.3	0	8.5	0.1	0	0	0
X_{T1}	42.5	0	1.5	0	0	0	0
Y_{T1}	35.9	0	0	0	0	0	0
Z_{T1}	4.9	0	0	0	0	0	0

Table 6.9 Flow rates through interceptor sewer sections at t = 15 minutes for selected solutions (S_{T1} – Z_{T1})

Solution	Interceptor sewer flow rates (m^3/s)						
	C1	C2	C3	C4	C5	C6	C7
S_{T1}	2.74	1.65	3.26	5.38	4.58	2.68	1.41
T_{T1}	2.70	1.60	3.26	3.19	2.73	0.92	0.17
U_{T1}	2.71	1.61	3.26	3.18	2.71	0.90	0.17
V_{T1}	2.71	1.61	3.11	2.76	2.10	0.53	0.07
W_{T1}	2.91	1.73	1.92	1.82	0.97	0.13	0
X_{T1}	2.72	1.61	0.77	0.42	0.06	0	0
Y_{T1}	2.47	1.44	0.42	0.11	0	0	0
Z_{T1}	0.39	0.10	0.01	0	0	0	0

Table 6.9 shows the flow rates through the interceptor sewer sections for solutions S_{T1} to Z_{T1}. It can be observed that the flow rates through conduits are less than or equal to the constrained maximum flow rates.

Table 6.10 Combined sewer overflow rates at CSO chambers at t = 15 minutes for selected solutions (S_{T1} – Z_{T1})

Solution	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
S_{T1}	0	0	0	0	0	0	0
T_{T1}	0	0	0	4.36	0	0	0
U_{T1}	0	0	0.95	4.35	0	0	0
V_{T1}	0	0	2.53	4.35	0	0	0
W_{T1}	0	0	3.76	4.33	0	0	0
X_{T1}	0	0	4.52	4.37	0	0	0
Y_{T1}	1.93	0	4.67	4.40	0	0	0
Z_{T1}	4.50	0	4.74	4.36	0	0	0

Table 6.10 presents the CSO rates for the selected solutions (S_{T1} to Z_{T1}). Solution S_{T1} corresponds to the minimum pollution load shows 0 CSO flow rates whereas the solution Z_{T1} corresponds to the minimum wastewater treatment cost gives the maximum CSO rates. Similar to the Table 6.5, T2 and T5 CSO chambers have no CSOs.

Table 6.11 Pollution loads at CSO chambers at t = 15 minutes for selected solutions ($S_{T1} - Z_{T1}$)

Solution	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
S_{T1}	0	0	0	0	0	0	0
T_{T1}	0	0	0.000	0.409	0	0	0
U_{T1}	0	0	0.139	0.409	0	0	0
V_{T1}	0	0	0.370	0.408	0	0	0
W_{T1}	0	0	0.550	0.405	0	0	0
X_{T1}	0	0	0.660	0.411	0	0	0
Y_{T1}	0.208	0	0.682	0.413	0	0	0
Z_{T1}	0.496	0	0.692	0.410	0	0	0

Table 6.11 demonstrates the pollution load to the receiving water from the CSO chambers for the solutions S_{T1} to Z_{T1}. Solution S_{T1} corresponds to minimum pollution load solution gives 0 pollution load.

Table 6.12 Wastewater depths at CSO chambers and storage tanks at t = 15 minutes for selected solutions ($A_{T1} - H_{T1}$)

Solution	Wastewater depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
S_{T1}	5.26	6.26	7.86	7.99	8.17	7.18	7.63	1.72	7.43
T_{T1}	5.39	6.26	7.91	8.45	8.17	7.18	7.63	1.71	7.42
U_{T1}	5.36	6.26	8.1	8.45	8.17	7.18	7.63	1.72	7.42
V_{T1}	5.37	6.26	8.24	8.45	8.17	7.18	7.63	1.73	7.44
W_{T1}	4.85	6.26	8.32	8.45	8.17	7.18	7.63	1.74	7.42
X_{T1}	5.26	6.26	8.38	8.45	8.18	7.18	7.63	1.77	7.4
Y_{T1}	5.65	6.26	8.39	8.45	8.18	7.18	7.62	1.77	7.26
Z_{T1}	5.84	6.26	8.39	8.45	8.18	7.18	7.62	1.78	7.24

Table 6.12 gives the wastewater depths at the CSO chambers and the storage tanks for the selected solutions. Storage tanks T8 and T9 show the wastewater depths less than geometric heights of the corresponding tanks.

Table 6.13 Pollution load and treatment cost for selected solutions at 15 minutes

Solution	Pollution load (kt/day)	Treatment cost (M.€/year)	Solution	Pollution load (kt/day)	Treatment cost (M.€/year)
A_{T1}	0.3289	2.423498	S_{T1}	0	2.063200
B_{T1}	0.5450	1.324528	T_{T1}	0.4091	0.517485
C_{T1}	0.7519	0.361279	U_{T1}	0.5479	0.516874
D_{T1}	0.9114	0.08208	V_{T1}	0.7779	0.284830
E_{T1}	0.9818	0.022299	W_{T1}	0.9552	0.041010
F_{T1}	1.0734	0.004486	X_{T1}	1.0713	0.004744
G_{T1}	1.3156	0.003997	Y_{T1}	1.3035	0.004002
H_{T1}	1.5980	0.003946	Z_{T1}	1.5986	0.003946

Table 6.13 gives the pollution load to the receiving water and the wastewater treatment cost at downstream treatment plant for the selected solutions for both constraint handling approaches. Solutions H_{T1} and Z_{T1} correspond to the minimum cost solutions show the same wastewater treatment cost. In addition, they have almost equal corresponding pollution loads. However, solutions A_{T1} and S_{T1} correspond to the minimum pollution load solutions show contrast values. As it wsa seen in Figure 6.4, the SWMM constraint handling approach outperforms the NSGA II constraint handling approach for the minimum pollution load solution. Furthermore, the corresponding treatment cost for SWMM constraint handling approach is less than that of NSGA II constraint handling approach.

6.7.3 Optimization results for 30 minutes

Snapshot optimization for 30 minutes using NSGA II constraint handling approach

Figure 6.17 illustrates the selected solutions along Pareto optimal front for 30 minutes snapshot optimization using the NSGA II constraint handling approach. Solutions A_{T2} corresponds to the minimum pollution load solution, whereas solution H_{T2} corresponds to the minimum treatment cost solution. Pareto optimal fronts obtained for different random seeds and the function evaluations for two objectives are given in the Appendix B.

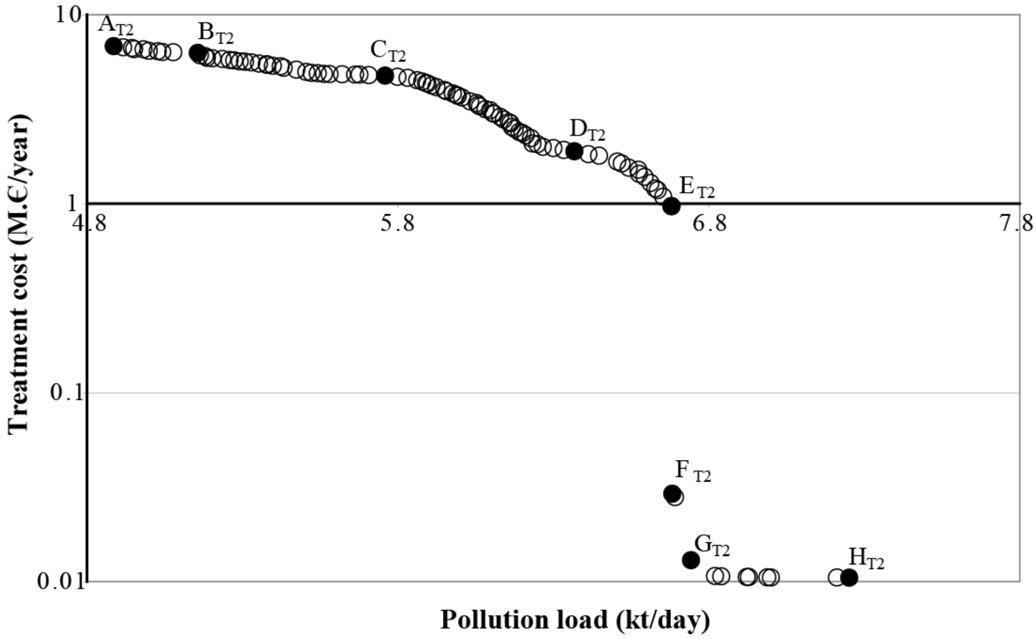

Figure 6.17 Selected solutions for T=30 minutes from NSGA II approach

Table 6.14 Orifice openings for selected solutions during 30 minutes (A_{T2} – H_{T2})

Solution	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
A_{T2}	41.5	0	0.2	20.8	0.2	14.6	0.1
B_{T2}	40.9	0	2.2	27.6	0.7	1.7	0
C_{T2}	41.2	0	0	12.0	0	0	0
D_{T2}	38.6	0	0	0	0	0	0
E_{T2}	25.6	0	0	0	0	0	0
F_{T2}	8.6	0	0	0	0	0	0
G_{T2}	5.4	0	0	0	0	0	0
H_{T2}	0	0	0	0	0	0	0

Table 6.14 gives the control settings (orifice openings) for these selected solutions, A_{T2} to H_{T2}, during the time period of 30 minutes. Hydraulic simulation results obtained for the selected solutions, using these control settings are given in the Appendix C.

Snapshot optimization for 30 minutes using SWMM constraint handling approach

Figure 6.18 presents the selected solutions along Pareto optimal front for 30 minutes snapshot optimization using the SWMM constraint handling approach. Pareto optimal fronts obtained for different random seeds for 30 minutes snapshot optimization and the function evaluations for two objectives are given in the Appendix B.

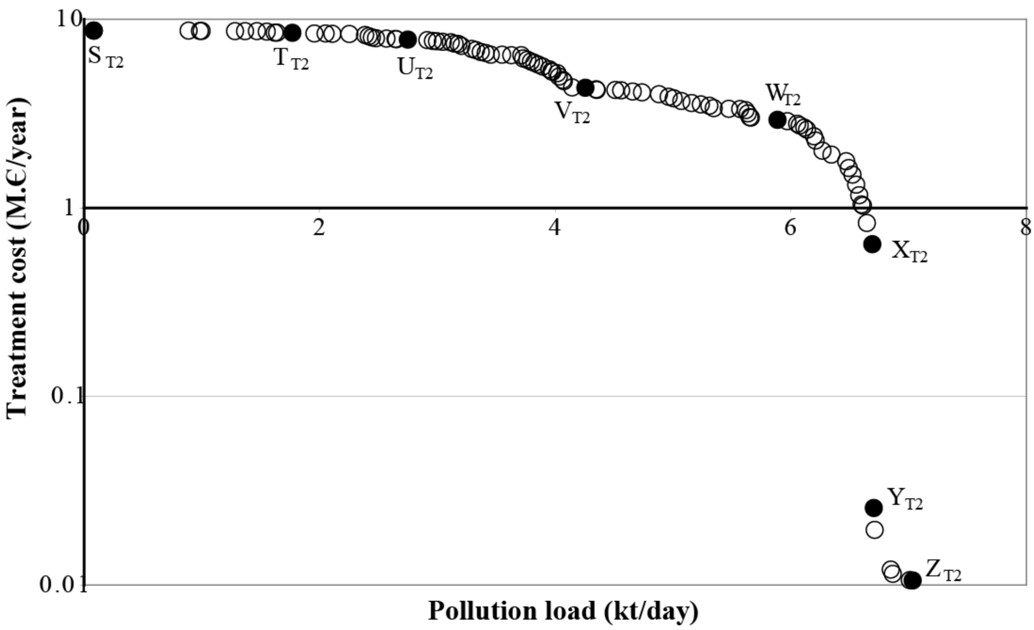

Figure 6.18 Selected solutions for T=30 minutes from SWMM approach

Control settings (orifice openings) for the selected solutions, S_{T2} to Z_{T2} for 30 minutes time period are given in the Table 6.15. Hydraulic simulation results obtained for the selected solutions, using these control settings are given in the Appendix C.

Table 6.15 Orifice openings for selected solutions during 30 minutes (S_{T2} – Z_{T2})

Solution	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
S_{T2}	136.9	51.0	62.5	62.5	123.1	18.9	59.7
T_{T2}	133.3	9.1	45.8	54.2	0	19.1	12.7
U_{T2}	111.1	8.2	35.9	54.3	0	0	0
V_{T2}	131.0	14.5	46.4	0	0	0	0
W_{T2}	93.2	0	0	0	0	0	0
X_{T2}	5.9	7.9	0	0	0	0	0
Y_{T2}	7.1	0.7	0	0	0	0	0
Z_{T2}	0.4	0.1	0	0	0	0	0

Comparison of constraint handling approaches for 30 minutes snapshot optimization

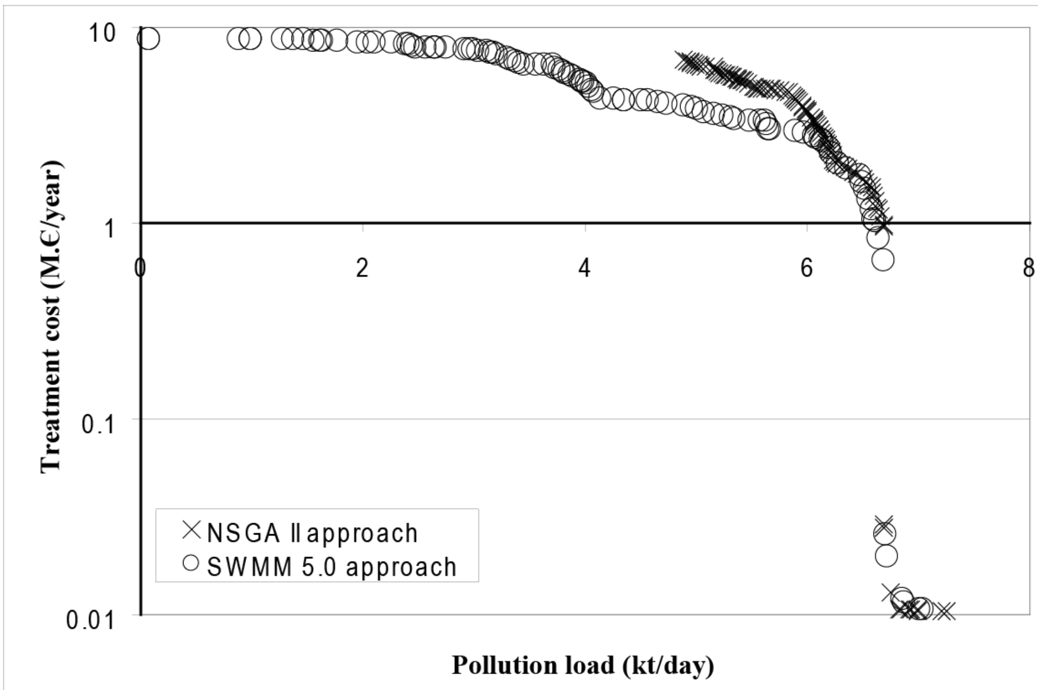

Figure 6.19 Pareto optimal fronts achieved for 30 minutes

Similar to the 15 minutes snapshot optimization Pareto optimal fronts, Figure 6.19 shows the Pareto optimal front obtained from SWMM constraint handling approach outperforms that of obtained from NSGA II constraint handling approach. Table 6.16 confirms this observation. A_{T2} and S_{T2} solutions correspond to minimum pollution load solutions show a considerable difference. However, the minimum treatment cost solutions, H_{T2} and Z_{T2}, show almost the same numerical values.

Table 6.16 Pollution load and treatment cost for selected solutions at 30 minutes

Solution	Pollution load (kt/day)	Treatment cost (M.€/year)	Solution	Pollution load (kt/day)	Treatment cost (M.€/year)
A_{T2}	4.886	6.827	S_{T2}	0.083	8.707
B_{T2}	5.156	6.278	T_{T2}	1.769	8.473
C_{T2}	5.759	4.761	U_{T2}	2.750	7.797
D_{T2}	6.368	1.896	V_{T2}	4.256	4.329
E_{T2}	6.680	0.972	W_{T2}	5.886	2.927
F_{T2}	6.683	0.029	X_{T2}	6.692	0.641
G_{T2}	6.743	0.013	Y_{T2}	6.704	0.026
H_{T2}	7.252	0.010	Z_{T2}	7.033	0.011

6.7.4 Optimization results for 1 hour

Snapshot optimization for 1 hour using NSGA II constraint handling approach

Solutions A_{T4} to H_{T4} were selected from the Pareto optimal front shown in Figure 6.20 for hydraulic simulations. The Pareto optimal front shown in Figure 6.20 is the best optimal front achieved from different random optimization runs. These different optimization runs use different random seeds and the function evaluations for two objectives are given in the Appendix B.

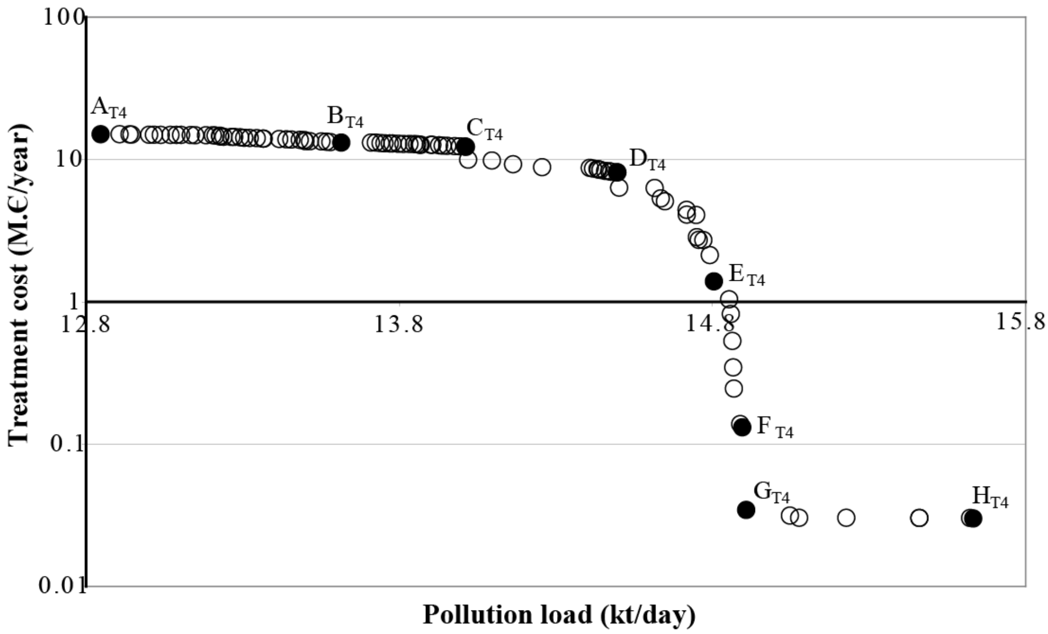

Figure 6.20 Selected solutions for T=1 hour from NSGA II approach

Table 6.17 Orifice openings for selected solutions during 1 hour (A_{T4} – H_{T4})

Solution	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
A_{T4}	28.1	7.7	0.1	13.8	0	14.3	0
B_{T4}	27.9	7.9	0	13.2	0	0	0
C_{T4}	8.8	7.9	0.6	0	0	13.8	0
D_{T4}	19.8	8.0	0.1	0	0	0.1	0
E_{T4}	2.1	0	0.9	0.1	0	0.1	0
F_{T4}	1.1	0	0	0	0	0	0
G_{T4}	0.2	0	0	0	0	0	0
H_{T4}	0	0	0	0	0	0	0

Table 6.17 gives the orifice openings during 1 hour time period. Solution A_{T4} corresponds to the minimum pollution load solution shows larger orifice openings compared to that of solution H_{T4} which corresponds to the minimum treatment cost solution. The hydraulic simulation results obtained using these orifice openings are shown in detail in the Appendix C.

Snapshot optimization for 1 hour using SWMM constraint handling approach

Solutions S_{T4} to Z_{T4} are the solutions selected to perform hydraulic simulations from the Pareto optimal front obtained to 1 hour snapshot optimization using SWMM constraint handling approach.

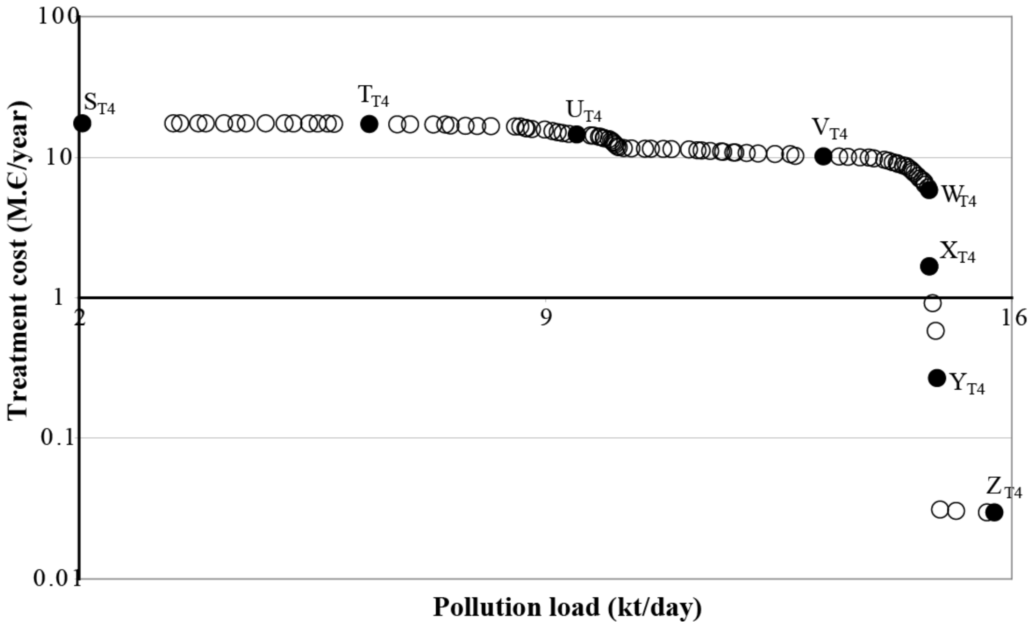

Figure 6.21 Selected solutions for T=1 hour from SWMM approach

Table 6.18 gives the control settings (orifice opening) for the selected solutions (S_{T4} – Z_{T4}). Solution S_{T4} corresponds to the minimum pollution load solution from SWMM constraint handling approach demonstrates significant orifice openings compared to solution A_{T4} corresponds to the minimum pollution load solution from NSGA II constraint handling approach. The hydraulic simulation results obtained using these orifice openings are shown in detail in the Appendix C.

Table 6.18 Orifice openings for selected solutions during 1 hour (S_{T4} – Z_{T4})

Solution	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
S_{T4}	145.0	33.8	62.5	62.5	133.4	41.2	53.1
T_{T4}	143.6	30.8	62.4	54.9	0	15.7	18.5
U_{T4}	144.1	20.2	58.1	0.2	0	14.1	0
V_{T4}	105.9	0	0	0	0	0	0
W_{T4}	10.6	7.5	0.1	0	0	0	0
X_{T4}	5.4	0.3	0	0	0	0	0
Y_{T4}	0.7	0.5	0	0	0	0	0
Z_{T4}	0	0	0	0	0	0	0

Comparison of constraint handling approaches for 1 hour snapshot optimization

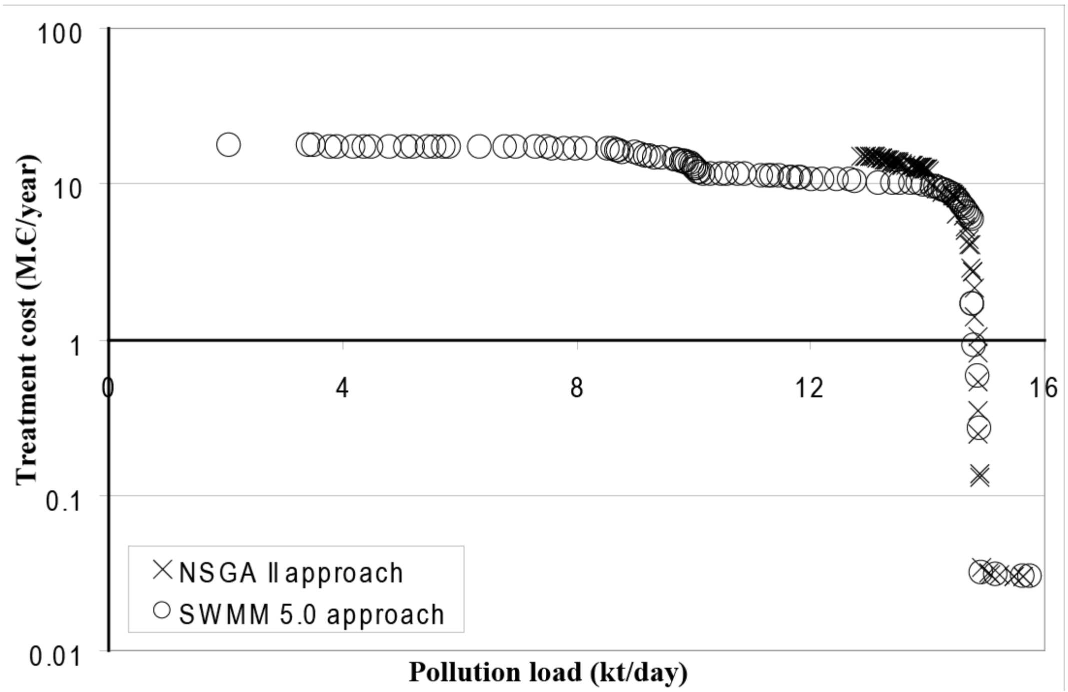

Figure 6.22 Pareto optimal fronts achieved for 1 hour

Similar to the comparisons for 15 minutes and 30 minutes snapshot optimization results, Figure 6.22 shows the Pareto optimal front obtained using SWMM constraint handling approach outperforms that of obtained from NSGA II constraint handling approach.

Table 6.19 Pollution load and treatment cost for selected solutions at 1 hour

Solution	Pollution load (kt/day)	Treatment cost (M.€/year)	Solution	Pollution load (kt/day)	Treatment cost (M.€/year)
A_{T4}	12.847	15.054	S_{T4}	2.050	17.415
B_{T4}	13.615	13.148	T_{T4}	6.342	17.208
C_{T4}	14.010	12.312	U_{T4}	9.472	14.493
D_{T4}	14.496	8.096	V_{T4}	13.174	10.143
E_{T4}	14.805	1.392	W_{T4}	14.761	5.833
F_{T4}	14.895	0.131	X_{T4}	14.762	1.670
G_{T4}	14.907	0.034	Y_{T4}	14.880	0.268
H_{T4}	15.632	0.030	Z_{T4}	15.738	0.030

Table 6.19 illustrates solutions A_{T4} and S_{T4} correspond to the minimum pollution load solution from both constraint handling approaches have significant differences in pollution loads. However, the solutions H_{T4} and Z_{T4} correspond to the minimum wastewater treatment cost solutions from both constraint handling approaches give similar treatment costs.

6.7.5 Optimization results for 2 hours and 30 minutes

Snapshot optimization for 2 hours and 30 minutes using NSGA II constraint handling approach

The best Pareto optimal front for the total storm period from different random seed snapshot optimization runs is shown in the Figure 6.23. Random seed runs can be found from the Appendix B.

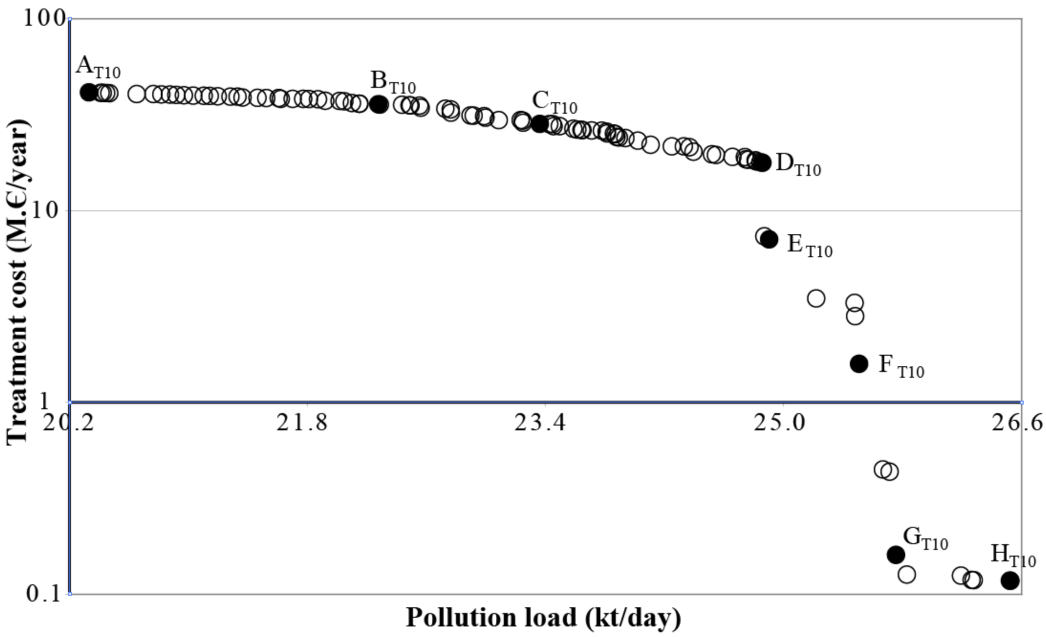

Figure 6.23 Selected solutions for T=2 hours and 30 minutes from NSGA II approach

Table 6.20 presents the orifice openings for the selected solutions from Figure 6.23 for total storm period. The hydraulic simulation results obtained using these orifice openings are shown in detail in the Appendix C.

Table 6.20 Orifice openings for selected solutions during 2 hours and 30 minutes (A_{T10} – H_{T10})

Solution	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
A_{T10}	20.7	0	11.2	12.9	0	14.6	3.0
B_{T10}	4.9	0	21.2	0	0	34.8	0
C_{T10}	17.7	0	10.8	0.1	0	0.1	0
D_{T10}	0.4	0	10.8	0.1	0	0.1	0
E_{T10}	0	0	2.6	0.1	0	0	0
F_{T10}	0	0	0.2	0	0	0	0
G_{T10}	0	0	0	0	0	0	0
H_{T10}	0	0	0	0	0	0	0

Snapshot optimization for 2 hours and 30 minutes using SWMM constraint handling approach

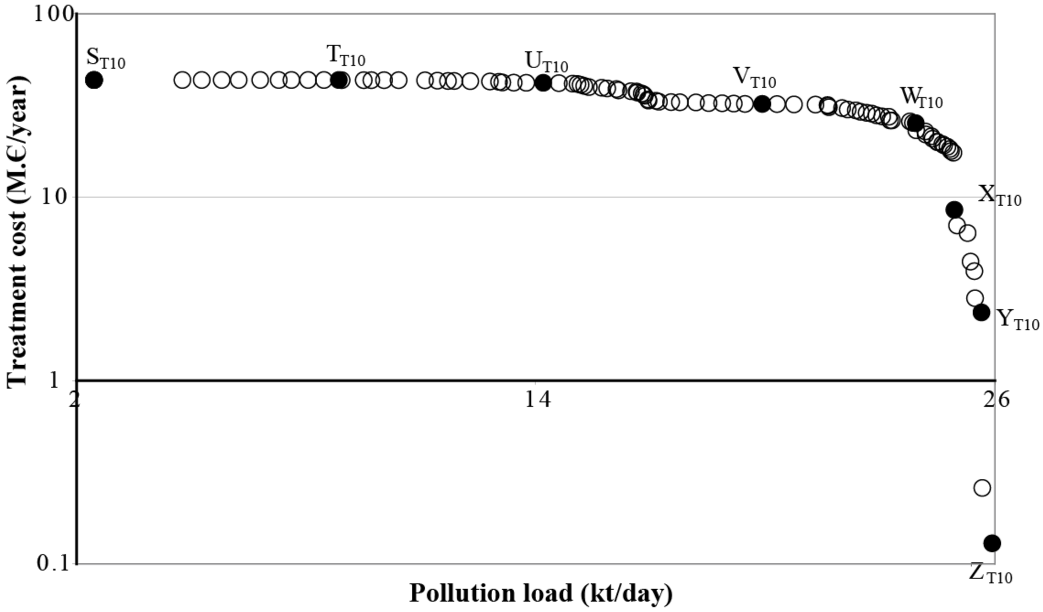

Figure 6.24 Selected solutions for T=2 hours and 30 minutes from SWMM approach

Figure 6.24 shows the best Pareto optimal front from SWMM constraint handling approach for the total storm period. Pareto optimal fronts from different random seeds are given in the Appendix B. Table 6.21 presents the orifice openings for selected solutions along the Pareto optimal front shown in Figure 6.24. The hydraulic simulation results obtained using these orifice openings are shown in detail in the Appendix C.

Table 6.21 Orifice openings for selected solutions during 2 hours and 30 minutes (S_{T10} – Z_{T10})

Solution	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
S_{T10}	145.0	58.9	62.5	62.5	132.4	50.3	62.3
T_{T10}	144.3	33.6	44.1	0	134.2	0	0
U_{T10}	141.1	25.3	38.6	50.6	0	0	0
V_{T10}	127.4	9.5	6.8	0	0	0	0
W_{T10}	12.5	0	10.9	0	0	0	0
X_{T10}	0	0.9	2.4	0	0	0.4	0
Y_{T10}	1.2	0	0	0	0	0	0
Z_{T10}	0	0	0	0	0	0	0

Comparison of Pareto optimal fronts for two constraint handling approaches

Similar comparisons can be identified from the Figure 6.25 and Table 6.22 that were indicated for comparisons of 15 minutes, 30 minutes and 1 hour snapshot optimization results. It can be clearly seen here that the SWMM constraint handling approach outperforms the NSGA II constraint handling approach.

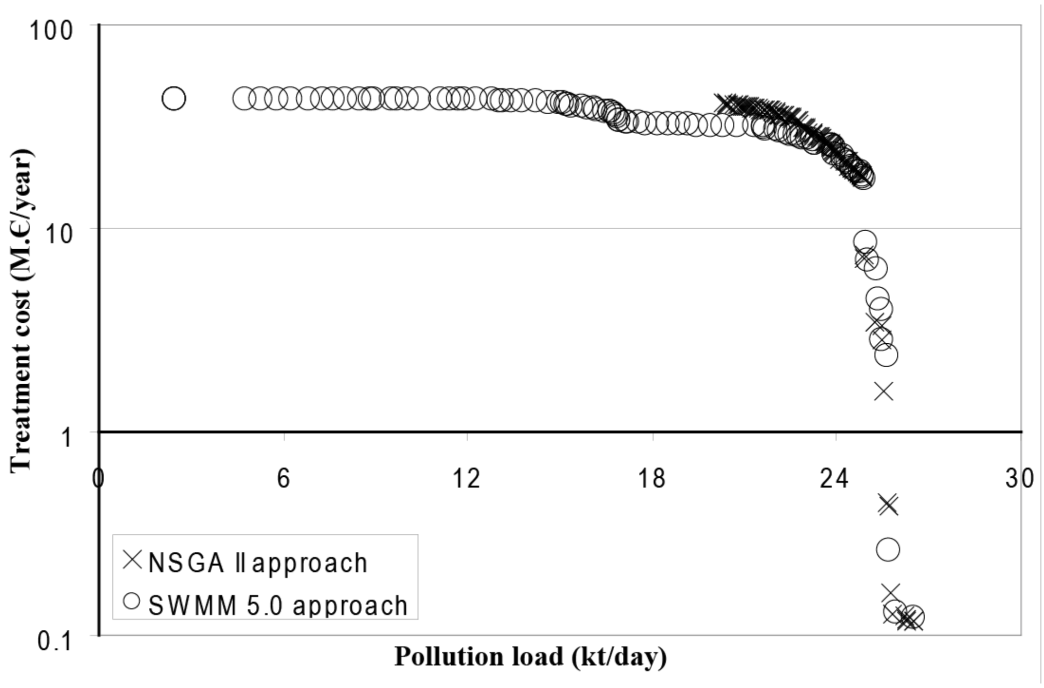

Figure 6.25 Pareto optimal fronts achieved 2 hours and 30 minutes

Table 6.22 Pollution load and treatment cost for selected solutions at 2 hours and 30 minutes

Solution	Pollution load (kt/day)	Treatment cost (M.€/year)	Solution	Pollution load (kt/day)	Treatment cost (M.€/year)
A_{T10}	20.331	41.316	S_{T10}	2.458	43.537
B_{T10}	22.288	35.670	T_{T10}	8.926	43.454
C_{T10}	23.383	28.265	U_{T10}	14.213	41.963
D_{T10}	24.845	17.852	V_{T10}	19.814	32.209
E_{T10}	24.904	7.064	W_{T10}	23.938	25.289
F_{T10}	25.606	1.590	X_{T10}	24.962	8.528
G_{T10}	25.702	0.160	Y_{T10}	25.627	2.350
H_{T10}	26.533	0.118	Z_{T10}	25.926	0.130

Table 6.23 presents the average simulation times taken for the snapshot optimization runs for different time steps. The optimization runs were performed on a Pentium 4 desktop computer with a Core 2 Duo processor and 4 GB of RAM.

Table 6.23 Simulation times for the snapshot optimization

Time step	Average simulation times (hr:min:sec)	
	NSGA II approach	SWMM approach
0-15 minutes	0:09:08	0:10:18
0-30 minutes	0:14:44	0:17:53
0-1 hour	0:24:50	0:30:31
0-2 hours & 30 minutes	1:01:33	1:11:02

6.8 Summary and conclusions

In this chapter, the formulation of a snapshot optimization approach for combined sewer systems has been presented. The multi-objective optimization approach was developed, considering the pollution load to the receiving water from CSOs and the wastewater treatment cost. The originality of the research presented in this Chapter is the development of the multi-objective optimization approach considering water qualities and the wastewater treatment cost. The proposed model gives the optimal CSO control settings where a single set of static control settings is used throughout the considered time period.

The developed multi-objective optimization algorithm has been applied to a simple interceptor sewer network. The results overall demonstrate the benefits of the multi-objective optimization approach and its potential to establish the key properties of a range of control strategies through an analysis of the various trade-offs involved.

Interesting observations can be seen for the mutation rate. The comparison of Pareto optimal fronts for different mutation rates suggests that the higher mutation rates are beneficial for the developed multi-objective optimization approach.

In addition, Pareto optimal fronts from the two different constraint handling approaches show that the SWMM constraint handling approach outperforms the NSGA II constraint handling approach in producing minimum solutions. This observation can be found in all the snapshot optimization trials tested for different times. Therefore, it can be concluded herein that the SWMM constraint handling approach produces better results than that of NSGA II constraint handling approaches. More importantly, obtaining infeasible solutions from the optimization runs were eliminated from this SWMM constraint handling approach.

Hydraulic simulation results further indicate that the developed multi-objective optimization approach produces feasible solutions. However, this model gives the optimal CSO control settings where a single set of static control settings is used throughout the considered time period. The next chapter, Chapter 7, introduces the dynamic optimization in controlling combined sewer system, where the dynamic control settings are used over the storm duration.

CHAPTER 7

DYNAMIC OPTIMIZATION

7.1 Introduction

This chapter extends the developed snapshot optimization approach in Chapter 6, to a dynamic optimization approach in controlling combined sewer systems. This is the main originality of the research work presented in Chapter 7. Snapshot optimization gives a single set of optimal control settings throughout the considered time period. However, the developed dynamic optimization approach gives dynamic optimal control settings in different time steps for the full duration of the storm. Similar to the Chapter 6, pollution load to the receiving water and the cost of wastewater treatment have incorporated in extending the dynamic optimization approach.

7.2 Multi-objective optimization problem formulation

7.2.1 Objective functions

The first objective function was formulated to minimize the pollution load to receiving water through the CSOs. *EQI* was used to obtain the pollution load to the receiving water from CSOs. Equation (7.1) shows the first objective function.

$$Minimize\ \ F1 = \sum_{t=1}^{T} \sum_{i=1}^{n_0} P_{i,t} \tag{7.1}$$

where T, t, n_0 and $P_{i,t}$ are the last time step of the hydraulic simulation, results reporting time step from hydraulic simulator, the number of interceptor nodes or

CSO chamber points and the pollution load to the receiving water from the i^{th} CSO chamber at time t respectively. $P_{i,t}$ can be expressed as shown in the Equation (7.2).

$$P_{i,t} = EQI_{i,t} \tag{7.2}$$

where $EQI_{i,t}$ is the effluent quality index at node i at time t. Detailed explanation of *EQI* is given in the Chapter 6, section 6.2.

The second objective function was formulated to minimize the cost of wastewater treatment at the downstream treatment plant.

$$Minimize \;\; F2 = \sum_{t=1}^{T} C_{T,t} \tag{7.3}$$

where $C_{T,t}$ is the treatment cost at the wastewater treatment plant at time t. A detailed explanation about the cost of wastewater treatment, including the calculation method is given in the Chapter 6, section 6.3 (Equations (6.2) to (6.6)).

7.2.2 Constraints

The same set of constraints given in the subsection 6.4.2, Chapter 6, is the set of constraints for the dynamic optimization approach. Figure 7.1 shows the schematic diagram of a typical interceptor sewer and a CSO chamber.

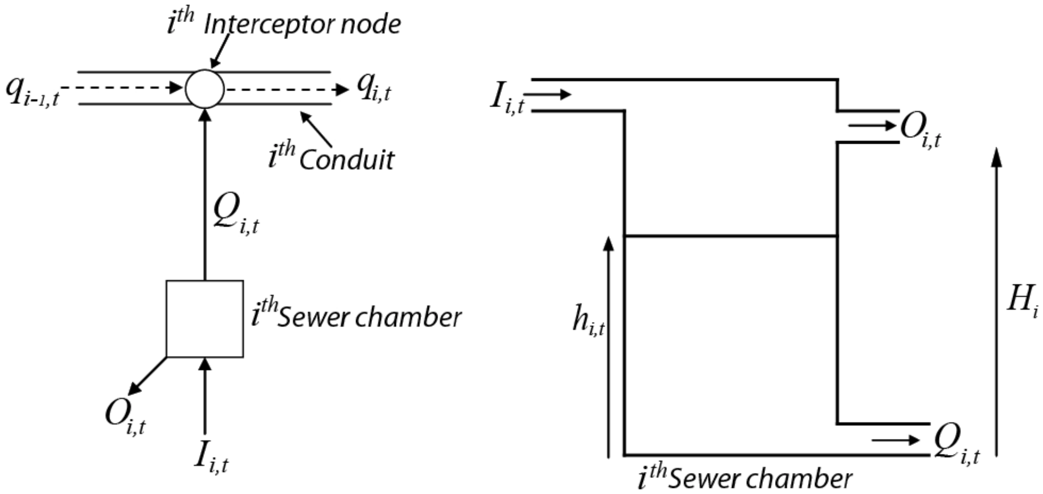

Figure 7.1 Schematic diagram of sewer chamber

$h_{i,t}$ – Water level in i^{th} sewer chamber at time t

H_i – Spill level of i^{th} sewer chamber

$I_{i,t}$ – Catchment inflow to node i at time t

$q_{i,t}$ – Through flow in interceptor sewer at node i at time t

$Q_{i,t}$ – Flow from i^{th} sewer chamber to interceptor node i at time t

$O_{i,t}$ – Combined sewer over flow discharge at node i at time t

With reference to Figure 7.1 the continuity equations are

$$Q_{i,t} + q_{i-1,t} - q_{i,t} = 0 \tag{7.4}$$

$$A_C \frac{\Delta h_{i,t}}{\Delta t} = I_{i,t} - Q_{i,t} \qquad ; h_{i,t} < H_i \tag{7.5}$$

$$A_C \frac{\Delta h_{i,t}}{\Delta t} = I_{i,t} - Q_{i,t} - O_{i,t}; \qquad h_{i,t} > H_i \tag{7.6}$$

$$0 \le q_{i,t} \le q_{\max,i} \tag{7.7}$$

where A_C is the surface area of the CSO chamber and $q_{max,i}$ is the maximum flow rate at i^{th} conduit.

7.3 Solutions to the multi-objective optimization approach

7.3.1 Solution algorithm for dynamic optimization

U.S. EPA SWMM 5.0 (Rossman, 2009), the hydraulic model was linked with NSGA II (Deb et al., 2002) using C programming language. Similar to the snapshot optimization approach, wastewater flow from CSO chamber to the interceptor sewer is controlled using an orifice at the bottom of the CSO chamber. The orifice openings at the first time step of the storm were initially generated randomly. Hence, the decision variables of the optimization approach, the flow rates through the interceptor sewer sections ($q_{i,t}$) were indirectly generated. Next, a full hydraulic simulation, including water quality routing was carried out for the first time step. The results obtained from the hydraulic simulation were used to calculate the two objective function, *F1* (pollution load) and *F2* (wastewater treatment cost). After that, the NSGA II optimization module was run to obtain the optimal solutions for the first time step. The obtained optimal solutions are plotted as a Pareto optimal front for the first time step.

Depending on the sewer network controller's aspirations, an optimal solution can be selected from the Pareto optimal front for the first time step. The orifice settings for the selected optimal solution were obtained. These orifice settings were fed to the hydraulic model as the input data when finding the optimal settings for the next time step. Next, the optimization model was run from the beginning to the end of the second time step to find new optimal solutions for the second time step. The same process was carried out until the last time step, where the orifice control settings were found throughout the total duration of the runoff hydrograph. The algorithm, which was used to find the solutions to the multi-objective optimization approach, is shown in the Figure 7.2.

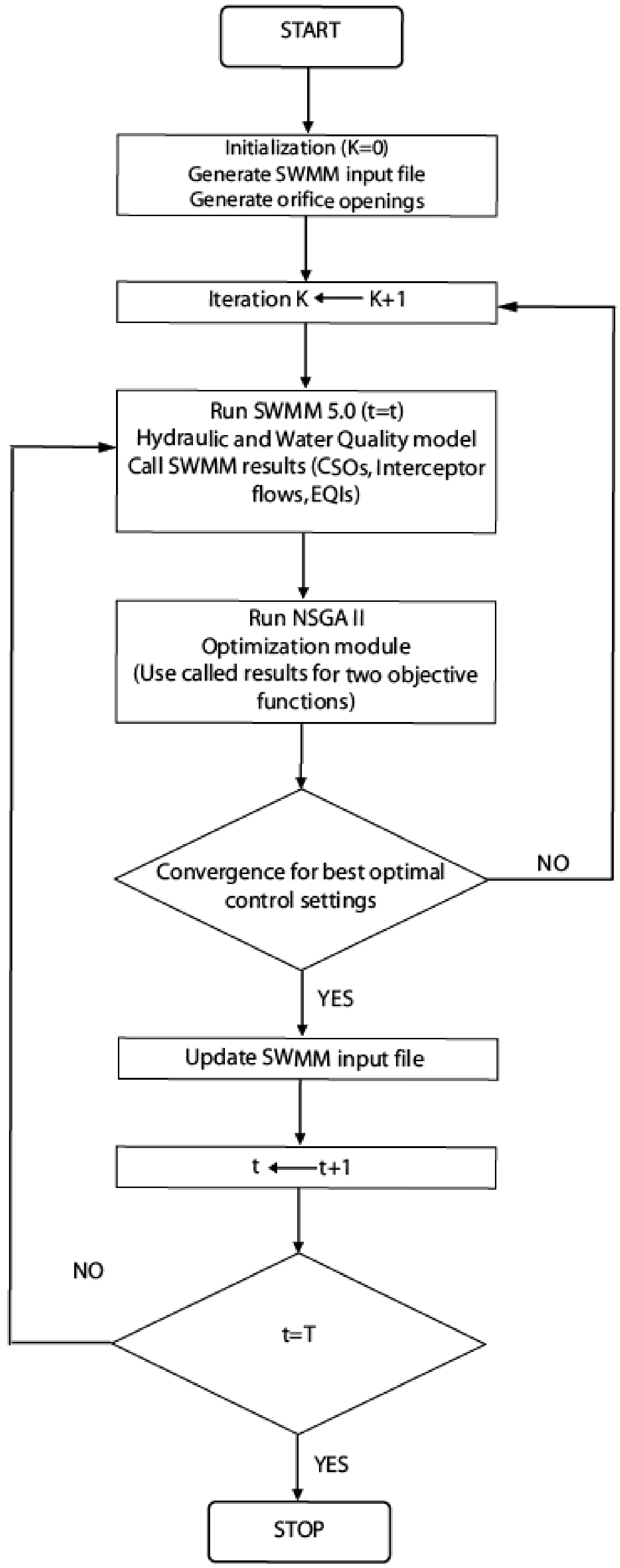

Figure 7.2 Solution algorithm for optimal control settings for dynamic optimization

7.3.2 Different constraint handling methods

Continuity equations shown in Equations (7.4) to (7.6) are automatically satisfied by the hydraulic model (SWMM 5.0). However, the flow rate constraints shown in Equation (7.7) were handled by two alternative formulations. NSGA II constraint handling approach and SWMM constraint handling approach are the two different approaches and detailed explanations about these two approaches can be found in the Chapter 6, subsection 6.5.2.

7.4 Case studies

7.4.1 Interceptor sewer system

The modified interceptor sewer system given in Chapter 6, was used to conduct the case studies for the above developed dynamic optimization approach. More details about this interceptor sewer system are found in the Chapter 6, subsection 6.6.1. However, the schematic view of the modified interceptor sewer system is presented in Figure 7.3.

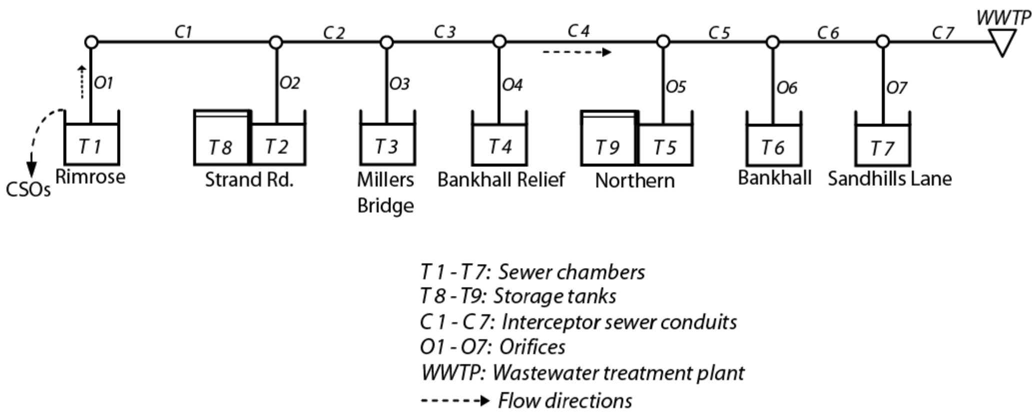

Figure 7.3 Modified interceptor sewer system

7.4.2 Dynamic optimization for single storm condition

The developed dynamic optimization approach was applied to the simplified interceptor sewer system. Flow rates through *C1* to *C3* and *C4* to *C7* were constrained to the maximum flow of 3.26 m^3/s and 7.72 m^3/s respectively. T8 and T9 on-line storage tanks were controlled, such that no overflows occur from these storage tanks. This was done using the control rules in SWMM 5.0.

DWFs (Table 6.2) and runoff hydrographs from single storm (Figure 5.2) were fed to the T1, T2, T3, T4, T5, T6, and T7 CSO chambers. As it was stated in the Chapter 6, subsection 6.6.2, the runoff hydrographs obtained from Thomas (2000) were for mild storms. Therefore, these runoff hydrographs were multiplied by a factor of five to obtain high intensity storm profiles (Thomas, 2000). A medium range of pollution level given in the Table 5.1 (TSS – 220 mg/L, COD – 500 mg/L, BOD – 220 mg/L, TKN – 25 mg/L and NOX – 40 mg/L) was fed with the DWFs. Generated pollutographs for single storm condition were fed according to the corresponding CSO chambers according to the assumed land-uses (Table 5.4). Generated pollutographs can be found in Chapter 5, subsection 5.4.3 and in Appendix A.

The real coded NSGA II program was used to perform the dynamic multi-objective optimization. The optimization process was carried out with a population of 100, 100 generations and a crossover probability of 1. The distribution indices for crossover and mutation operators were kept at 20. Mutation probabilities of 0.6 and 0.4 were separately used in NSGA II and SWMM constraint handling approaches respectively. 15 minutes snapshot optimization results from the previous chapter show that these two mutation probabilities give the best Pareto optimal fronts for two different constraint handling approaches. Many optimization runs with different random seeds were conducted.

Routing time-step in SWMM 5.0 was kept at 30 seconds, and the optimization model was run for 15 minutes (first time step). Two extreme feasible solutions, minimum pollution load to the receiving water and minimum wastewater treatment cost, were selected from the Pareto optimal front, which is obtained at 15 minutes. Control settings (orifice openings during the 15 minutes) were obtained for these two extreme solutions. These orifice settings were separately fed to the hydraulic model as the input data when finding the corresponding optimal solutions in the next time step. Next, the optimization model was run twice from the beginning to the end of the second time step (30 minutes) with two different control settings during the first 15 minutes. The optimal control settings, from 15 minutes to 30 minutes, were obtained from the corresponding Pareto optimal fronts. The same process was carried out for the duration of 2 hours and 30 minutes in 15 minutes time steps. Finally, two sets of orifice openings, varying with the time, were found for the minimum pollution load and minimum treatment cost approaches.

Two separate dynamic optimization runs were carried out for the two different constraint handling approaches which are stated in subsection 7.3.2. Obtained orifice openings for different scenarios for different constraint handling approaches were then used perform the hydraulic simulations.

7.4.3 Dynamic optimization for two consecutive storms

Dynamic optimization approach was applied to the same interceptor sewer system, but with different inflow characteristics. DWFs and pollution levels of the constituents were kept the same as stated in the subsection 7.4.2. However, runoff hydrographs (Chapter 5, subsection 5.5.1) and pollutographs (Chapter 5, subsection 5.5.2 & Appendix A) generated for two consecutive storms were fed to the corresponding CSO chambers. Runoff hydrographs from mild storm event were multiplied by a factor of 5 to make them high intensity storms. Same flow constraints and controls of storage tanks stated in the subsection 7.4.2 were kept in the optimization process.

Dynamic optimization process was carried out for 100 generations with 100 population size. The crossover probability and distribution indices were kept at 1 and 20 respectively. During the first 15 minutes, the runoff to the CSO chambers from single storm and two consecutive storms are the same. Therefore, the same mutation rates were applied to the optimization process, i.e. 0.6 for the NSGA II constraint handling approach and 0.4 for the SWMM constraint handling approach.

Dynamic optimization runs were performed in 15 minute time steps to the minimum pollution load solution and the minimum treatment cost solutions from both constraint handling approaches.

7.4.4 Dynamic optimization for migrating storms

Dynamic optimization approach was applied to the same interceptor sewer system for the migrating storms. DWFs and pollution levels of the constituents were kept the same as stated in the subsection 7.4.2. However, runoff hydrographs (Chapter 5, subsection 5.6.1) and pollutographs (Chapter 5, subsection 5.6.2 & Appendix A) generated for migrating downstream and upstream storms were separately fed to the corresponding CSO chambers. Runoff hydrographs from mild storm event were multiplied by a factor of 5 to make them high intensity storms. Same flow constraints and controls of storage tanks stated in the subsection 7.4.2 were kept in the optimization process.

For each dynamic optimization process, 100 generations of 100 population size were performed. The crossover probability and distribution indices were kept at 1 and 20 respectively. In order to find the best mutation probability for the migrating storm conditions, different mutation probabilities were tried in different runs. Dynamic optimization runs were performed in 15 minute time steps for both constraint handling approaches under the best mutation probabilities found from different runs. Results for the two extreme solutions, minimum pollution load and minimum treatment cost, are presented in the next section.

7.5 Results and discussion

7.5.1 Optimization results for single storm using NSGA II constraint handling approach

Figure 7.4 shows the Pareto optimal front archived at 15 minutes duration using the NSGA II constraint handling approach. Solutions $A_{T1}(S)$ and $B_{T1}(S)$ are the two extreme solutions, which were selected for the dynamic optimization. Solution $A_{T1}(S)$ gives the minimum pollution load solution at 15 minutes, whereas solution $B_{T1}(S)$ gives the minimum treatment cost solution.

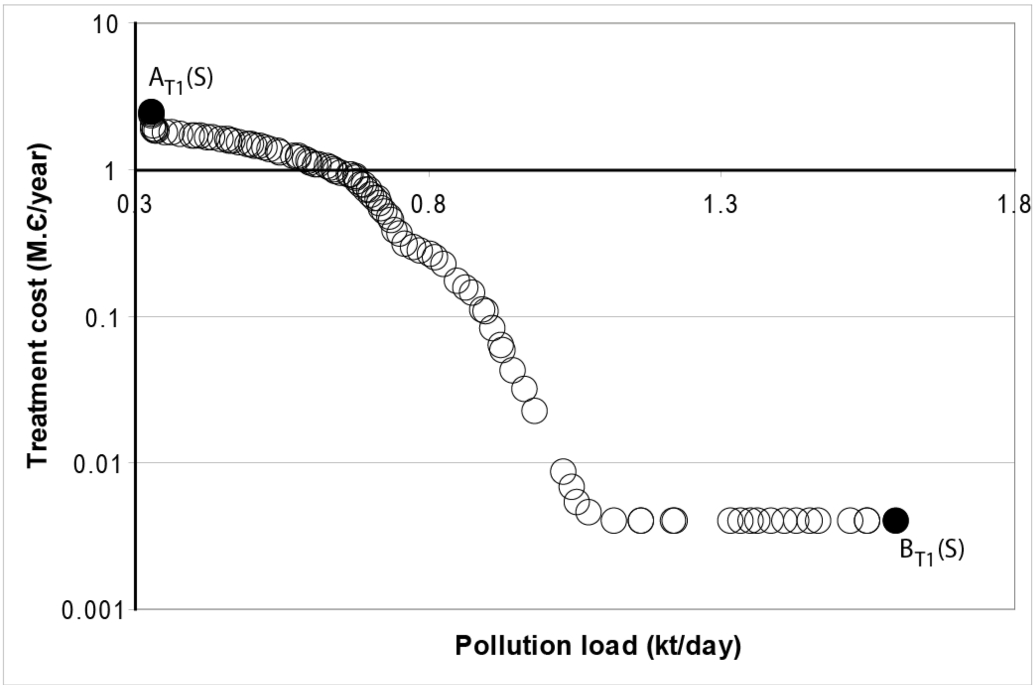

Figure 7.4 Pareto optimal front for T=15 minutes from NSGA II approach

Minimum pollution load solution

Figure 7.5 presents the Pareto optimal front for 30 minutes for the minimum pollution load approach. Orifice openings obtained for solution $A_{T1}(S)$ were fed as input parameters during the 0-15 minutes interval, in order to run the optimization process for 0-30 minutes. Minimum pollution load solution from the Figure 7.5, $A_{T2}(S)$ was selected to next time step optimization process.

Figure 7.6 illustrates the Pareto optimal front for 45 minutes for the minimum pollution load approach. Similar to the explanation given in the preceding paragraph, solution $A_{T3}(S)$ represent the minimum pollution load solution at 45 minutes. Orifice openings for solution $A_{T3}(S)$ were fed to the systems as input parameters during 30-45 minutes time step for next optimization runs. Obtained Pareto optimal fronts during the total runoff duration are presented in the Appendix D. In addition, the optimal fronts obtained from the random seed analysis are presented in the same appendix.

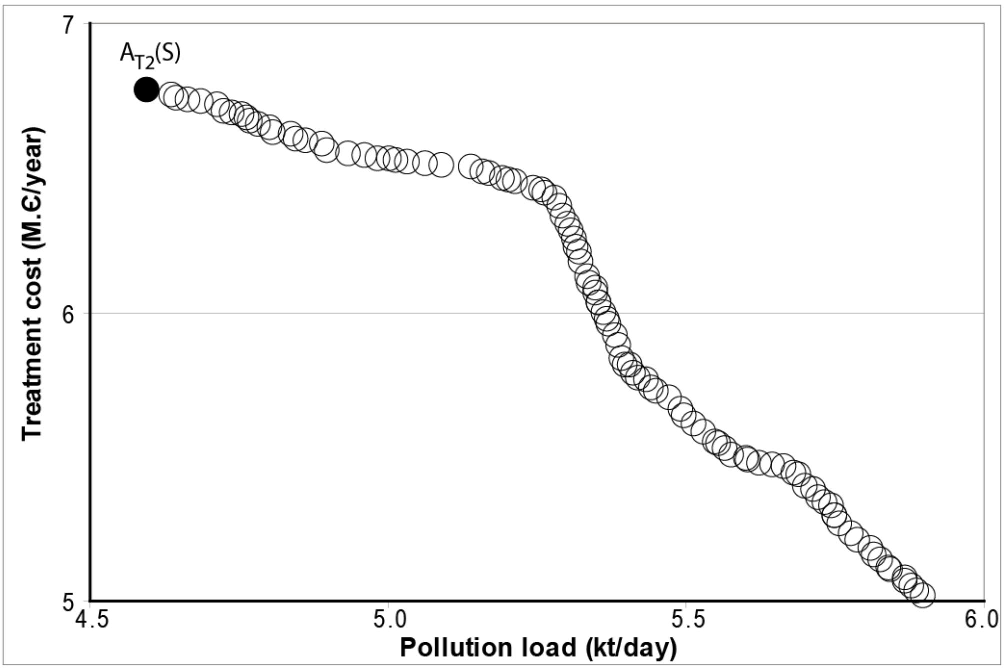

Figure 7.5 Pareto optimal front for T=30 minutes for minimum pollution load approach

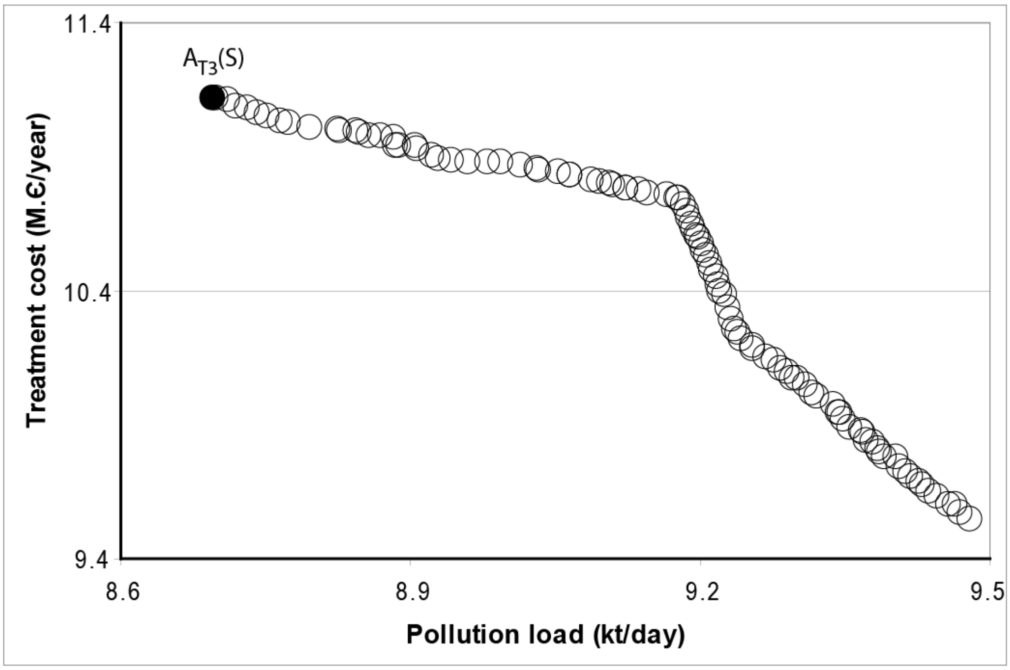

Figure 7.6 Pareto optimal front for T=45 minutes for minimum pollution load approach

Table 7.1 Orifice openings for minimum pollution load solutions under NSGA II constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	41.98	0	21.84	24.08	0	0.02	0
15-30	21.06	13.45	0	12.34	1.64	18.11	0
30-45	39.63	0	1.18	0.03	0	6.58	26.63
45-60	40.58	0	0.03	0.34	0.01	16.45	14.85
60-75	14.34	0.05	13.85	0.04	0.25	15.00	11.56
75-90	25.05	0.56	11.37	1.81	14.70	55.49	0
90-105	29.32	0.01	9.17	0.10	20.87	0.00	0
105-120	38.41	0.02	3.53	0.03	19.67	5.01	0.39
120-135	41.71	0.93	0.77	0.06	21.89	0.67	0
135-150	19.07	0.01	62.39	4.95	15.17	0.33	0

Table 7.1 gives the control settings (orifice openings) obtained for the minimum pollution load solution throughout the total duration of the storm runoff hydrograph.

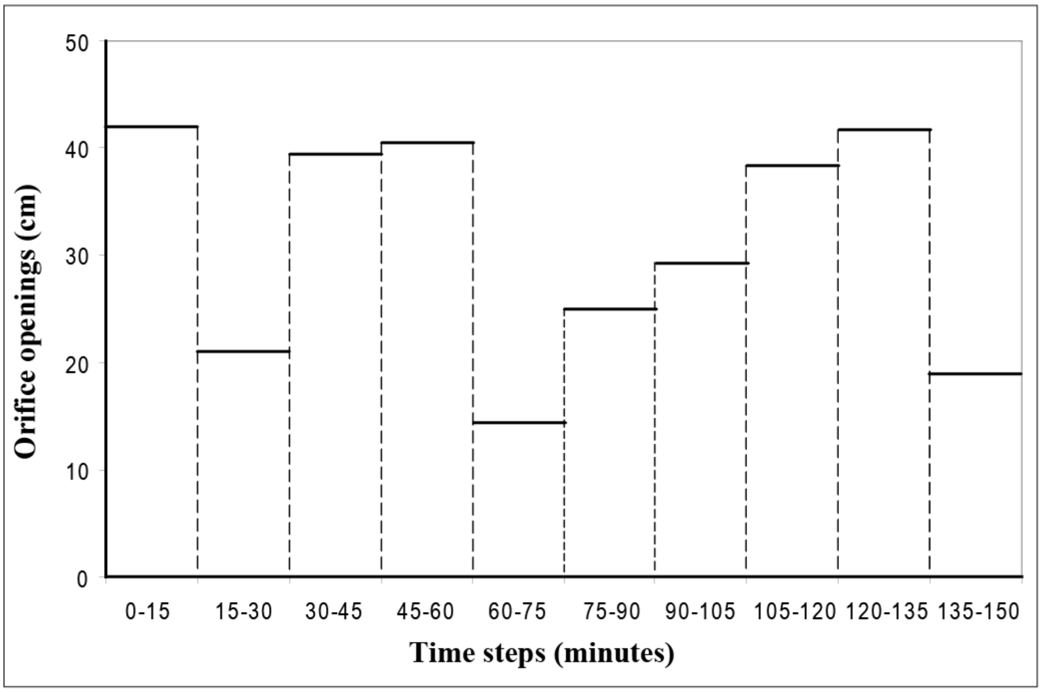

Figure 7.7 Orifice openings for O1

To further illustrate the dynamic behavior of the control settings, orifice openings corresponding to O1 were plotted against the time. The variation can be clearly seen from the Figure 7.7.

Orifice openings for O1 for minimum pollution load approach from snapshot optimization and dynamic optimization were plotted against the time. These two settings can be seen in the Figure 7.8. They both are for the single storm condition under the NSGA II constraint handling approach. As it was already stated in the previous chapter, snapshot optimization gives a single set of optimal CSO control settings throughout the considered time period. This can be clearly visualized from the following figure. However, the dynamic optimization approach gives the dynamic optimal control settings in different time steps. Figure 7.8 is a fine explanation for the difference between snapshot and dynamic optimization.

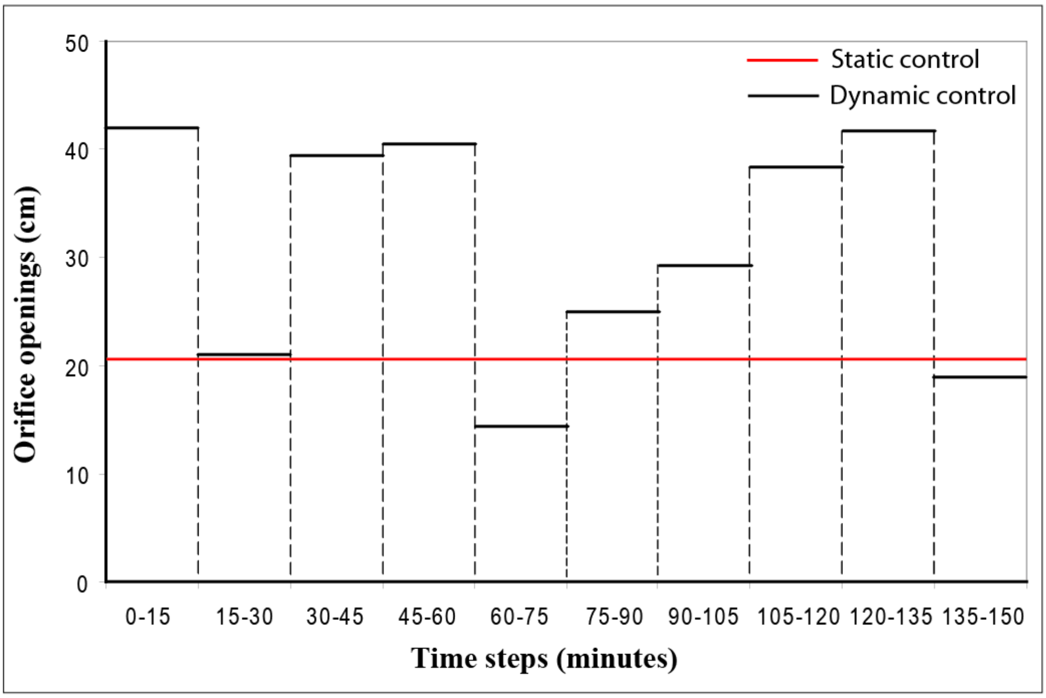

Figure 7.8 Orifice openings for O1 from snapshot and dynamic optimizations

Minimum treatment cost solution

Orifice openings for the minimum treatment cost solution from Figure 7.4, $B_{T1}(S)$ was used to obtain the following Pareto optimal front (Figure 7.9) at 30 minutes. Figure 7.10 presents the optimal front for 45 minutes. Rest of the Pareto optimal fronts from dynamic optimization under the NSGA II constraint handling approach are given in the Appendix D.

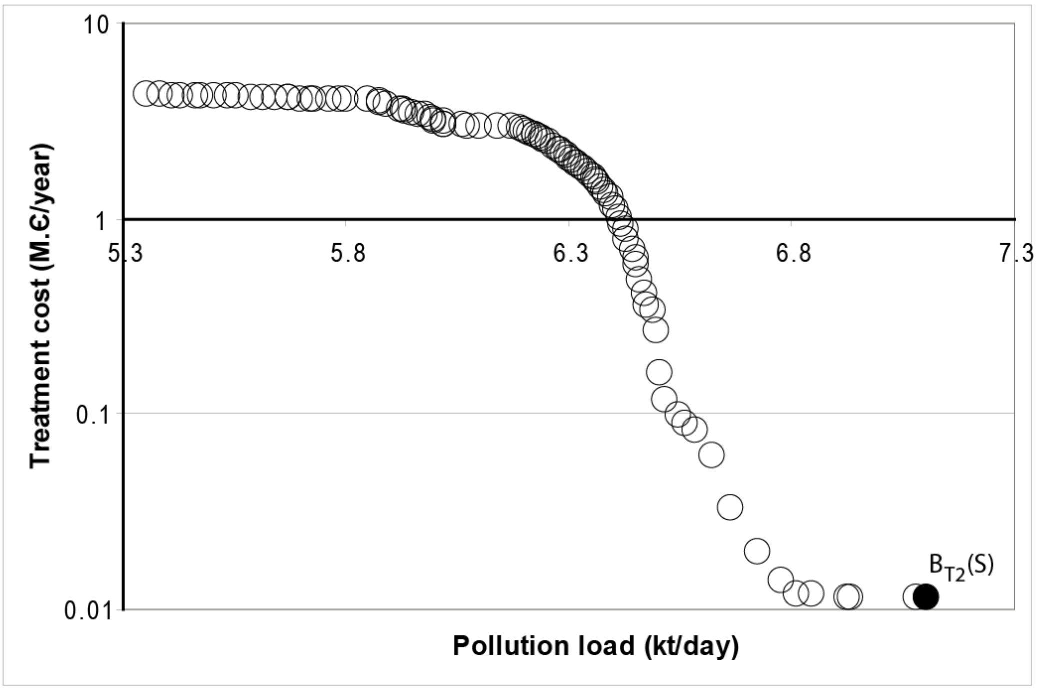

Figure 7.9 Pareto optimal front for T=30 minutes for minimum treatment cost approach

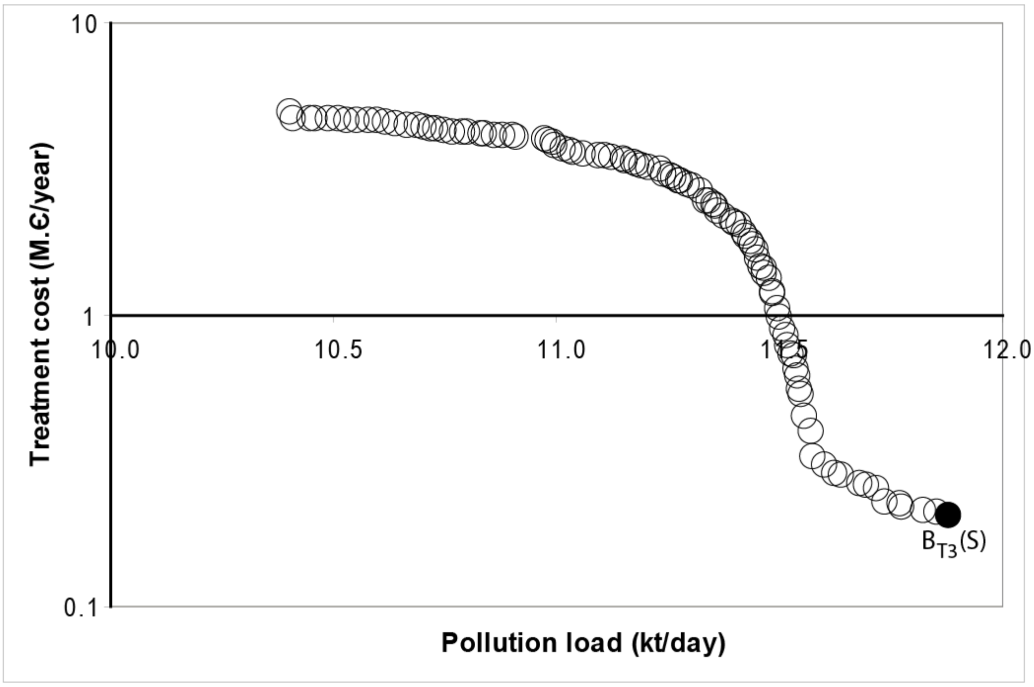

Figure 7.10 Pareto optimal front for T=45 minutes for minimum treatment cost approach

Table 7.2 gives the orifice openings for the minimum treatment cost solution throughout the total duration of the storm runoff hydrograph. Most of the orifices throughout the total storm period were kept at the minimum opening levels to obtain the minimum wastewater treatment cost at downstream wastewater treatment plant.

Table 7.2 Orifice openings for minimum treatment cost solutions under NSGA II constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	5.02	0	0	0	0	0	0
15-30	1.40	0.02	0	0	0	0	0
30-45	0	0	0	0	0	0	0
45-60	0.01	0	0	0	0	0	0
60-75	0	0	0	0	0	0	0
75-90	0	0	0	0	0	0	0
90-105	0	0	0	0	0	0	0
105-120	0.03	0	0	0	0	0	0
120-135	0	0	0	0	0	0	0
135-150	0.07	0	0	0	0	0	0

7.5.2 Hydraulic simulation results

Minimum pollution load solution

Results from the hydraulic simulations for minimum pollution load solution are presented in the following tables. Table 7.3 gives the interceptor sewer flow rates. It can be clearly seen that the maximum flow rates through these conduits are less than that of constrained values.

Table 7.3 Flow rates through interceptor sewer sections for minimum pollution load approach

Time (hr:min:sec)	Interceptor sewer flow rates (m^3/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.74	1.66	3.24	5.84	4.89	3.02	1.81
00:30:00	1.73	3.24	3.19	5.14	5.42	7.70	7.59
00:45:00	3.20	3.16	3.21	3.20	3.15	4.10	7.69
01:00:00	3.24	3.25	3.24	3.30	3.29	5.62	7.69
01:15:00	1.20	1.43	3.22	3.32	3.57	5.95	7.66
01:30:00	1.99	2.00	3.21	3.43	6.48	7.55	7.60
01:45:00	2.28	2.26	3.24	3.33	7.49	7.42	7.39
02:00:00	2.92	2.87	3.19	3.21	7.05	7.67	7.70
02:15:00	3.07	3.19	3.26	3.22	7.56	7.59	7.56
02:30:00	1.53	1.69	3.26	4.02	7.27	7.58	7.71

Tables 7.4 and 7.5 present the CSO rates from CSO chambers and pollution loads to the receiving water from corresponding CSO chambers for the minimum pollution load approach under the NSGA II constraint handling technique respectively.

Table 7.4 Combined sewer overflow rates at CSO chambers for minimum pollution load approach

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	2.869	0	0	0	0
00:30:00	7.134	0.002	7.892	7.621	12.065	0	1.051
00:45:00	8.594	3.290	7.162	16.579	16.579	3.113	0
01:00:00	7.773	2.411	4.553	16.579	16.579	1.079	0.081
01:15:00	5.484	1.309	1.074	16.579	16.579	0	0.101
01:30:00	1.376	0.772	0	9.140	10.080	0	0.822
01:45:00	0.745	0.839	0.288	4.183	5.336	0	1.482
02:00:00	0.128	0.838	0.864	1.626	1.209	0	0.230
02:15:00	0	0.730	0.179	1.583	0.806	0.928	0.120
02:30:00	1.477	0.833	0	0.834	2.059	0.494	0.172

Table 7.5 Pollution loads at CSO chambers for minimum pollution load approach

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.420	0	0	0	0
00:30:00	0.735	0	0.606	1.014	1.492	0	0.107
00:45:00	0.589	0.225	0.463	1.572	1.231	0.241	0
01:00:00	0.432	0.138	0.320	1.081	1.013	0.062	0.004
01:15:00	0.351	0.082	0.100	0.793	0.886	0	0.006
01:30:00	0.129	0.063	0	0.395	0.611	0	0.062
01:45:00	0.089	0.078	0.054	0.199	0.369	0	0.136
02:00:00	0.016	0.082	0.163	0.146	0.129	0	0.023
02:15:00	0	0.073	0.034	0.180	0.099	0.092	0.013
02:30:00	0.185	0.085	0	0.102	0.259	0.049	0.019

Table 7.6 demonstrates the wastewater depths at CSO chambers and storage tanks throughout the single storm duration. Wastewater depths of the two storage tanks show that they are less than the geometric depths of the corresponding storage tanks.

Table 7.6 Wastewater depths at CSO chambers and storage tanks for minimum pollution load approach

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.26	6.25	8.26	5.54	8.17	7.16	7.63	1.75	7.42
00:30:00	5.99	6.89	8.56	8.64	8.99	8.88	10.07	6.91	8.18
00:45:00	6.07	7.25	8.52	9.04	9.18	8.98	9.41	6.93	8.18
01:00:00	6.02	7.19	8.37	9.04	9.18	8.93	10.06	6.93	8.18
01:15:00	5.9	7.09	8.11	9.04	9.18	8.81	10.06	6.93	8.18
01:30:00	5.61	7.04	7.91	8.71	8.9	1.7	10.07	6.93	8.18
01:45:00	5.54	7.05	8.02	8.44	8.65	7.07	10.06	6.93	8.18
02:00:00	5.46	7.05	8.09	8.25	8.35	8.89	10.06	6.93	8.18
02:15:00	5.26	7.04	7.94	8.25	8.31	8.93	10.06	6.93	8.18
02:30:00	5.62	7.05	1.41	8.17	8.43	8.93	10.07	6.93	8.18

Minimum treatment cost solution

Table 7.7 gives the flow rates through interceptor sewer sections throughout runoff period. Flow rates are less than the constrained values. This shows the developed multi-objective dynamic optimization model produces feasible solutions. CSOs, pollution loads and wastewater depths of the chambers obtained from hydraulic simulations are presented in Appendix E.

Table 7.7 Flow rates through interceptor sewer sections for minimum treatment cost approach

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.39	0.10	0.01	0	0	0	0
00:30:00	0.13	0.19	0.16	0.12	0.04	0	0
00:45:00	0.02	0.08	0.13	0.14	0.15	0.09	0.04
01:00:00	0.01	0.03	0.06	0.07	0.10	0.12	0.13
01:15:00	0	0.02	0.03	0.04	0.05	0.08	0.09
01:30:00	0	0.01	0.02	0.02	0.03	0.05	0.06
01:45:00	0	0.01	0.01	0.01	0.02	0.03	0.04
02:00:00	0	0	0.01	0.01	0.01	0.02	0.03
02:15:00	0	0	0.01	0.01	0.01	0.01	0.02
02:30:00	0	0	0	0	0.01	0.01	0.01

7.5.3 Optimization results for single storm using SWMM constraint handling approach

Dynamic optimization results for the single storm condition under the SWMM constraints handling approach are given in this section. Figure 7.11 shows the Pareto optimal front archived at 15 minutes duration under the SWMM constraint handling approach. Solutions $U_{T1}(S)$ and $V_{T1}(S)$ are the two extreme solutions, which were selected for the dynamic optimization. Solution $U_{T1}(S)$ gives the minimum pollution load solution at 15 minutes, whereas solution $V_{T1}(S)$ gives the minimum treatment cost solution.

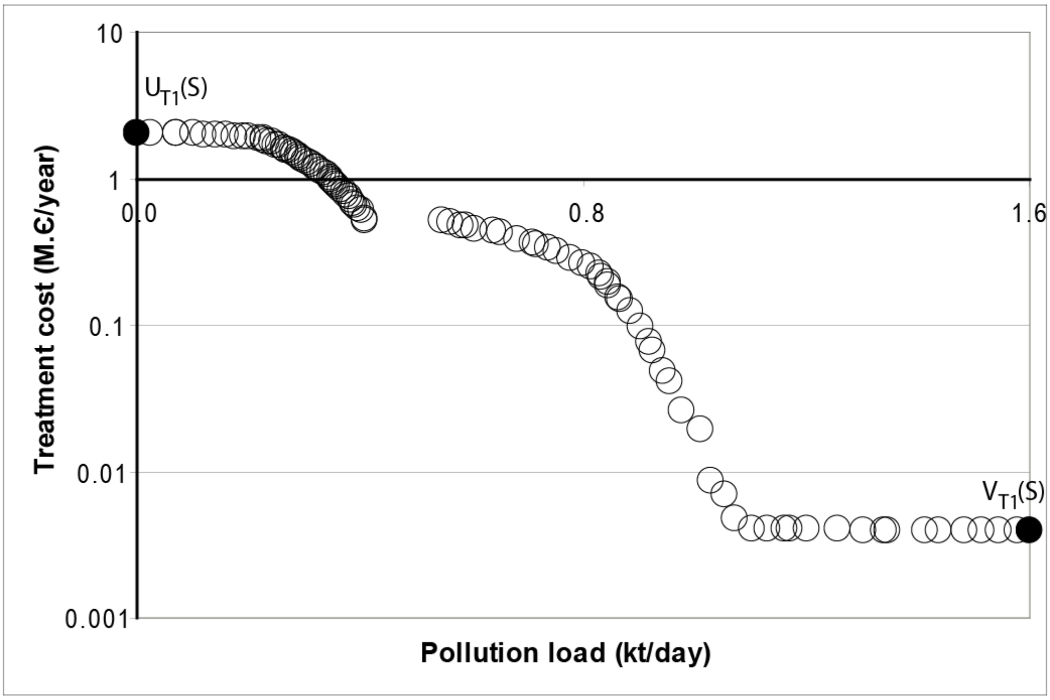

Figure 7.11 Pareto optimal front for T=15 minutes from SWMM approach

Minimum pollution load solution

Figure 7.12 shows the Pareto optimal front obtained for minimum pollution load solution under SWMM constraint handling approach at 30 minutes. Solution $U_{T2}(S)$ gives the minimum pollution load from the Pareto optimal front and the corresponding orifice openings were used to carry out the optimization process for the next time step.

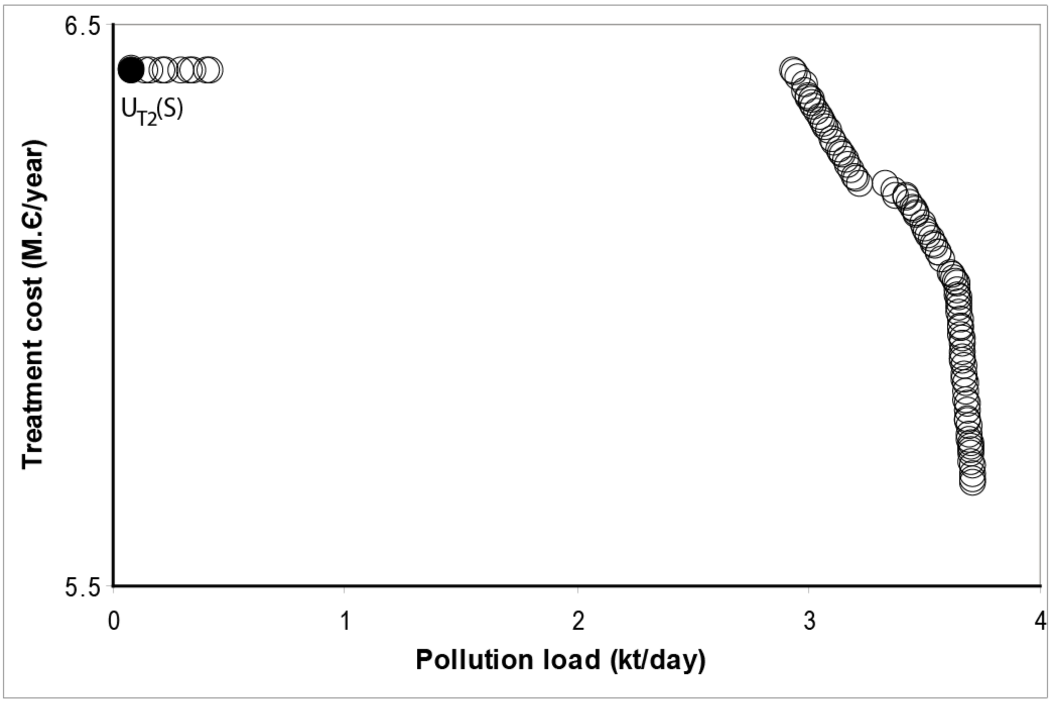

Figure 7.12 Pareto optimal front for T=30 minutes for minimum pollution load approach

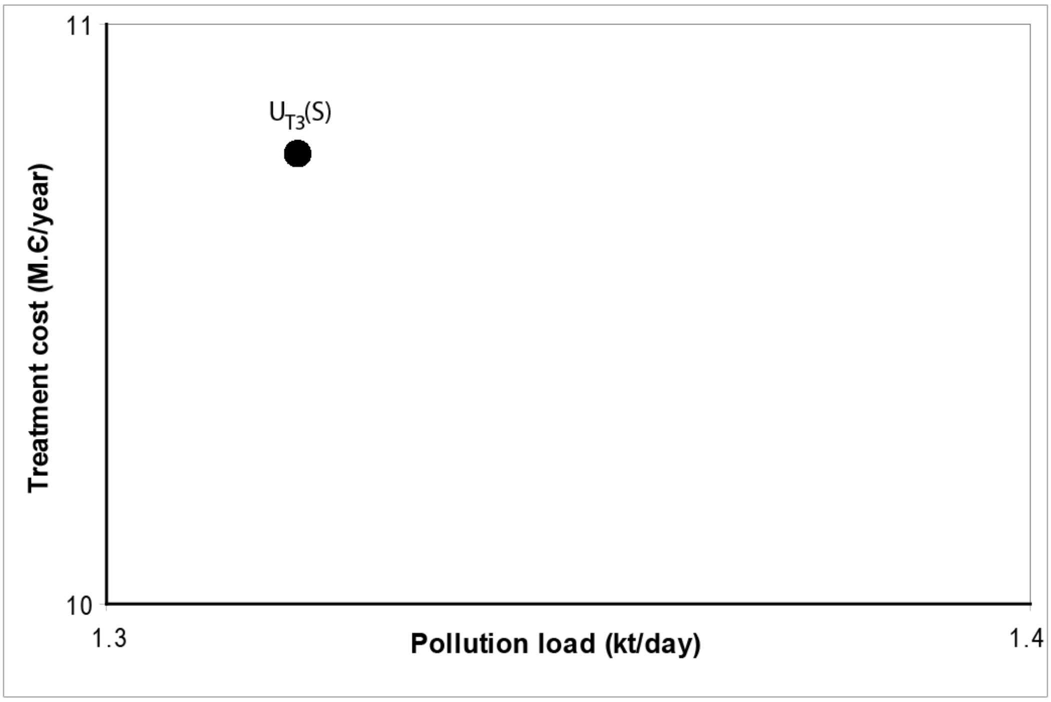

Figure 7.13 Pareto optimal front for T=45 minutes for minimum pollution load approach

Figure 7.13 shows an interesting observation. Instead of a set of solutions from the multi-objective optimization run, a single solution was found at 45 minutes. The fitness values correspond to this solution were further investigated. It was found that the multi-objective optimization approach converged to a set of solutions, which has the same fitness values, but with different decision variables. Table 7.8 gives few examples of this particular observation.

Table 7.8 Orifice openings for random solutions at 100th generation

Orifice openings (cm)							Pollution load (kt/day)	Cost (M.€/year)
O1	O2	O3	O4	O5	O6	O7		
145	61.48	62.5	62.5	145	62.49	62.46	1.32084	10.7738
145	53.64	62.5	62.5	145	62.5	62.36	1.32084	10.7738
145	56.77	62.5	62.5	143.62	62.42	46.81	1.32084	10.7738
145	56.65	62.5	62.5	143.64	62.35	54.84	1.32084	10.7738

In such cases the selection of a set of decision variables corresponds to the minimum pollution load approach was not straightforward process. Overall system storage was considered the basis in such cases. The set of decision variables, which gives the minimum overall system storage, was picked to the next time step optimization. The similar situation was observed during the rest of the time steps in minimum pollution load approach and the Pareto optimal fronts are presented in the Appendix D. Table 7.9 gives the orifice openings obtained for the minimum pollution load approach according to the above stated environment.

Table 7.9 Orifice openings for minimum pollution load solutions under SWMM constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	41.88	0	35.73	13.97	0	0	0
15-30	136.81	40.41	62.5	62.41	137.91	56.38	30.46
30-45	145	62.49	62.5	62.5	145	62.5	62.5
45-60	145	37.55	62.5	62.5	145	27.11	62.5
60-75	145	40.7	58.74	62.5	122.14	58.1	58.38
75-90	83.63	38.64	33.38	62.5	143.62	47.71	0.04
90-105	92.78	37.8	57.71	62.5	111.46	4.37	50.97
105-120	129.59	62.2	22.45	62.39	137.03	46.06	56.58
120-135	53.36	11.09	55.85	59.04	24.29	9.87	50
135-150	129.59	62.2	22.45	62.39	137.03	46.06	56.58

Minimum treatment cost solution

Figure 7.14 shows the Pareto optimal front obtained at 30 minutes for the minimum treatment cost solution under the SWMM constraint handing techniques. Solution V_{T2}(S) corresponds to the identified minimum treatment cost solution at 30 minutes. Similar process was carried out for the other time steps of the dynamic optimization. Figure 7.15 is the Pareto optimal front for the 45 minutes. Pareto optimal fronts for the other time steps are given in the Appendix D.

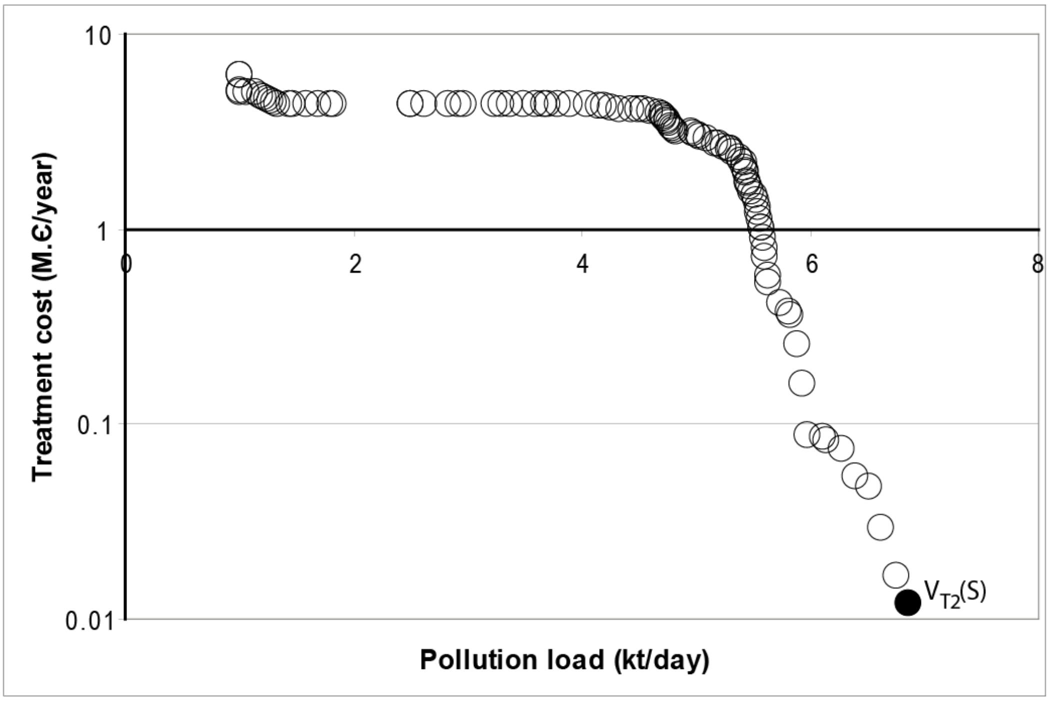

Figure 7.14 Pareto optimal front for T=30 minutes for minimum treatment cost approach

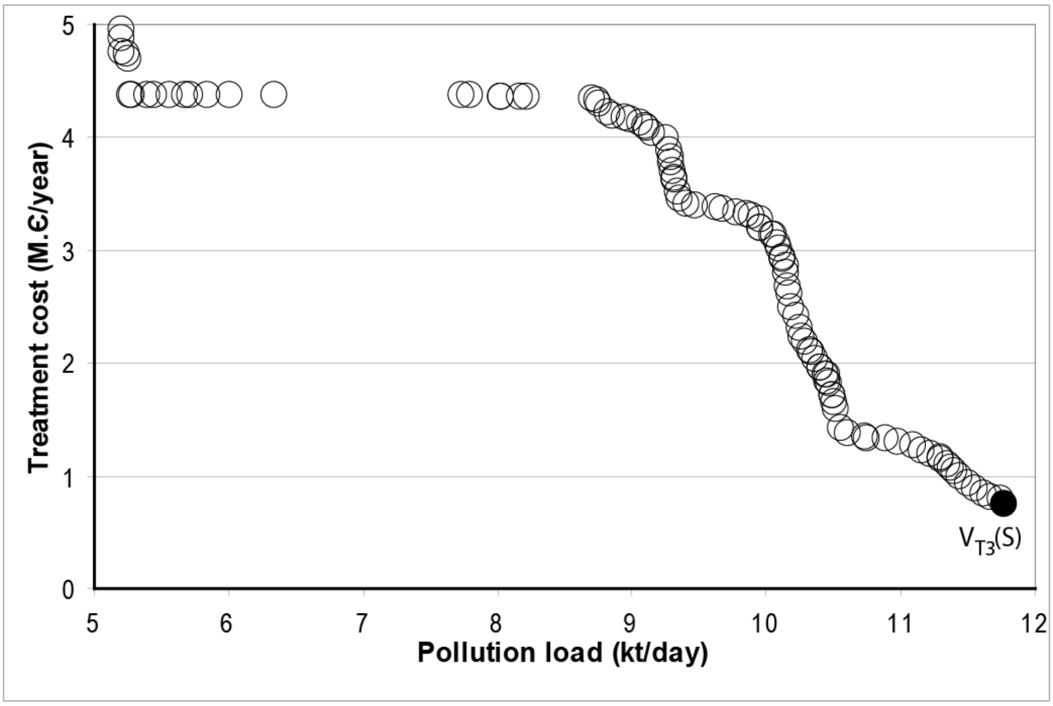

Figure 7.15 Pareto optimal front for T=45 minutes for minimum treatment cost approach

Table 7.10 Orifice openings for minimum treatment cost solutions under SWMM constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	4.94	0	0	0	0	0	0
15-30	9.28	0.05	0	0.01	0	0	0
30-45	0.04	0.01	0	0	0	0	0
45-60	0.01	0	0	0	0	0	0
60-75	1.19	0.05	0	0	0	0	0
75-90	0.02	0.01	0	0	0	0	0
90-105	0.20	0.03	0.01	0	0	0	0
105-120	0.31	0.03	0	0	0	0	0
120-135	0.12	0.03	0.01	0	0	0	0
135-150	0.44	0	0.01	0	0	0	0

Table 7.10 presents the achieved orifice openings throughout the total storm duration for the minimum wastewater treatment cost approach. Hydraulic simulations from these control settings are presented in the Appendix E.

7.5.4 Comparison of the two constraint handling approaches for single storm

Table 7.11 gives the minimum pollution load and the corresponding wastewater treatment costs along the storm duration under both constraint handling approaches. It can be clearly seen that the SWMM constraint handling approach produces much better solutions. For example, the minimum pollution load solution from NSGA II constraint handling approach at 02:30:00 is 17.71919 kt/day whereas that from SWMM constraint handling approach is 2.458298 kt/day. This shows a significant reduction. More importantly, the corresponding wastewater treatment costs for these two solutions are virtually the same. This finding is graphically visualized in Figures 7.16 and 7.17.

Table 7.11 Minimum pollution load solutions for single storm

Time (hr:min:sec)	NSGA II approach		SWMM approach	
	Pollution load (kt/day)	Corresponding Cost (M.€ /year)	Pollution load (kt/day)	Corresponding Cost (M.€ /year)
00:15:00	0.328905	2.423498	0	2.0632
00:30:00	4.595415	6.768871	0.083328	6.420127
00:45:00	8.695957	11.120941	1.320841	10.773808
01:00:00	11.743592	15.471457	2.049974	15.127489
01:15:00	13.955019	19.81973	2.434666	19.48117
01:30:00	15.172698	24.161904	2.458298	23.834852
01:45:00	15.962518	28.48332	2.458298	28.188532
02:00:00	16.530712	32.834588	2.458298	32.542212
02:15:00	17.209994	37.172832	2.458298	36.895896
02:30:00	17.71919	41.525772	2.458298	41.249576

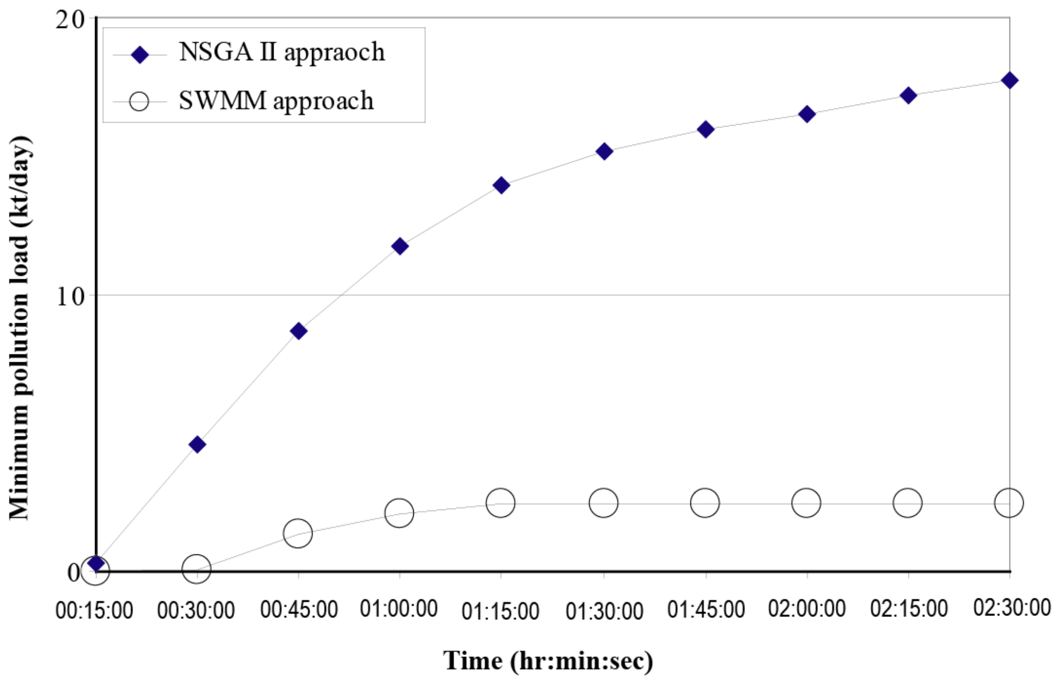

Figure 7.16 Minimum pollution loads at different time steps

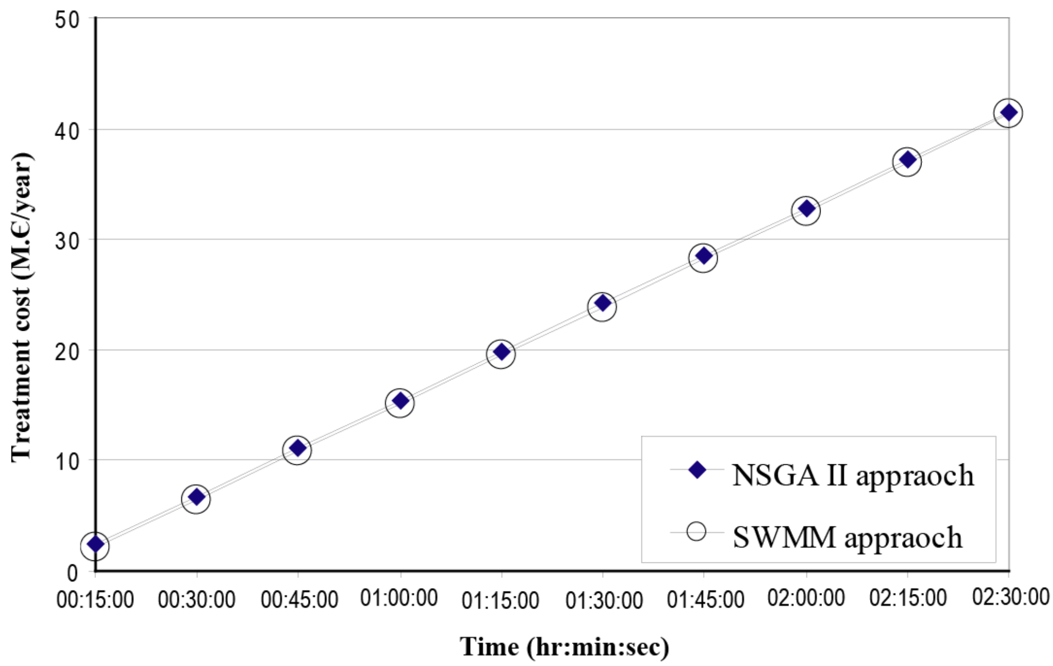

Figure 7.17 Corresponding treatment cost at different time steps

Table 7.12 illustrates the minimum wastewater treatment and the corresponding pollution load along the storm duration. In contrast to the minimum pollution load solution, NSGA II constraint handling approach gives better results in minimum treatment cost solution than that of SWMM approach. For example, the minimum wastewater treatment cost solution from SWMM constraint handling approach at 2 hours is 3.050101 M.€/year, whereas that from NSGA II constraint handling approach is 1.538147 M.€/year. However, the corresponding pollution loads from two different constraint handling approaches show virtually the same values. This interesting finding is graphically visualized in Figures 7.18 and 7.19.

Table 7.12 Minimum treatment cost solutions for single storm

Time (hr:min:sec)	NSGA II approach		SWMM approach	
	Cost (M.€ /year)	Corresponding Pollution load (kt/day)	Cost (M.€ /year)	Corresponding Pollution load (kt/day)
00:15:00	0.003946	1.598053	0.003946	1.598749
00:30:00	0.011524	7.103383	0.011941	6.859553
00:45:00	0.204151	11.879752	0.751892	11.775123
01:00:00	0.624853	15.343641	1.579422	15.353309
01:15:00	0.960471	17.996102	2.112044	17.987658
01:30:00	1.209283	19.95056	2.491067	19.955672
01:45:00	1.39384	21.688686	2.796524	21.720168
02:00:00	1.538147	23.293754	3.050101	23.33668
02:15:00	1.651255	25.012212	3.259397	25.038756
02:30:00	1.742049	26.52894	3.44223	26.538

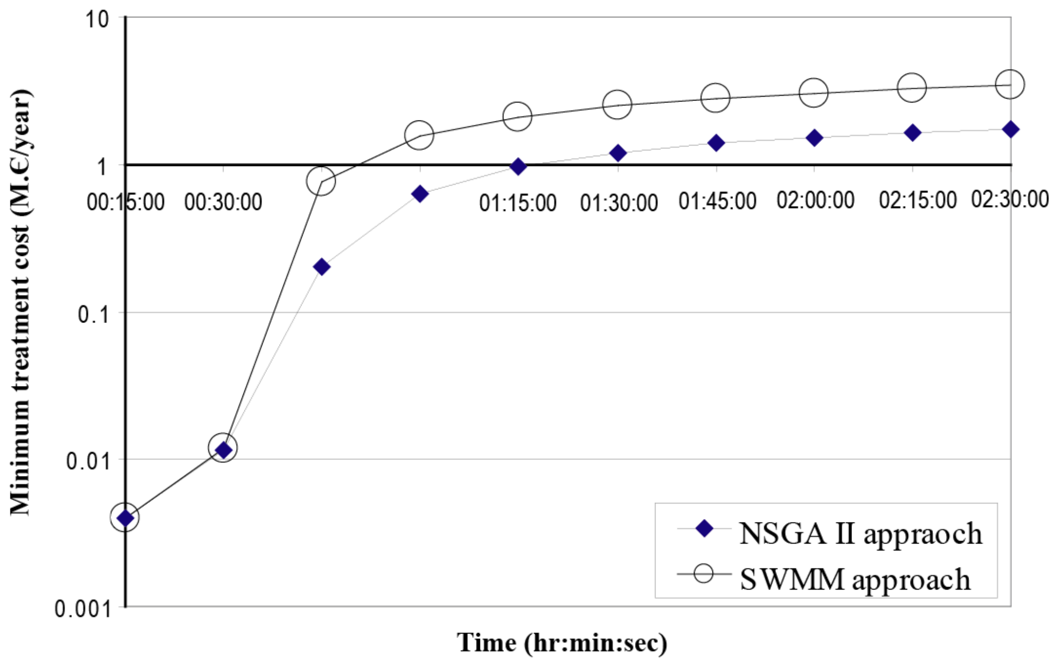

Figure 7.18 Minimum treatment cost at different time steps

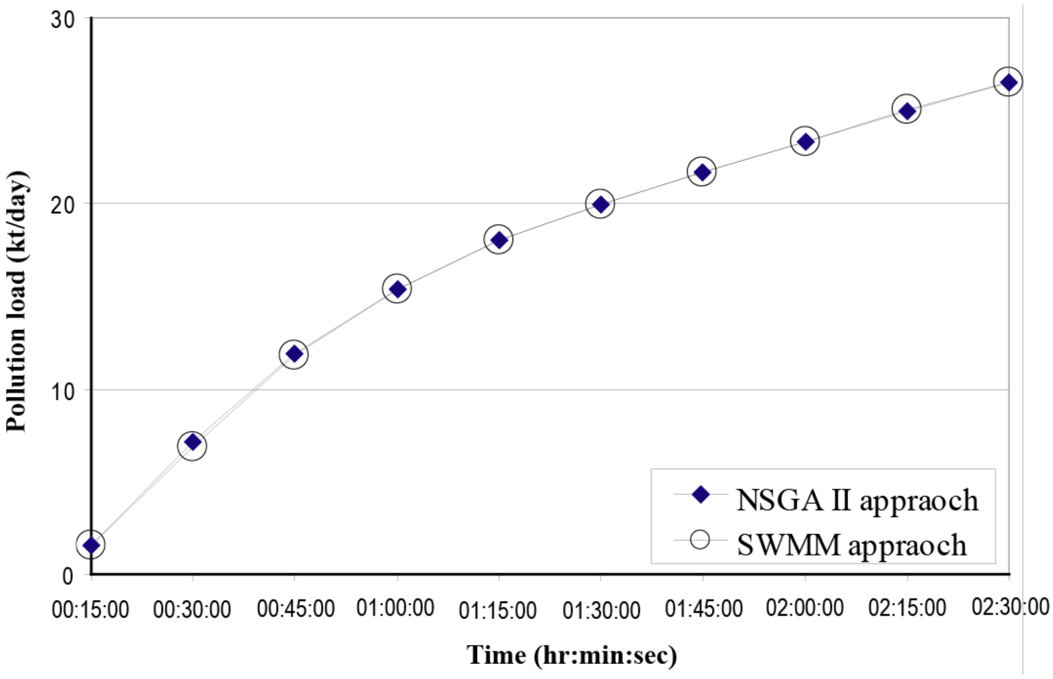

Figure 7.19 Corresponding pollution loads at different time steps

Table 7.13 presents the average simulation times taken for the single storm conditions. The optimization runs were performed on a Pentium 4 desktop computer with a Core 2 Duo processor and 4 GB of RAM. Simulation times for the SWMM constraint handling approaches are slightly higher than that of for NSGA II constraint handling approach.

Table 7.13 Average simulation times for single storm

Time step (hr:min:sec)	Simulation times (hr:min:sec)			
	NSGA II approach		SWMM approach	
	Min Cost	Min Pol	Min Cost	Min Pol
00:15:00	00:09:42		00:10:18	
00:30:00	00:12:47	00:18:42	00:14:50	00:20:40
00:45:00	00:12:30	00:25:05	00:12:02	00:34:34
01:00:00	00:12:58	00:31:34	00:13:46	00:35:24
01:15:00	00:14:06	00:38:47	00:14:54	00:39:26
01:30:00	00:14:56	00:46:00	00:16:18	00:47:10
01:45:00	00:15:29	00:53:15	00:18:23	00:54:06
02:00:00	00:16:12	01:00:51	00:18:36	01:01:05
02:15:00	00:17:12	01:08:17	00:20:27	01:08:17
02:30:00	00:18:06	01:15:57	00:20:40	01:16:14

7.5.5 Optimization results for two consecutive storms using NSGA II constraint handling approach

Figure 7.20 presents the Pareto optimal front archived at 15 minutes duration for two consecutive storm conditions, using the NSGA II constraint handling approach. Solutions A_{T1}(TC) and B_{T1}(TC) are the two extreme solutions which were selected for the dynamic optimization. Solution A_{T1}(TC) is the minimum pollution load solution at 15 minutes, whereas solution B_{T1}(TC) is the minimum treatment cost solution.

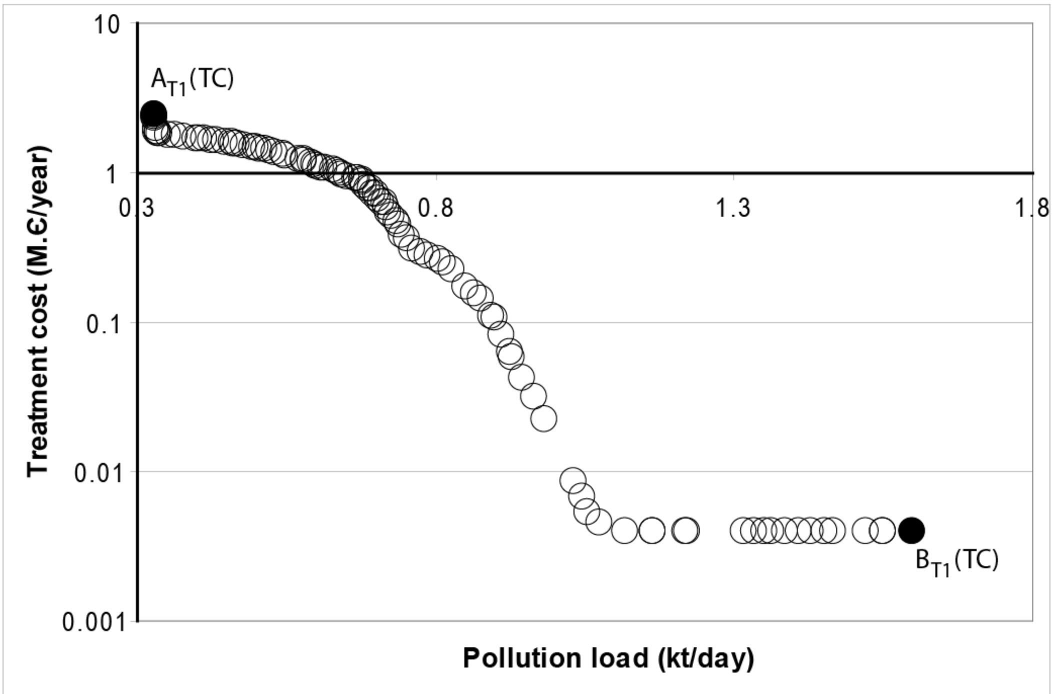

Figure 7.20 Pareto optimal front for T=15 minutes from NSGA II approach

Similar to the single storm conditions, picked two extreme solution were used to carry out the dynamic optimization in two separate approaches, minimum pollution load solution and minimum treatment cost solution. Figures 7.21 and 7.22 are couple of Pareto optimal fronts obtained for minimum pollution load solution at 2 hours and 3 hours & 30 minutes respectively. Figures 7.23 and 7.24 are two Pareto optimal fronts achieved for minimum treatment cost solution at 1 hour & 15 minutes and 3 hours & 30 minutes respectively. Pareto optimal fronts for the different time steps for the total storm periods can be found in the Appendix D.

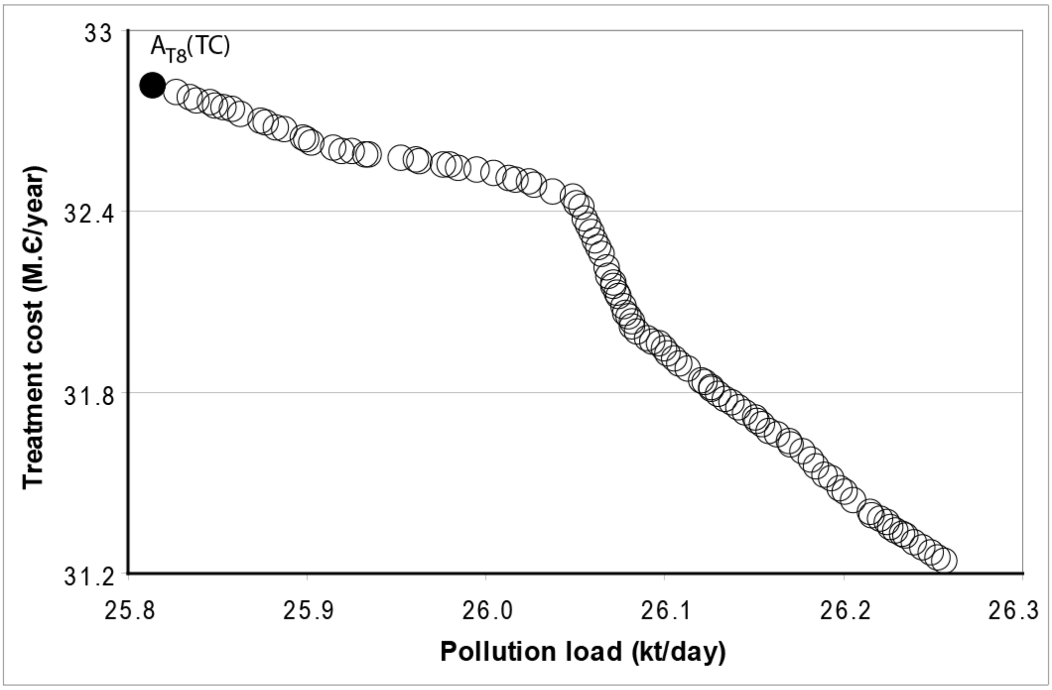

Figure 7.21 Pareto optimal front at T=2 hours for minimum pollution load approach

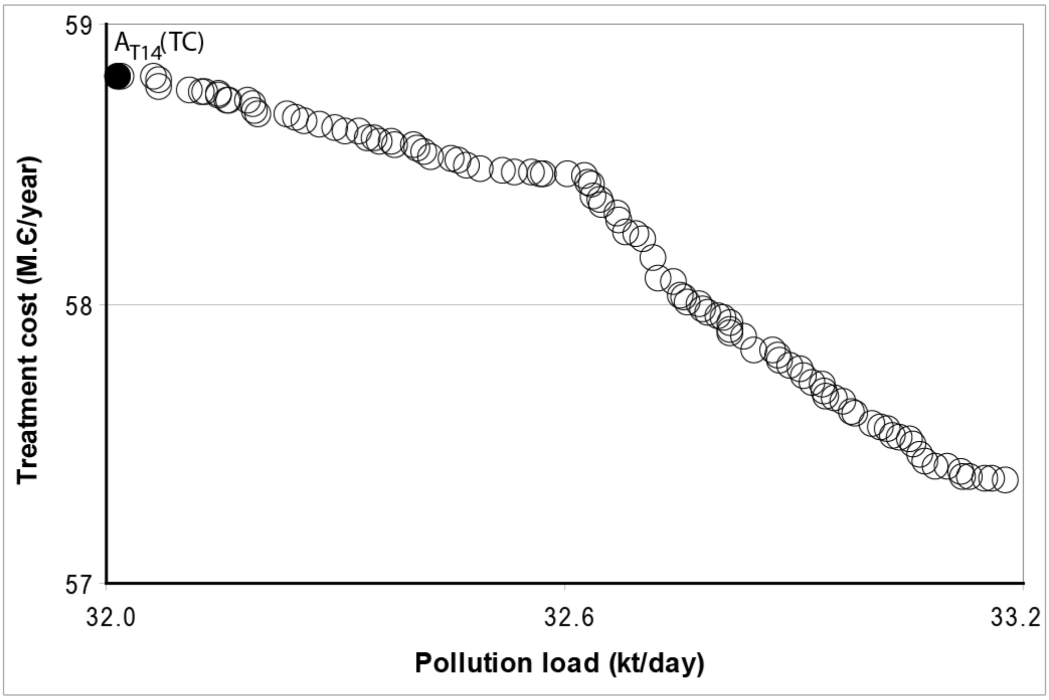

Figure 7.22 Pareto optimal front at T=3 hours & 30 minutes for minimum pollution load approach

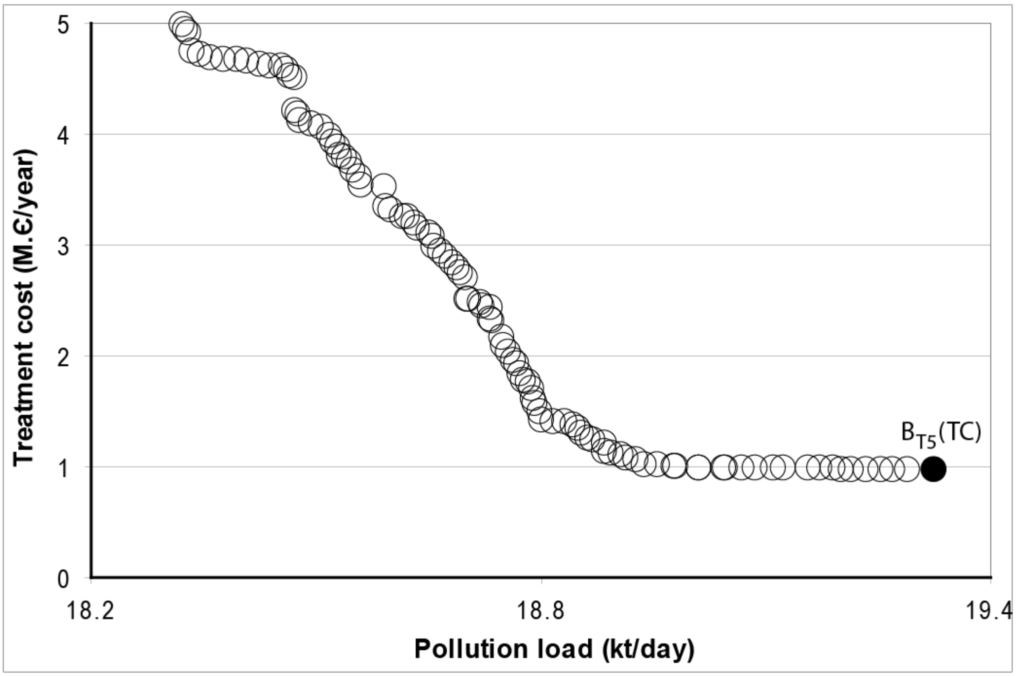

Figure 7.23 Pareto optimal front at T=1 hour & 15 minutes for minimum treatment cost approach

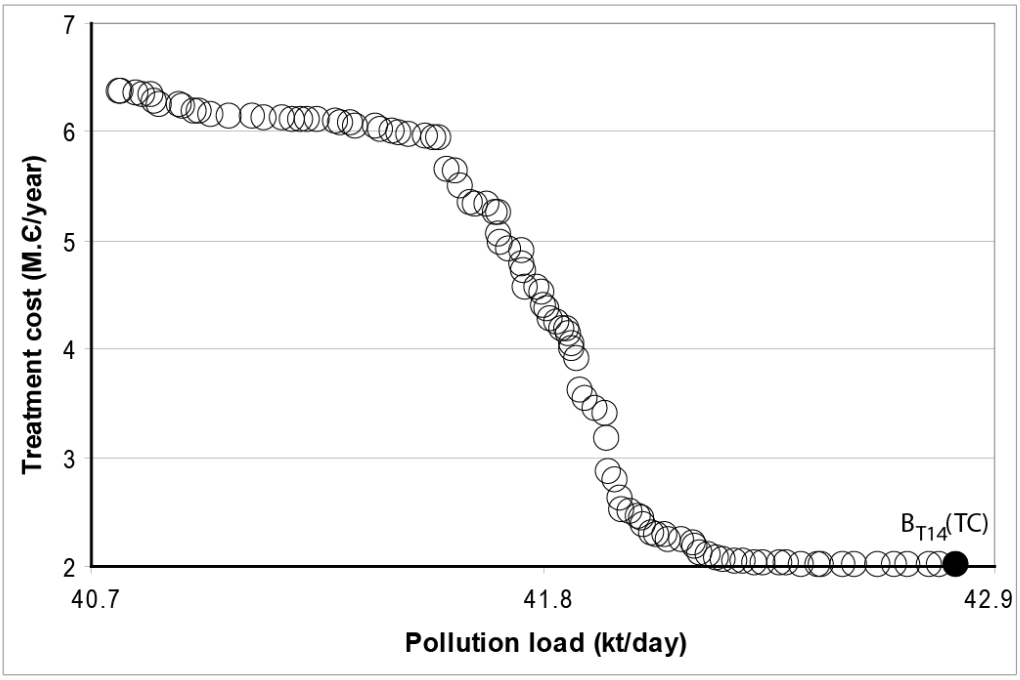

Figure 7.24 Pareto optimal front at T=3 hours & 30 minutes for minimum treatment cost approach

Tables 7.14 and 7.15 give the dynamic orifice openings correspond to the minimum pollution load and minimum treatment cost solution respectively. Minimum treatment cost solution shows smaller orifice openings compared to that of for the minimum pollution load solution. Hydraulic simulation results carried out using these orifice openings are given in the Appendix E.

Table 7.14 Orifice openings for minimum pollution load solutions under NSGA II constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	41.98	0	21.84	24.08	0	0.02	0
15-30	21.06	13.45	0	12.34	1.64	18.11	0
30-45	39.63	0	1.18	0.03	0	6.58	26.63
45-60	40.58	0	0.03	0.34	0.01	16.45	14.85
60-75	14.23	0	14.17	0.02	0.01	14.64	11.97
75-90	16.96	16.54	0.02	0.21	2.97	18.64	15.83
90-105	38.83	0.89	0	0.03	0.01	14.64	16.32
105-120	39.07	0.05	0.01	0.01	0	16.13	15.69
120-135	13.67	0.07	14.56	0.01	0	12.90	14.49
135-150	23.71	0	10.85	0.03	22.17	0	2.23
150-165	27.23	0	10.38	0.01	17.02	3.55	0.13
165-180	40.33	0.06	3.52	0.09	17.40	6.77	0.06
180-195	38.58	0.64	1.11	0.29	17.79	5.32	0.02
195-210	18.71	0.29	59.40	0	17.25	2.08	0.01

Table 7.15 Orifice openings for minimum treatment cost solutions under NSGA II constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	5.02	0	0	0	0	0	0
15-30	1.40	0.02	0	0	0	0	0
30-45	0	0	0	0	0	0	0
45-60	0.01	0	0	0	0	0	0
60-75	0.02	0	0	0	0	0	0
75-90	0.01	0	0	0	0	0	0
90-105	0.05	0	0	0	0	0	0
105-120	0.02	0	0	0	0	0	0
120-135	0.04	0	0	0	0	0	0
135-150	0.03	0	0	0	0	0	0
150-165	0.02	0	0	0	0	0	0
165-180	0.06	0	0	0	0	0	0
180-195	0.11	0	0	0	0	0	0
195-210	0.10	0	0	0	0	0	0

7.5.6 Optimization results for two consecutive storms using SWMM constraint handling approach

The following figures and tables give the results from the dynamic optimization for two consecutive storms using the SWMM constraint handling approach. Figure 7.25 is the Pareto optimal front at 15 minutes. Solutions U_{T1}(TC) corresponds to the minimum pollution load solution, whereas V_{T1}(TC) corresponds to the minimum treatment cost solution at 15 minutes. Figures 7.26 and 7.27 are the optimal fronts obtained at 30 minutes and 3 hours & 30 minutes for the minimum pollution load solutions. However, the similar observation shown in Figure 7.13 can be seen from the Figure 7.27. Figures 7.28 and 7.29 are the optimal front obtained at 30 minutes and 3 hours and 30 minutes for the minimum treatment cost solutions. Full set of Pareto optimal fronts for both minimum pollution load and treatment cost approaches for the total duration of the two consecutive storms are given in the Appendix D.

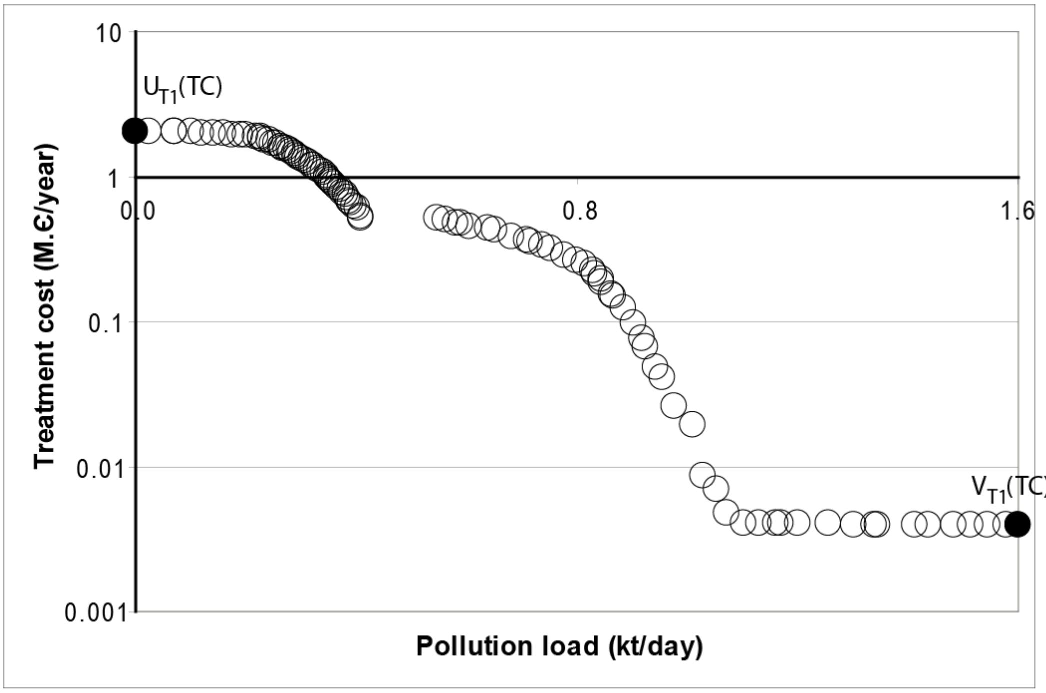

Figure 7.25 Pareto optimal front for T=15 minutes from SWMM approach

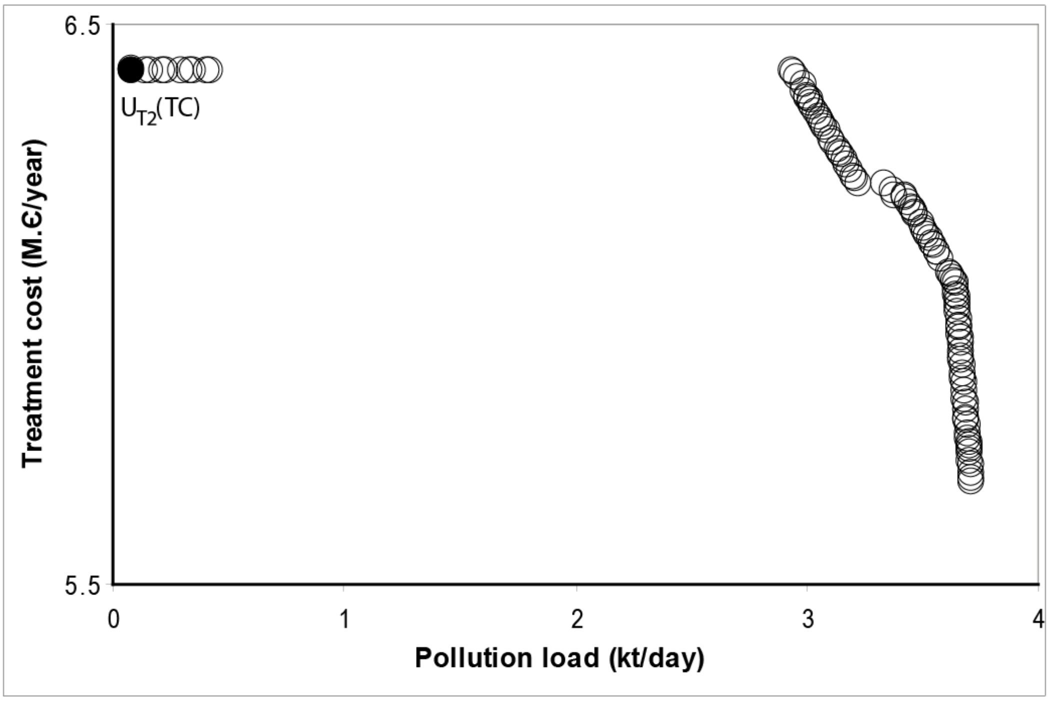

Figure 7.26 Pareto optimal front at T=30 minutes for minimum pollution load approach

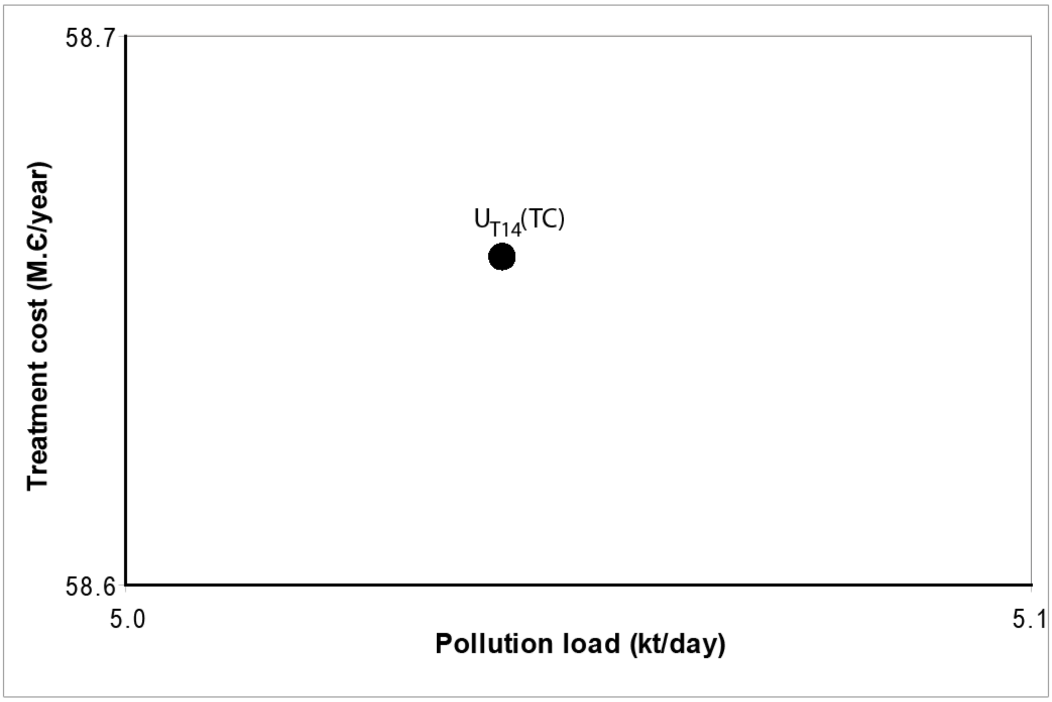

Figure 7.27 Pareto optimal front at T=3 hours & 30 minutes for minimum pollution load approach

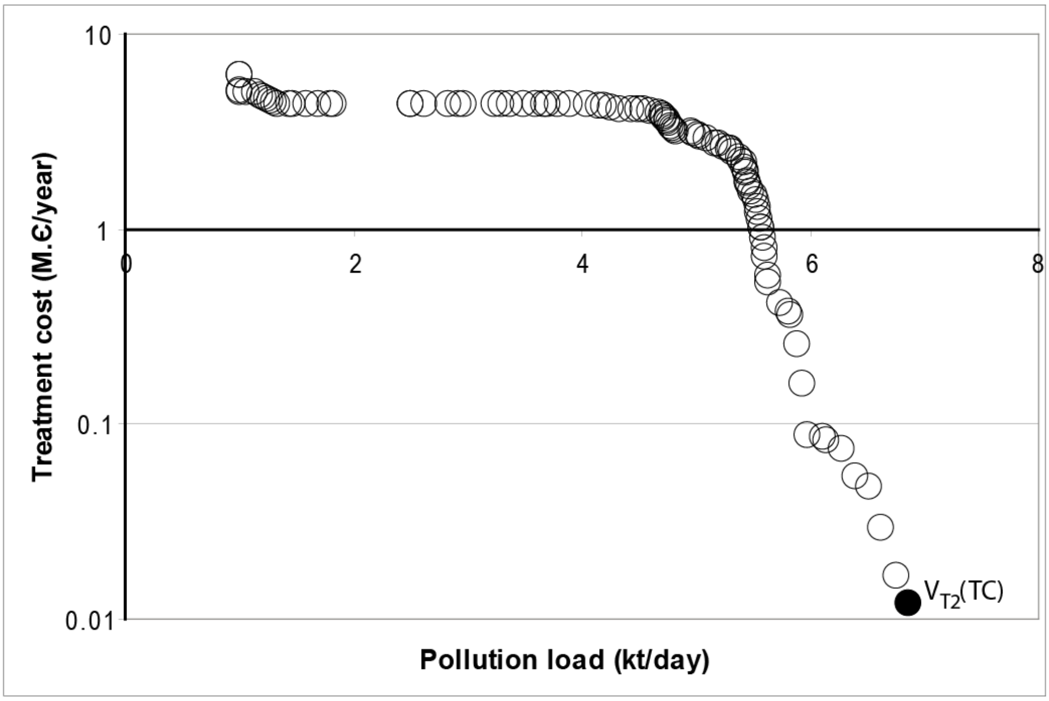

Figure 7.28 Pareto optimal front at T= 30 minutes for minimum treatment cost approach

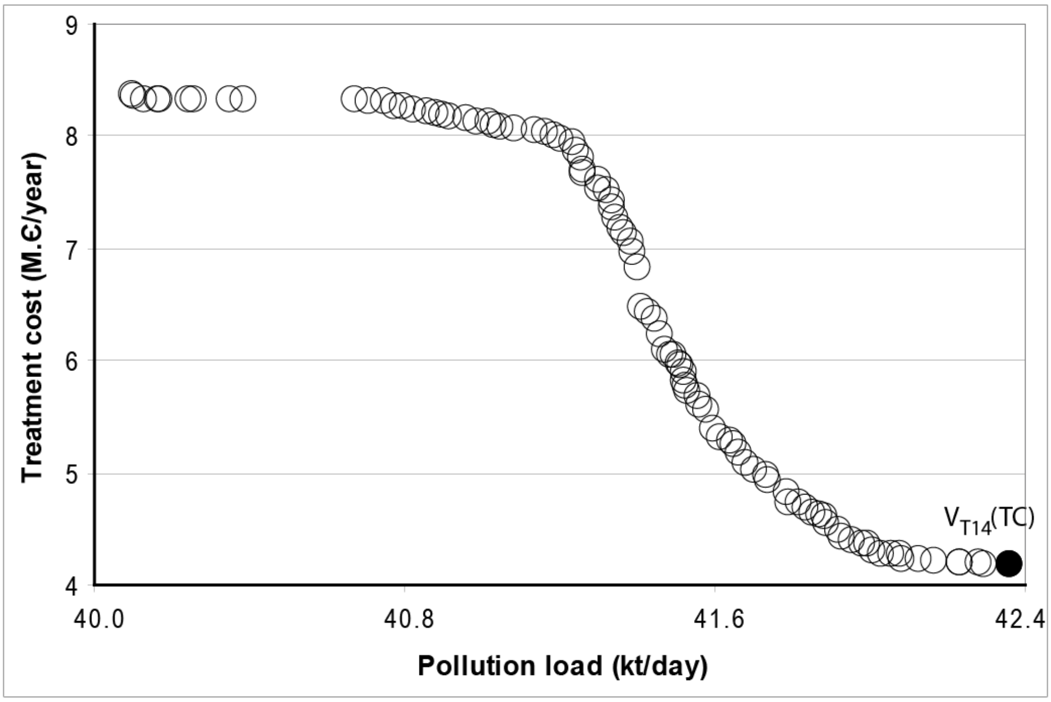

Figure 7.29 Pareto optimal front at T=3 hours & 30 minutes for minimum treatment cost approach

Tables 7.16 and 7.17 give the dynamic control settings throughout the two consecutive storm duration for minimum pollution load and minimum treatment cost solution respectively. Similar to the control settings obtained from NSGA II constraint handling approach, Table 7.17 shows smaller orifice openings compared to the Table 7.16. Hydraulic simulation results obtained using these orifice openings are shown in the Appendix E.

Table 7.16 Orifice openings for minimum pollution load solution under SWMM constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	41.88	0	35.73	13.97	0	0	0
15-30	136.81	40.41	62.5	62.41	137.91	56.38	30.46
30-45	145	62.49	62.5	62.5	145	62.5	62.5
45-60	145	37.55	62.5	62.5	145	27.11	62.5
60-75	145	61.51	34.33	62.5	144.99	59.35	11.82
75-90	135.07	62.48	62.5	62.5	87.74	33.83	38.33
90-105	145	62.5	62.5	62.5	145	62.5	62.48
105-120	145	62.5	62.49	62.5	145	62.46	61.95
120-135	145	62.18	29.4	62.5	142.13	62.5	62.44
135-150	58.53	34.32	62.42	62.5	145	62.22	62.46
150-165	144.24	29.53	39.04	62.5	144.42	49.43	62.38
165-180	145	59.46	46.2	50	144.5	39.11	48.13
180-195	144.21	61.01	35.15	60.78	91.49	12.62	3.63
195-210	144.74	62.06	61.71	61.07	144.95	59.92	58.81

Table 7.17 Orifice openings for minimum treatment cost solutions under SWMM constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	4.94	0	0	0	0	0	0
15-30	9.28	0.05	0	0.01	0	0	0
30-45	0.04	0.01	0	0	0	0	0
45-60	0.01	0	0	0	0	0	0
60-75	0.06	0.02	0	0	0	0	0
75-90	0.29	0.01	0	0	0	0	0
90-105	0.44	0.01	0	0	0	0	0
105-120	0.70	0.03	0	0	0	0	0
120-135	0.74	0.25	0	0	0	0	0
135-150	0.03	0.01	0.01	0	0	0	0
150-165	0.26	0	0	0	0	0	0
165-180	0.92	0	0	0	0	0	0
180-195	0.09	0.06	0	0	0	0	0
195-210	0.24	0.01	0	0	0	0	0

Table 7.18 gives the minimum pollution load and the corresponding wastewater treatment costs along the storm duration under both constraint handling approaches. Minimum pollution loads and corresponding treatment cost against the time were plotted and can be seen from the Figures 7.30 and 7.31 respectively.

Table 7.18 Minimum pollution load solutions for two consecutive storms

Time (hr:min:sec)	NSGA II approach		SWMM 5.0 approach	
	Pollution load (kt/day)	Corresponding Cost (M.€ /year)	Pollution load (kt/day)	Corresponding Cost (M.€ /year)
00:15:00	0.328905	2.423498	0	2.0632
00:30:00	4.595415	6.768871	0.083328	6.420127
00:45:00	8.695957	11.120941	1.320841	10.773808
01:00:00	11.743592	15.471457	2.049974	15.127489
01:15:00	15.360203	19.806018	2.658589	19.476516
01:30:00	18.437036	24.15867	2.800985	23.830196
01:45:00	22.748084	28.468092	3.814362	28.183878
02:00:00	25.814284	32.815484	4.578728	32.53756
02:15:00	27.903804	37.168296	4.990722	36.89124
02:30:00	29.163988	41.509656	5.041671	41.24492
02:45:00	30.174724	45.819264	5.041671	45.5986
03:00:00	30.775848	50.145708	5.041671	49.952284
03:15:00	31.464588	54.480264	5.041671	54.305964
03:30:00	32.014896	58.813452	5.041671	58.659644

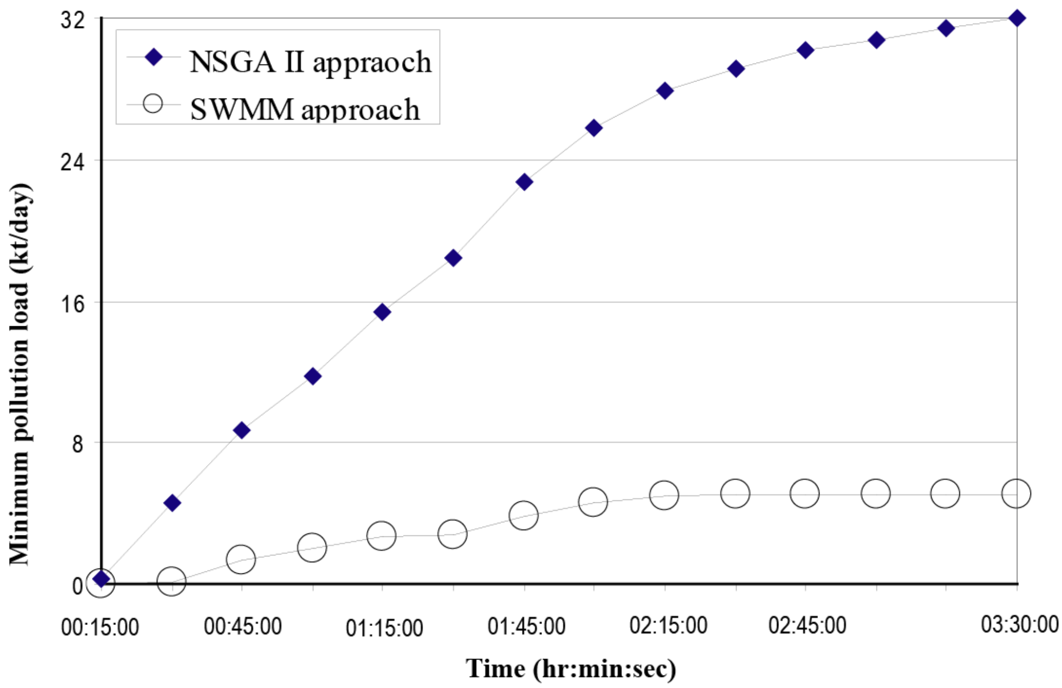

Figure 7.30 Minimum pollution loads at different time steps for two consecutive storms

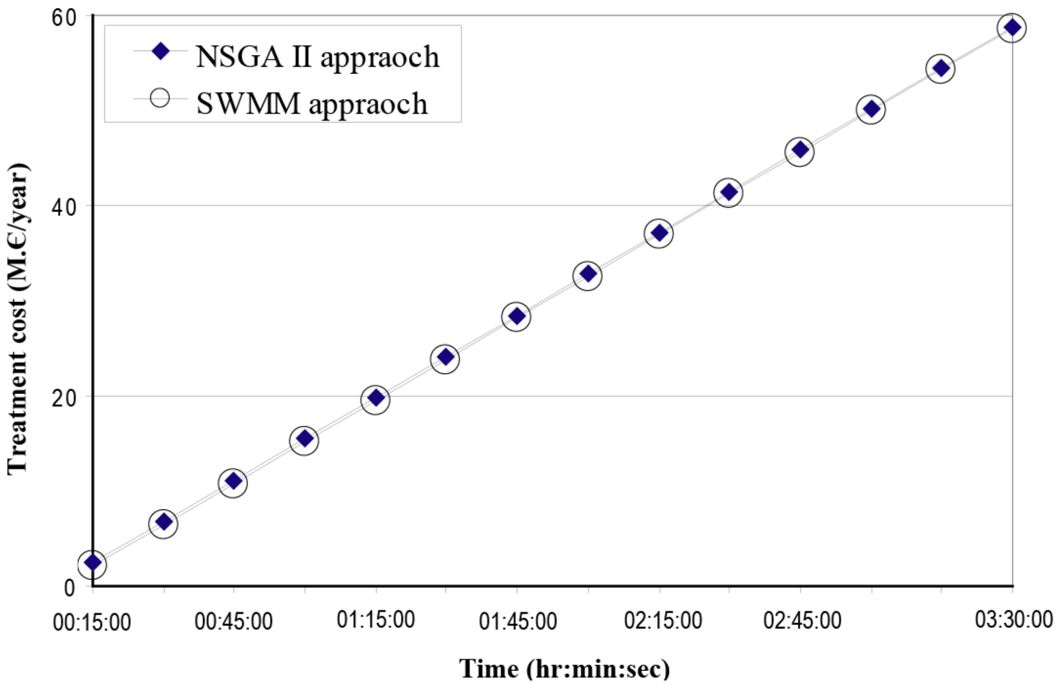

Figure 7.31 Corresponding treatment cost at different time steps

Table 7.19 gives the minimum treatment cost and the corresponding pollution load along the two consecutive storms duration under both constraint handling approaches. Minimum wastewater treatment cost and the corresponding pollution load against the time were plotted and can be seen from the Figures 7.32 and 7.33

respectively. Similar to the minimum solutions of single storm condition, the two consecutive storm conditions show the similar observations. SWMM constraint handling approach produces much better results in minimum pollution load approach compared to that of in NSGA II constraint handling approach. However, NSGA II constraint handling approach is better in obtaining minimum treatment cost solutions compared to SWMM approach.

Table 7.19 Minimum treatment cost solutions for two consecutive storms

Time (hr:min:sec)	NSGA II approach		SWMM 5.0 approach	
	Treatment cost (M.€ /year)	Corresponding Pollution load (kt/day)	Treatment cost (M.€ /year)	Corresponding Pollution load (kt/day)
00:15:00	0.003946	1.598053	0.003946	1.598749
00:30:00	0.011524	7.103383	0.011941	6.859553
00:45:00	0.204151	11.879752	0.751892	11.775123
01:00:00	0.624853	15.343641	1.579422	15.353309
01:15:00	0.968559	19.323788	2.104411	19.446212
01:30:00	1.218142	23.33559	2.459187	23.455462
01:45:00	1.403429	28.2377	2.713592	28.227824
02:00:00	1.548696	31.759176	2.91088	31.668536
02:15:00	1.66376	34.282168	3.0881	34.271984
02:30:00	1.757029	36.221356	3.287551	36.119776
02:45:00	1.835907	38.03232	3.52984	37.906656
03:00:00	1.904342	39.54714	3.766469	39.402136
03:15:00	1.965298	41.242952	3.975778	41.04704
03:30:00	2.020638	42.808128	4.178886	42.524588

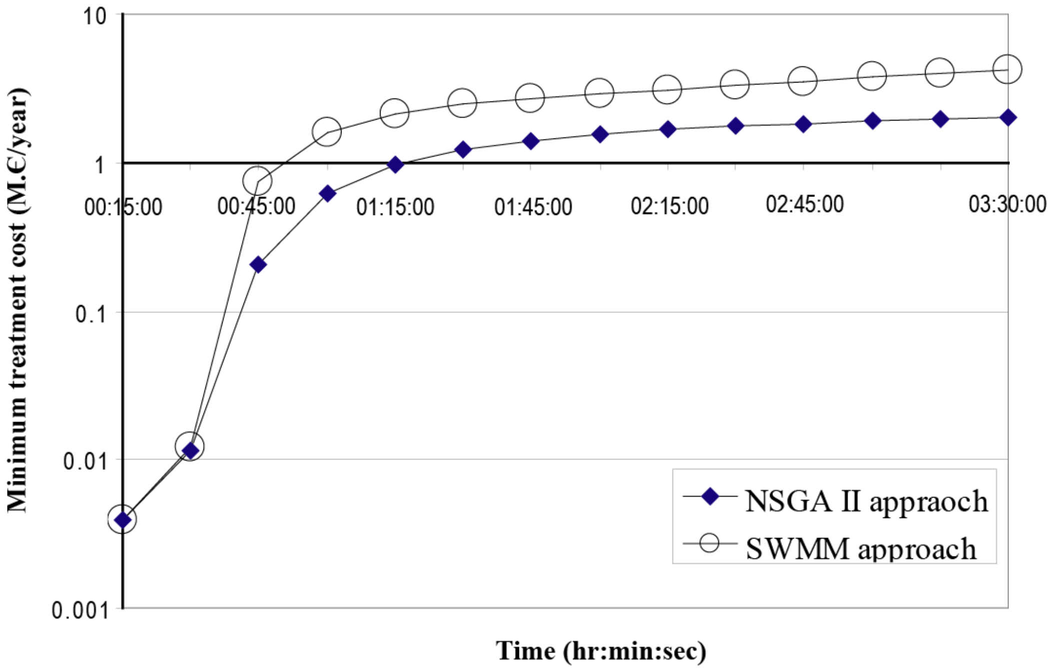

Figure 7.32 Minimum treatment cost at different time steps for two consecutive storms

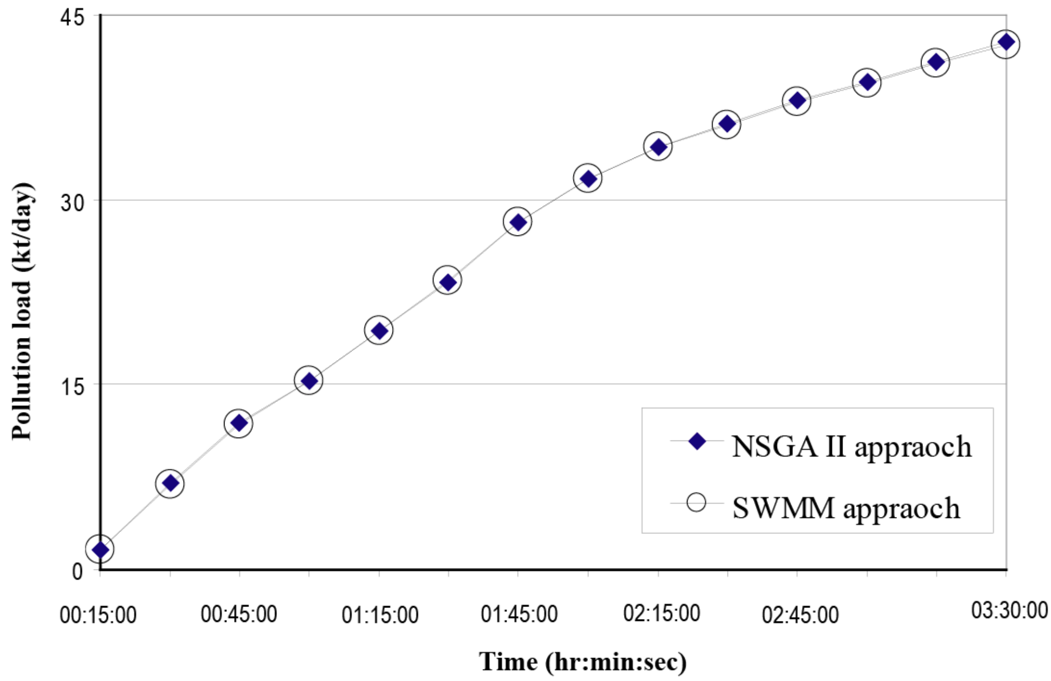

Figure 7.33 Corresponding pollution loads at different time steps

Table 7.20 gives the average simulation times for both solutions using the two constraint handling approaches. Simulation times were slightly higher than in the SWMM constraint handling approach compared to that of NSGA II constraint handling approach.

Table 7.20 Average simulation times for two consecutive storms

Time step (hr:min:sec)	Simulation times (hr:min:sec)			
	NSGA II approach		SWMM approach	
	Min Cost	Min Pol	Min Cost	Min Pol
00:15:00	00:09:42		00:10:18	
00:30:00	00:12:47	00:18:42	00:14:50	00:20:40
00:45:00	00:12:30	00:25:05	00:12:02	00:34:34
01:00:00	00:12:58	00:31:34	00:13:46	00:35:24
01:15:00	00:15:39	00:39:36	00:16:38	00:39:28
01:30:00	00:16:14	00:46:57	00:16:33	00:44:52
01:45:00	00:16:45	00:52:15	00:18:05	00:54:23
02:00:00	00:17:49	00:59:04	00:19:09	01:03:26
02:15:00	00:19:07	01:08:19	00:20:48	01:11:14
02:30:00	00:20:24	01:14:23	00:22:19	01:18:23
02:45:00	00:20:32	01:20:54	00:23:26	01:26:54
03:00:00	00:21:25	01:28:42	00:26:00	01:35:42
03:15:00	00:22:22	01:36:14	00:25:19	01:44:14
03:30:00	00:23:38	01:45:22	00:27:22	01:50:22

7.5.7 Optimization results for migrating downstream storms using NSGA II constraint handling approach

Figure 7.34 shows the Pareto optimal fronts archived at 15 minutes, for the migrating downstream storms under the NSGA II constraint handling approach, for different mutation probabilities. These Pareto optimal fronts were developed with the feasible solutions. It can be seen from the Figure 7.34 that the performance of mutation probability 0.4, over the entire population of solutions, is better than any of other. Similar to the other storm conditions, this observation shows that the higher mutation probability is beneficial for this multi-objective optimization problem.

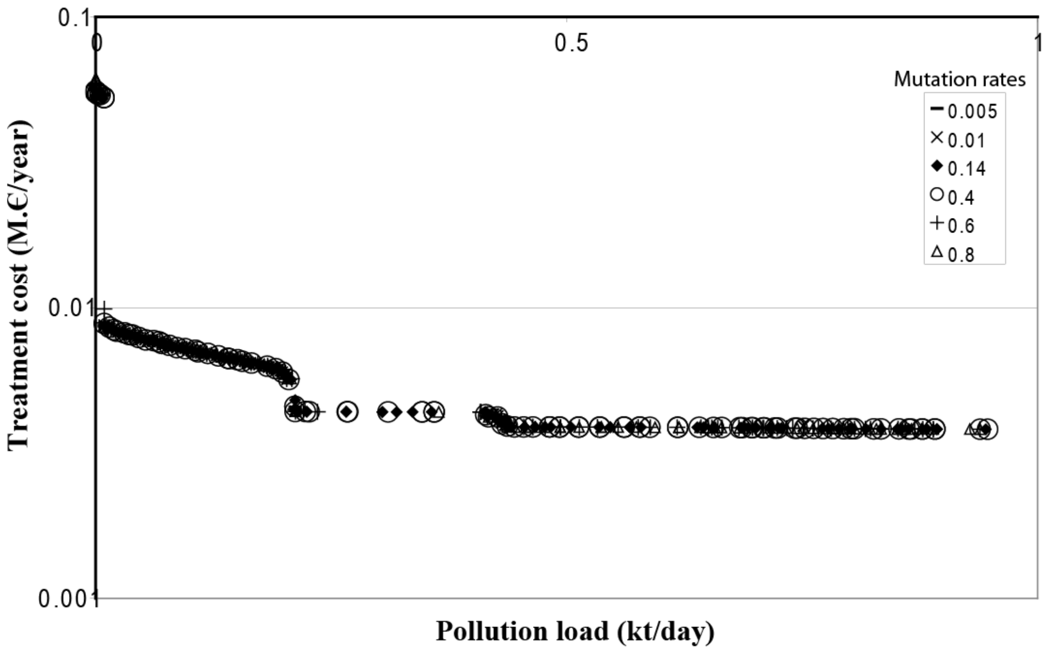

Figure 7.34 Pareto optimal fronts for different mutation rates for NSGA II approach for 15 minutes

Figures 7.35 to 7.37 show the results from the dynamic optimization for migrating downstream storms using the NSGA II constraint handling approach. Figure 7.35 is the Pareto optimal front at 15 minutes. Solutions A_{T1}(MDS) corresponds to the minimum pollution load solution, whereas B_{T1}(MDS) corresponds to the minimum treatment cost solution at 15 minutes. Figure 7.36 is the optimal fronts obtained at 30 minutes for the minimum pollution load solution. Figure 7.37 is the optimal front obtained at 30 minutes for the minimum treatment cost solutions. Full set of Pareto optimal fronts for both minimum pollution load and treatment cost approaches for the total storm duration are given in the Appendix D.

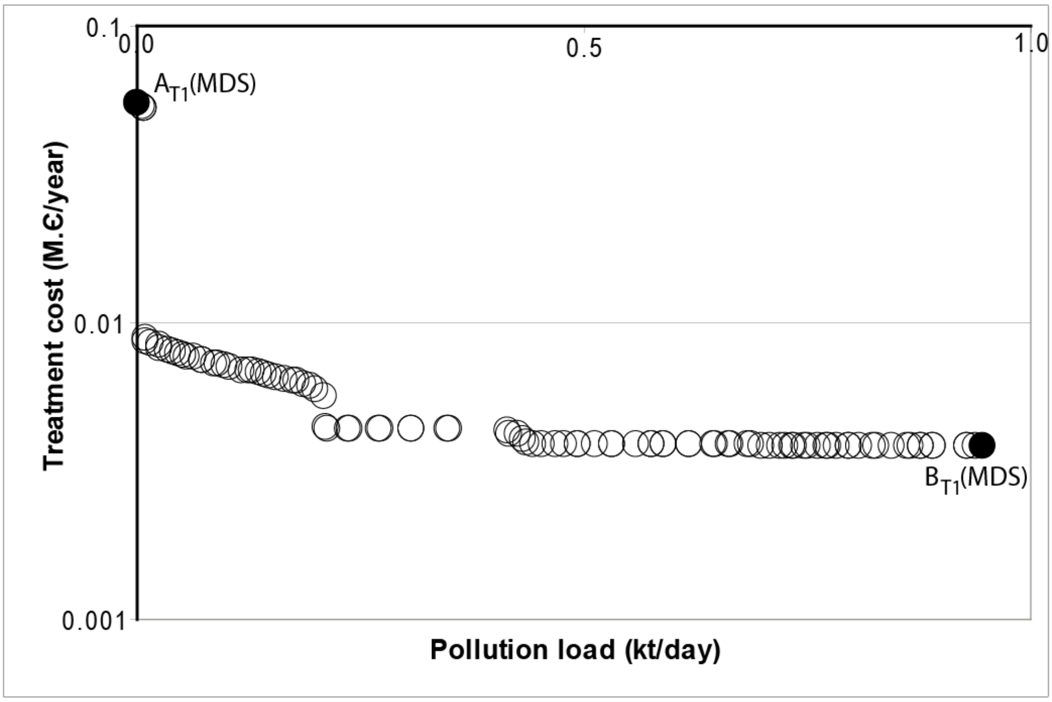

Figure 7.35 Best Pareto optimal front for T=15 minutes from NSGA II approach

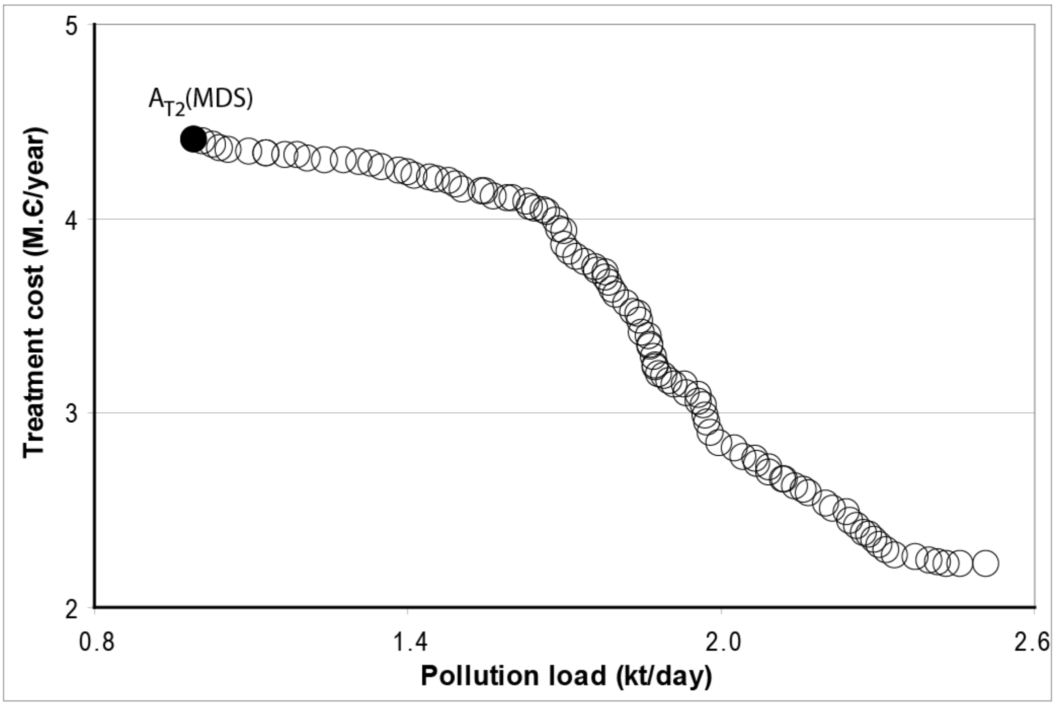

Figure 7.36 Pareto optimal front at T=30 minutes for minimum pollution load approach

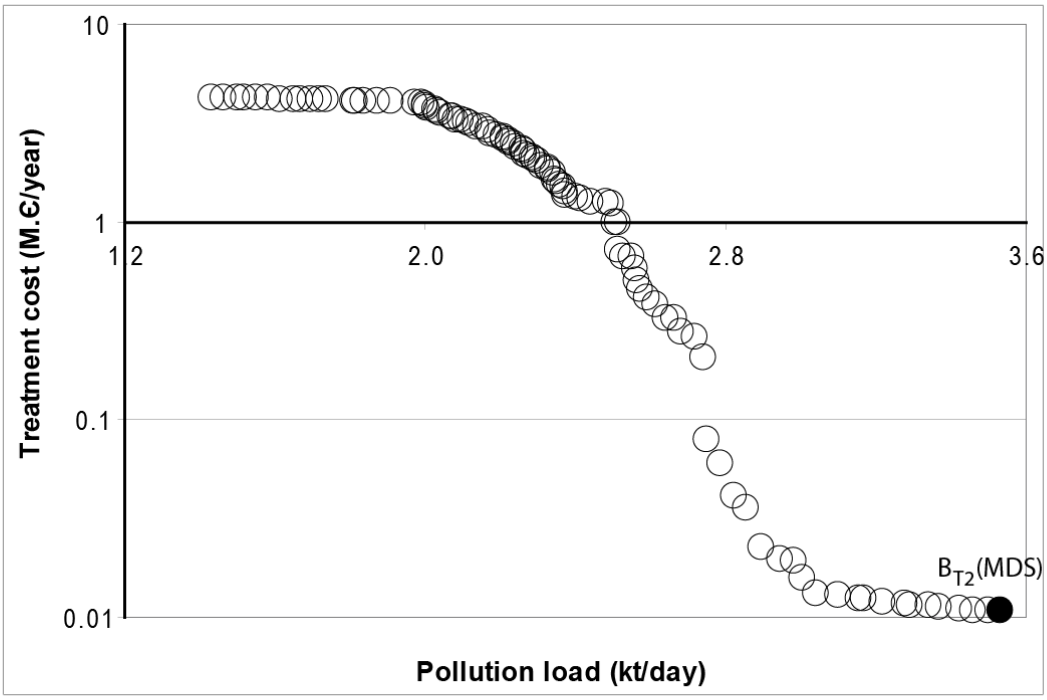

Figure 7.37 Pareto optimal front at T=30 minutes for minimum treatment cost approach

Tables 7.21 and 7.22 give the control settings (orifice openings) for the minimum pollution load and minimum treatment cost solutions respectively. Hydraulic simulation results using these orifice openings are given in the Appendix E.

Table 7.21 Orifice openings for minimum pollution load solution under NSGA II constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	43.31	0.09	8.70	0.93	0	0	0
15-30	10.17	0.95	25.39	2.61	20.72	2.58	0.25
30-45	27.61	8.26	0.11	32.43	0.01	8.09	0.14
45-60	5.24	0.22	0.70	3.95	11.19	15.45	3.36
60-75	40.60	0	4.03	0	0	19.29	22.20
75-90	42.53	0.08	0.05	0.13	0.04	15.09	16.63
90-105	40.32	0.09	1.17	0.01	0	14.86	16.19
105-120	33.27	0.09	4.98	1.34	8.40	7.74	8.32
120-135	21.33	0.14	44.62	0.13	14.51	4.64	0.73
135-150	30.07	0.02	10.06	0.03	19.39	4.19	0.02
150-165	38.53	0.03	2.77	0.35	23.24	0.04	0.61
165-180	19.07	0.25	62.19	8.53	12.82	0.03	0

Table 7.22 Orifice openings for minimum treatment cost solution under NSGA II constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	4.56	0	0	0	0	0	0
15-30	0	0	0	0	0	0	0
30-45	0.12	0	0	0	0	0	0
45-60	0.05	0	0	0	0	0	0
60-75	0.09	0	0	0	0	0	0
75-90	0.05	0.01	0	0	0	0	0
90-105	0	0	0	0	0	0	0
105-120	0.04	0	0	0	0	0	0
120-135	0.03	0	0	0	0	0	0
135-150	0.01	0	0	0	0	0	0
150-165	0.04	0	0	0	0	0	0
165-180	0.75	0.04	0	0	0	0	0

7.5.8 Optimization results for migrating downstream storms using SWMM constraint handling approach

Similar to the NSGA II constraint handling approach, a sensitivity analysis was carried out to find the best mutation probability to the multi-objective optimization problem under the SWMM constraint handling approach. Figure 7.38 gives some of the test mutation probabilities. Similar to the other cases, a higher mutation probability, 0.3 shows the better Pareto optimal front over the others.

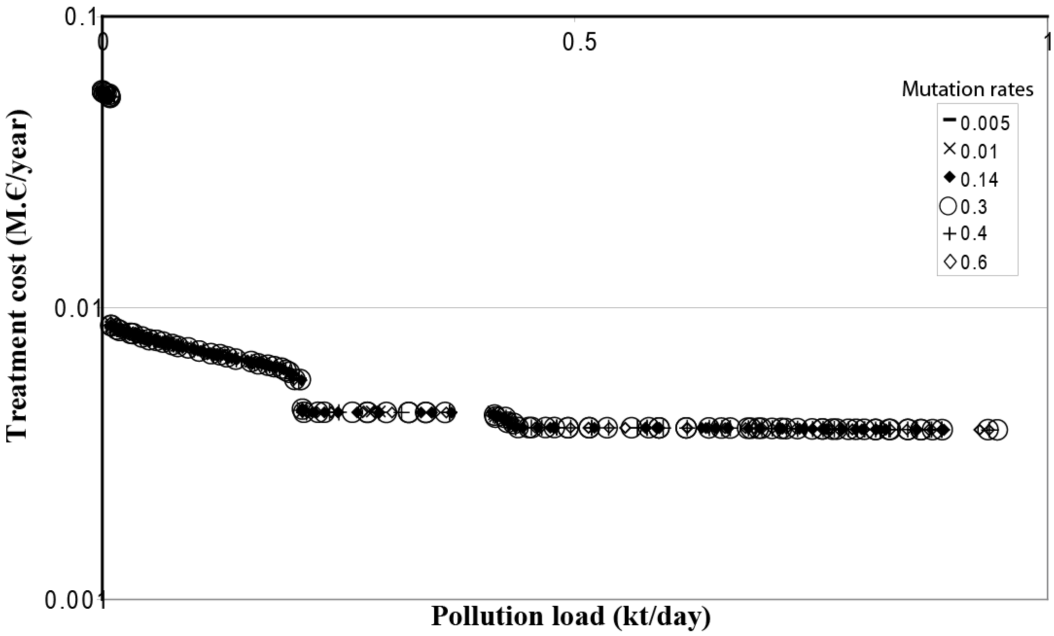

Figure 7.38 Pareto optimal fronts for different mutation rates for SWMM approach for 15 minutes

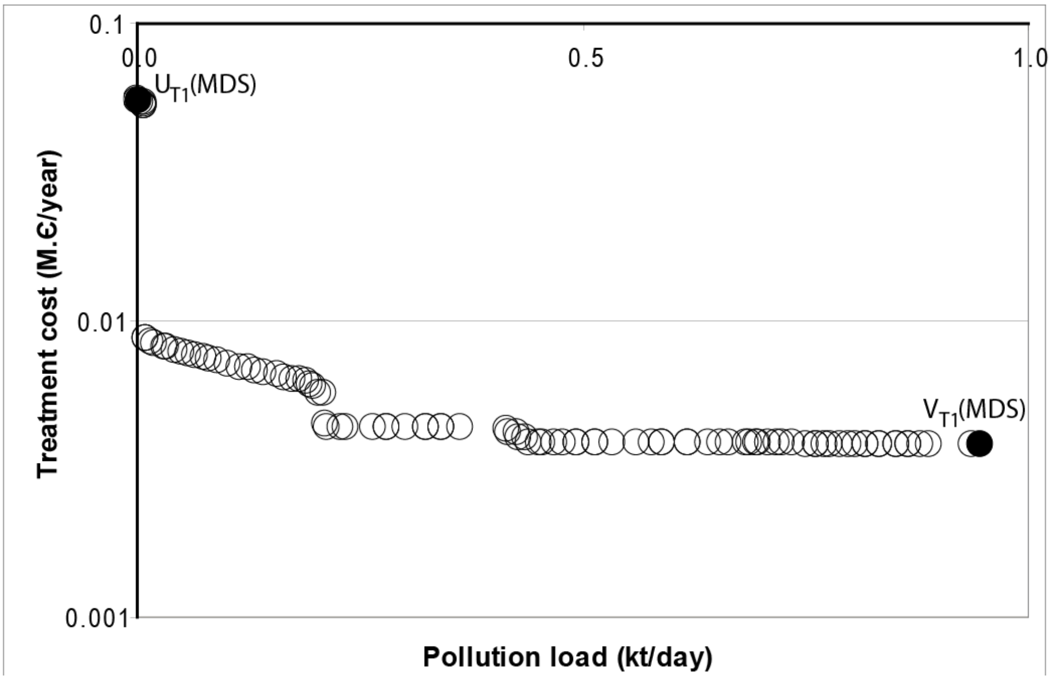

Figure 7.39 Best Pareto optimal front for T=15 minutes from SWMM approach

Figure 7.39 is the best Pareto optimal front achieved at the mutation probability of 0.3. U_{T1}(MDS) and V_{T1}(MDS) are the two extreme solutions selected for the dynamic optimization at 15 minutes. Solution U_{T1}(MDS) is the minimum pollution load solution, whereas solution V_{T1}(MDS) is the minimum treatment cost solution at

15 minutes. Figures 7.40 and 7.41 are couple of Pareto optimal fronts at 30 minutes for the minimum pollution load and the minimum treatment cost solutions respectively. However, the full sets of Pareto optimal fronts for both solutions are given in the Appendix D. From 45 minutes onwards, the optimization solutions for the minimum pollution load show the convergence to a single solution. However, the similar procedure has been followed in such cases as stated in the previous cases under the SWMM constraint handling techniques.

Tables 7.23 and 7.24 give the orifice openings for the two extreme solutions. Similar to the minimum treatment cost solutions in other storm conditions, Table 7.24 gives the smaller orifice openings compared to Table 7.23. Results from the hydraulic simulations using these orifice openings are given in the Appendix E.

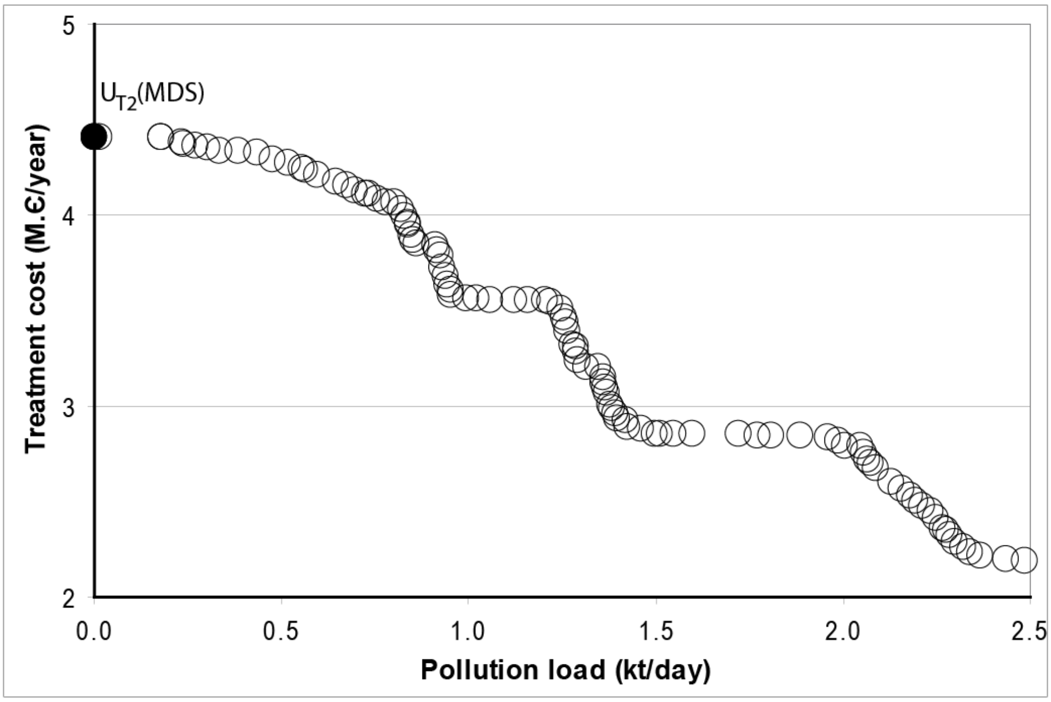

Figure 7.40 Pareto optimal front at T=30 minutes for minimum pollution load approach

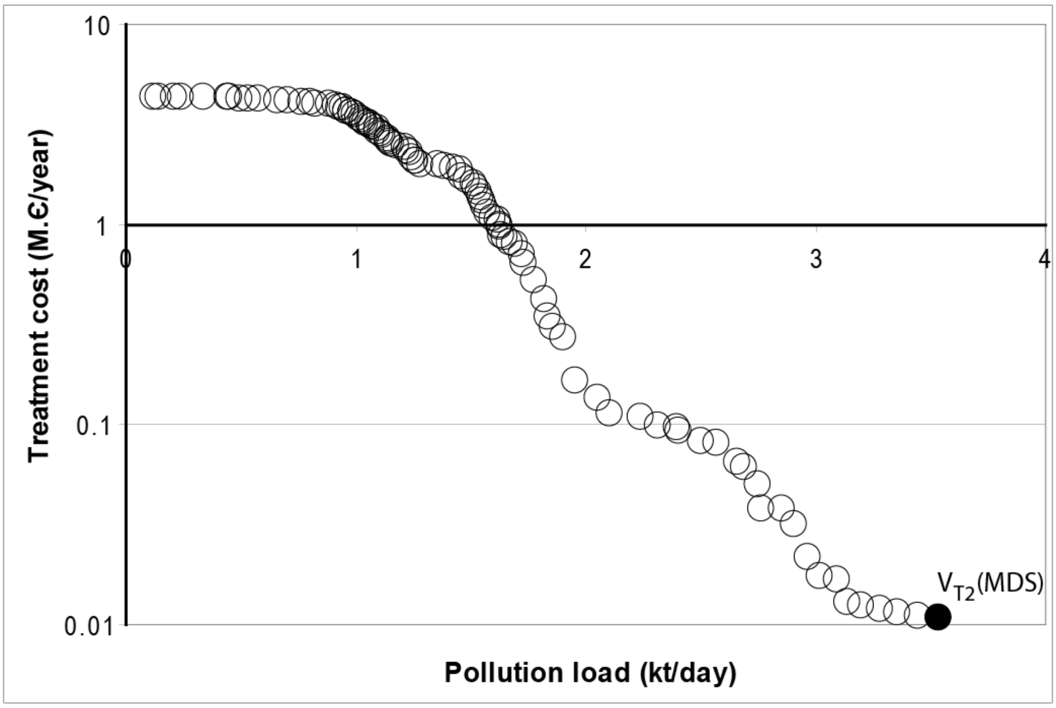

Figure 7.41 Pareto optimal front at T=30 minutes for minimum treatment cost approach

Table 7.23 Orifice openings for minimum pollution load solution under SWMM constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	43.31	0.09	8.70	0.93	0	0	0
15-30	10.17	0.95	25.39	2.61	20.72	2.58	0.25
30-45	27.61	8.26	0.11	32.43	0.01	8.09	0.14
45-60	5.24	0.22	0.70	3.95	11.19	15.45	3.36
60-75	40.60	0	4.03	0	0	19.29	22.20
75-90	42.53	0.08	0.05	0.13	0.04	15.09	16.63
90-105	40.32	0.09	1.17	0.01	0	14.86	16.19
105-120	33.27	0.09	4.98	1.34	8.40	7.74	8.32
120-135	21.33	0.14	44.62	0.13	14.51	4.64	0.73
135-150	30.07	0.02	10.06	0.03	19.39	4.19	0.02
150-165	38.53	0.03	2.77	0.35	23.24	0.04	0.61
165-180	19.07	0.25	62.19	8.53	12.82	0.03	0

Table 7.24 Orifice openings for minimum treatment cost solution under SWMM constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	4.56	0	0	0	0	0	0
15-30	0.04	0	0.01	0	0	0	0
30-45	0.21	0.02	0	0	0	0	0
45-60	0.78	0.10	0	0	0	0	0
60-75	0.85	0.02	0.01	0	0	0	0
75-90	0.33	0	0	0	0	0	0
90-105	0.03	0	0	0	0	0	0
105-120	0.04	0.15	0.01	0	0	0	0
120-135	1.78	0.13	0.04	0	0	0	0
135-150	0.48	0.01	0.00	0	0	0	0
150-165	0.02	0	0	0	0	0	0
165-180	0.16	0.08	0.01	0	0	0	0

Table 7.25 presents the minimum pollution load solutions and the corresponding wastewater treatment cost at different time steps for the migrating downstream storms. A significant reduction of minimum pollution load at similar time steps can be seen from the SWMM constraint handling approach. However, the corresponding costs for minimum pollution load solutions are almost equal. Figures 7.42 and 7.43 graphically illustrate these findings.

Table 7.25 Minimum pollution load solutions for migrating downstream storms

Time (hr:min:sec)	NSGA II approach		SWMM 5.0 approach	
	Pollution load (kt/day)	Corresponding Cost (M.€/year)	Pollution load (kt/day)	Corresponding Cost (M.€/year)
00:15:00	0	0.055061	0	0.055585
00:30:00	0.991524	4.40198	0	4.409326
00:45:00	3.564882	8.719388	0.204397	8.763007
01:00:00	8.279937	13.067719	0.872601	13.116688
01:15:00	11.708351	17.421892	1.886202	17.470368
01:30:00	14.230565	21.770632	2.49509	21.82405
01:45:00	16.047465	26.119696	2.691522	26.177732
02:00:00	17.396376	30.466436	2.700692	30.531412
02:15:00	18.189402	34.794528	2.700692	34.885092
02:30:00	18.685128	39.110796	2.700692	39.238776
02:45:00	19.320306	43.450644	2.700692	43.592456
03:00:00	19.861742	47.79842	2.700692	47.946136

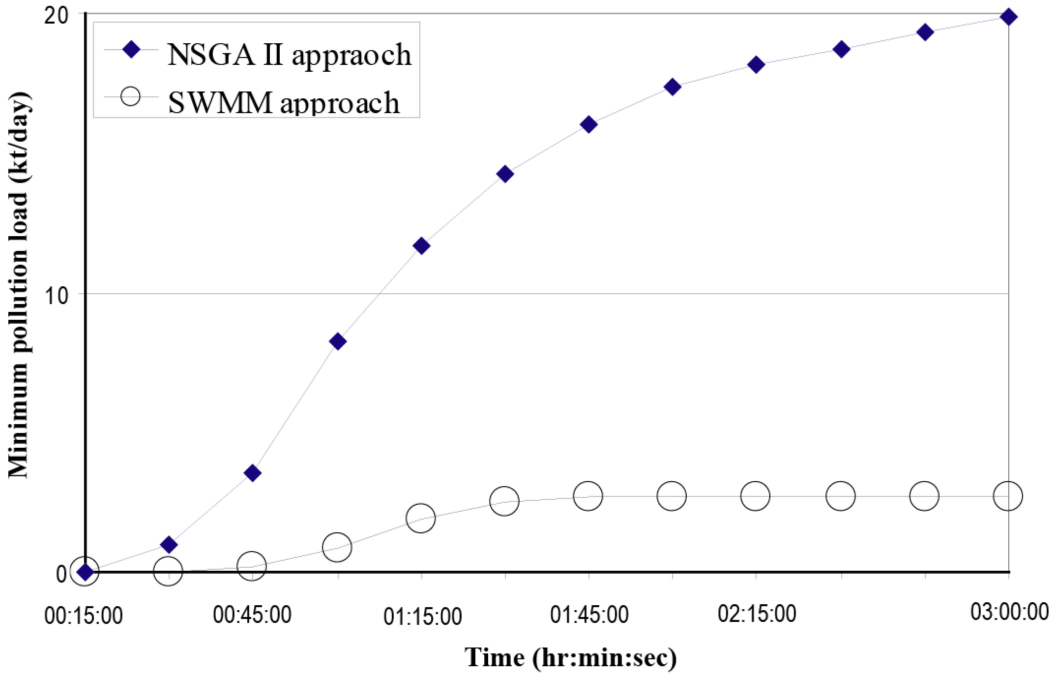

Figure 7.42 Minimum pollution loads at different time steps for migrating downstream storms

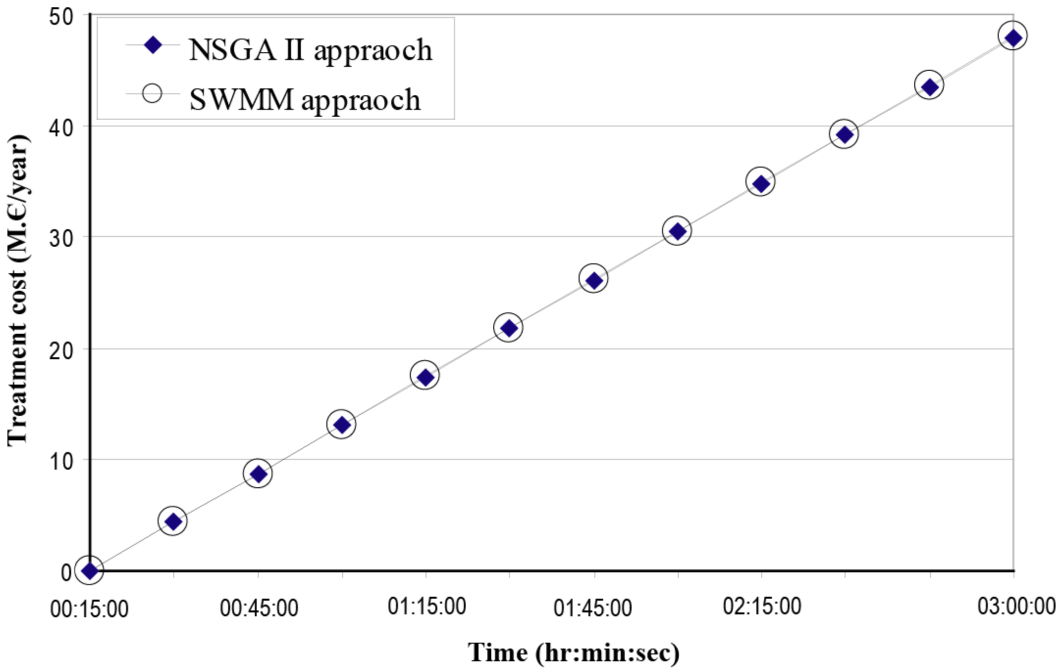

Figure 7.43 Corresponding treatment cost at different time steps

Table 7.26 presents the minimum wastewater treatment costs and corresponding pollution load at different time steps. During the initial time steps both constraint handling approach has produced almost the same treatment costs. However, towards the end of the storm, NSGA II constraint handling approach outperformed the SWMM approach. This can be clearly visualized in the Figures 7.44 and 7.45.

Table 7.26 Minimum treatment cost solutions for migrating downstream storms

Time (hr:min:sec)	NSGA II approach		SWMM 5.0 approach	
	Treatment cost (M.€/year)	Corresponding Pollution load (kt/day)	Treatment cost (M.€/year)	Corresponding Pollution load (kt/day)
00:15:00	0.003817	0.946995	0.003817	0.94693
00:30:00	0.01081	3.533958	0.010826	3.532967
00:45:00	0.118884	7.161026	0.121638	7.15851
01:00:00	0.402939	12.88111	0.414848	12.872538
01:15:00	0.667026	17.186718	0.705898	17.175436
01:30:00	0.871605	20.367472	0.979423	20.35453
01:45:00	1.03466	22.780016	1.246283	22.818612
02:00:00	1.168921	24.800034	1.489817	24.835754
02:15:00	1.277777	26.44316	1.701729	26.384444
02:30:00	1.369076	28.097394	1.908052	27.993994
02:45:00	1.447443	29.817248	2.15925	29.679768
03:00:00	1.515952	31.31866	2.435504	31.200756

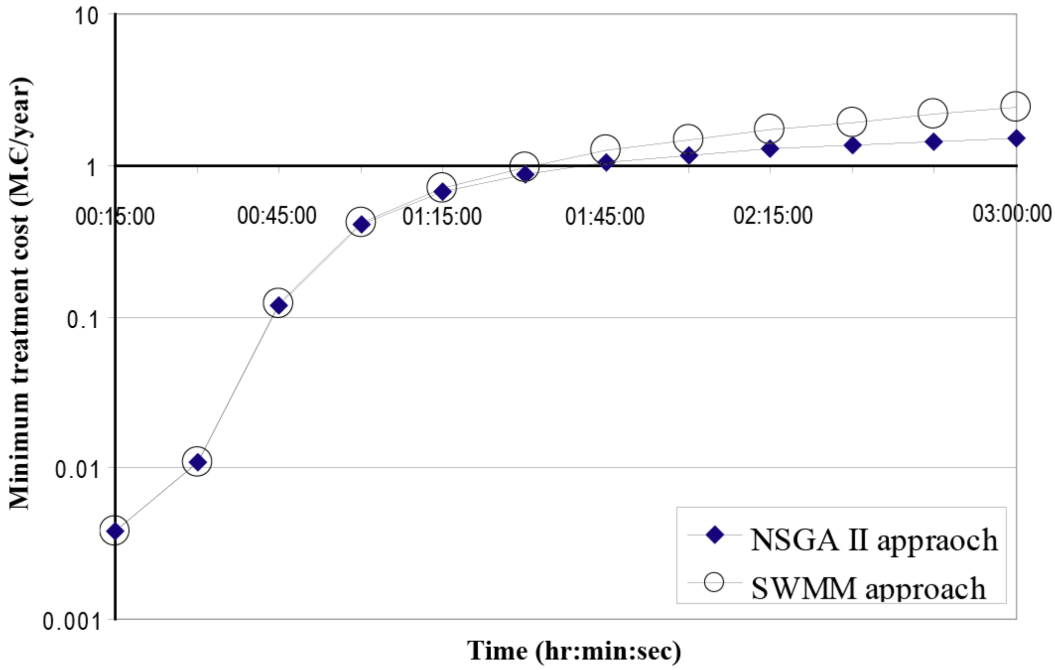

Figure 7.44 Minimum treatment cost at different time steps for migrating downstream storms

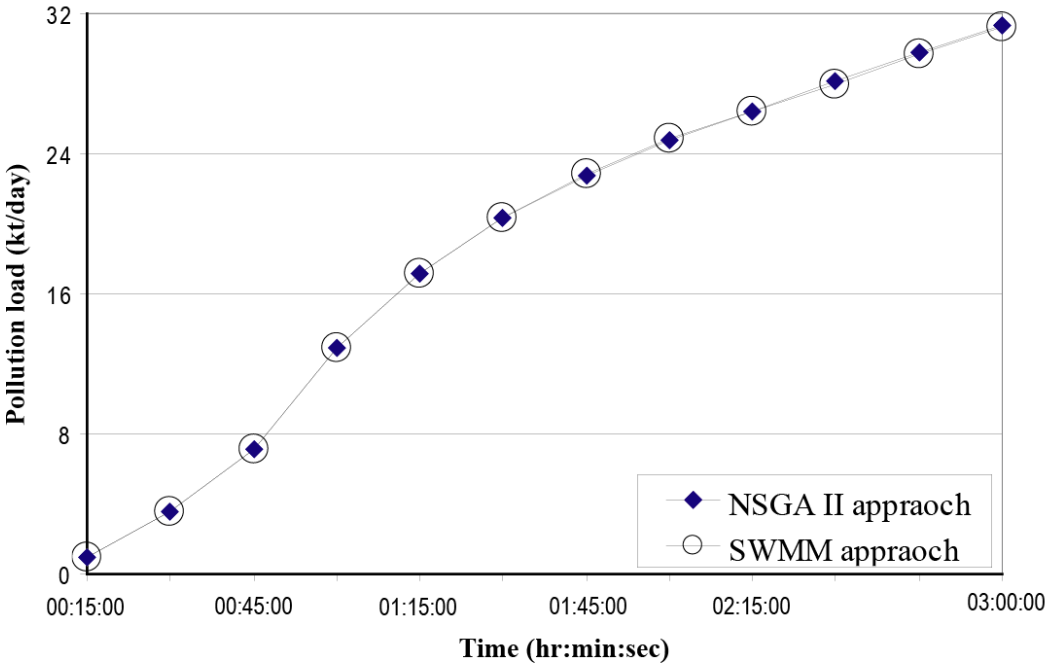

Figure 7.45 Corresponding pollution loads at different time steps

Table 7.27 presents the average simulation times for the migrating downstream storms. Simulation times for both constraint handling approaches are quite similar, but the SWMM approach has larger simulation times.

Table 7.27 Average simulation times for migrating downstream storms

Time step (hr:min:sec)	Simulation times (hr:min:sec)			
	NSGA II approach		SWMM approach	
	Min Cost	Min Pol	Min Cost	Min Pol
00:15:00	00:07:37		00:07:43	
00:30:00	00:11:39	00:14:21	00:12:18	00:14:12
00:45:00	00:13:11	00:21:51	00:11:27	00:23:29
01:00:00	00:13:37	00:30:26	00:14:43	00:42:12
01:15:00	00:13:08	00:36:38	00:13:35	00:56:08
01:30:00	00:14:18	00:42:13	00:14:47	00:56:13
01:45:00	00:15:24	00:50:09	00:15:25	00:57:44
02:00:00	00:16:46	00:57:11	00:16:37	00:59:16
02:15:00	00:16:50	01:04:46	00:19:53	01:12:02
02:30:00	00:18:31	01:12:17	00:19:37	01:22:56
02:45:00	00:19:03	01:19:14	00:20:17	01:28:34
03:00:00	00:19:26	01:27:28	00:21:03	01:33:33

7.5.9 Optimization results for migrating upstream storms using NSGA II constraint handling approach

Sensitivity analysis shows that the mutation probability of 0.4 gives best Pareto optimal front under NSGA II constraint handling approach. Figure 7.46 presents some of the mutation probability tested.

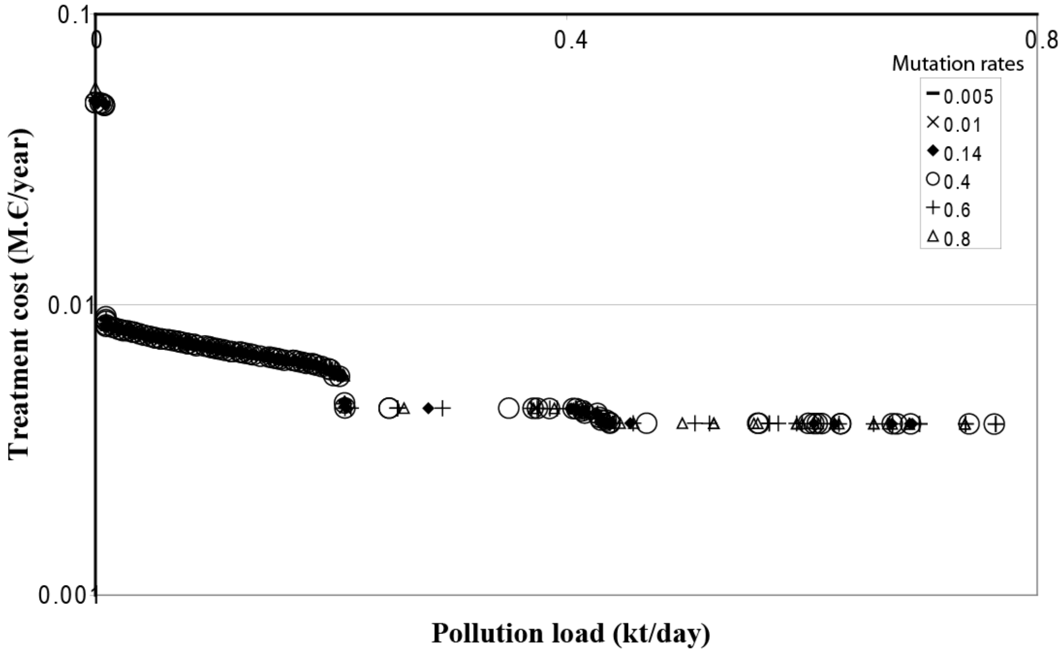

Figure 7.46 Pareto optimal fronts for different mutation rates for NSGA II approach for 15 minutes

Figure 7.47 is the best Pareto optimal front achieved under the NSGA II constraint handling approach. A_{T1}(MUS) and B_{T1}(MUS) are the two extreme solutions, minimum pollution load and minimum treatment cost solutions, selected for the dynamic optimization process. Figure 7.48 shows the minimum pollution load Pareto optimal front at 30 minutes, whereas Figure 7.49 presents the minimum treatment cost Pareto optimal front at 30 minutes.

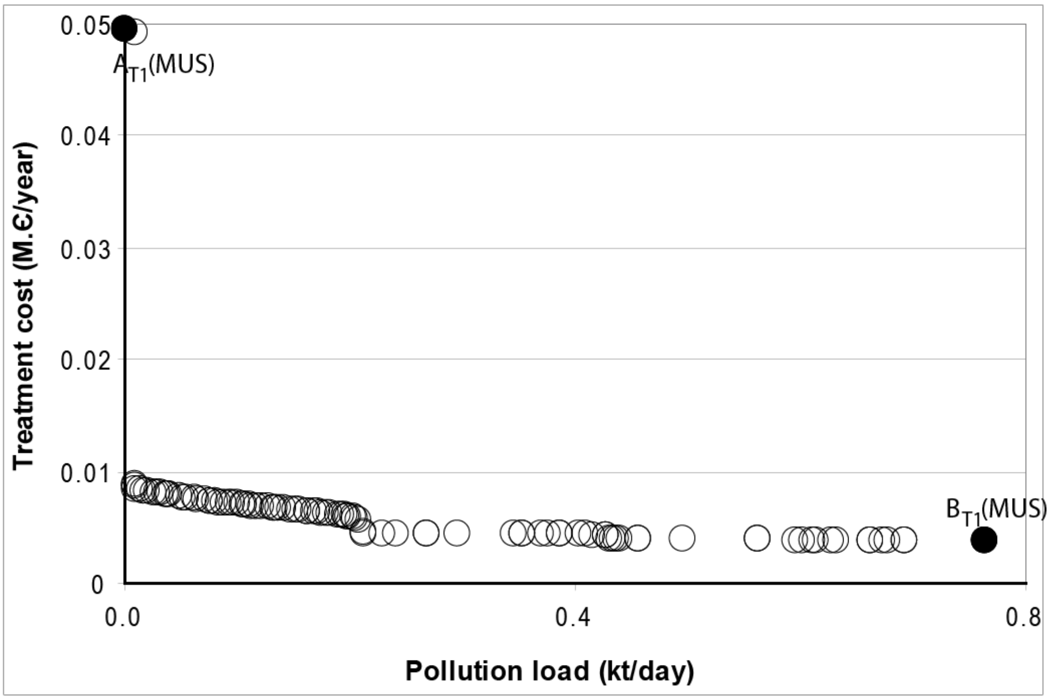

Figure 7.47 Best Pareto optimal front for T=15 minutes from NSGA II approach

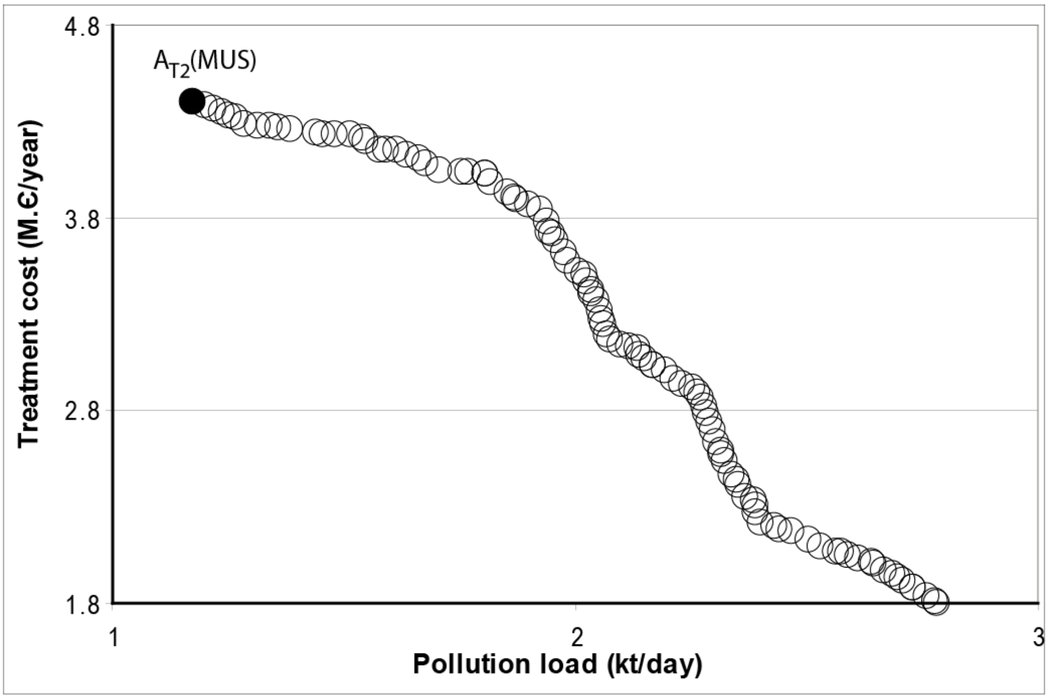

Figure 7.48 Pareto optimal front at T=30 minutes for minimum pollution load approach

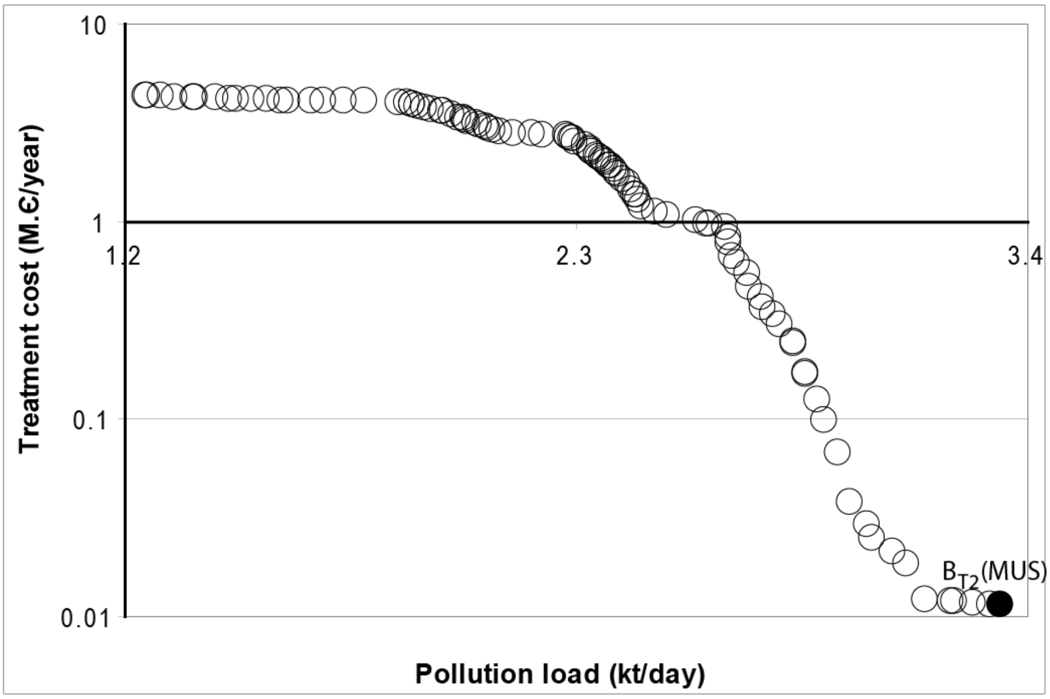

Figure 7.49 Pareto optimal front at T=30 minutes for minimum treatment cost approach

Tables 7.28 and 7.29 give the dynamic control settings over the total storm period for the minimum pollution load and minimum treatment cost solutions. Hydraulic simulation results obtained using these orifice openings are given in the Appendix E.

Table 7.28 Orifice openings for minimum pollution load solution under NSGA II constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	25.91	0.02	8.64	0.94	0	0	0
15-30	21.86	0.34	16.47	16.53	0.03	9.9	8.8
30-45	36.23	4.82	0.28	13.99	12.63	0.14	3.24
45-60	41.96	0.18	0.36	0.01	0	5.83	25.32
60-75	40.73	0.26	0.04	0.02	0.02	16.8	13.74
75-90	24.82	0.01	8.41	0.01	0.01	17.23	44.61
90-105	27.09	0.02	8.94	0	18.51	4.84	0.06
105-120	19.03	1.65	60.42	0.03	12.81	6.17	2.91
120-135	27.96	0.02	10.22	0.06	19.66	5.36	0.18
135-150	39.69	0.55	3.14	0.25	22.82	0.93	0.09
150-165	18.49	0.02	61.59	0.16	14.43	6.2	0.07
165-180	29.73	0.24	10.49	0.46	22.72	2.29	0.18

Table 7.29 Orifice openings for minimum treatment cost solution under NSGA II constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	5.11	0.05	0	0	0	0	0
15-30	0.03	0	0	0	0	0	0
30-45	0	0	0	0	0	0	0
45-60	0	0	0	0	0	0	0
60-75	0.01	0	0	0	0	0	0
75-90	0.01	0	0	0	0	0	0
90-105	0.63	0	0	0	0	0	0
105-120	0.05	0	0	0	0	0	0
120-135	0	0.01	0	0	0	0	0
135-150	0.03	0	0	0	0	0	0
150-165	0.09	0	0	0	0	0	0
165-180	0.01	0	0	0	0	0	0

7.5.10 Optimization results for migrating upstream storms using SWMM constraint handling approach

Sensitivity analysis shows that the mutation probability of 0.3 gives best Pareto optimal front under SWMM constraint handling approach. Figure 7.50 presents some of the mutation probability tested. Figure 7.51 shows the best Pareto optimal front obtained from the sensitivity analysis. Solutions U_{T1}(MUS) and V_{T1}(MUS) are the minimum pollution load and minimum treatment cost solutions selected for dynamic optimization respectively. Figures 7.52 and 7.53 are couple of Pareto optimal fronts from the dynamic optimization at 30 minutes for both minimum pollution load and minimum treatment cost approaches.

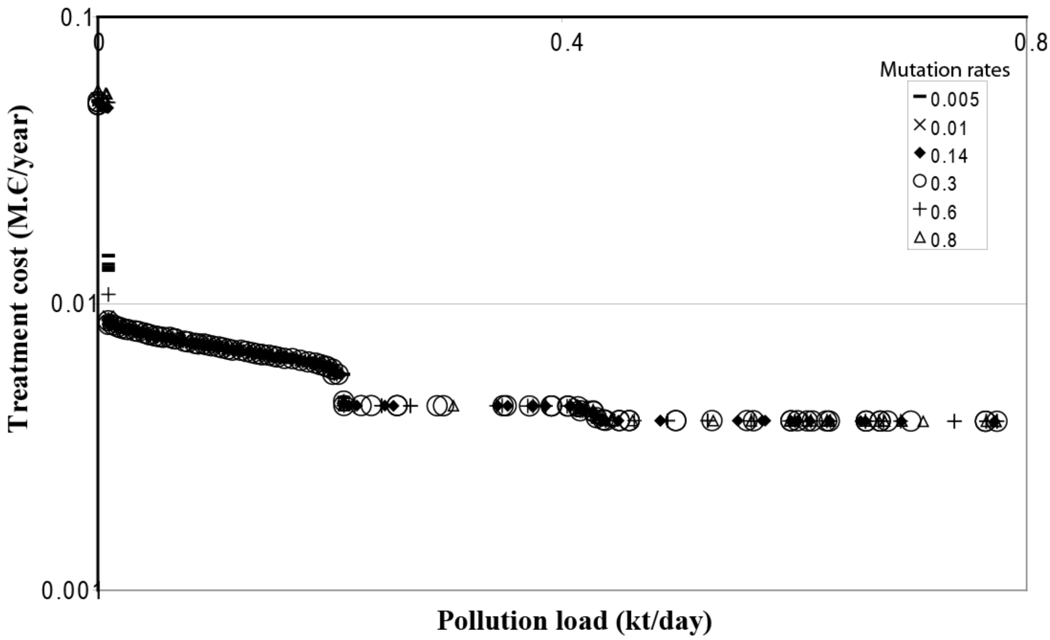

Figure 7.50 Pareto optimal fronts for different mutation rates for SWMM approach for 15 minutes

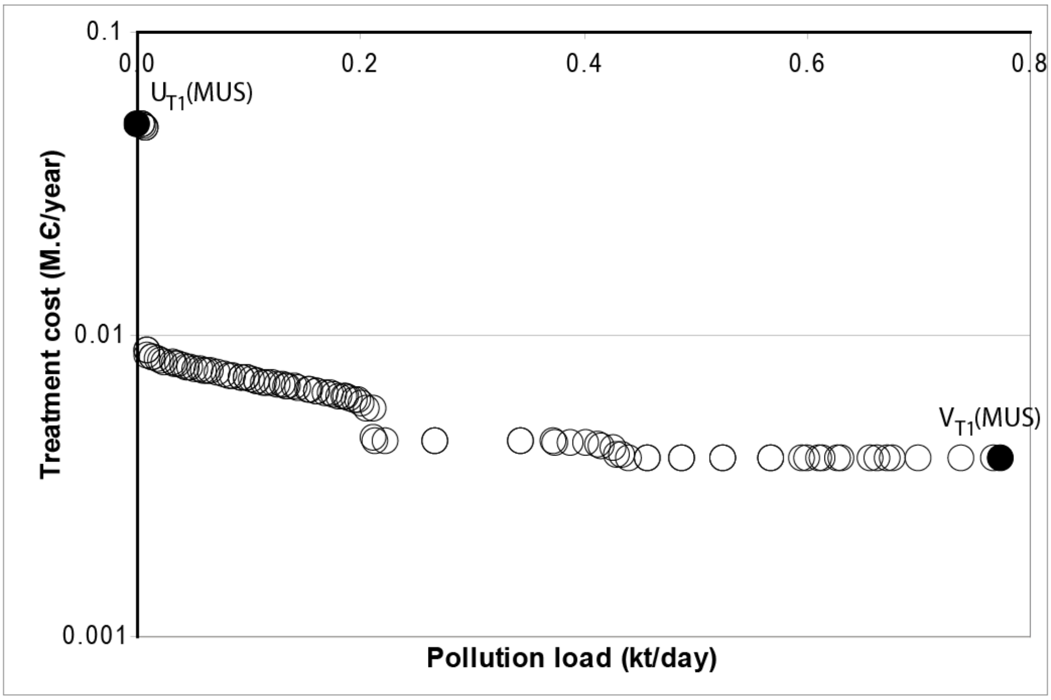

Figure 7.51 Best Pareto optimal fronts at 15 minutes

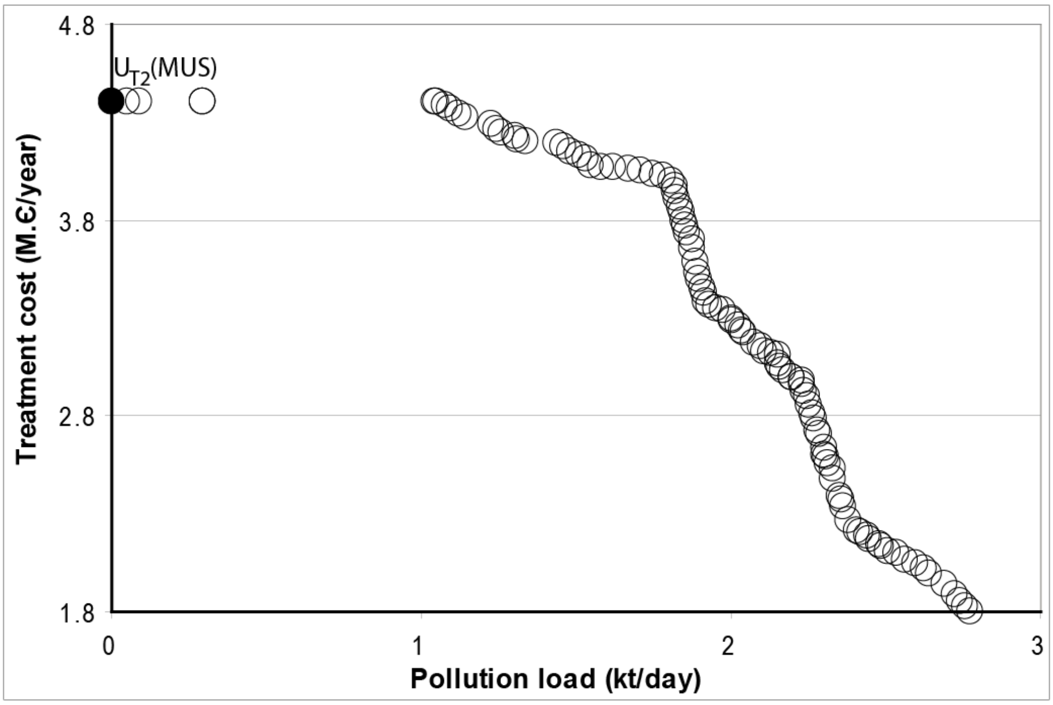

Figure 7.52 Pareto optimal front at T=30 minutes for minimum pollution load approach

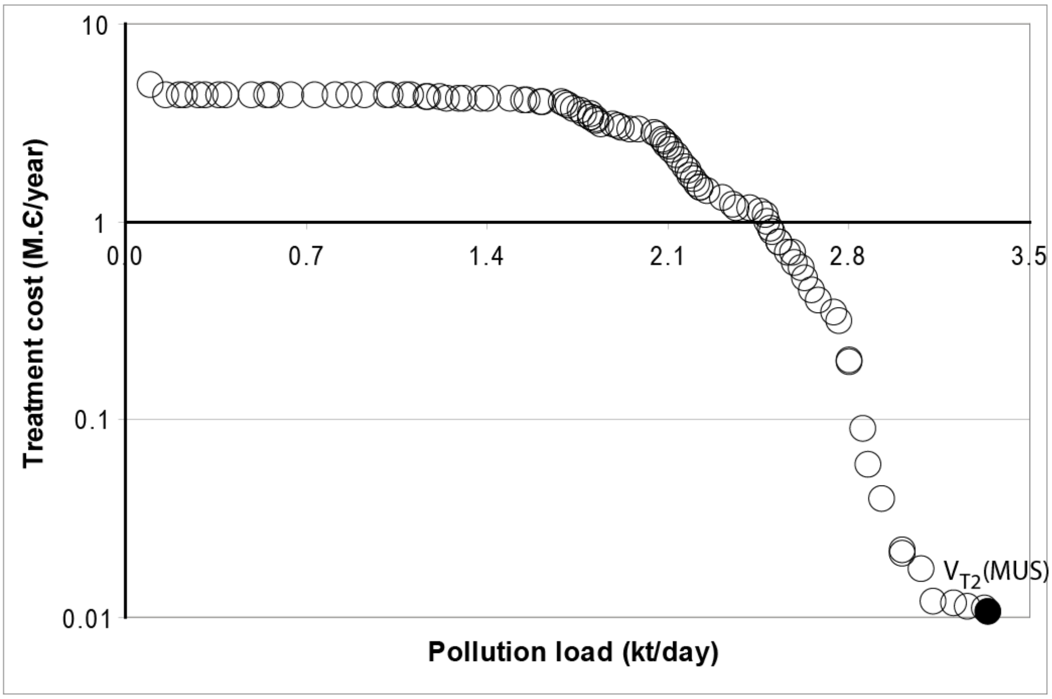

Figure 7.53 Pareto optimal front at T=30 minutes for minimum treatment cost approach

Tables 7.30 and 7.31 present the orifice openings for the minimum pollution load and minimum treatment cost solutions respectively. Results from the hydraulic simulations carried out using these orifice openings are given in the Appendix E.

Table 7.30 Orifice openings for minimum pollution load solution under SWMM constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	25.9	0	8.7	0.9	0	0	0
15-30	77.8	48.2	20.3	32	32	16.6	10.3
30-45	142	21.9	62.5	53.8	100.8	29	42.3
45-60	141.3	62.5	62.5	62.5	145	62.5	62.5
60-75	145	58.6	62.5	62.5	145	62.5	62.4
75-90	145	62.2	38.5	62.5	134.6	45.5	27.9
90-105	145	57.5	61.7	62.5	95	0.4	56.2
105-120	141	62.3	46.2	62.5	139.3	60.8	61.3
120-135	129.6	62.2	22.4	62.4	137	46.1	56.6
135-150	53.4	11.1	55.9	59	24.3	9.9	50
150-165	129.6	62.2	22.4	62.4	137	46.1	56.6
165-180	53.4	11.1	55.9	59	24.3	9.9	50

Table 7.31 Orifice openings for minimum treatment cost solution under SWMM constraint handling approach

Time steps (min)	Orifice openings (cm)						
	O1	O2	O3	O4	O5	O6	O7
0-15	4.12	0	0	0	0	0	0
15-30	0.07	0	0	0	0	0	0
30-45	0.59	0.03	0.01	0	0	0	0
45-60	0.23	0	0	0	0	0	0
60-75	0.06	0.01	0	0	0	0	0
75-90	0.15	0	0	0	0	0	0
90-105	0.31	0	0.01	0	0	0	0
105-120	0.63	0.06	0	0	0	0	0
120-135	0.10	0.03	0	0	0	0	0
135-150	0.09	0	0	0	0	0	0
150-165	0.08	0.01	0	0	0	0	0
165-180	0.61	0.01	0	0	0	0	0

Tables 7.32 and 7.33 present the minimum pollution load and minimum treatment cost solutions from both constraints handling approaches respectively. SWMM

constraint handling outperforms the NSGA II approach in obtaining minimum pollution load solutions. This can be clearly seen from the Table 7.32 and Figure 7.54. However, the corresponding treatment costs at particular time steps for both constraint handling approaches are virtually the same. This finding is graphically visualized in the Figure 7.55. However, both NSGA II and SWMM constraint handling approaches produce almost similar solutions for minimum treatment cost approach. Table 7.33 and Figures 7.56 and 7.57 present these findings.

Table 7.32 Minimum pollution load solutions for migrating upstream storms

Time (hr:min:sec)	NSGA II approach		SWMM 5.0 approach	
	Pollution load (kt/day)	Corresponding Cost (M.Euro/year)	Pollution load (kt/day)	Corresponding Cost (M.Euro/year)
00:15:00	0	0.049468	0	0.049388
00:30:00	1.172942	4.39646	0	4.407687
00:45:00	4.323471	8.744026	0.079881	8.761368
01:00:00	8.419285	13.097483	1.203331	13.115049
01:15:00	11.828638	17.436822	2.031405	17.46873
01:30:00	14.433578	21.731692	2.531862	21.822412
01:45:00	15.944289	26.082772	2.559878	26.176092
02:00:00	16.964994	30.429664	2.559878	30.529772
02:15:00	17.43175	34.764892	2.559878	34.883456
02:30:00	18.038516	39.10372	2.559878	39.237136
02:45:00	18.622194	43.453472	2.559878	43.590816
03:00:00	19.104036	47.797588	2.559878	47.944496

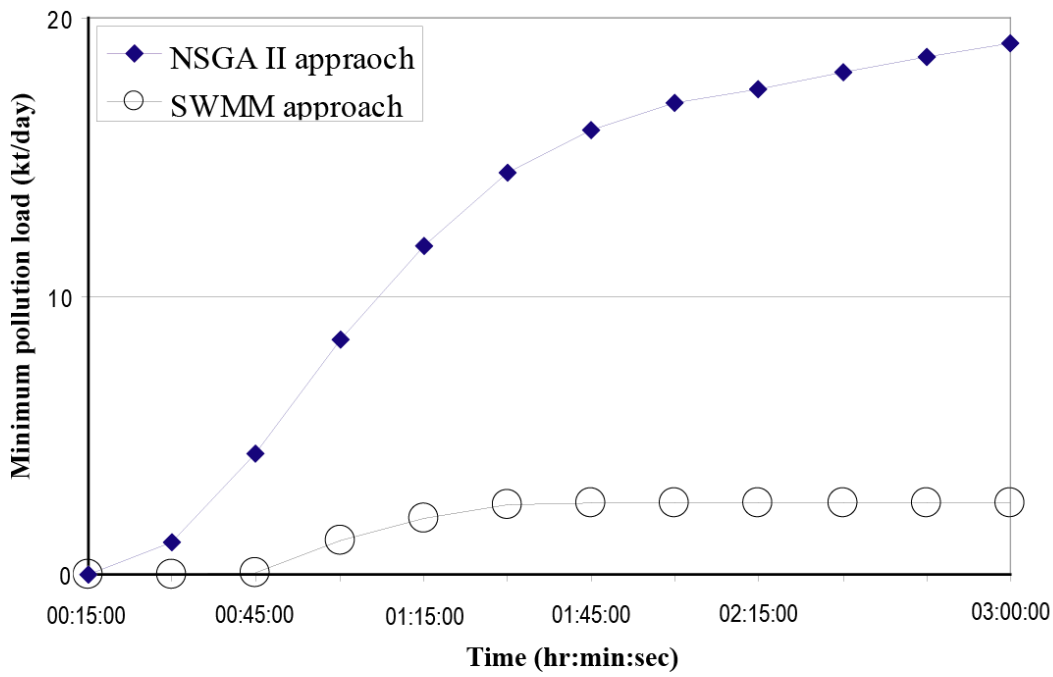

Figure 7.54 Minimum pollution loads at different time steps for migrating upstream storms

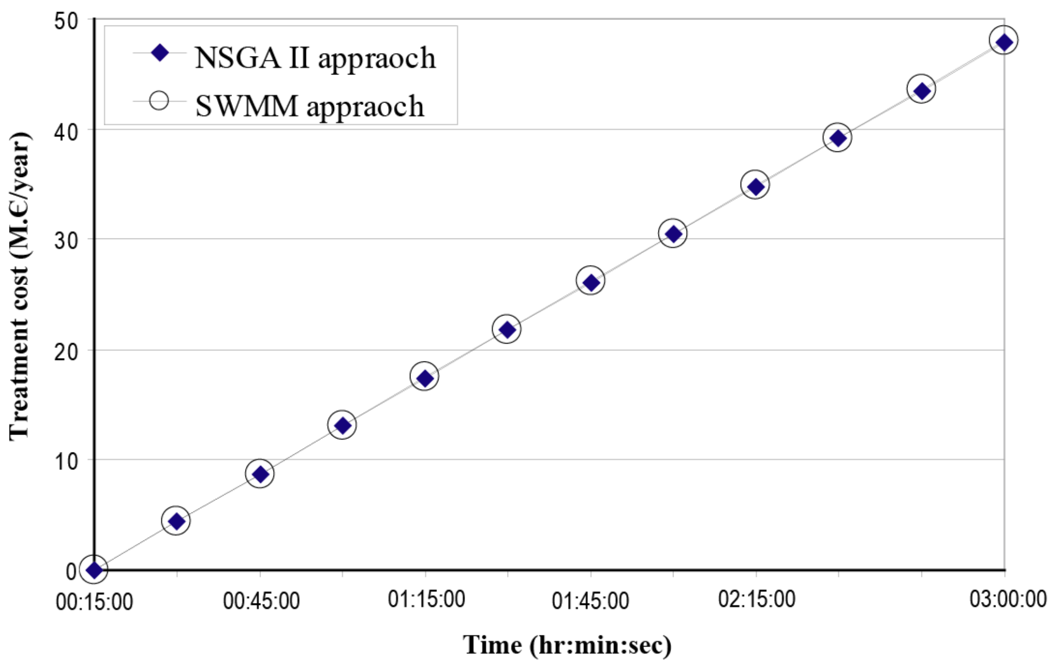

Figure 7.55 Corresponding treatment cost at different time steps

Table 7.33 Minimum treatment cost solutions for migrating upstream storms

Time (hr:min:sec)	NSGA II approach		SWMM 5.0 approach	
	Cost (M.Euro/year)	Corresponding Pollution load (kt/day)	Cost (M.Euro/year)	Corresponding Pollution load (kt/day)
00:15:00	0.003882	0.76355	0.003881	0.773717
00:30:00	0.01156	3.333873	0.010714	3.338644
00:45:00	0.161166	7.42299	0.094259	7.421005
01:00:00	0.48767	12.13121	0.36065	12.127734
01:15:00	0.759739	16.053975	0.64576	16.053535
01:30:00	0.96361	18.956008	0.885494	19.07074
01:45:00	1.121194	21.003236	1.079296	21.2104
02:00:00	1.248948	22.660942	1.24263	23.004748
02:15:00	1.358005	23.991974	1.39105	24.497042
02:30:00	1.465346	25.55568	1.541773	26.285008
02:45:00	1.573311	27.169886	1.701346	28.09163
03:00:00	1.675593	28.823258	1.856641	29.947784

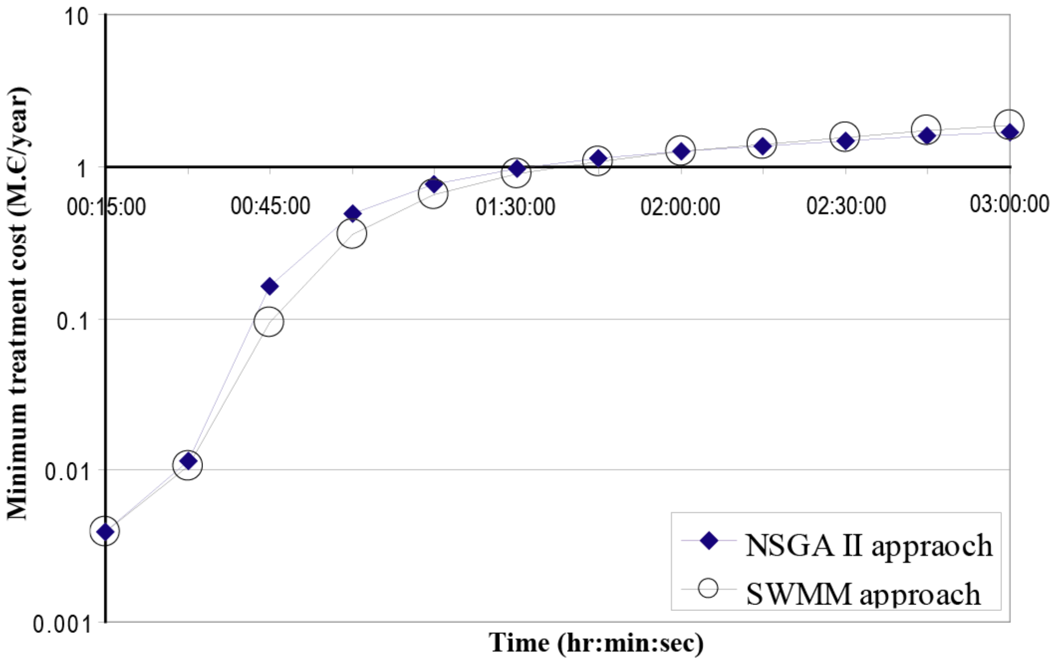

Figure 7.56 Minimum treatment cost at different time steps for migrating upstream storms

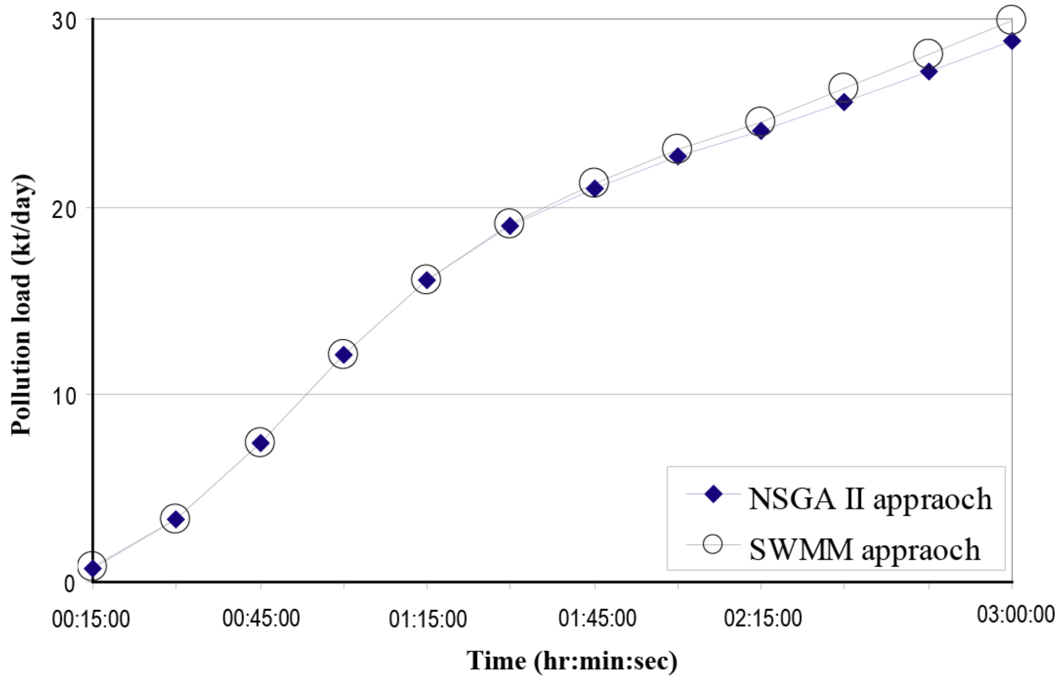

Figure 7.57 Corresponding pollution loads at different time steps

Table 7.34 presents the average CPU simulation times for optimization runs at different time steps. Similar to the other storm conditions, SWMM constraint handling approach requires slightly higher CPU simulation times to obtain the optimal solutions.

Table 7.34 Average simulation times for migrating upstream storms

Time step (hr:min:sec)	Simulation times (hr:min:sec)			
	NSGA II approach		SWMM approach	
	Min Cost	Min Pol	Min Cost	Min Pol
00:15:00	0:07:32		00:07:28	
00:30:00	0:11:34	0:14:09	00:14:01	00:15:28
00:45:00	00:13:08	00:22:37	00:14:23	00:26:04
01:00:00	00:12:33	00:29:40	00:13:34	00:44:23
01:15:00	00:13:09	00:35:57	00:14:57	00:45:14
01:30:00	00:14:47	00:42:40	00:16:16	00:49:11
01:45:00	00:15:41	00:48:52	00:17:09	00:49:33
02:00:00	00:16:17	00:54:31	00:18:06	00:51:35
02:15:00	00:17:18	01:01:29	00:19:35	01:06:25
02:30:00	00:19:21	01:10:48	00:19:31	01:16:39
02:45:00	00:18:37	01:16:02	00:20:17	01:23:33
03:00:00	00:19:46	01:22:56	00:21:09	01:28:45

7.5.11 Identified drawback in dynamic optimization

Some of the hydraulic results obtained at a particular time step cannot be fed as the input data for the next time step simulation. Therefore, the hydraulic analysis has to be started from the beginning of the storm in each time step for the dynamic optimization. The following example shows the drawback in detail.

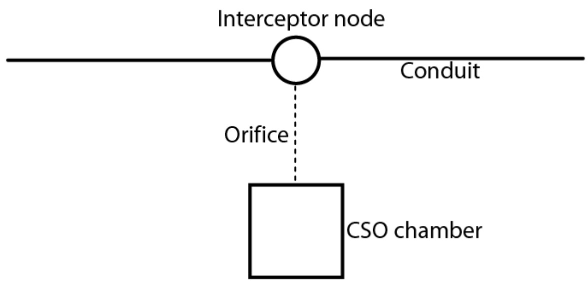

Figure 7.58 Schematic diagram of an interceptor node

Figure 7.58 shows the schematics of an interceptor node. Flow rate, flow depth, velocity, Froude number, capacity of conduits and the concentrations of the constituents are the hydraulic and water quality results, which can be obtained for a conduit after a hydraulic simulation. Wastewater depth, head, volume, lateral inflows, total inflows, flooding and concentrations of constituents are the hydraulic results for a node after a simulation.

It is assumed herein that the above stated results are obtained at 15 minutes for the 0-15 minutes hydraulic simulation. In case, if someone needs to run the hydraulic model for 15-30 minutes, the results obtained at 15 minutes should be fed to the system as input data, so that the conservation of mass and momentum can be satisfied. However, SWMM 5.0 is incapable of setting the water quality concentrations in both nodes and links at given time steps. This is a major drawback in dynamic optimization. Therefore, to satisfy the continuity, the hydraulic model should be run for 0-30 minutes with the basic inputs, instead of 15-30 minutes.

7.5.12 Hydraulic verification of dynamic optimization

Table 7.35, 7.36 and 7.37 give some of the hydraulic simulation results for minimum treatment cost solutions at 2 hours & 15 minutes, 2 hours and 1 hour. These are for the single storm condition under the SWMM constraint handling approach. These three tables clearly show that the different time step simulations produce same results. Therefore, the developed dynamic multi-objective optimization approach can be hydraulically verified.

Table 7.35 Flow rates through C1 and C3 conduits at different time steps optimization simulations

Time steps (hr:min:sec)	C1 Flow rate (m3/s)			C3 Flow rate (m3/s)		
	02:15:00 simulation	02:00:00 simulation	01:00:00 simulation	02:15:00 simulation	02:00:00 simulation	01:00:00 simulation
0:15:00	0.39	0.39	0.39	0	0	0
0:30:00	0.78	0.78	0.78	0.44	0.44	0.44
0:45:00	0.05	0.05	0.05	0.37	0.37	0.37
1:00:00	0.01	0.01	0.01	0.13	0.13	0.13
1:15:00	0.09	0.09		0.08	0.08	
1:30:00	0.02	0.02		0.06	0.06	
1:45:00	0.02	0.02		0.04	0.04	
2:00:00	0.07	0.07		0.03	0.03	
2:15:00	3.14			2.35		

Table 7.36 Wastewater depths of T2 CSO chamber and T9 storage tank at different time steps optimization simulations

Time steps (hr:min:sec)	T2 Water depth (m)			T9 Water depth (m)		
	02:15:00 simulation	02:00:00 simulation	01:00:00 simulation	02:15:00 simulation	02:00:00 simulation	01:00:00 simulation
0:15:00	6.28	6.28	6.28	8.39	8.39	8.39
0:30:00	7.26	7.26	7.26	8.59	8.59	8.59
0:45:00	7.25	7.25	7.25	8.59	8.59	8.59
1:00:00	7.19	7.19	7.19	8.59	8.59	8.59
1:15:00	7.09	7.09		8.59	8.59	
1:30:00	7.05	7.05		8.59	8.59	
1:45:00	7.05	7.05		8.59	8.59	
2:00:00	7.05	7.05		8.59	8.59	
2:15:00	7.05			8.59		

Table 7.37 Combined sewer overflow rates at T3 and T4 CSO chambers at different time steps optimization simulations

Time steps (hr:min:sec)	T3 Overflows (m3/s)			T4 Overflows (m3/s)		
	02:15:00 simulation	02:00:00 simulation	01:00:00 simulation	02:15:00 simulation	02:00:00 simulation	01:00:00 simulation
0:15:00	4.76	4.76	4.76	4.33	4.33	4.33
0:30:00	7.91	7.91	7.91	9.51	9.51	9.51
0:45:00	7.29	7.29	7.29	16.58	16.58	16.58
1:00:00	4.56	4.56	4.56	16.58	16.58	16.58
1:15:00	2.67	2.67		16.58	16.58	
1:30:00	1.27	1.27		9.44	9.44	
1:45:00	1.26	1.26		4.2	4.2	
2:00:00	1.23	1.23		1.63	1.63	
2:15:00	1.2			1.63		

7.6 Summary and conclusions

In this chapter, the formulation of a dynamic optimization approach for combined sewer systems, which is novel, has been presented. The multi-objective optimization approach was developed, considering the pollution load to the receiving water from CSOs and the wastewater treatment cost. The developed optimization model gives the dynamic optimal control settings varying along the full duration of the storm.

The developed multi-objective optimization algorithm has been applied to a simple interceptor sewer network under several storm conditions. Irrespective of the types of the storms, the results overall demonstrate the benefits of the multi-objective optimization approach. Developed multi-objective optimization approach produces better results with higher mutation rates.

Comparison results for two different constraint handling approaches show that the SWMM constraint handling approach outperforms the NSGA II constraint handling approach for the minimum pollution load solutions. However, NSGA II constraint handling approach produces better solutions for the minimum wastewater treatment cost over the SWMM approach. Nevertheless, the differences in minimum treatment cost solutions from both constraint handling approaches are not significant, when it compared against the difference in minimum pollution load solutions. In addition, CPU simulation times are in the same orders, but with slight increments in SWMM constraint handling approach. Therefore, it can be concluded herein that the SWMM constraint handling approach outperform the NSGA II constraint handling approach at minimum pollution load solutions.

Hydraulic simulation results further indicate that the developed dynamic multi-objective optimization approach produces feasible solutions. In addition, the hydraulic verification results denote the dynamic approach developed in the multi-objective optimization process produces accurate results.

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Introduction

Urban wastewater systems are complex systems. Rapid urbanization, population growth and climate change have increased the complexity of the urban wastewater systems. Therefore, separate sewer systems for storm water and wastewater are encouraged in most of the developed countries. However, apart from some cities and few counties, most of the other cities have combined sewer systems. High construction cost in making these combined sewer system separated and the disruptions to the inhabitants have forced to search some alternative methods to deal the problems in combined sewer systems.

Reducing storm runoff to the combined sewer system is one option. Introduction of sustainable urban drainage systems (SUDS) is an example for reducing storm runoff on-site. Efficient control of exiting combined sewer systems is another alternative. Finding the optimal control strategies to the existing combined sewer systems was the main aim of this research. Chapter 1 has introduced the problems in combined sewer systems. It is well understood that the CSOs into receiving waters has significant environmental concerns.

Because of the complexity of combined sewers, usage of optimization approaches cannot be given up. Genetic algorithms have been proven to be well appropriated in solving complex multi-objective optimization problems. Therefore, a multi-objective optimization approach was proposed in searching the optimal controlling strategies of combined sewer systems. The developed optimization approach considers the flows and water quality in combined sewers and the economic aspects of wastewater treatment.

8.2 Summary and conclusions

Receiving water quality due to the CSOs is a concerned topic. However, a comprehensive analysis was not implemented in the literature. Therefore, the research work presented in Chapter 5 was carried out to accomplish the identified gap. The originality of the research work presented in Chapter 5 lies along the lines of receiving water quality due to CSOs. Pollutographs for important water quality constituents were developed. Temporal and spatial variations were incorporated in generating these pollutographs.

Chapter 6 presents the developed multi-objective approach in controlling combined sewer networks. It was developed, considering the pollution load to the receiving water from CSOs and the wastewater treatment cost. This is the novelty of the research work presented in Chapter 6. The proposed model gives the optimal CSO control settings where a single set of static control settings is used throughout the considered time period.

Case study results conclude that both constraints handling approaches produce feasible solutions. However, the SWMM constraint handling approach outperforms the NSGA II constraint handling approach in producing minimum pollution load solutions. In this approach, no constraints were handled inside the NSGA II, which has the benefit of obtaining all feasible solutions from the optimization runs.

However, this model gives the optimal CSO control settings where a single set of static control settings is used throughout the considered time period. Therefore, this approach can be practiced if the storm conditions are known ahead of the controlling. Given that the climate predictions are accurate enough, this snapshot optimization may be a good solution in controlling combined sewer systems.

Snapshot optimization was then implemented to the dynamic optimization approach and presented in Chapter 7. Dynamic optimization for control sewer systems is novel. Unlike a set of static control settings from snapshot optimization, a dynamic set of control settings at considered time intervals can be obtained from this approach.

Case study results conclude that the dynamic optimization approach produces feasible results from both constraint handling approaches. However, SWMM constraint handling outperforms the NSGA II approach for the minimum pollution load solutions. Nevertheless, NSGA II approach produces better results for the minimum wastewater treatment cost solutions. Therefore, unlike the snapshot optimization approach, it is difficult to conclude which constraint handling approach is better than the other. Depending on the aspirations of the sewer controllers, the better approach should be selected.

Nevertheless the larger CPU times for simulations, this approach can be fit to the real time controls in combined sewer systems. Improvements, which can be implemented in reducing the simulation times are, discussed in the future work section. However, given that, the technology is there to measure water quality parameters and flows and then to send feedback to a control location, this dynamic optimization approach is a good solution in RTC of combined sewer systems.

Sensitivity analysis of mutation rates given in Chapter 6 and 7 show that the higher mutation probabilities are beneficial for the developed multi-objective optimization approaches. This observation can be seen irrespective of the type of the storm. Therefore, this proves that the necessity of the calibration of the genetic algorithm. Furthermore, it is evident that the developed multi-objective optimization approaches converge to optimal solutions before the 100^{th} generation. Therefore, the number of simulation runs carried out in this thesis work is comparably enough for obtaining the optimal solutions. However, the population size can be effective in producing results. Larger population sizes often produces better solutions and better diversity in solutions. Population size used throughout this thesis was 100. This was based on the literature. Though it is time-consuming for larger population sizes, it is advisable to have test simulation runs for different population sizes and then to provide a detailed sensitivity analysis of the population size similar to the mutation probability. This will enhance the calibration of the genetic algorithm. However, due to the available time, population size calibration was not carried out in this research.

However, overall, the results in Chapters 6 and 7 have illustrated that the benefits of using optimal controlling strategies in combined sewer systems. Further research should be carried out to find a universal solution with real-time control, but this is an initiation.

8.3 Limitations of the work

The main limitation of both optimization approaches is the large CPU time when it comes to the end of the storms. However, given that the usage of high performance computers (HPCs) and parallel computing in real world problems, larger simulation times should be able to reduce dramatically.

Due to the limitations in the hydraulic model, SWMM 5.0 the dynamic optimization approach had to be run from the beginning of the storm to the considered time interval in each time. This has increased the simulation times to obtain the optimal solutions.

In addition, SWMM doesn’t have functions to call a specific input variable to the NSGA II optimization module and then to assign a decision variable to that particular variable. Therefore, it is necessary to generate the whole input file of the corresponding sewer example in the NSGA II optimization module and then to assign the decision variables. This requires a considerable amount of simulation time to run an optimization run.

The ultimate goal of this research work is to apply the optimization model in much bigger sewer networks in real-time control. Given that, there is enough speed in obtaining solutions (with the aid of HPC and parallel computing) the model should be able to apply in real-time control. However, in order to apply the model in RTC, there should be enough resources in the considered urban sewer networks. This is a concerned problem. Proving high tech equipments to all over the sewer network in measuring and reporting to the central processing unit is not easy. Sensors to measure the water quality parameters and flow rates in CSO locations should be readily available. In addition, the controllers should be readily available at the actuators of the sewer system. Orifice gate openings are the actuators in this research. Therefore, controllers like PID and PLC should be readily available. Furthermore, usage of satellites and mobile network can be used in feedback and again they should be reachable to all the locations of the sewer network. However, all these components in real-time control require a huge capital cost and maintenance cost.

8.4 Future work

Agricultural lands usually carry a considerable amount of phosphorous. It is identified as a nonpoint source pollutant that causes eutrophication in surface waters. Even though, phosphorous is less mobile than nitrogen, soil erosion in agricultural lands leads to increase the phosphorous levels in surface water. Therefore, it is always better to consider phosphorous when considering the receiving water quality due to CSOs. This modified equation can be implemented to the developed multi-objective optimization module with further research.

Dynamic optimization results from two different constraint handling approaches show interesting results. Minimum treatment solutions from these two constraint handling approaches are comparable. However, the minimum pollution load solutions show significant differences. Therefore, it is important to conduct a detailed research on the SWMM constraint handling approach.

In addition, interfacing source codes of SWMM 5.0 can be improved with further research. As it was already stated in the previous section, SWMM 5.0 hasn't got functions to call and assign decision variables to a particular input. Source code improvement are required to generate such functions. This will lead to reduce the simulation time in multi-objective optimization. Therefore, it will be a major advantage, when it comes to the application of real world problems with larger networks.

Furthermore, usage of artificial neural networks (ANN) in controlling sewer networks is a possible direction for future research. ANN is a useful tool in controlling complex problems. They are efficient and require a little knowledge about the complex system. Therefore, a neural network can be trained to the developed optimization problem. Application of artificial neural networks will reduce the CPU time for the simulations. This is a great advantage for the real-time control of combined sewer systems, particularly for large combined sewer networks. Therefore, further research is required in the directions of training a neural network to the problem. Darsono et al. (2007) and Fu et al. (2010a) have showed the application of ANN to the combined sewer systems.

There is enough research evidence to show that the climate change is occurring. There is no argument that the climate change creates major challenges in water resource management. Some areas may have higher precipitations compared to the precipitations that they have had in couple of years back. However, some areas may have less precipitation. Therefore, the research in controlling urban wastewater systems has to be implemented along the lines of climate change. Proposed further research areas stated in above paragraphs, together with improvements of climatic scenarios should also be considered in developing a model, which can lead to obtain improved solutions in controlling combined sewer networks.

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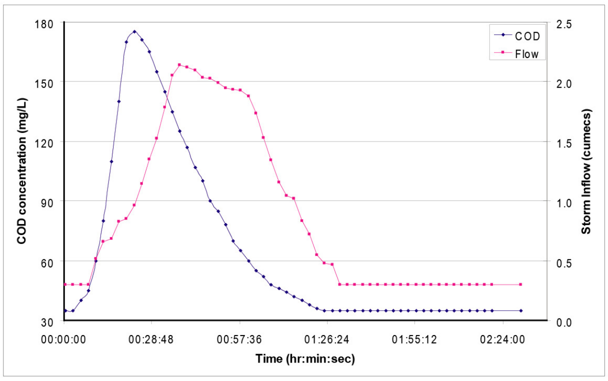
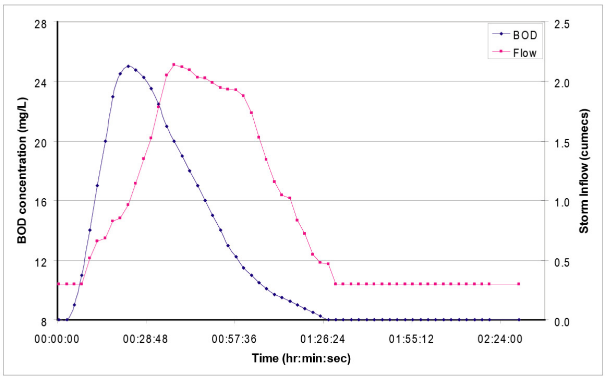
APPENDIX A - POLLUTOGRAPHS

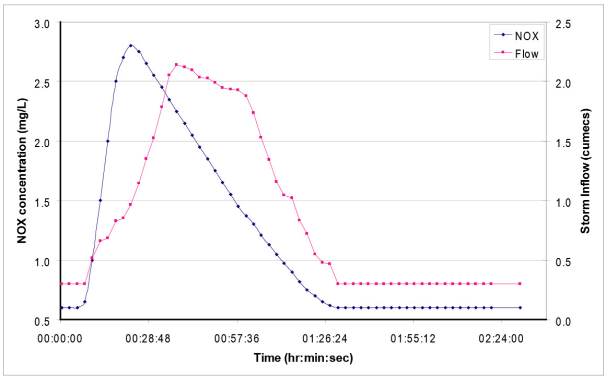
A.1 Pollutographs for single storm



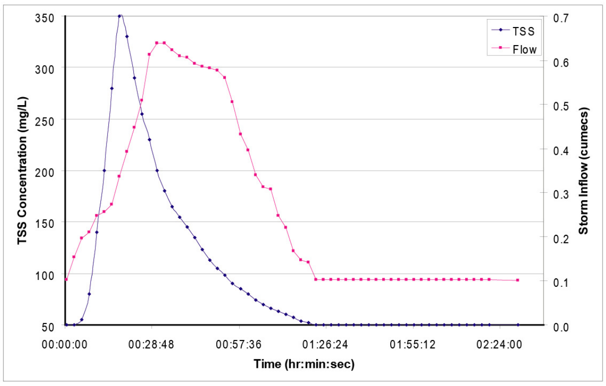
COD pollutograph for Rimrose



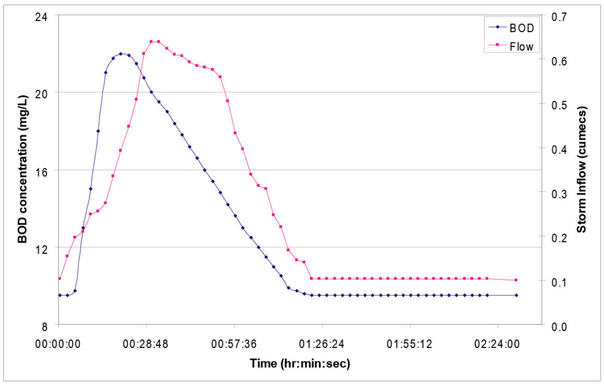
BOD pollutograph for Rimrose



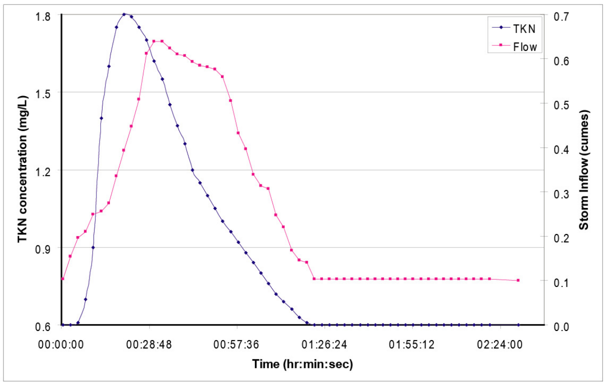
NOX pollutograph for Rimrose



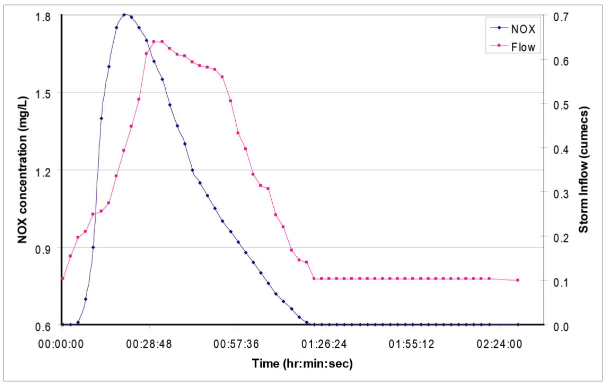
TSS pollutograph for Strand Road



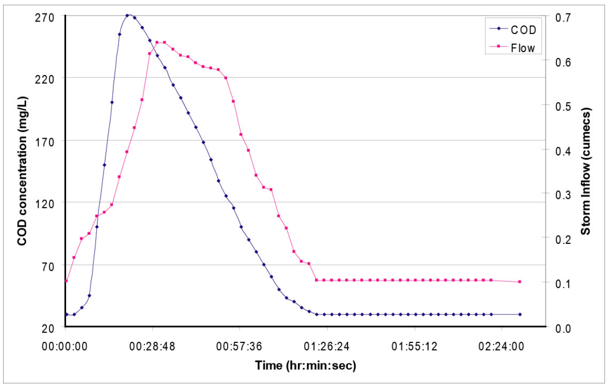
BOD pollutograph for Strand Road



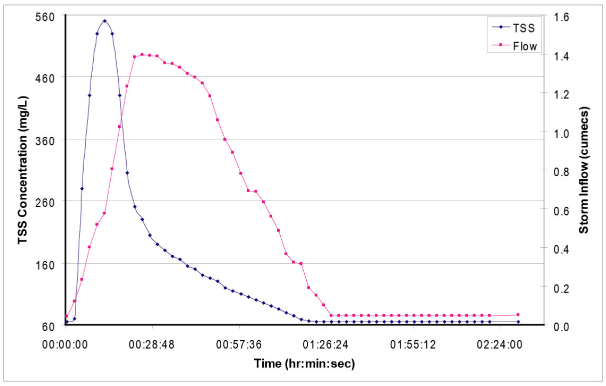
TKN pollutograph for Strand Road



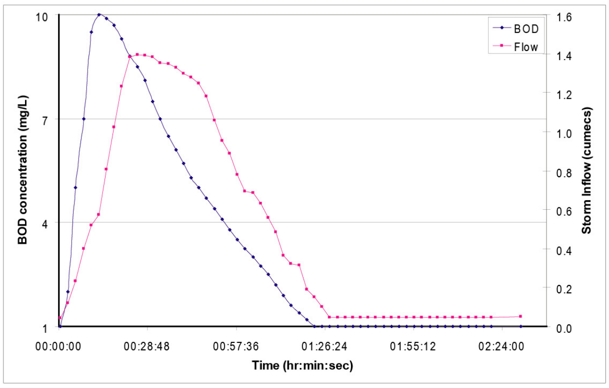
NOX pollutograph for Strand Road



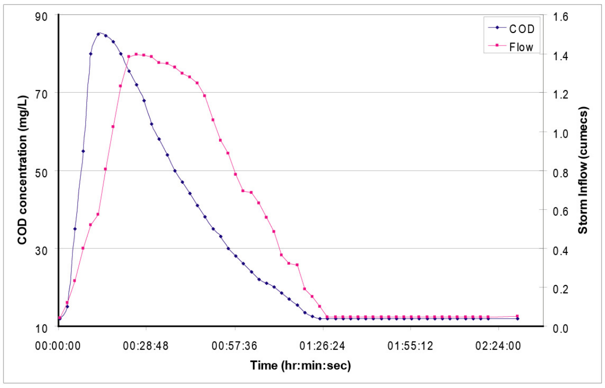
COD pollutograph for Strand Road



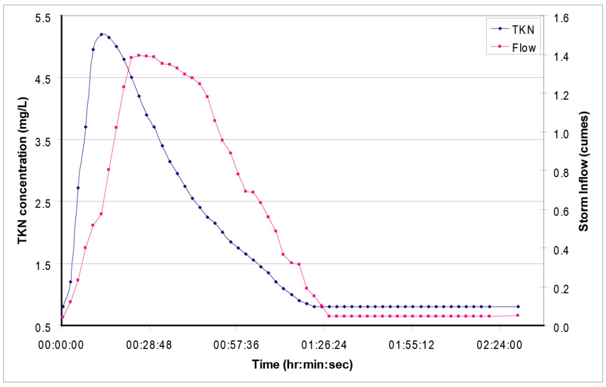
TSS pollutograph for Millers Bridge



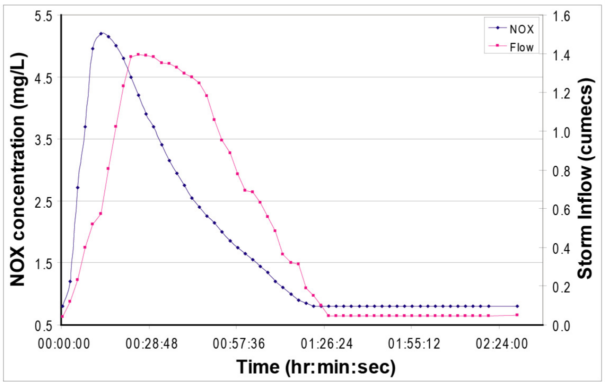
BOD pollutograph for Millers Bridge



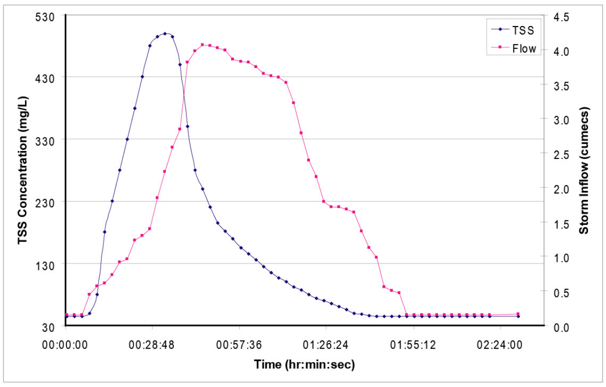
COD pollutograph for Millers Bridge



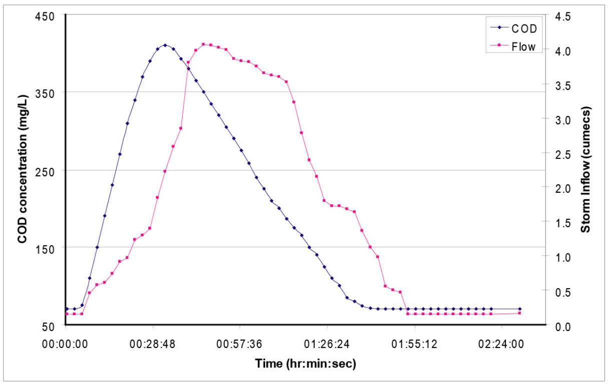
TKN pollutograph for Millers Bridge



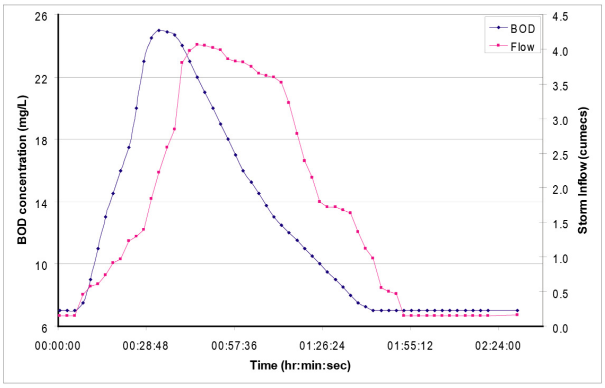
NOX pollutograph for Millers Bridge



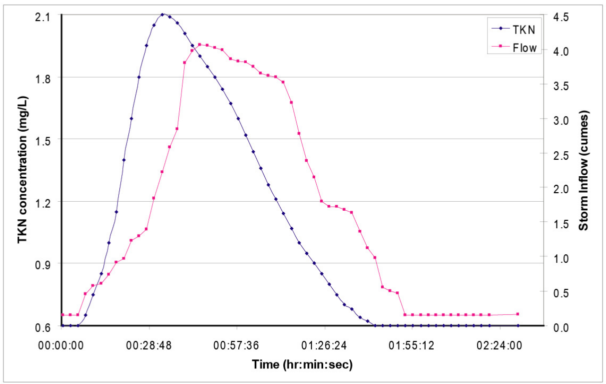
TSS pollutograph for Bankhall Relief



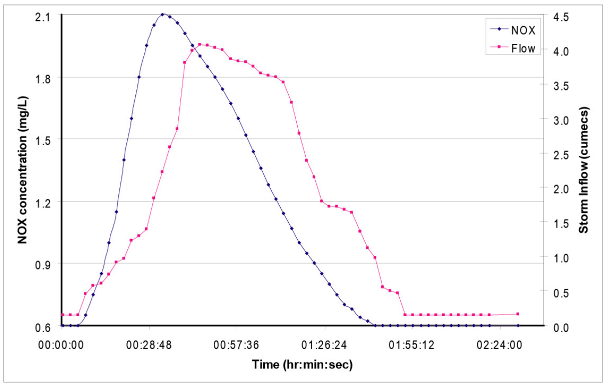
COD pollutograph for Bankhall Relief



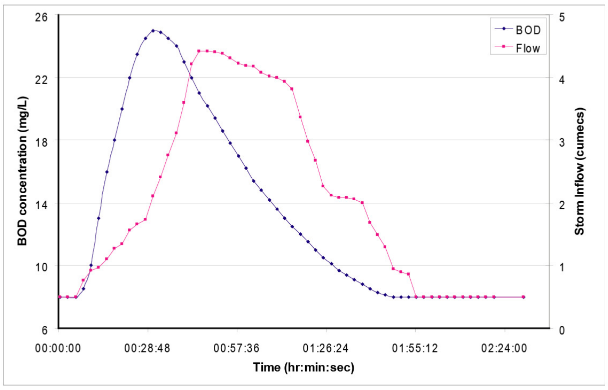
BOD pollutograph for Bankhall Relief



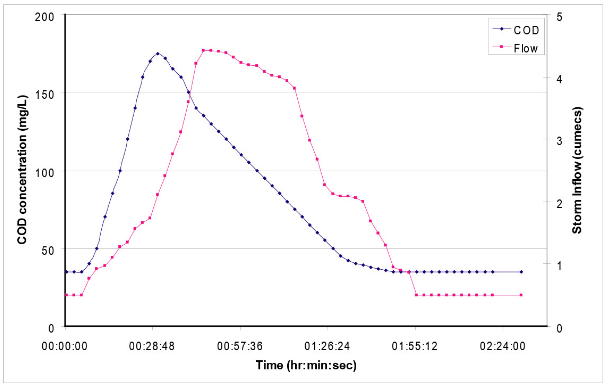
TKN pollutograph for Bankhall Relief



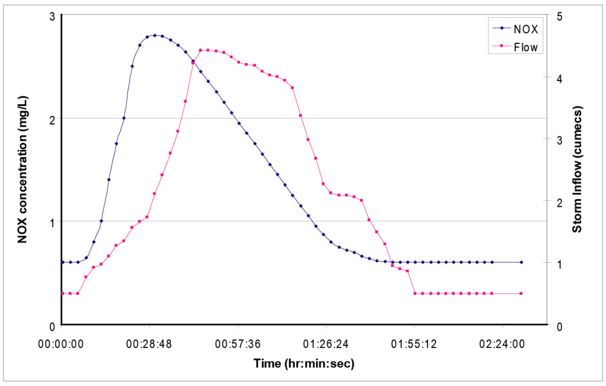
NOX pollutograph for Bankhall Relief



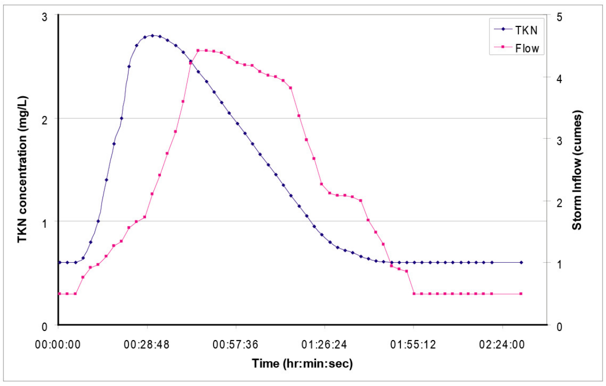
BOD pollutograph for Northern



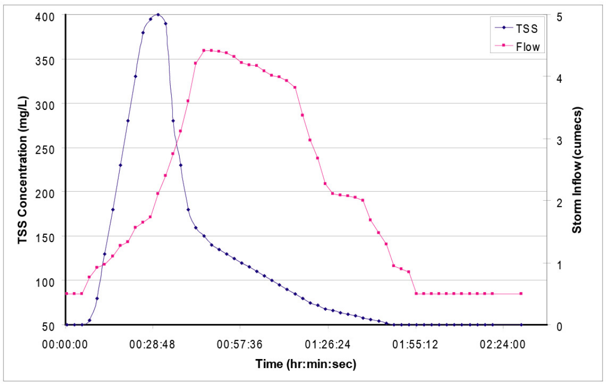
COD pollutograph for Northern



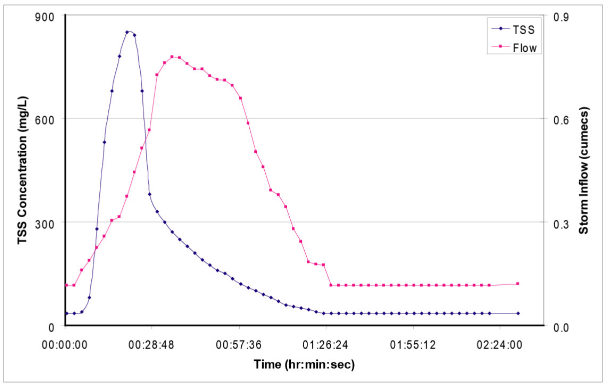
NOX pollutograph for Northern



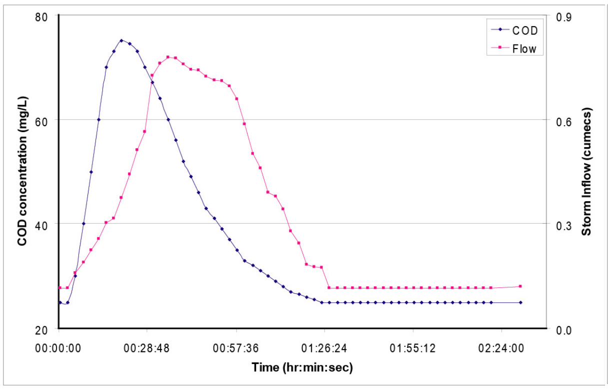
TKN pollutograph for Northern



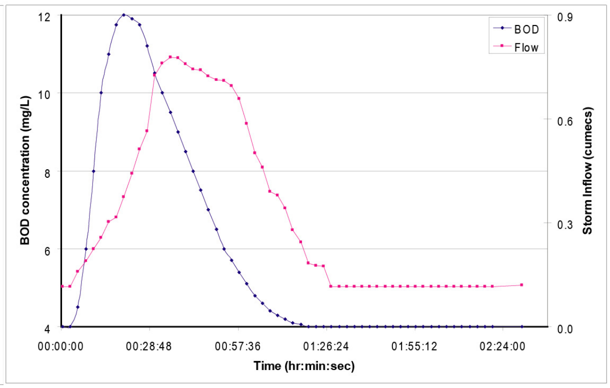
TSS pollutograph for Northern



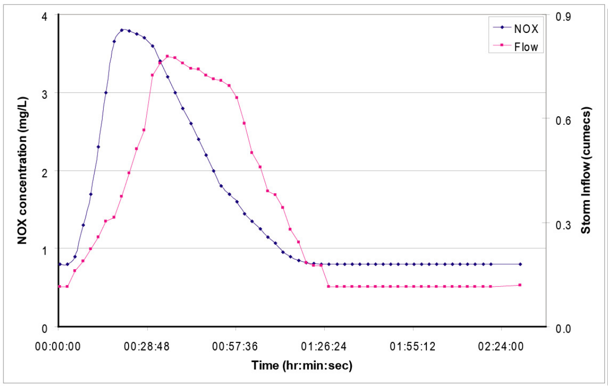
TSS pollutograph for Bankhall



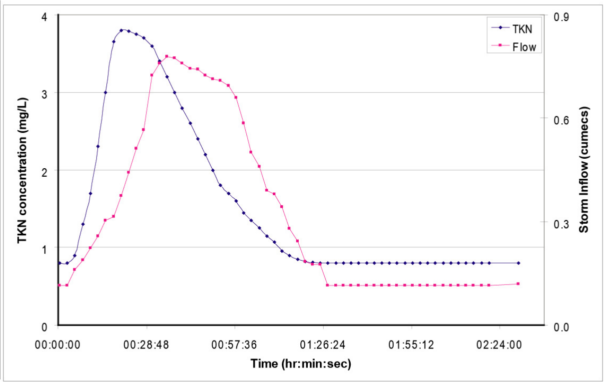
COD pollutograph for Bankhall



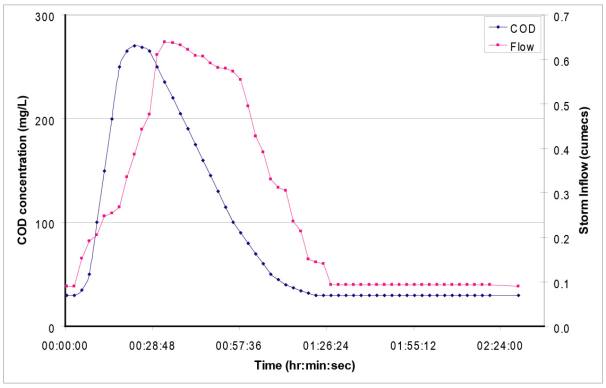
BOD pollutograph for Bankhall



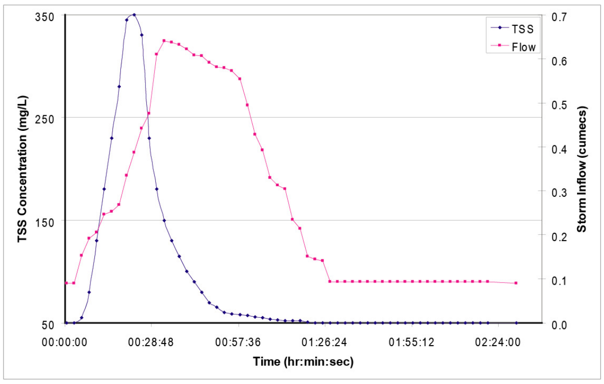
NOX pollutograph for Bankhall



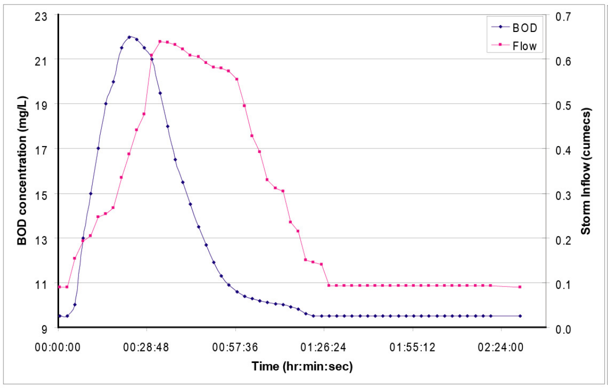
TKN pollutograph for Bankhall



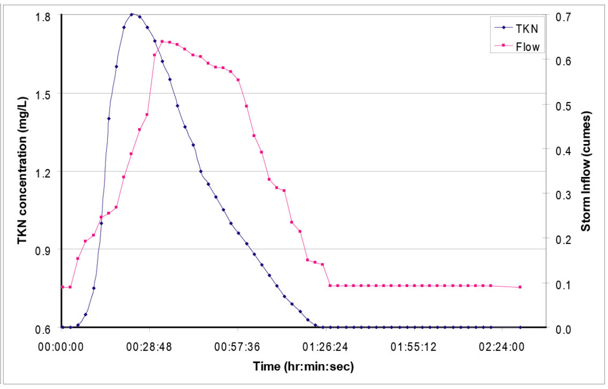
COD pollutograph for Sandhills Lane



TSS pollutograph for Sandhills Lane

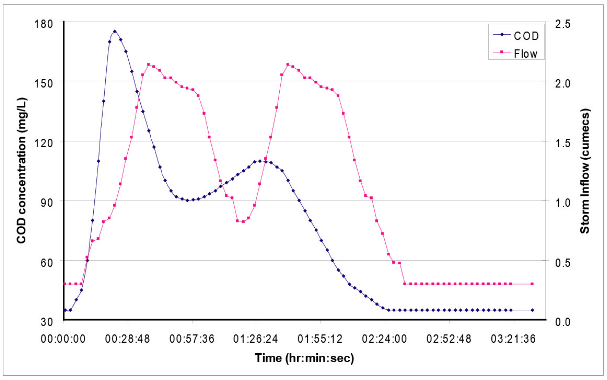


BOD pollutograph for Sandhills Lane

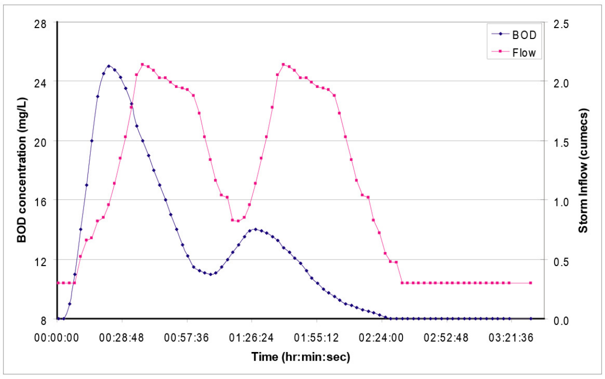


TKN pollutograph for Sandhills Lane

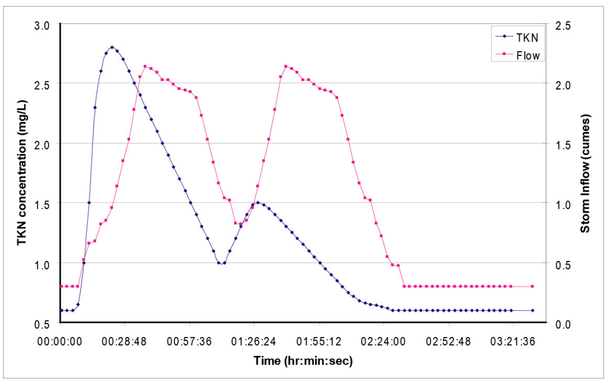
A.2 Pollutographs for two consecutive storms



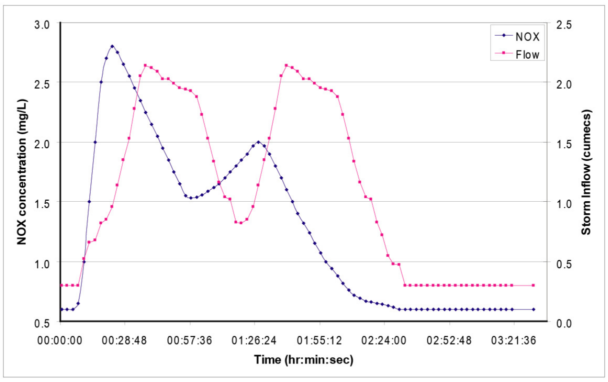
COD pollutograph for Rimrose



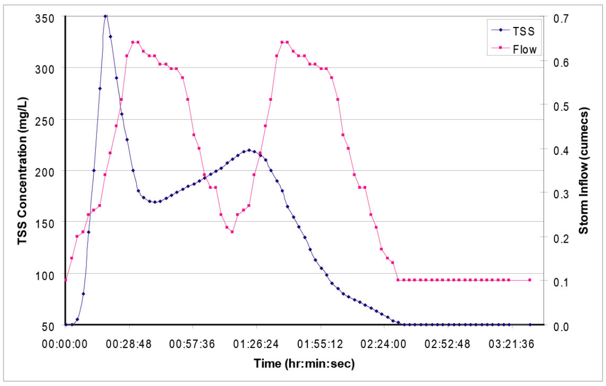
BOD pollutograph for Rimrose



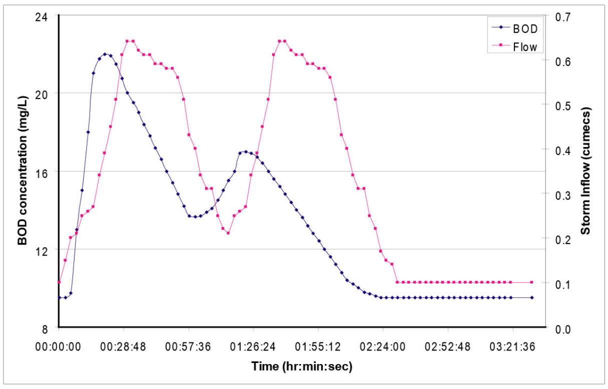
TKN pollutograph for Rimrose



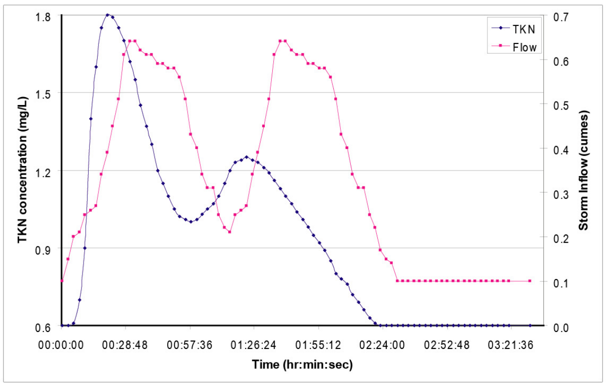
NOX pollutograph for Rimrose



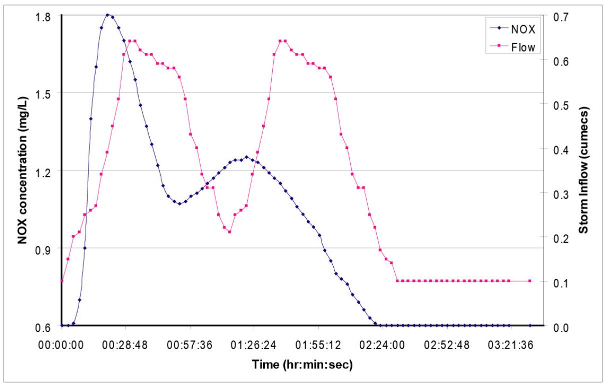
TSS pollutograph for Strand Road



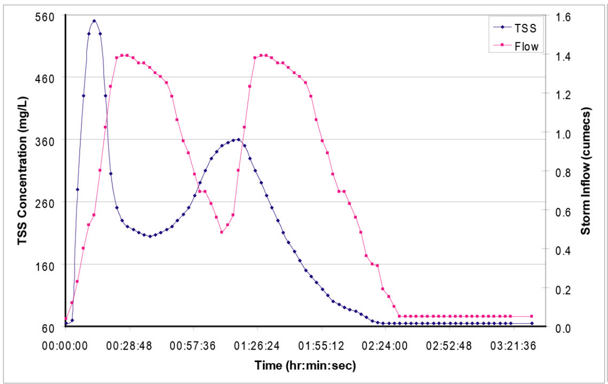
BOD pollutograph for Strand Road



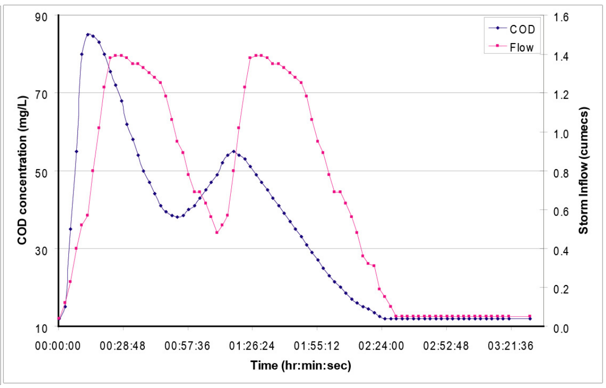
TKN pollutograph for Strand Road



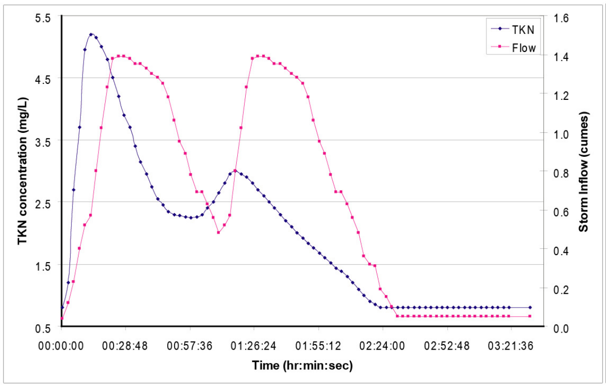
NOX pollutograph for Strand Road



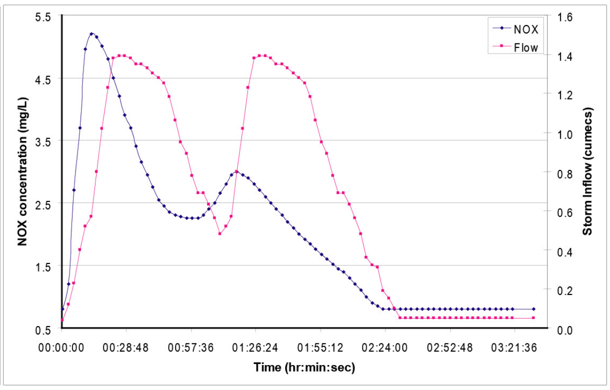
TSS pollutograph for Millers Bridge



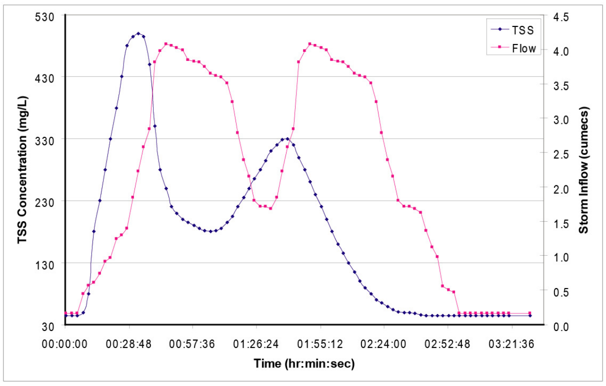
COD pollutograph for Millers Bridge



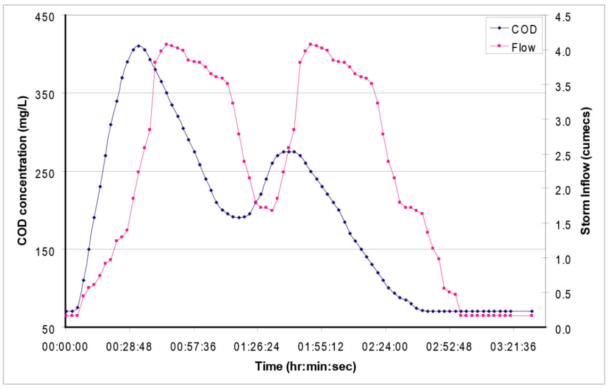
TKN pollutograph for Millers Bridge



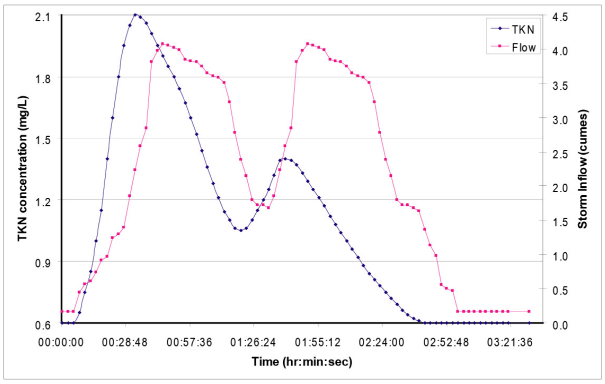
NOX pollutograph for Millers Bridge



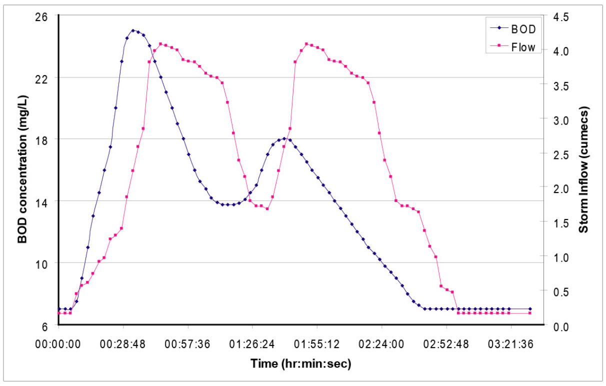
TSS pollutograph for Bankhall Relief



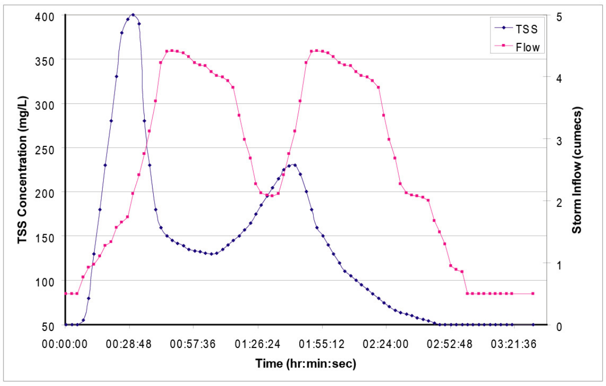
COD pollutograph for Bankhall Relief



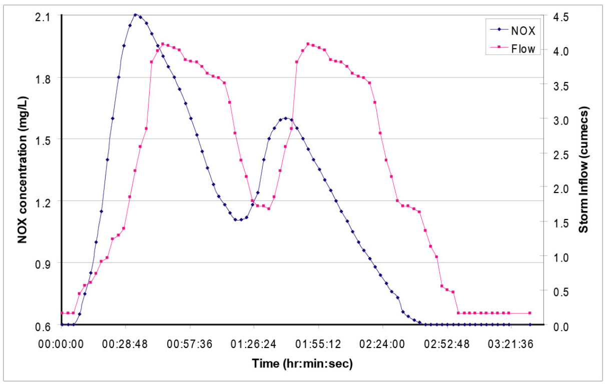
TKN pollutograph for Bankhall Relief



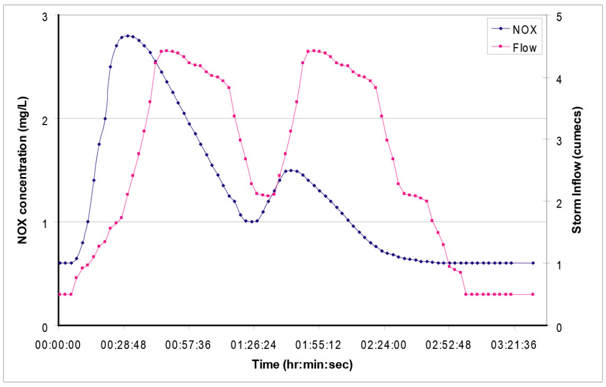
BOD pollutograph for Bankhall Relief



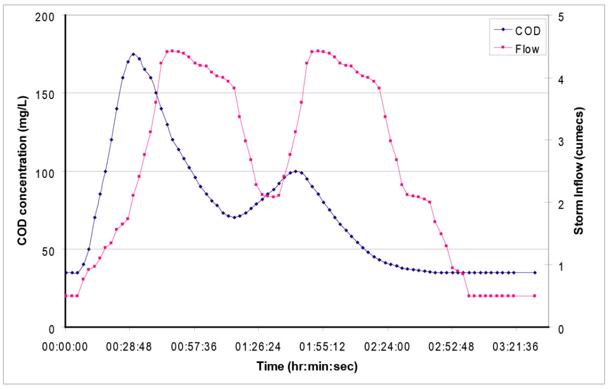
TSS pollutograph for Northern



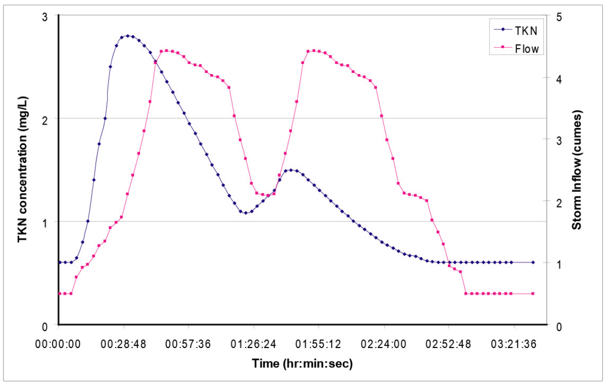
NOX pollutograph for Bankhall Relief



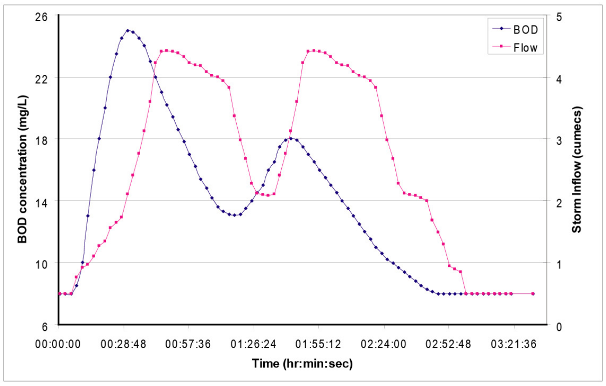
NOX pollutograph for Northern



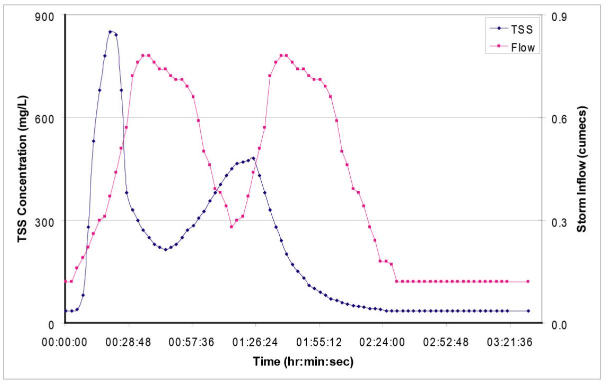
COD pollutograph for Northern



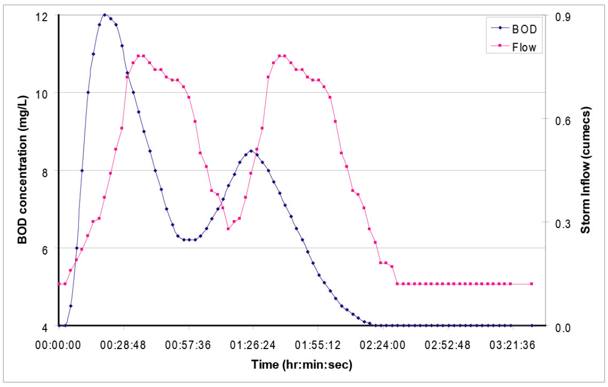
TKN pollutograph for Northern



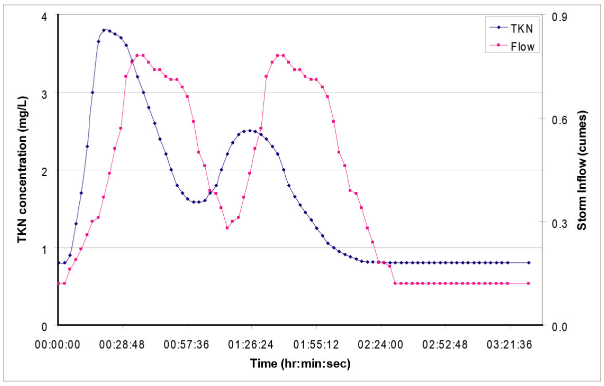
BOD pollutograph for Northern



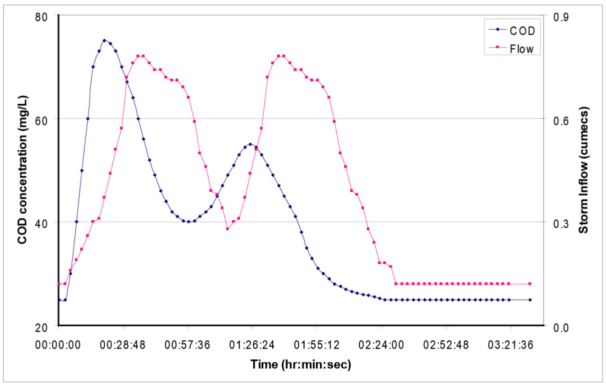
TSS pollutograph for Bankhall



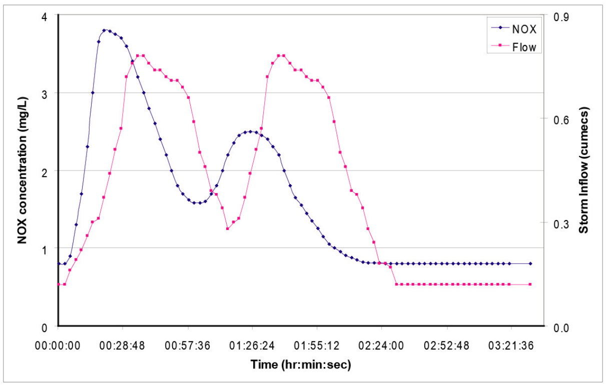
BOD pollutograph for Bankhall



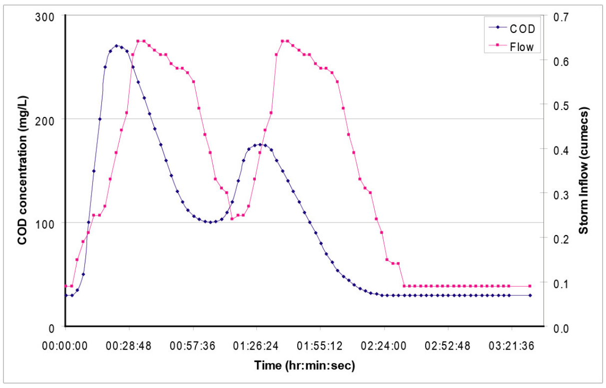
TKN pollutograph for Bankhall



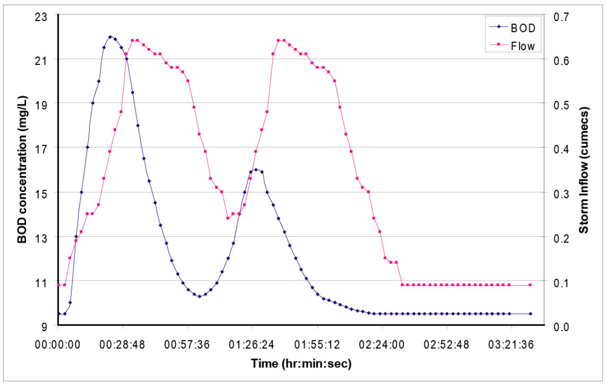
COD pollutograph for Bankhall



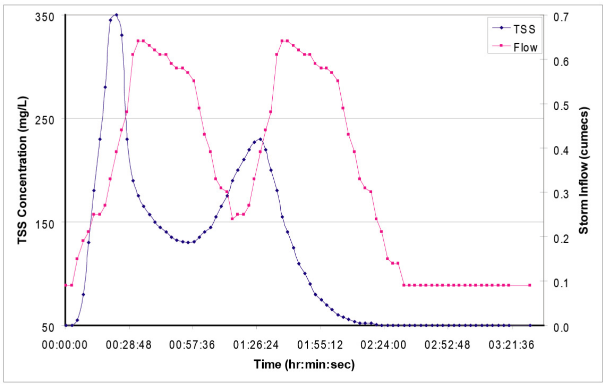
NOX pollutograph for Bankhall



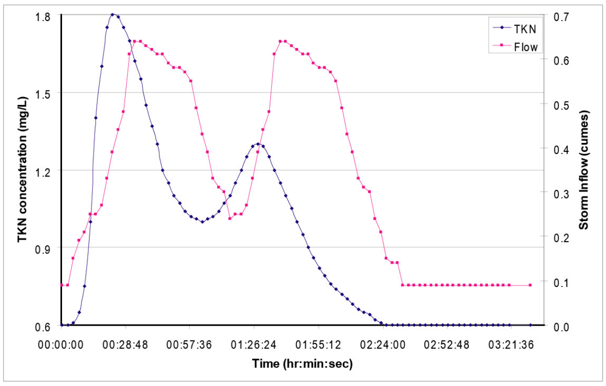
COD pollutograph for Sandhills Lane



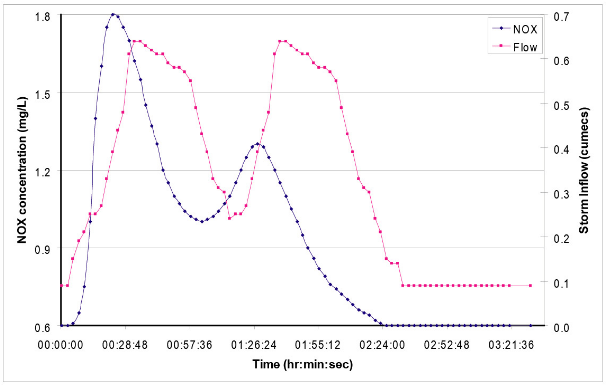
BOD pollutograph for Sandhills Lane



TSS pollutograph for Sandhills Lane

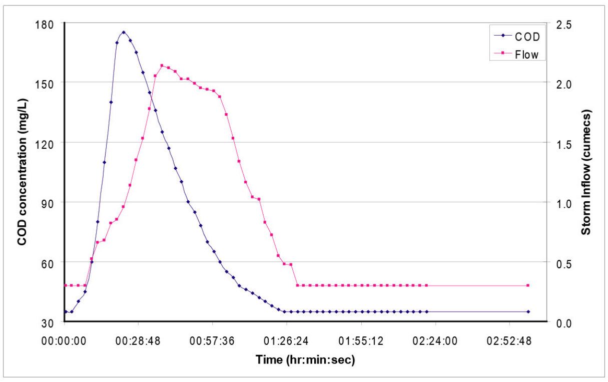


TKN pollutograph for Sandhills Lane

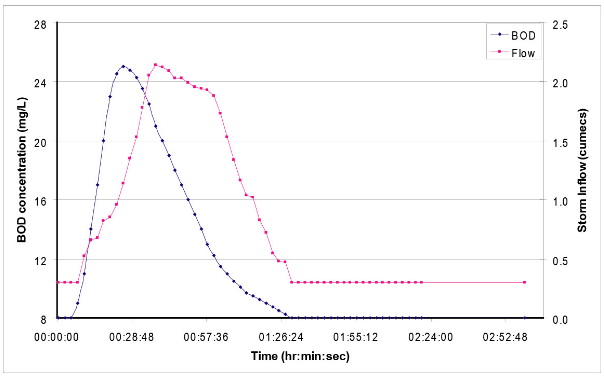


NOX pollutograph for Sandhills Lane

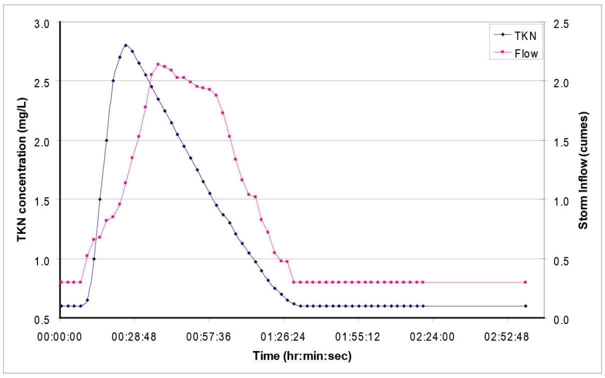
A.3 Pollutographs for migrating downstream storms



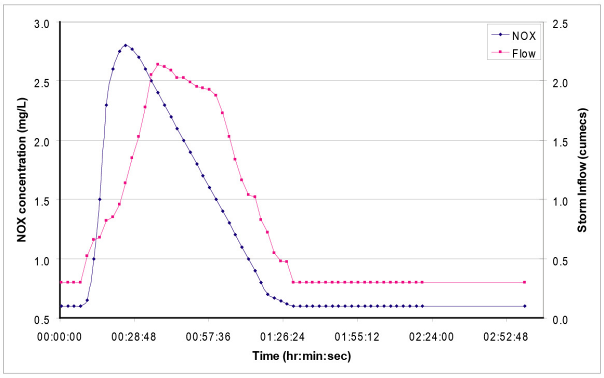
COD pollutograph for Rimrose



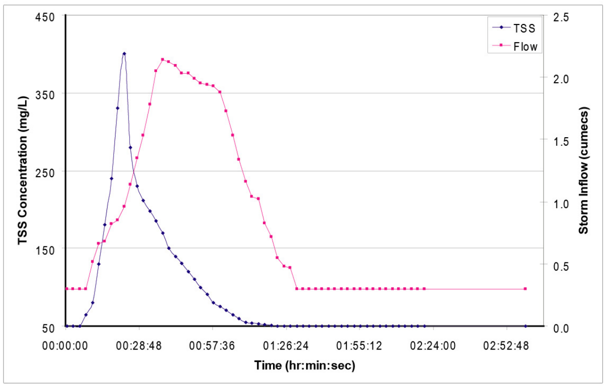
BOD pollutograph for Rimrose



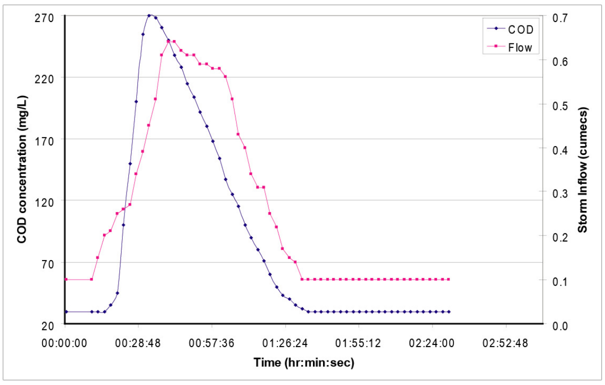
TKN pollutograph for Rimrose



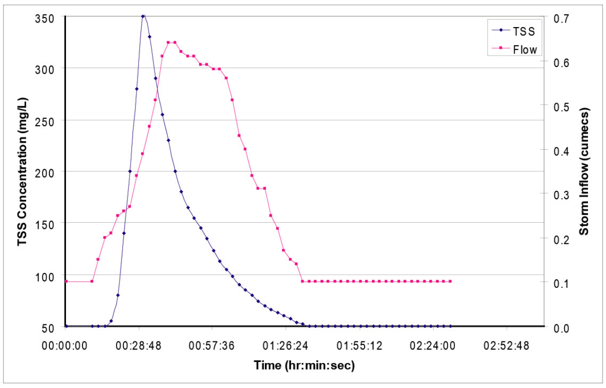
NOX pollutograph for Rimrose



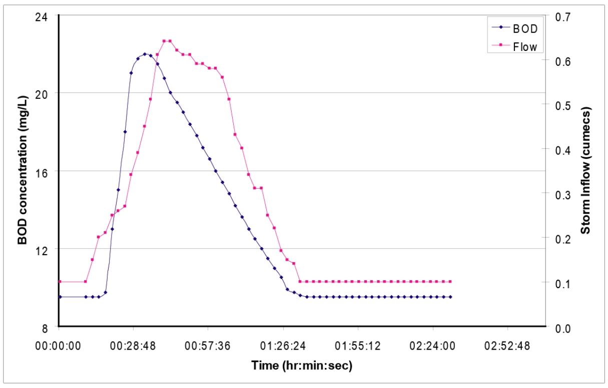
TSS pollutograph for Rimrose



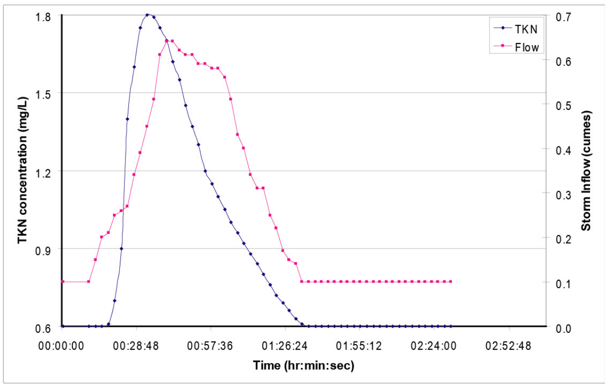
COD pollutograph for Strand Road



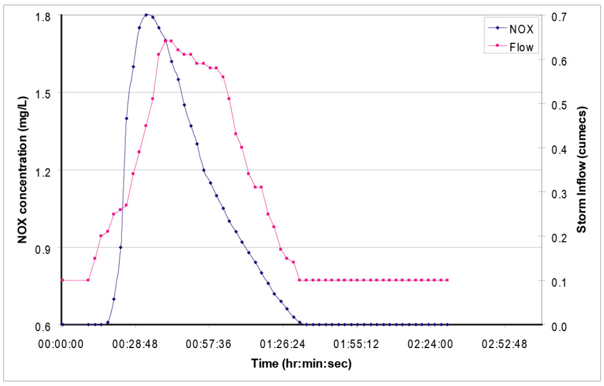
TSS pollutograph for Strand Road



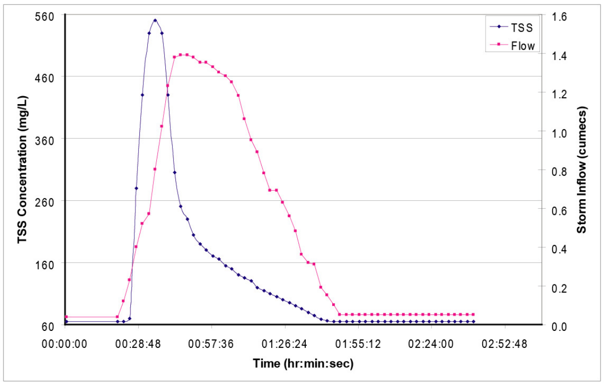
BOD pollutograph for Strand Road



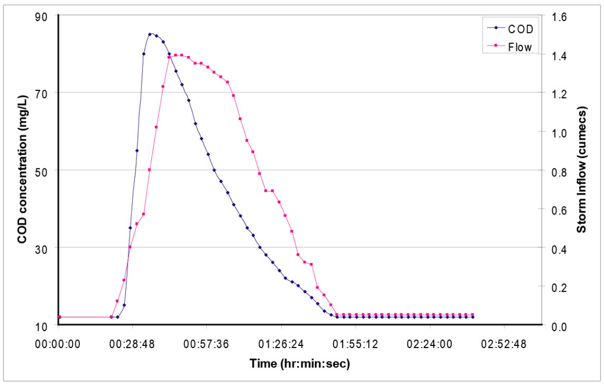
TKN pollutograph for Strand Road



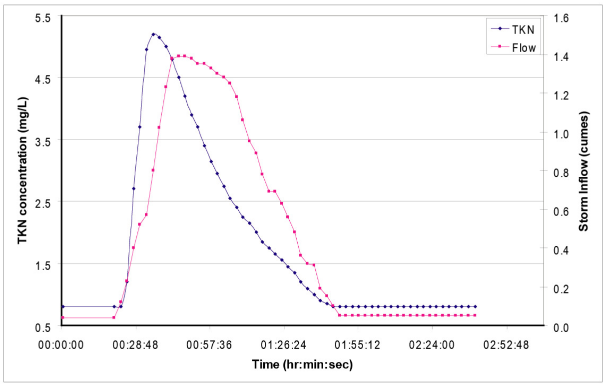
NOX pollutograph for Strand Road



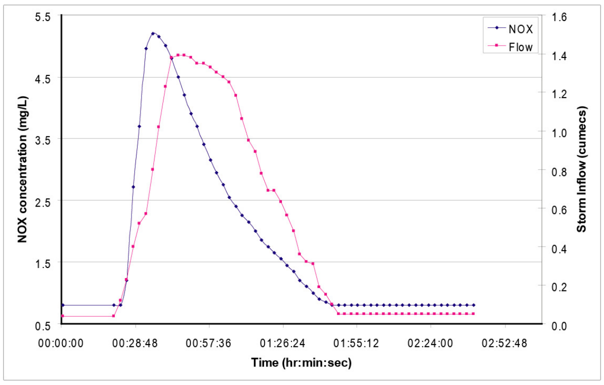
TSS pollutograph for Millers Bridge



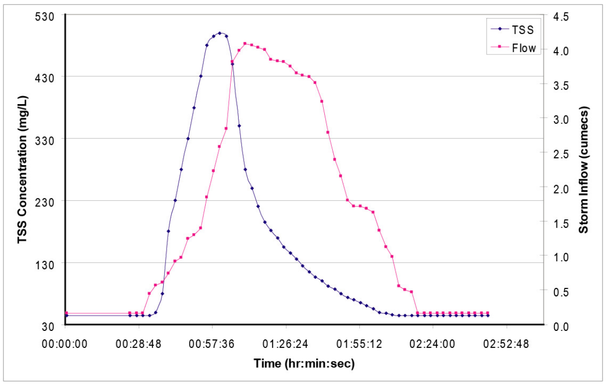
COD pollutograph for Millers Bridge



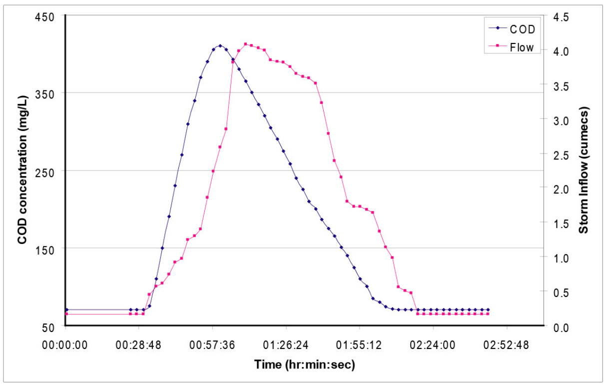
TKN pollutograph for Millers Bridge



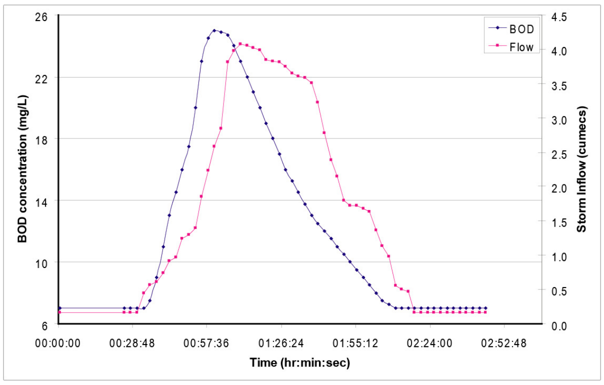
NOX pollutograph for Millers Bridge



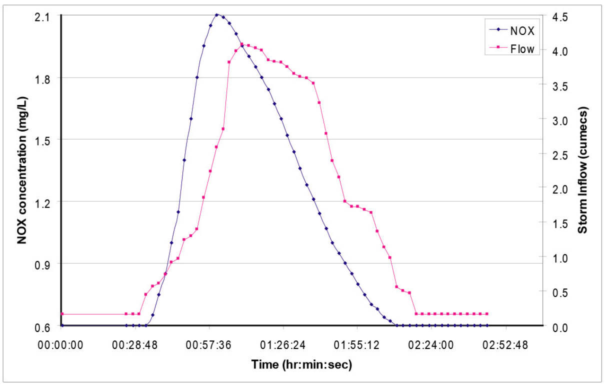
TSS pollutograph for Bankhall Relief



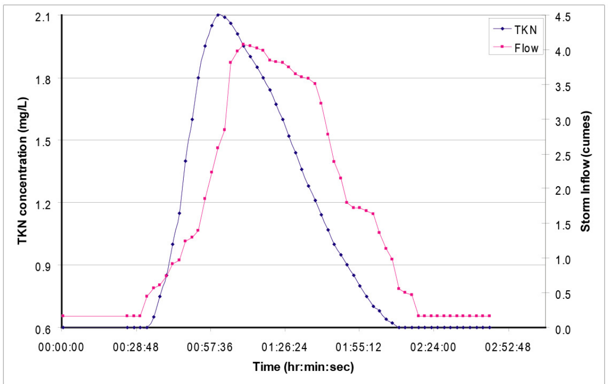
COD pollutograph for Bankhall Relief



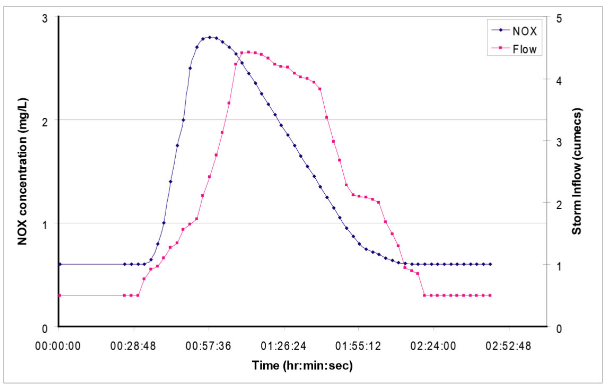
BOD pollutograph for Bankhall Relief



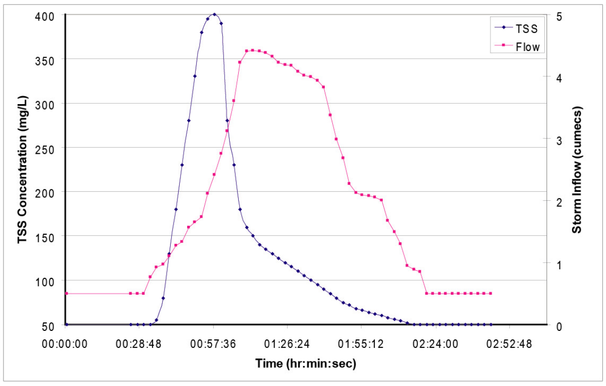
NOX pollutograph for Bankhall Relief



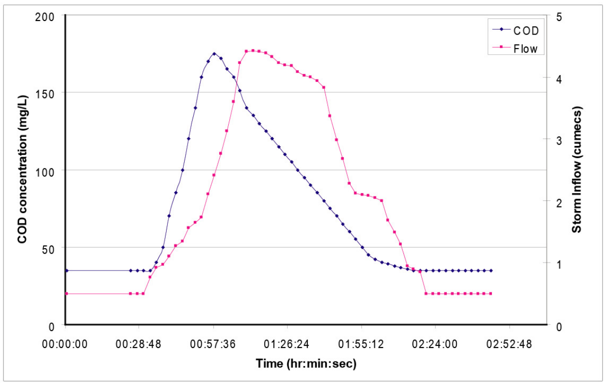
TKN pollutograph for Bankhall Relief



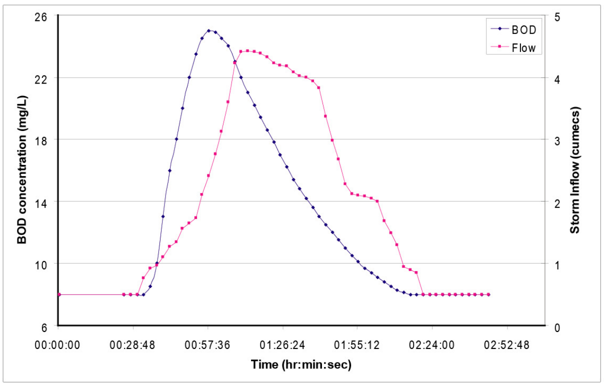
NOX pollutograph for Northern



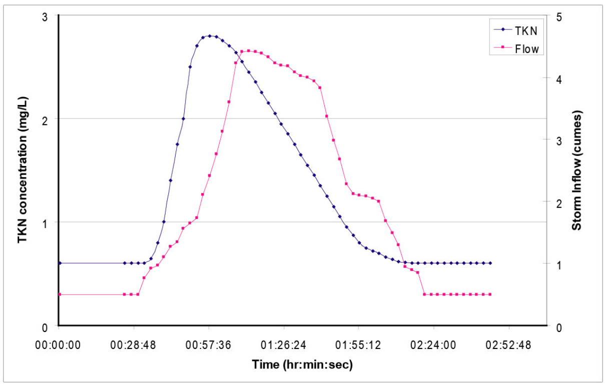
TSS pollutograph for Northern



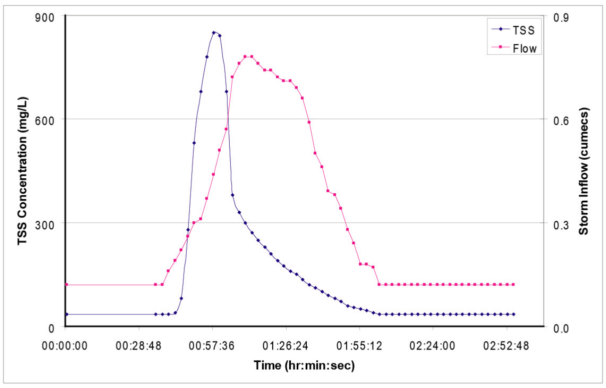
COD pollutograph for Northern



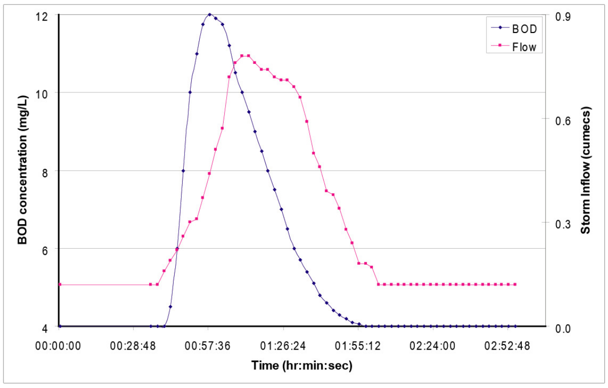
BOD pollutograph for Northern



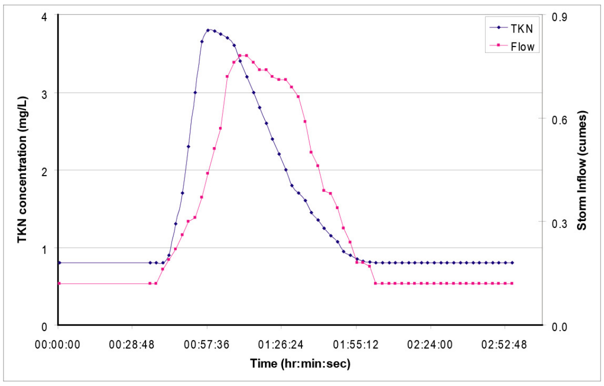
TKN pollutograph for Northern



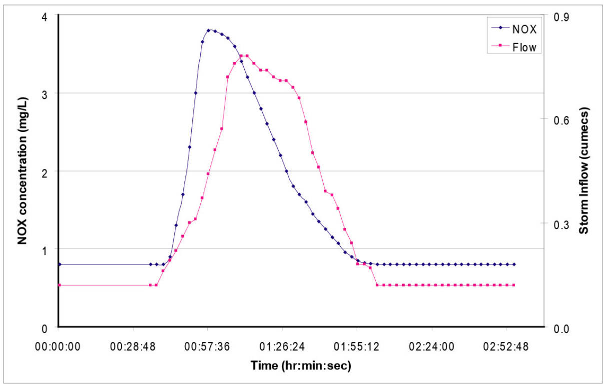
TSS pollutograph for Bankhall



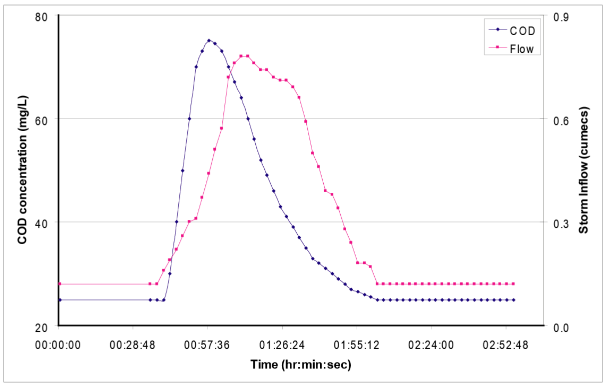
BOD pollutograph for Bankhall



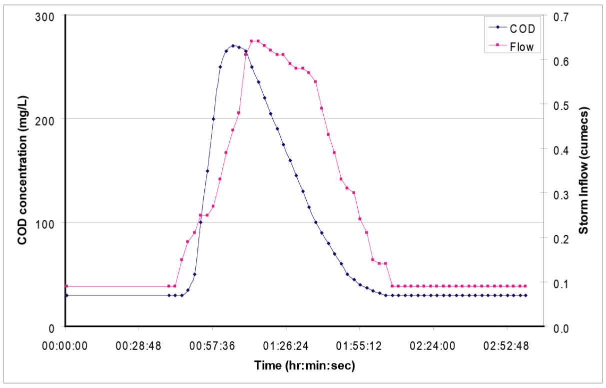
TKN pollutograph for Bankhall



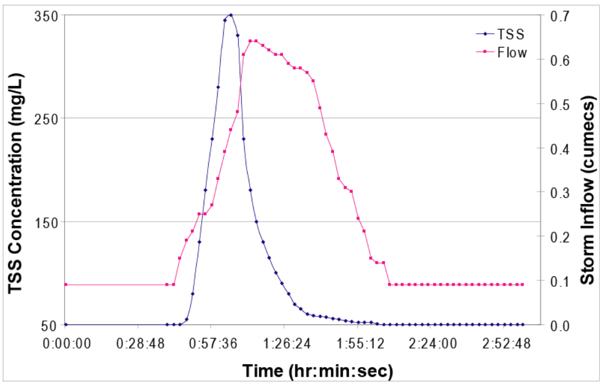
NOX pollutograph for Bankhall



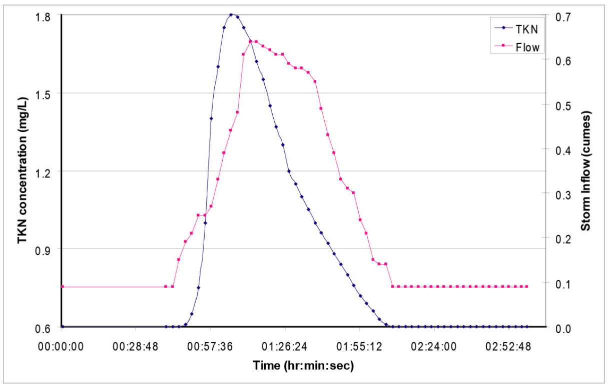
COD pollutograph for Bankhall



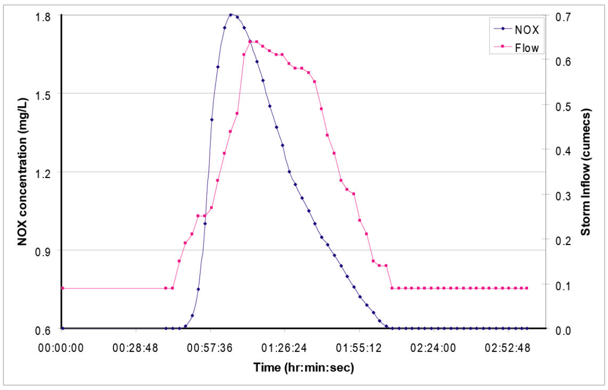
COD pollutograph for Sandhills Lane



TSS pollutograph for Sandhills Lane

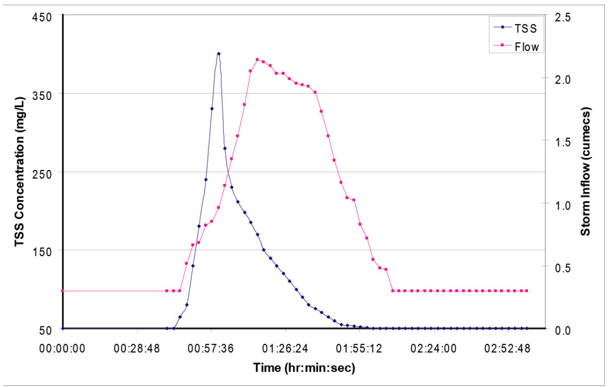


TKN pollutograph for Sandhills Lane

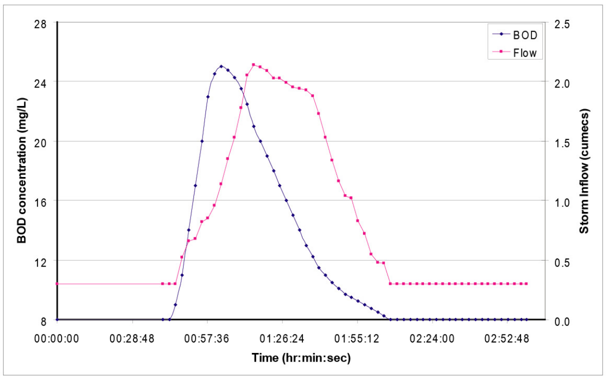


NOX pollutograph for Sandhills Lane

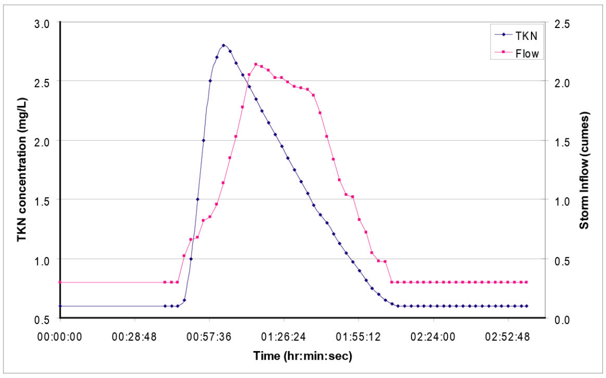
A.4 Pollutographs for migrating downstream storms



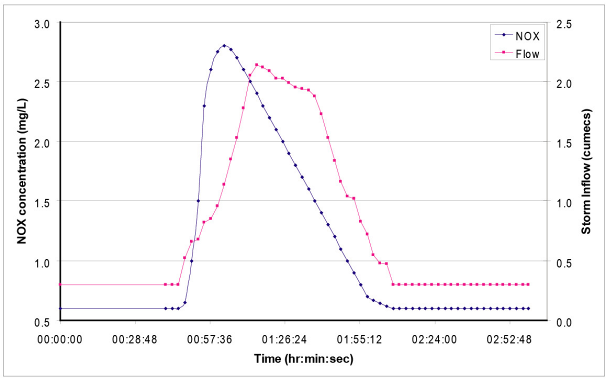
TSS pollutograph for Rimrose



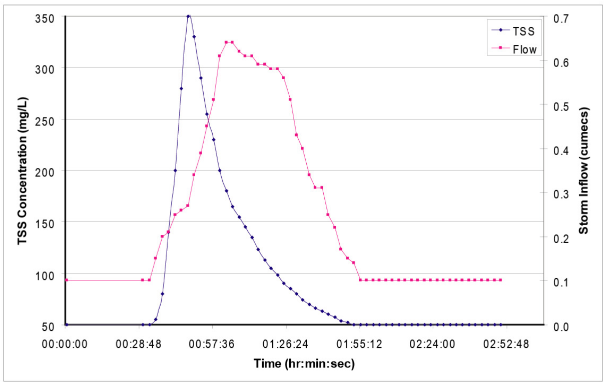
BOD pollutograph for Rimrose



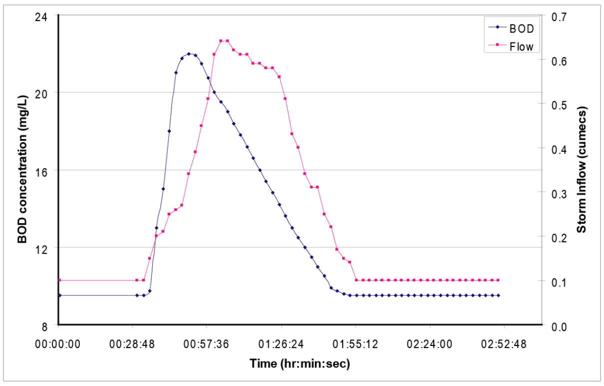
TKN pollutograph for Rimrose



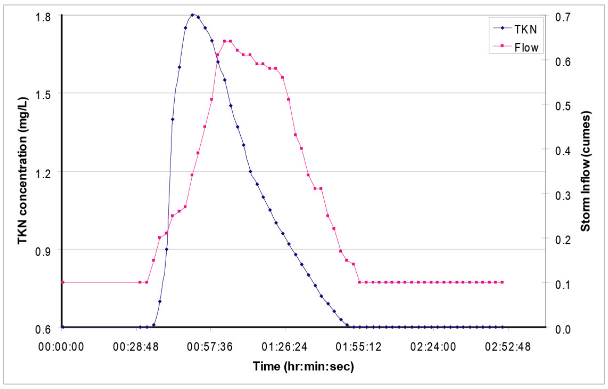
NOX pollutograph for Rimrose



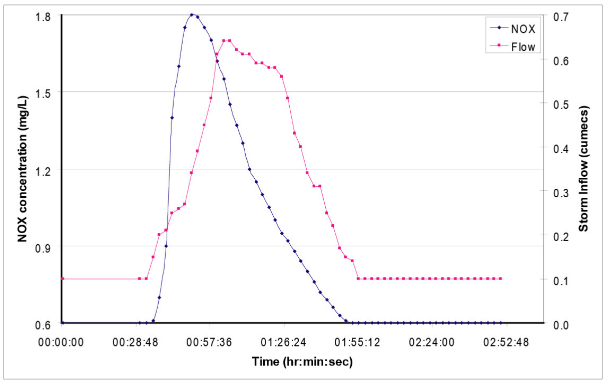
TSS pollutograph for Strand Road



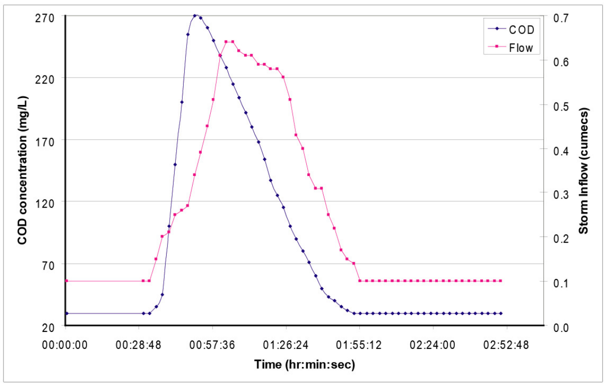
BOD pollutograph for Strand Road



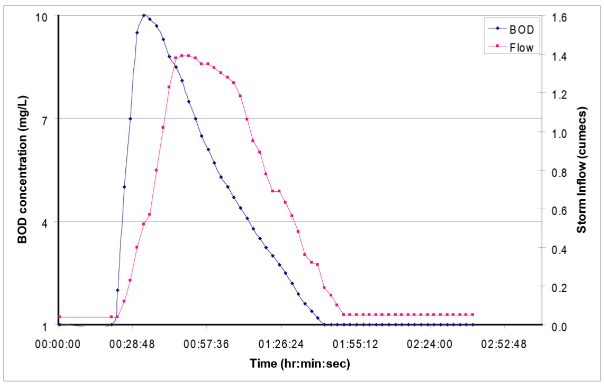
TKN pollutograph for Strand Road



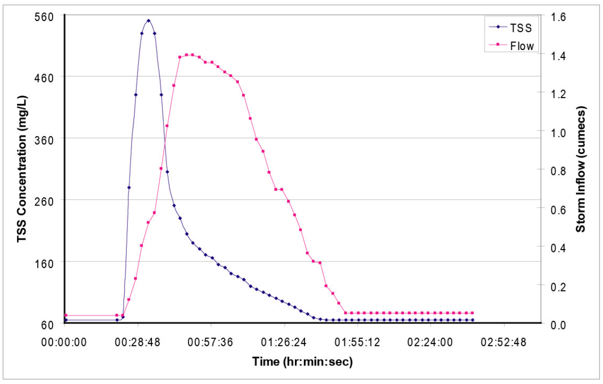
NOX pollutograph for Strand Road



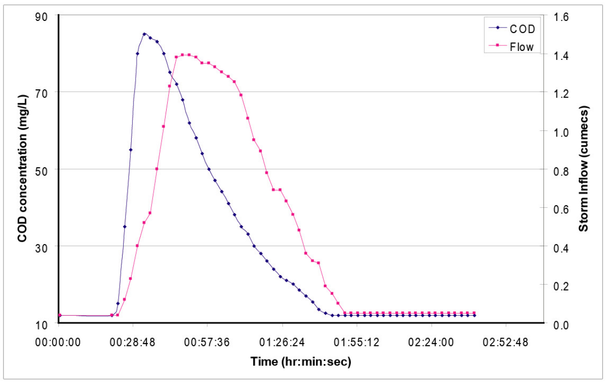
COD pollutograph for Strand Road



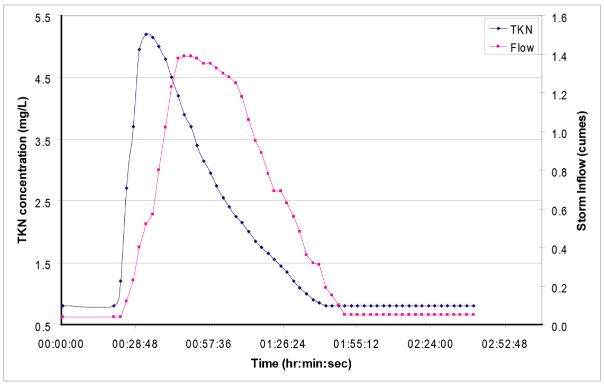
BOD pollutograph for Millers Bridge



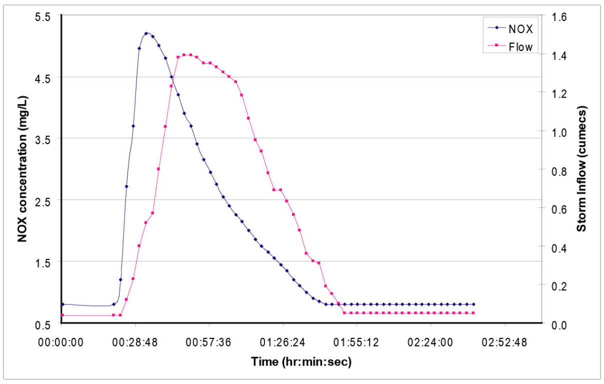
TSS pollutograph for Millers Bridge



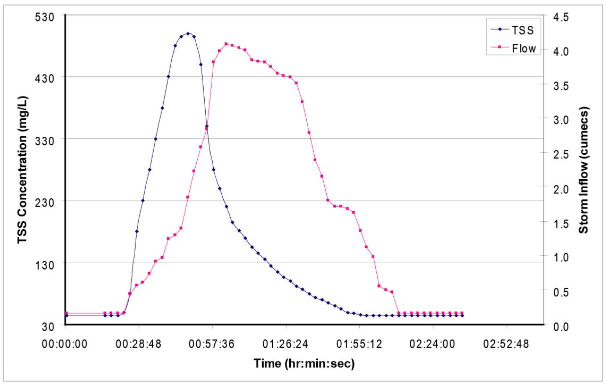
COD pollutograph for Millers Bridge



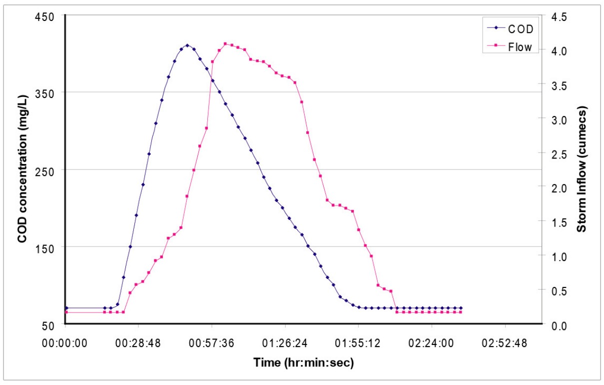
TKN pollutograph for Millers Bridge



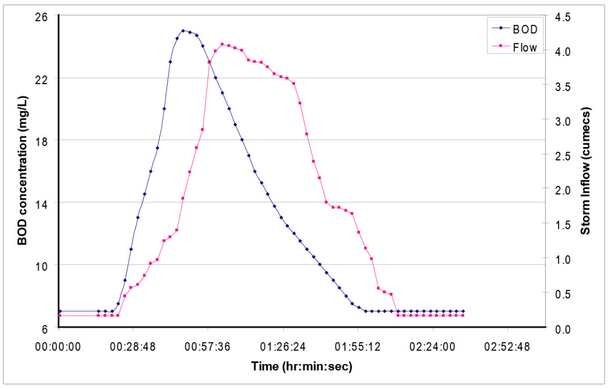
NOX pollutograph for Millers Bridge



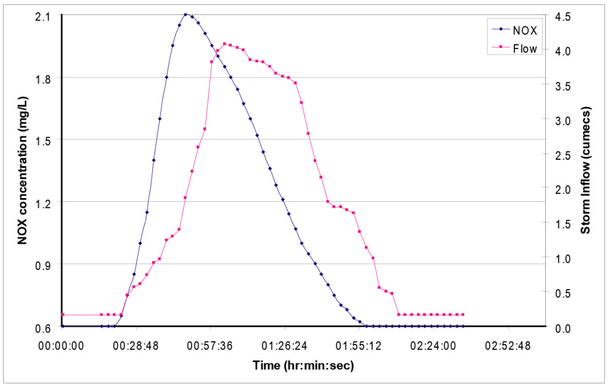
TSS pollutograph for Bankhall Relief



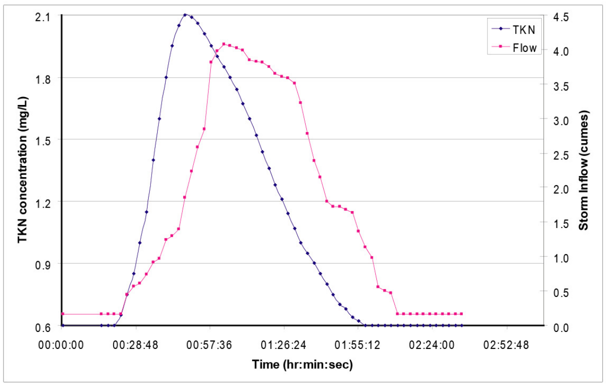
COD pollutograph for Bankhall Relief



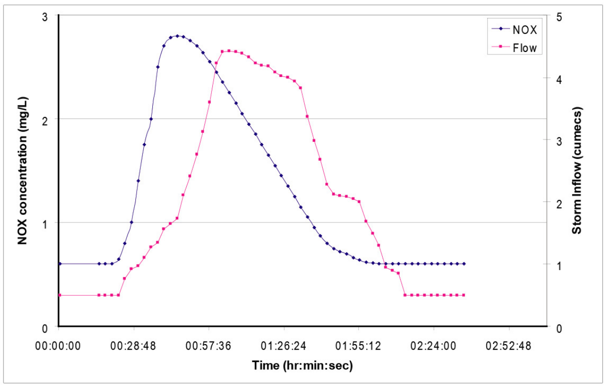
BOD pollutograph for Bankhall Relief



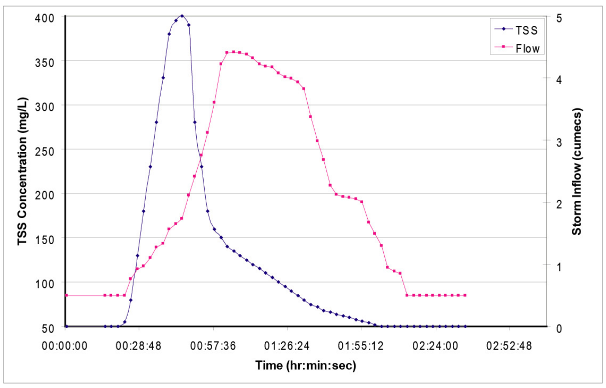
NOX pollutograph for Bankhall Relief



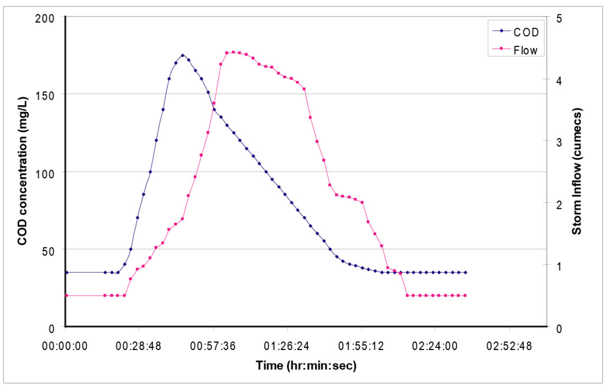
TKN pollutograph for Bankhall Relief



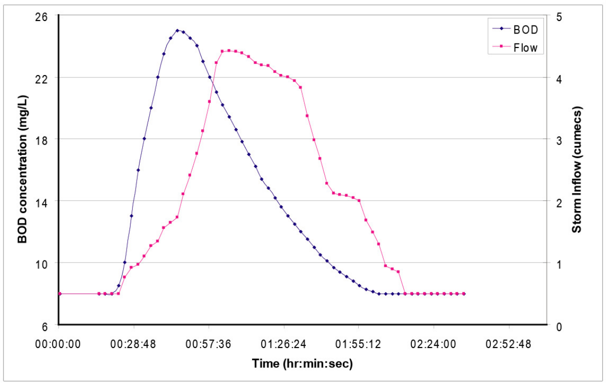
NOX pollutograph for Northern



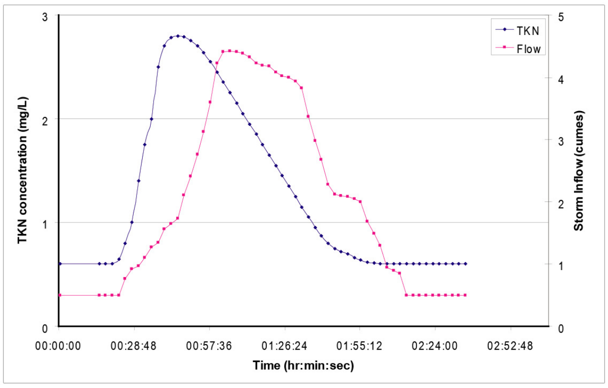
TSS pollutograph for Northern



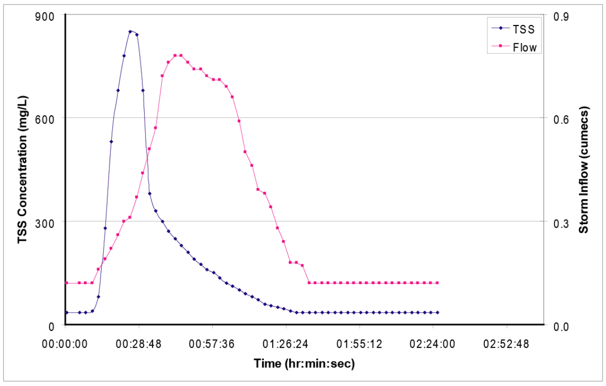
COD pollutograph for Northern



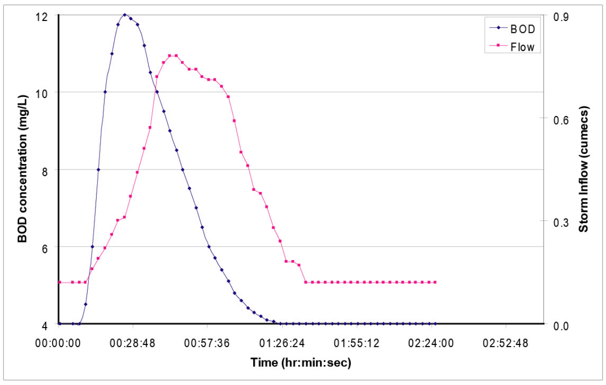
BOD pollutograph for Northern



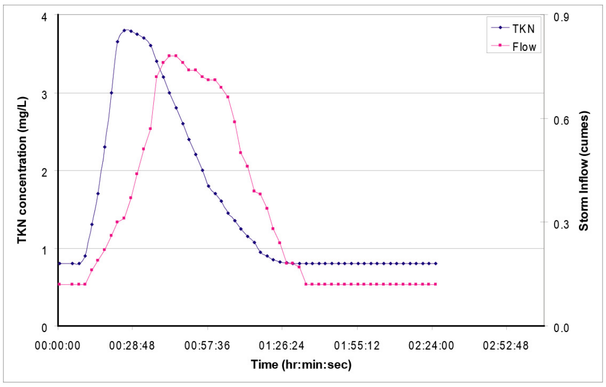
TKN pollutograph for Northern



TSS pollutograph for Bankhall



BOD pollutograph for Bankhall



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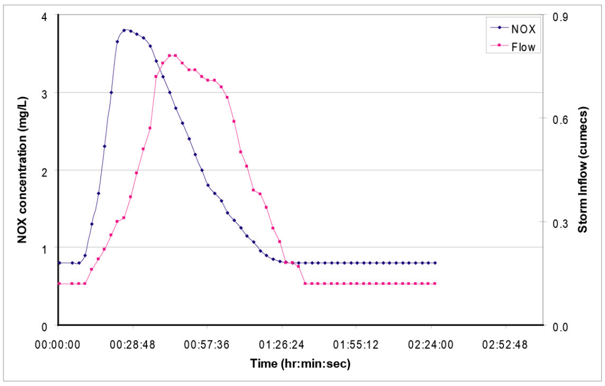
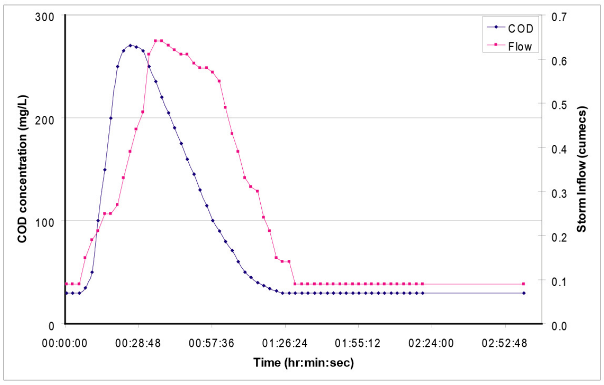
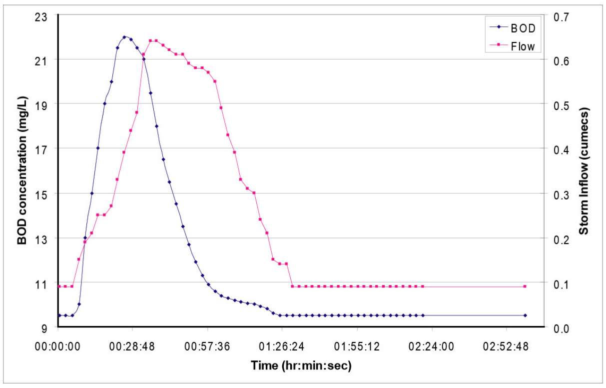


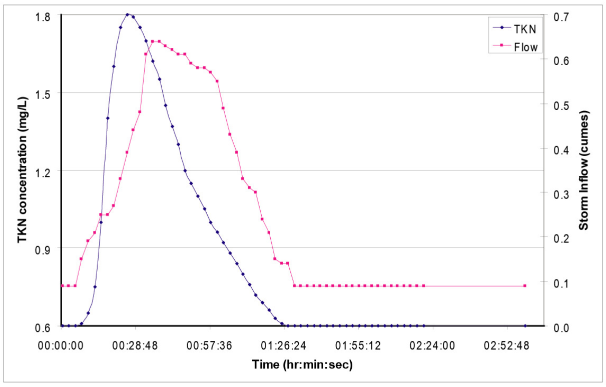
NOX pollutograph for Bankhall



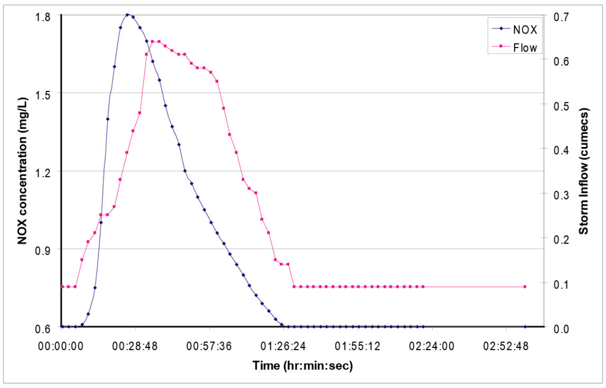
COD pollutograph for Sandhills Lane



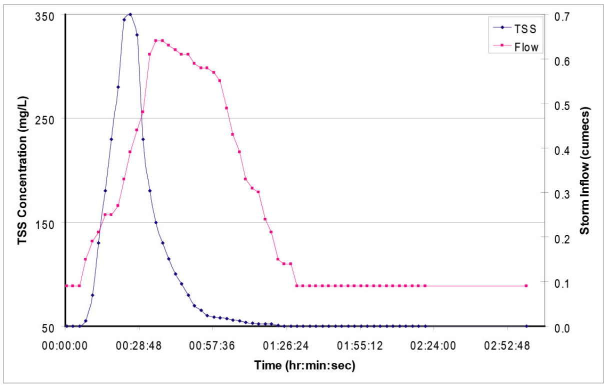
BOD pollutograph for Sandhills Lane



TKN pollutograph for Sandhills Lane



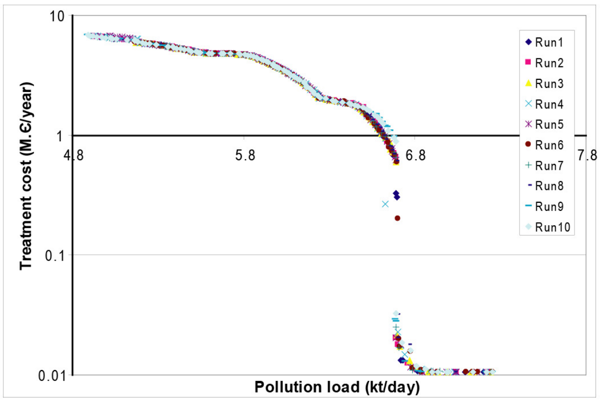
NOX pollutograph for Sandhills Lane



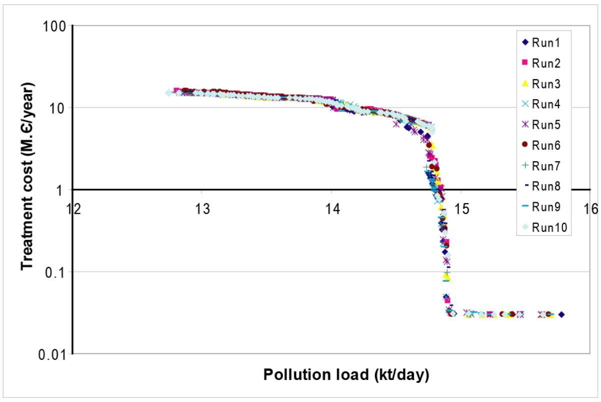
TSS pollutograph for Sandhills Lane

APPENDIX B – RANDOM RUNS FOR SNAPSHOT OPTIMIZATION

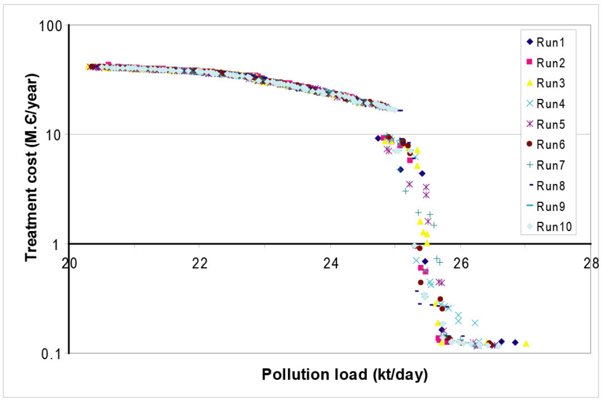
B.1 Random runs for NSGA II constraints handling approach



Random runs for T=30 minutes

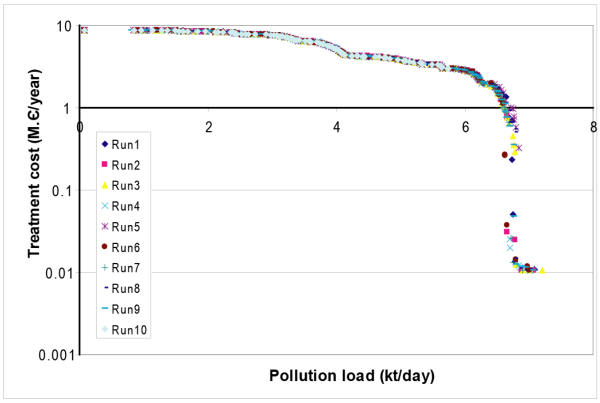


Random runs for T=1 hour

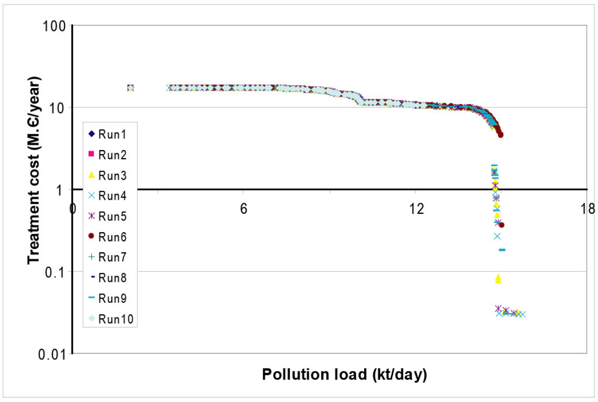


Random runs for T=2 hours & 30 minutes

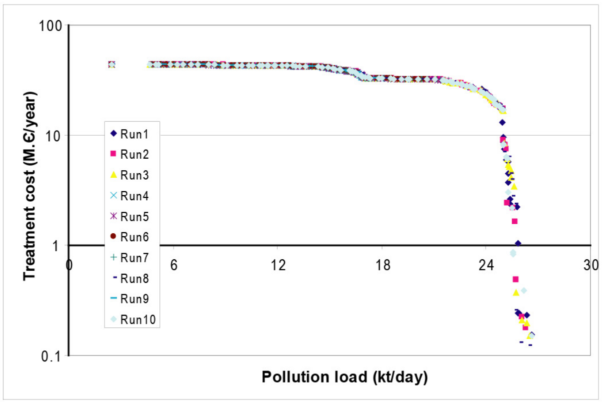
B.2 Random runs for SWMM 5.0 constraints handling approach



Random runs for T=30 minutes

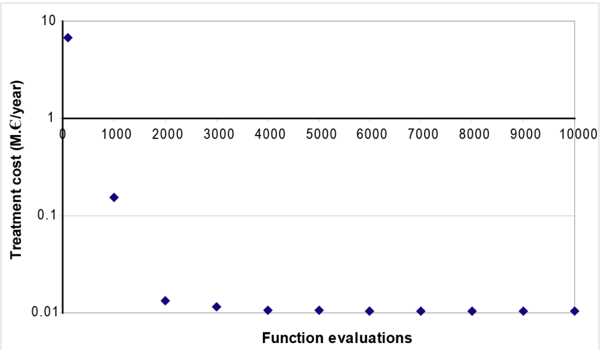


Random runs for T=1 hour

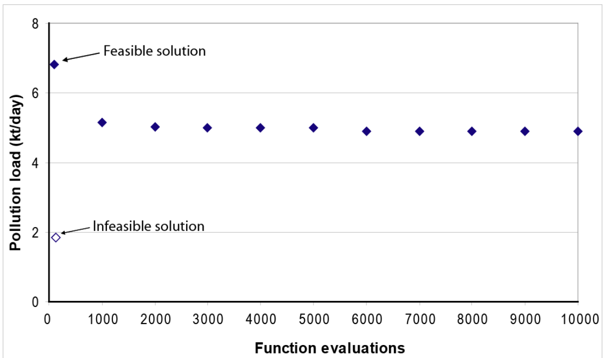


Random runs for T=2 hours & 30 minutes

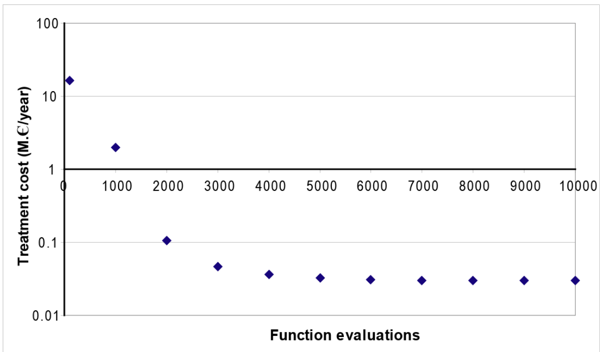
B.3 Function evaluations for NSGA II approach

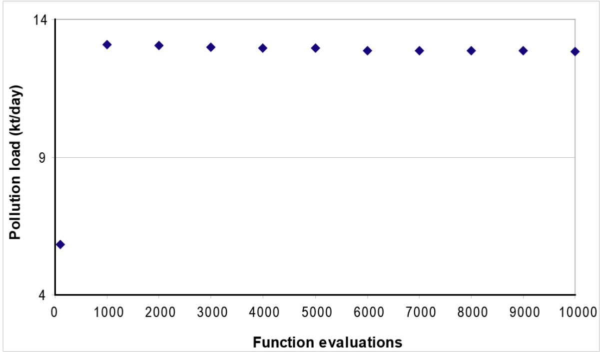


Function evaluation for minimum treatment cost T=30 minutes



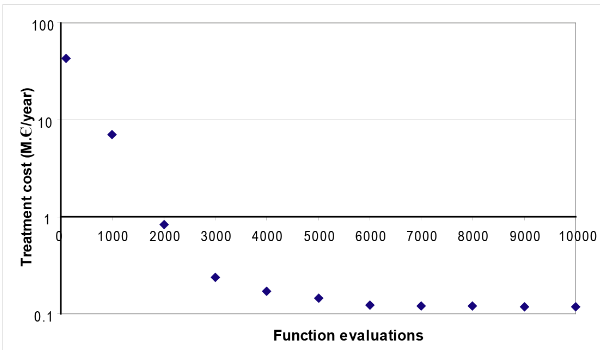
Function evaluation for minimum pollution load T=30 minutes

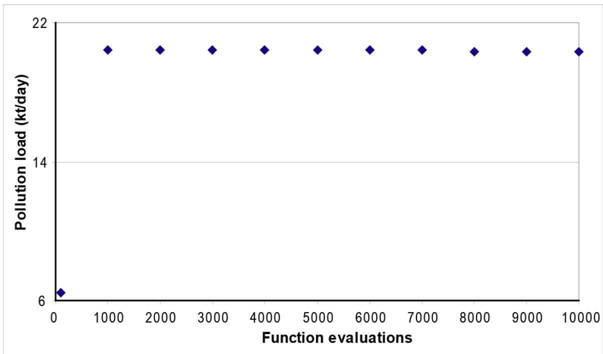




Function evaluation for minimum treatment cost T=1 hour

Function evaluation for minimum pollution load T=1 hour

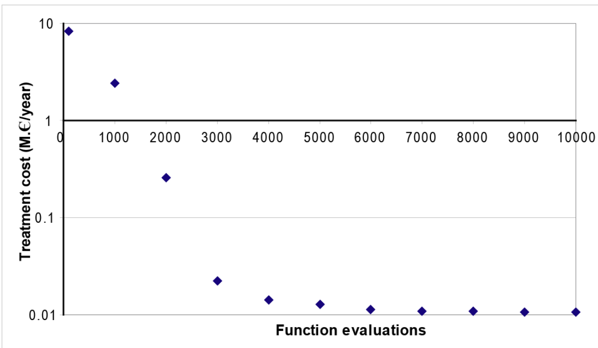


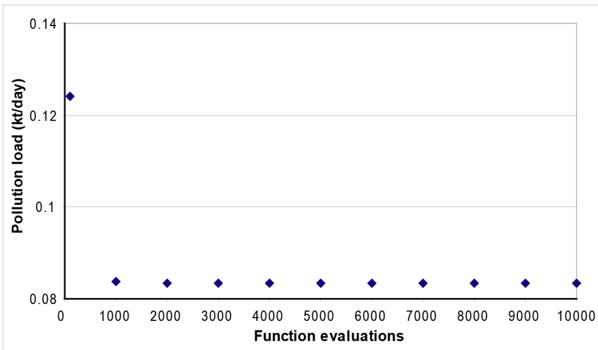


Function evaluation for minimum treatment cost T=2 hours & 30 minutes

Function evaluation for minimum pollution load T=2 hours & 30 minutes

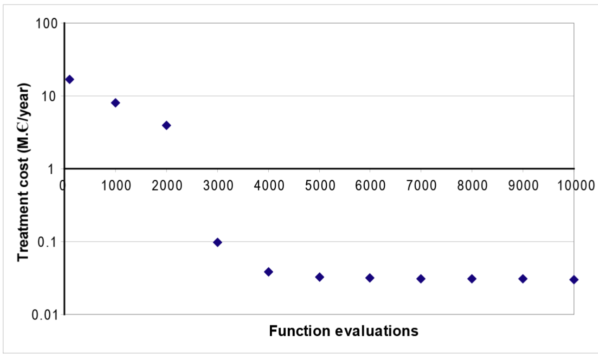
B.4 Function evaluations for SWMM approach



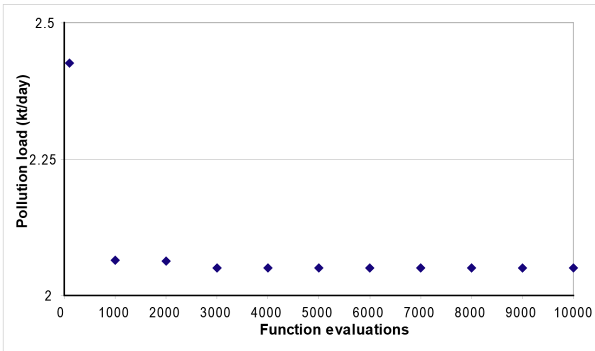


Function evaluation for minimum treatment cost T=30 minutes

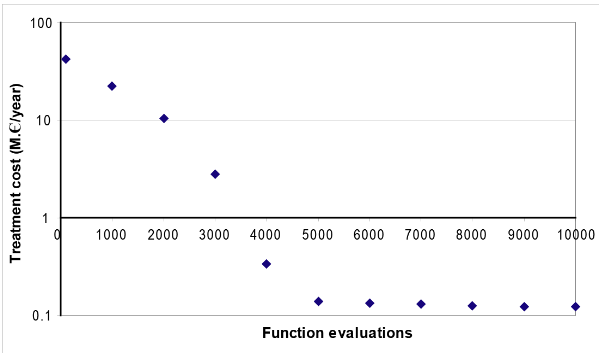
Function evaluation for minimum pollution load T=30 minutes



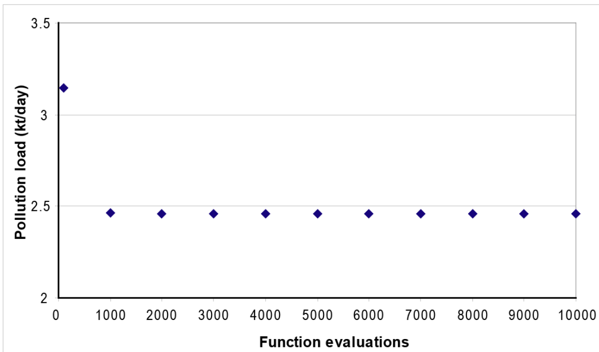
Function evaluation for minimum treatment cost T=1 hour



Function evaluation for minimum pollution load T=1 hour



Function evaluation for minimum treatment cost T=2 hours &30 minutes



Function evaluation for minimum pollution load T=2 hours & 30 minutes

APPENDIX C – HYDRAULIC SIMULATION RESULTS FOR SELECTED SOLUTIONS FROM SNAPSHOT OPTIMIZATION

C.1 Hydraulic simulation results for 30 min

Hydraulic simulation results for solution A_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.6865	1.6523	0.6439	3.135	2.5137	2.5099	1.8844
00:30:00	3.2537	3.2126	3.0298	6.279	6.1444	7.7148	7.51

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	4.73	0	0	0	0
00:30:00	5.64	3.47	7.90	6.17	12.36	0	3.30

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.693	0	0	0	0
00:30:00	0.581	0.319	0.606	0.822	1.528	0	0.337

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.3	6.25	8.38	6.01	8.17	3.15	7.57	1.71	7.37
00:30:00	5.91	7.26	8.56	8.56	9	8.79	10.08	6.91	8.19

Hydraulic simulation results for solution B_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.6708	1.5925	0.9268	3.7831	3.1914	2.1036	1.3212
00:30:00	3.2108	3.1541	3.2242	7.4981	7.4546	7.3792	7.2164

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
02:15:00	0	0	4.49	0	0	0	0
02:30:00	5.69	3.47	7.66	5.15	12.27	3.58	3.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.657	0	0	0	0
02:30:00	0.585	0.319	0.588	0.686	1.517	0.466	0.339

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.35	6.26	8.37	4.78	8.16	6.49	7.62	1.74	7.29
00:30:00	5.91	7.26	8.55	8.5	9	9	10.08	6.95	8.2

Hydraulic simulation results for solution C_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.6769	1.647	0.6083	2.3011	1.7142	0.8191	0.3103
00:30:00	3.2331	3.1813	3.0187	4.9052	4.7454	4.3352	4.099

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7

02:15:00	0	0	4.76	2.02	0	0	0
02:30:00	5.67	3.47	7.92	7.55	12.41	3.81	3.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.697	0.181	0	0	0
02:30:00	0.583	0.319	0.608	1.005	1.534	0.494	0.338

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.33	6.26	8.39	8.29	8.17	7.17	7.62	1.74	7.43
00:30:00	5.91	7.26	8.56	8.63	9.01	9.01	10.08	6.95	8.21

Hydraulic simulation results for solution D_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.5719	1.5249	0.4636	0.1404	0.0036	0.0003	0.0002
00:30:00	3.0458	3.004	2.8873	2.8212	2.6302	1.8791	1.2367

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
02:15:00	0.88	0.00	4.80	4.34	0.00	0.00	0.00
02:30:00	5.86	3.47	7.92	9.51	12.41	3.82	3.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.094	0	0.702	0.408	0	0	0
02:30:00	0.602	0.320	0.608	1.266	1.534	0.495	0.339

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.56	6.26	8.39	8.45	8.18	7.18	7.62	1.76	7.24
00:30:00	5.92	7.26	8.56	8.73	9.01	9.01	10.08	6.94	8.3

Hydraulic simulation results for solution E_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	1.9519	1.0232	0.2401	0.0477	0.0017	0.0004	0.0002
00:30:00	2.0748	2.0527	1.9615	1.8724	1.7493	1.0146	0.4475

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
02:15:00	2.87	0	4.74	4.36	0	0	0
02:30:00	6.84	3.47	7.92	9.51	12.41	3.82	3.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.312	0.000	0.692	0.410	0	0	0
02:30:00	0.704	0.320	0.608	1.266	1.535	0.495	0.339

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.73	6.26	8.39	8.45	8.18	7.18	7.62	1.78	7.24
00:30:00	5.98	7.26	8.56	8.73	9.01	9.01	10.08	6.92	8.31

Hydraulic simulation results for solution F_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.6948	0.2323	0.0274	0.0024	0.0006	0.0002	0.0003
00:30:00	0.7304	0.7012	0.5841	0.51	0.2791	0.0232	0.0015

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
02:15:00	4.20	0	4.70	4.36	0	0	0
02:30:00	8.21	3.48	7.92	9.52	12.44	3.82	0.58

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.463	0	0.687	0.410	0	0	0
02:30:00	0.845	0.319	0.608	1.266	1.537	0.491	0.059

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.82	6.25	8.39	8.45	8.18	7.18	7.62	1.57	7.42
00:30:00	6.04	7.26	8.57	8.72	9.01	9.01	10.14	7.16	8.3

Hydraulic simulation results for solution G_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.4216	0.1092	0.0101	0.0008	0.0003	0.0002	0.0001
00:30:00	0.4646	0.4209	0.3011	0.2295	0.0797	0.003	0.0004

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7

02:15:00	4.46	0	4.71	4.36	0	0	0
02:30:00	8.48	3.48	7.93	9.52	12.44	3.82	0.59

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.492	0	0.687	0.410	0	0	0
02:30:00	0.873	0.319	0.609	1.266	1.537	0.491	0.06

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.84	6.25	8.39	8.45	8.18	7.18	7.63	1.57	7.41
00:30:00	6.06	7.26	8.57	8.72	9.01	9.01	10.14	7.16	8.3

Hydraulic simulation results for solution H_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0002	0.0001	0.0001	0.0002	0.0002	0.0001	0.0001
00:30:00	0.0005	0.0002	0.0002	0.0003	0.0004	0.0003	0.0002

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
02:15:00	4.91	0	4.74	4.36	0	0	0
02:30:00	8.94	3.48	7.97	9.52	12.45	3.81	4.56

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.692	0.410	0	0	0
02:30:00	0.921	0.319	0.612	1.265	1.537	0.491	0.464

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.87	6.26	8.39	8.45	8.18	7.18	7.62	1.79	7.24
00:30:00	6.08	7.26	8.57	8.72	9.01	9.01	10.1	7.06	8.31

Hydraulic simulation results for solution S_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.26	0.6802	4.529	7.72	7.72	7.72
00:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	1.09	0	0	0	0

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0.083	0	0	0	0

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.21	1.62	4.33	3.56	3.14	3.26	2.17	0	0
00:30:00	4.33	3.41	8.11	7.07	4.07	8.21	3.55	0	0

Hydraulic simulation results for solution T_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	2.96	3.26	7.02	6.64	5.69	5.32
00:30:00	3.26	3.26	0.29	7.72	6.39	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	2.86	0.12	12.41	0	0

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0.220	0.016	1.534	0	0

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.23	5.13	6.06	2.86	8.17	2.78	3.8	0	7.42
00:30:00	4.44	6.57	8.26	8.08	9.01	7.81	10.02	6.58	8.2

Hydraulic simulation results for solution U_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	2.92	3.26	6.98	6.48	4.50	3.08
00:30:00	3.26	3.26	0.29	7.72	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m3/s)						
	T1	T2	T3	T4	T5	T6	T7

00:15:00	0	0	0.51	0	0	0	0
00:30:00	0	0.01	3.93	0.05	12.41	3.81	3.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.074	0	0	0	0
00:30:00	0	0	0.302	0.007	1.534	0.494	0.338

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.49	5.39	8.05	2.82	8.17	7.18	7.63	0	7.42
00:30:00	5.35	6.91	8.33	8.06	9.01	9.01	10.08	6.91	8.19

Hydraulic simulation results for solution V_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.15	3.26	3.22	3.13	1.31	0.30
00:30:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	4.35	0	0	0
00:30:00	0	0	2.80	9.51	12.41	3.82	3.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0.409	0	0	0
00:30:00	0	0	0.215	1.265	1.534	0.494	0.338

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)

	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.25	3.83	6	8.45	8.17	7.18	7.62	0	7.44
00:30:00	4.53	6.31	8.26	8.73	9.01	9.01	10.08	4.21	8.19

Hydraulic simulation results for solution W_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	2.46	1.10	0.54	0.02	0.00	0.00
00:30:00	3.26	3.25	3.21	3.17	3.11	2.70	2.40

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	4.73	4.36	0	0	0
00:30:00	2.19	3.47	7.92	9.51	12.41	3.82	3.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.691	0.410	0	0	0
00:30:00	0.224	0.319	0.608	1.265	1.534	0.495	0.338

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.85	6.26	8.39	8.45	8.18	7.18	7.62	1.76	7.24
00:30:00	5.68	7.26	8.56	8.73	9.01	9.01	10.08	6.96	8.21

Hydraulic simulation results for solution X_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.44	0.77	0.25	0.08	0.0033	0.0004	0.0002
00:30:00	0.50	1.40	1.32	1.26	1.12	0.58	0.24

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.42	0	4.68	4.37	0	0	0
00:30:00	8.42	0	7.92	9.51	12.41	3.81	3.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.487	0	0.684	0.411	0	0	0
00:30:00	0.869	0	0.608	1.265	1.534	0.495	0.338

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.83	5.41	8.39	8.45	8.18	7.18	7.63	0	7.4
00:30:00	6.06	6.87	8.56	8.73	9.01	9.01	10.08	6.85	8.32

Hydraulic simulation results for solution Y_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.57	0.24	0.03	0.0047	0.0006	0.0002	0.0002
00:30:00	0.61	0.67	0.56	0.49	0.28	0.03	0.00

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.31	0	4.71	4.36	0	0	0
00:30:00	8.33	3.42	7.93	9.52	12.44	3.82	0.58

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.476	0	0.687	0.410	0	0	0
00:30:00	0.857	0.313	0.609	1.265	1.537	0.491	0.059

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.83	6.24	8.39	8.45	8.18	7.18	7.62	1.43	7.41
00:30:00	6.05	7.26	8.57	8.72	9.01	9.01	10.14	7.31	8.3

Hydraulic simulation results for solution Z_{T2}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0209	0.0051	0.0003	0.0002	0.0002	0.0001	0.0001
00:30:00	0.0373	0.0266	0.006	0.0014	0.0005	0.0003	0.0002

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.87	0	4.74	4.36	0	0	0
00:30:00	8.90	3.46	7.96	9.52	12.44	3.81	2.52

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.538	0	0.692	0.410	0	0	0
00:30:00	0.917	0.317	0.611	1.265	1.536	0.491	0.256

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.86	6.25	8.39	8.45	8.18	7.18	7.62	1.62	7.24
00:30:00	6.08	7.26	8.57	8.72	9.01	9.01	10.1	7.02	8.31

C.2 Hydraulic simulation results for 1 hr

Hydraulic simulation results for solution A_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.11	1.90	0.85	2.77	2.00	2.08	1.51
00:30:00	2.26	3.09	3.00	5.18	5.07	6.66	6.45
00:45:00	2.31	3.23	3.22	5.51	5.50	7.49	7.47
01:00:00	2.30	3.23	3.24	5.53	5.54	7.56	7.56

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	2.68	0	4.74	0	0	0	0
00:30:00	6.64	0.33	7.90	7.26	12.41	0	3.31
00:45:00	9.47	2.36	7.28	16.58	16.58	2.07	3.45
01:00:00	8.72	1.50	4.54	16.58	16.58	1.40	2.91

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.291	0	0.694	0	0	0	0
00:30:00	0.684	0.031	0.607	0.967	1.534	0	0.338
00:45:00	0.650	0.161	0.470	1.571	1.231	0.161	0.237
01:00:00	0.485	0.085	0.320	1.081	1.013	0.080	0.156

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.72	5.51	8.39	7.89	8.17	3.17	7.63	0	7.42
00:30:00	5.96	6.99	8.56	8.62	9.01	8.83	10.08	6.94	8.21
00:45:00	6.11	7.18	8.53	9.04	9.18	8.95	10.08	6.94	8.21
01:00:00	6.07	7.11	8.37	9.04	9.18	8.93	10.08	6.94	8.21

Hydraulic simulation results for solution B_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.11	1.92	0.84	2.61	1.94	0.94	0.37
00:30:00	2.25	3.11	3.00	5.09	4.99	4.69	4.52
00:45:00	2.30	3.25	3.23	5.42	5.41	5.37	5.35
01:00:00	2.29	3.24	3.25	5.44	5.44	5.44	5.44

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	2.69	0	4.76	0.15	0	0	0
00:30:00	6.65	0.01	7.92	7.36	12.41	3.82	3.31
00:45:00	9.48	2.34	7.29	16.58	16.58	4.10	3.45
01:00:00	8.72	1.47	4.56	16.58	16.58	3.43	2.91

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.292	0	0.696	0.014	0	0	0
00:30:00	0.685	0.001	0.608	0.979	1.534	0.494	0.338
00:45:00	0.650	0.160	0.471	1.571	1.231	0.317	0.237
01:00:00	0.485	0.084	0.321	1.081	1.013	0.196	0.156

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.72	5.44	8.39	8.09	8.17	7.18	7.63	0	7.44
00:30:00	5.97	6.89	8.56	8.62	9.01	9.01	10.08	6.9	8.21
00:45:00	6.11	7.18	8.53	9.04	9.18	9.03	10.08	6.95	8.21
01:00:00	6.07	7.11	8.37	9.04	9.18	9	10.08	6.95	8.21

Hydraulic simulation results for solution C_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.69	0.92	0.40	0.19	0.02	0.91	0.71
00:30:00	0.74	1.64	1.64	1.61	1.43	2.92	2.63
00:45:00	0.75	1.73	1.79	1.79	1.78	3.76	3.74
01:00:00	0.75	1.73	1.80	1.81	1.82	3.82	3.82

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.18	0	4.63	4.36	0	0	0
00:30:00	8.18	0	7.85	9.50	12.40	0	3.31
00:45:00	11.02	2.31	7.22	16.58	16.58	2.09	3.45
01:00:00	10.26	1.45	4.49	16.58	16.58	1.42	2.91

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.460	0	0.676	0.410	0	0	0
00:30:00	0.843	0	0.602	1.264	1.533	0	0.338
00:45:00	0.756	0.158	0.467	1.572	1.231	0.163	0.237
01:00:00	0.571	0.083	0.316	1.081	1.013	0.081	0.156

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.82	5.41	8.39	8.45	8.18	3.21	7.63	0	7.43
00:30:00	6.05	6.81	8.56	8.73	9.01	8.81	10.08	6.83	8.3
00:45:00	6.18	7.18	8.53	9.04	9.18	8.95	10.08	6.91	8.3
01:00:00	6.15	7.11	8.37	9.04	9.18	8.93	10.08	6.91	8.3

Hydraulic simulation results for solution D_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	1.58	1.50	0.56	0.23	0.01	0	0
00:30:00	1.62	2.51	2.45	2.39	2.28	1.71	1.23
00:45:00	1.65	2.63	2.63	2.63	2.61	2.59	2.58
01:00:00	1.65	2.63	2.64	2.64	2.64	2.66	2.66

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	3.31	0	4.70	4.36	0	0	0
00:30:00	7.29	0	7.91	9.51	12.41	3.80	3.31
00:45:00	10.12	2.31	7.28	16.58	16.58	4.09	3.45
01:00:00	9.37	1.45	4.54	16.58	16.58	3.42	2.91

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.363	0	0.687	0.410	0	0	0
00:30:00	0.751	0	0.607	1.265	1.534	0.494	0.338
00:45:00	0.694	0.158	0.470	1.572	1.231	0.316	0.237
01:00:00	0.521	0.083	0.320	1.081	1.013	0.195	0.156

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.76	5.41	8.39	8.45	8.18	7.14	7.62	0	7.44
00:30:00	6	6.82	8.56	8.73	9.01	9.01	10.08	6.83	8.3
00:45:00	6.14	7.18	8.53	9.04	9.18	9.03	10.08	6.92	8.3
01:00:00	6.1	7.11	8.37	9.04	9.18	9	10.08	6.92	8.3

Hydraulic simulation results for solution E_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.15	0.03	0.08	0.08	0.01	0	0
00:30:00	0.18	0.14	0.20	0.19	0.14	0.06	0.02
00:45:00	0.19	0.18	0.27	0.29	0.27	0.22	0.18
01:00:00	0.19	0.19	0.29	0.31	0.31	0.31	0.30

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.73	0	4.61	4.34	0	0	0
00:30:00	8.76	3.50	7.81	9.50	12.44	3.81	0.61
00:45:00	11.58	3.29	7.13	16.58	16.58	4.10	0.78
01:00:00	10.82	2.37	4.39	16.58	16.58	3.43	0.55

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.522	0	0.674	0.408	0	0	0
00:30:00	0.901	0.320	0.600	1.263	1.537	0.490	0.062
00:45:00	0.796	0.225	0.460	1.574	1.234	0.318	0.053
01:00:00	0.603	0.136	0.309	1.081	1.013	0.196	0.029

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.85	6.25	8.39	8.45	8.18	7.16	7.62	1.61	7.44
00:30:00	6.07	7.26	8.57	8.72	9.01	9.01	10.14	7.36	8.3
00:45:00	6.21	7.25	8.52	9.04	9.18	9.03	10.1	7.36	8.3
01:00:00	6.17	7.19	8.37	9.04	9.18	9	10.02	7.36	8.3

Hydraulic simulation results for solution F_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.06	0.01	0	0	0	0	0
00:30:00	0.09	0.06	0.02	0.01	0	0	0
00:45:00	0.10	0.09	0.07	0.05	0.02	0	0
01:00:00	0.09	0.10	0.10	0.09	0.07	0.03	0.01

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.82	0	4.72	4.36	0	0	0
00:30:00	8.85	3.50	7.83	9.52	12.44	3.81	0.68
00:45:00	11.67	3.28	7.28	16.58	16.58	4.11	0.83
01:00:00	10.91	2.37	4.52	16.58	16.58	3.43	0.69

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.532	0	0.689	0.410	0	0	0
00:30:00	0.911	0.320	0.601	1.266	1.537	0.490	0.069
00:45:00	0.802	0.225	0.471	1.574	1.234	0.319	0.057
01:00:00	0.608	0.135	0.318	1.081	1.013	0.196	0.037

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.86	6.25	8.39	8.45	8.18	7.18	7.62	1.7	7.24
00:30:00	6.08	7.26	8.57	8.72	9.01	9.01	10.14	7.33	8.33
00:45:00	6.21	7.25	8.52	9.04	9.18	9.03	10.1	7.33	8.33
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.01	7.33	8.33

Hydraulic simulation results for solution G_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.01	0	0	0	0	0	0
00:30:00	0.01	0	0	0	0	0	0
00:45:00	0.02	0.01	0.01	0	0	0	0
01:00:00	0.02	0.01	0.01	0.01	0	0	0

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.89	0	4.71	4.36	0	0	0
00:30:00	8.92	3.48	7.93	9.52	12.44	3.82	0.59
00:45:00	11.75	3.29	7.24	16.58	16.58	4.11	0.76
01:00:00	10.99	2.41	4.52	16.58	16.58	3.43	0.60

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.541	0	0.687	0.410	0	0	0
00:30:00	0.919	0.319	0.609	1.266	1.537	0.491	0.060
00:45:00	0.808	0.225	0.468	1.574	1.234	0.319	0.052
01:00:00	0.612	0.138	0.318	1.081	1.013	0.197	0.032

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.86	6.25	8.39	8.45	8.18	7.18	7.63	1.56	7.41
00:30:00	6.08	7.26	8.57	8.72	9.01	9.01	10.14	7.16	8.3
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.1	7.16	8.3
01:00:00	6.18	7.19	8.38	9.04	9.18	9	10.02	7.16	8.3

Hydraulic simulation results for solution H_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0007	0.0001	0.0001	0.0002	0.0002	0.0001	0.0001
00:30:00	0.0023	0.0004	0.0003	0.0004	0.0004	0.0003	0.0002
00:45:00	0.0030	0.0008	0.0004	0.0005	0.0006	0.0005	0.0004
01:00:00	0.0035	0.0014	0.0007	0.0007	0.0007	0.0006	0.0005

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.91	0	4.73	4.36	0	0	0
00:30:00	8.94	3.48	7.97	9.52	12.45	3.81	2.45
00:45:00	11.76	3.29	7.28	16.58	16.58	4.11	5.01
01:00:00	11.01	2.41	4.57	16.58	16.58	3.44	4.80

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.691	0.410	0	0	0
00:30:00	0.920	0.319	0.612	1.265	1.537	0.491	0.249
00:45:00	0.808	0.225	0.470	1.574	1.235	0.318	0.345
01:00:00	0.613	0.138	0.322	1.081	1.013	0.197	0.257

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.86	6.26	8.39	8.45	8.18	7.18	7.62	1.77	7.24
00:30:00	6.08	7.26	8.56	8.72	9.01	9.01	10.11	6.99	8.32
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.13	6.99	8.32
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.11	6.99	8.32

Hydraulic simulation results for solution S_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.26	0.67	4.54	7.72	7.72	7.72
00:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
00:45:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	1.09	0	0	0	0
00:45:00	1.45	0	0.50	11.67	0	0	0
01:00:00	0.78	0	0	10.53	0	0	0

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0.083	0	0	0	0
00:45:00	0.099	0	0.032	1.106	0	0	0
01:00:00	0.043	0	0	0.686	0	0	0

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.15	2.02	4.33	3.56	3.11	2.33	2.19	0	0
00:30:00	4.09	5	8.11	7.07	3.85	4.65	3.79	0	0
00:45:00	5.62	5.76	8.05	8.83	7.37	6.19	4.34	0	0.07
01:00:00	5.55	4.46	4.78	8.78	7.44	5.55	3.96	0	4.1

Hydraulic simulation results for solution T_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.26	3.26	7.06	6.75	5.79	5.59
00:30:00	3.26	3.26	0.26	7.72	6.57	7.72	7.72
00:45:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	1.10	0	12.41	0	0
00:45:00	1.54	0	0.51	12.77	16.58	2.03	1.02
01:00:00	0.86	0	0	11.63	16.58	1.36	0.42

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0.084	0	1.534	0	0
00:45:00	0.105	0	0.033	1.209	1.230	0.158	0.070
01:00:00	0.048	0	0	0.758	1.013	0.077	0.023

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.15	2.12	4.31	2.85	8.17	3.16	2.96	0	7.41
00:30:00	4.13	5.46	8.11	7.97	9.01	8.87	7.86	0	8.19
00:45:00	5.63	5.99	8.05	8.88	9.18	8.95	10.07	1.24	8.19
01:00:00	5.56	5.11	4.8	8.83	9.18	8.93	10.06	1.8	8.19

Hydraulic simulation results for solution U_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.26	3.26	3.21	3.02	2.45	1.51
00:30:00	3.26	3.26	3.26	3.30	3.30	5.14	5.06
00:45:00	3.26	3.26	3.26	3.30	3.30	5.33	5.33
01:00:00	3.26	3.26	3.26	3.30	3.30	5.33	5.33

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	4.32	0	0	0
00:30:00	0	0	1.56	9.48	12.41	0	3.31
00:45:00	1.51	0.91	0.96	16.58	16.58	2.07	3.45
01:00:00	0.84	0.20	0	16.58	16.58	1.39	2.91

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0.406	0	0	0
00:30:00	0	0	0.119	1.261	1.534	0	0.338
00:45:00	0.103	0.062	0.062	1.572	1.231	0.161	0.237
01:00:00	0.046	0.011	0	1.081	1.013	0.080	0.156

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.15	2.95	4.66	8.45	8.17	3.2	7.62	0	7.44
00:30:00	4.12	6.21	8.16	8.73	9.01	8.89	10.08	1.43	8.19
00:45:00	5.62	7.05	8.1	9.04	9.18	8.95	10.08	6.96	8.19
01:00:00	5.56	6.96	5.34	9.04	9.18	8.93	10.08	6.96	8.19

Hydraulic simulation results for solution V_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	2.52	1.17	0.59	0.03	0.00	0.00
00:30:00	3.26	3.25	3.22	3.19	3.12	2.73	2.47
00:45:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26
01:00:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	4.73	4.36	0	0	0
00:30:00	0.92	3.47	7.92	9.51	12.41	3.82	3.31
00:45:00	4.05	3.29	7.29	16.58	16.58	4.10	3.45
01:00:00	3.33	2.41	4.56	16.58	16.58	3.43	2.91

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.691	0.410	0	0	0
00:30:00	0.094	0.319	0.608	1.265	1.534	0.495	0.338
00:45:00	0.277	0.225	0.471	1.572	1.231	0.317	0.237
01:00:00	0.185	0.138	0.321	1.081	1.013	0.196	0.156

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.59	6.26	8.39	8.45	8.17	7.18	7.62	1.74	7.4
00:30:00	5.57	7.26	8.56	8.73	9.01	9.01	10.08	6.91	8.2
00:45:00	5.81	7.25	8.53	9.04	9.18	9.03	10.08	6.91	8.2
01:00:00	5.76	7.19	8.37	9.04	9.18	9	10.08	6.91	8.2

Hydraulic simulation results for solution W_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.85	0.99	0.33	0.11	0	0	0
00:30:00	0.89	1.74	1.67	1.60	1.48	0.90	0.47
00:45:00	0.91	1.83	1.83	1.82	1.81	1.78	1.75
01:00:00	0.91	1.83	1.84	1.84	1.84	1.84	1.84

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.03	0	4.67	4.37	0	0	0
00:30:00	8.03	0.08	7.91	9.51	12.41	3.82	3.31
00:45:00	10.87	2.37	7.28	16.58	16.58	4.10	3.45
01:00:00	10.11	1.50	4.55	16.58	16.58	3.43	2.91

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.444	0	0.682	0.411	0	0	0
00:30:00	0.828	0.007	0.607	1.265	1.534	0.495	0.338
00:45:00	0.746	0.162	0.471	1.572	1.231	0.317	0.237
01:00:00	0.562	0.086	0.320	1.081	1.013	0.196	0.156

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.81	5.57	8.39	8.45	8.18	7.18	7.62	0	7.4
00:30:00	6.04	6.94	8.56	8.73	9.01	9.01	10.08	6.92	8.32
00:45:00	6.18	7.18	8.53	9.04	9.18	9.03	10.08	6.92	8.32
01:00:00	6.14	7.11	8.37	9.04	9.18	9	10.08	6.92	8.32

Hydraulic simulation results for solution X_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.42	0.13	0.01	0	0	0	0
00:30:00	0.47	0.46	0.34	0.28	0.11	0.01	0
00:45:00	0.47	0.50	0.50	0.49	0.47	0.33	0.22
01:00:00	0.47	0.50	0.51	0.51	0.52	0.51	0.50

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.46	0	4.71	4.36	0	0	0
00:30:00	8.48	3.46	7.93	9.52	12.44	3.82	0.65
00:45:00	11.30	3.25	7.25	16.58	16.58	4.11	0.81
01:00:00	10.54	2.34	4.51	16.58	16.58	3.43	0.68

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.492	0	0.687	0.410	0	0	0
00:30:00	0.872	0.318	0.609	1.265	1.537	0.491	0.066
00:45:00	0.776	0.223	0.468	1.574	1.234	0.319	0.056
01:00:00	0.587	0.134	0.317	1.081	1.013	0.197	0.037

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.84	6.25	8.39	8.45	8.18	7.18	7.62	1.65	7.42
00:30:00	6.06	7.26	8.57	8.72	9.01	9.01	10.14	7.32	8.3
00:45:00	6.2	7.25	8.53	9.04	9.18	9.03	10.1	7.32	8.3
01:00:00	6.16	7.18	8.37	9.04	9.18	9	9.92	7.32	8.3

Hydraulic simulation results for solution Y_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.04	0.03	0	0	0	0	0
00:30:00	0.06	0.09	0.05	0.03	0	0	0
00:45:00	0.06	0.13	0.11	0.09	0.06	0.01	0
01:00:00	0.06	0.13	0.13	0.13	0.12	0.08	0.05

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.85	0	4.70	4.36	0	0	0
00:30:00	8.88	3.43	7.92	9.52	12.44	3.81	0.63
00:45:00	11.71	3.22	7.24	16.58	16.58	4.11	0.79
01:00:00	10.95	2.30	4.50	16.58	16.58	3.43	0.56

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.536	0	0.687	0.410	0	0	0
00:30:00	0.914	0.314	0.608	1.266	1.537	0.490	0.064
00:45:00	0.805	0.220	0.468	1.574	1.234	0.319	0.055
01:00:00	0.610	0.132	0.317	1.081	1.013	0.196	0.030

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.86	6.25	8.39	8.45	8.18	7.18	7.63	1.52	7.42
00:30:00	6.08	7.26	8.57	8.72	9.01	9.01	10.14	7.3	8.3
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.1	7.3	8.3
01:00:00	6.18	7.18	8.38	9.04	9.18	9	10.02	7.3	8.3

Hydraulic simulation results for solution Z_{T4}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0	0	0	0	0
00:45:00	0	0	0	0	0	0	0
01:00:00	0	0	0	0	0	0	0

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.91	0	4.74	4.36	0	0	0
00:30:00	8.94	3.48	7.97	9.52	12.45	3.81	4.28
00:45:00	11.77	3.29	7.26	16.58	16.58	4.11	4.17
01:00:00	11.01	2.41	4.54	16.58	16.58	3.44	4.42

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.692	0.410	0	0	0
00:30:00	0.920	0.319	0.611	1.265	1.537	0.491	0.435
00:45:00	0.809	0.225	0.469	1.575	1.235	0.318	0.287
01:00:00	0.613	0.138	0.319	1.081	1.013	0.197	0.238

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.87	6.26	8.39	8.45	8.18	7.18	7.62	1.79	7.24
00:30:00	6.08	7.26	8.57	8.72	9.01	9.01	10.1	7.06	8.31
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.12	7.06	8.31
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.11	7.06	8.31

C.3 Hydraulic simulation results for 2 hr & 30 min

Hydraulic simulation results for solution A_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	1.64	0.85	1.68	3.47	2.92	2.68	2.28
00:30:00	1.70	1.64	2.86	4.92	4.83	6.55	6.83
00:45:00	1.73	1.73	3.02	5.17	5.16	7.20	7.61
01:00:00	1.72	1.73	3.03	5.18	5.18	7.25	7.67
01:15:00	1.69	1.71	3.00	5.15	5.16	7.23	7.66
01:30:00	1.66	1.67	2.95	5.05	5.06	6.79	7.26
01:45:00	1.65	1.65	2.92	4.97	4.99	6.26	6.72
02:00:00	1.65	1.65	2.91	4.82	4.85	5.97	6.42
02:15:00	1.65	1.65	2.91	4.66	4.68	5.72	6.17
02:30:00	1.65	1.65	2.91	4.59	4.60	5.62	6.05

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	3.24	0	3.43	1.12	0	0	0
00:30:00	7.22	3.47	6.61	7.40	12.41	0	2.89
00:45:00	10.05	3.29	5.98	16.58	16.58	2.04	3.02
01:00:00	9.29	2.41	3.27	16.58	16.58	1.37	2.49
01:15:00	4.98	1.31	1.37	15.18	16.58	0	1.39
01:30:00	1.69	0.84	0	7.34	13.14	0	0.42
01:45:00	1.39	0.84	0	2.18	9.53	0	0.42
02:00:00	1.39	0.84	0	0	5.13	0	0.42
02:15:00	1.39	0.84	0	0	5.13	0	0.42
02:30:00	1.38	0.83	0	0	5.11	0	0.25

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.355	0	0.502	0.100	0	0	0
00:30:00	0.743	0.319	0.507	0.985	1.534	0	0.295
00:45:00	0.689	0.225	0.387	1.571	1.231	0.158	0.208
01:00:00	0.517	0.137	0.230	1.081	1.013	0.078	0.133

01:15:00	0.319	0.082	0.128	0.726	0.886	0	0.080
01:30:00	0.158	0.068	0	0.317	0.796	0	0.032
01:45:00	0.166	0.078	0	0.104	0.658	0	0.039
02:00:00	0.172	0.082	0	0	0.543	0	0.043
02:15:00	0.174	0.084	0	0	0.626	0	0.045
02:30:00	0.172	0.085	0	0	0.641	0	0.028

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.76	6.25	8.3	8.21	8.17	3.16	6.36	1.71	7.42
00:30:00	6	7.26	8.49	8.63	9.01	8.78	10.08	6.92	8.19
00:45:00	6.14	7.25	8.46	9.04	9.18	8.95	10.08	6.92	8.19
01:00:00	6.1	7.19	8.29	9.04	9.18	8.93	10.07	6.92	8.19
01:15:00	5.87	7.09	8.14	8.98	9.18	8.87	10.07	6.92	8.19
01:30:00	5.64	7.05	7.94	8.62	9.04	5.98	10.06	6.92	8.19
01:45:00	5.61	7.05	7.9	8.3	8.87	3.9	10.06	6.92	8.19
02:00:00	5.61	7.05	7.89	7.24	8.64	3.26	10.06	6.92	8.19
02:15:00	5.61	7.05	7.88	6.29	8.64	3.08	10.06	6.92	8.19
02:30:00	5.61	7.05	7.88	5.88	8.64	3.03	10.06	6.92	8.19

Hydraulic simulation results for solution B_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.38	0.09	2.42	2.31	1.69	2.09	1.53
00:30:00	0.43	0.35	2.81	2.79	2.74	5.31	5.13
00:45:00	0.43	0.42	2.90	2.89	2.88	6.89	6.86
01:00:00	0.43	0.43	2.89	2.89	2.89	6.81	6.84
01:15:00	0.42	0.42	2.85	2.86	2.87	5.46	5.58
01:30:00	0.41	0.42	2.19	2.29	2.47	3.91	4.06
01:45:00	0.41	0.41	1.68	1.70	1.73	2.82	2.86
02:00:00	0.41	0.41	1.67	1.67	1.67	2.68	2.68
02:15:00	0.41	0.41	1.67	1.67	1.67	2.67	2.67
02:30:00	0.41	0.41	1.67	1.67	1.67	2.67	2.67

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.49	0	2.28	4.35	0	0	0
00:30:00	8.50	3.47	5.43	9.51	12.41	0	3.31
00:45:00	11.35	3.29	4.81	16.58	16.58	0	3.45
01:00:00	10.58	2.41	2.11	16.58	16.58	0	2.91
01:15:00	6.25	1.32	0.24	16.58	16.58	0	1.66
01:30:00	2.91	0.84	0	9.43	13.14	0	0.86
01:45:00	2.63	0.84	0	4.20	9.53	0	0.37
02:00:00	2.63	0.84	0	1.63	5.13	0	0.38
02:15:00	2.63	0.84	0	1.63	5.13	0	0.21
02:30:00	2.57	0.83	0	1.58	5.08	0	0.26

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.496	0	0.334	0.409	0	0	0
00:30:00	0.877	0.319	0.417	1.266	1.534	0	0.338
00:45:00	0.779	0.225	0.311	1.572	1.231	0	0.237
01:00:00	0.589	0.138	0.148	1.081	1.013	0	0.156
01:15:00	0.400	0.082	0.022	0.793	0.886	0	0.096
01:30:00	0.271	0.069	0	0.408	0.797	0	0.065
01:45:00	0.313	0.078	0	0.199	0.658	0	0.034
02:00:00	0.325	0.083	0	0.147	0.543	0	0.039
02:15:00	0.328	0.085	0	0.185	0.626	0	0.023
02:30:00	0.321	0.085	0	0.193	0.638	0	0.030

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.84	6.26	8.22	8.45	8.17	1.51	7.63	1.72	7.42
00:30:00	6.06	7.26	8.43	8.73	9.01	3.97	10.08	6.96	8.2
00:45:00	6.2	7.25	8.39	9.04	9.18	6.45	10.08	6.96	8.2
01:00:00	6.16	7.19	8.2	9.04	9.18	5.89	10.08	6.96	8.2
01:15:00	5.94	7.09	8.01	9.04	9.18	2.88	10.07	6.96	8.2
01:30:00	5.73	7.05	3.36	8.73	9.04	1.37	10.06	6.96	8.2
01:45:00	5.71	7.05	2.63	8.44	8.87	1.17	10.06	6.96	8.2
02:00:00	5.71	7.05	2.62	8.25	8.64	1.14	10.06	6.96	8.2
02:15:00	5.71	7.05	2.62	8.25	8.64	1.14	10.06	6.96	8.2
02:30:00	5.7	7.05	2.61	8.25	8.63	1.13	10.06	6.96	8.2

Hydraulic simulation results for solution C_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	1.42	0.69	1.56	1.35	1.00	0.19	0.02
00:30:00	1.47	1.40	2.61	2.59	2.52	2.29	2.13
00:45:00	1.49	1.48	2.74	2.76	2.75	2.75	2.74
01:00:00	1.48	1.49	2.74	2.76	2.77	2.79	2.79
01:15:00	1.46	1.47	2.72	2.74	2.75	2.77	2.78
01:30:00	1.43	1.44	2.67	2.70	2.70	2.73	2.74
01:45:00	1.42	1.42	2.65	2.67	2.67	2.69	2.70
02:00:00	1.42	1.42	2.65	2.66	2.67	2.68	2.69
02:15:00	1.42	1.42	2.65	2.66	2.67	2.68	2.69
02:30:00	1.42	1.42	2.65	2.66	2.67	2.68	2.69

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	3.47	0	3.48	4.34	0	0	0
00:30:00	7.46	3.47	6.65	9.50	12.41	3.80	3.31
00:45:00	10.29	3.29	6.02	16.58	16.58	4.08	3.45
01:00:00	9.53	2.41	3.30	16.58	16.58	3.42	2.91
01:15:00	5.22	1.31	1.41	16.58	16.58	1.85	1.66
01:30:00	1.92	0.84	0.03	9.41	13.14	0.98	1.08
01:45:00	1.62	0.84	0.03	4.18	9.53	0.98	0.45
02:00:00	1.62	0.84	0.03	1.61	5.13	0.98	0.31
02:15:00	1.62	0.84	0.03	1.61	5.13	0.98	0.19
02:30:00	1.59	0.83	0.02	1.58	5.09	0.62	0.53

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.380	0	0.510	0.406	0	0	0
00:30:00	0.768	0.319	0.510	1.263	1.534	0.492	0.338
00:45:00	0.706	0.225	0.389	1.572	1.231	0.316	0.237
01:00:00	0.530	0.138	0.233	1.081	1.013	0.195	0.156
01:15:00	0.334	0.082	0.131	0.793	0.886	0.104	0.096
01:30:00	0.179	0.068	0.005	0.407	0.796	0.069	0.081
01:45:00	0.193	0.078	0.006	0.199	0.658	0.082	0.041

02:00:00	0.201	0.082	0.006	0.145	0.543	0.089	0.031
02:15:00	0.202	0.084	0.006	0.183	0.626	0.093	0.020
02:30:00	0.198	0.085	0.005	0.193	0.639	0.060	0.059

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.77	6.26	8.3	8.45	8.17	7.12	7.62	1.75	7.34
00:30:00	6.01	7.26	8.49	8.73	9.01	9.01	10.08	6.96	8.21
00:45:00	6.15	7.25	8.46	9.04	9.18	9.03	10.08	6.96	8.21
01:00:00	6.11	7.19	8.29	9.04	9.18	9	10.08	6.96	8.21
01:15:00	5.88	7.09	8.14	9.04	9.18	8.94	10.07	6.96	8.21
01:30:00	5.66	7.05	7.97	8.73	9.04	8.93	10.07	6.96	8.21
01:45:00	5.63	7.05	7.97	8.44	8.87	8.93	10.06	6.96	8.21
02:00:00	5.63	7.05	7.97	8.25	8.64	8.93	10.06	6.96	8.21
02:15:00	5.63	7.05	7.97	8.25	8.64	8.93	10.06	6.96	8.21
02:30:00	5.63	7.05	7.96	8.25	8.63	8.93	10.06	6.96	8.21

Hydraulic simulation results for solution D_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.02	0	1.24	1.10	0.90	0.17	0.01
00:30:00	0.04	0.01	1.30	1.31	1.32	1.29	1.25
00:45:00	0.04	0.02	1.32	1.32	1.32	1.34	1.34
01:00:00	0.04	0.03	1.31	1.33	1.33	1.35	1.35
01:15:00	0.04	0.04	1.31	1.32	1.32	1.34	1.34
01:30:00	0.04	0.04	1.29	1.31	1.31	1.33	1.33
01:45:00	0.04	0.04	1.29	1.30	1.30	1.32	1.32
02:00:00	0.04	0.04	1.29	1.30	1.30	1.32	1.32
02:15:00	0.04	0.04	1.29	1.30	1.30	1.32	1.32
02:30:00	0.04	0.04	1.29	1.30	1.30	1.32	1.32

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.87	0	3.48	4.35	0	0	0
00:30:00	8.89	3.47	6.62	9.50	12.41	3.80	3.31

00:45:00	11.74	3.29	6.00	16.58	16.58	4.08	3.45
01:00:00	10.97	2.41	3.28	16.58	16.58	3.42	2.91
01:15:00	6.63	1.31	1.39	16.58	16.58	1.85	1.74
01:30:00	3.28	0.84	0.01	9.42	13.14	0.98	0.76
01:45:00	3.00	0.84	0.01	4.19	9.53	0.98	0.76
02:00:00	3.00	0.84	0.01	1.62	5.13	0.98	0.76
02:15:00	3.00	0.84	0.01	1.62	5.13	0.98	0.76
02:30:00	2.93	0.83	0.00	1.57	5.07	0.55	0.55

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.537	0	0.509	0.407	0	0	0
00:30:00	0.917	0.319	0.508	1.264	1.534	0.492	0.338
00:45:00	0.806	0.225	0.388	1.572	1.231	0.316	0.237
01:00:00	0.610	0.138	0.231	1.081	1.013	0.195	0.156
01:15:00	0.424	0.082	0.129	0.793	0.886	0.104	0.101
01:30:00	0.306	0.069	0.001	0.407	0.797	0.069	0.057
01:45:00	0.357	0.078	0.001	0.199	0.658	0.082	0.070
02:00:00	0.371	0.082	0.001	0.146	0.543	0.089	0.078
02:15:00	0.374	0.084	0.001	0.184	0.626	0.093	0.082
02:30:00	0.367	0.085	0.001	0.191	0.637	0.054	0.062

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.86	6.26	8.3	8.45	8.17	7.12	7.62	1.72	7.35
00:30:00	6.08	7.26	8.49	8.73	9.01	9.01	10.08	6.93	8.22
00:45:00	6.22	7.25	8.46	9.04	9.18	9.03	10.08	6.93	8.22
01:00:00	6.18	7.19	8.29	9.04	9.18	9	10.08	6.93	8.22
01:15:00	5.96	7.09	8.14	9.04	9.18	8.94	10.07	6.93	8.22
01:30:00	5.76	7.05	7.96	8.73	9.04	8.93	10.06	6.93	8.22
01:45:00	5.74	7.05	7.96	8.44	8.87	8.93	10.06	6.93	8.22
02:00:00	5.74	7.05	7.96	8.25	8.64	8.93	10.06	6.93	8.22
02:15:00	5.74	7.05	7.96	8.25	8.64	8.93	10.06	6.93	8.22
02:30:00	5.73	7.05	7.95	8.24	8.63	8.93	10.06	6.93	8.22

Hydraulic simulation results for solution E_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0006	0.0005	0.2742	0.2358	0.0911	0.0046	0.0008
00:30:00	0.0012	0.0014	0.3275	0.3346	0.3334	0.2385	0.1513
00:45:00	0.0014	0.0025	0.3254	0.3388	0.3493	0.3556	0.3514
01:00:00	0.0015	0.0036	0.3249	0.3373	0.3453	0.3519	0.3544
01:15:00	0.0016	0.0044	0.3233	0.3357	0.3438	0.3500	0.3521
01:30:00	0.0016	0.0050	0.3210	0.3333	0.3415	0.3480	0.3503
01:45:00	0.0016	0.0054	0.3207	0.3325	0.3401	0.3461	0.3482
02:00:00	0.0016	0.0056	0.3210	0.3326	0.3399	0.3455	0.3474
02:15:00	0.0016	0.0058	0.3212	0.3328	0.3401	0.3456	0.3474
02:30:00	0.0016	0.0059	0.3213	0.3329	0.3402	0.3457	0.3476

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.91	0	4.39	4.35	0	0	0
00:30:00	8.94	3.50	7.47	9.51	12.44	3.81	0.72
00:45:00	11.76	3.29	7.07	16.58	16.58	4.10	0.80
01:00:00	11.01	2.37	4.28	16.58	16.58	3.43	0.61
01:15:00	6.67	1.25	2.34	16.58	16.58	2.12	0.35
01:30:00	3.30	0.84	0.95	9.42	13.13	0.82	0.02
01:45:00	3.04	0.84	0.95	4.16	9.52	0.82	0.02
02:00:00	3.04	0.84	0.95	1.62	5.12	0.82	0.02
02:15:00	3.04	0.84	0.95	1.62	5.12	0.82	0.02
02:30:00	2.33	0.55	0.77	1.16	4.46	0.67	0.02

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.640	0.409	0	0	0
00:30:00	0.920	0.321	0.574	1.264	1.536	0.490	0.074
00:45:00	0.809	0.225	0.457	1.574	1.234	0.318	0.055
01:00:00	0.613	0.136	0.301	1.081	1.013	0.196	0.033
01:15:00	0.426	0.079	0.217	0.793	0.886	0.120	0.020
01:30:00	0.308	0.068	0.154	0.407	0.795	0.058	0.002
01:45:00	0.361	0.078	0.176	0.199	0.658	0.069	0.002

02:00:00	0.375	0.082	0.179	0.146	0.542	0.075	0.002
02:15:00	0.378	0.084	0.178	0.183	0.625	0.078	0.002
02:30:00	0.298	0.057	0.148	0.143	0.571	0.065	0.002

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.87	6.26	8.37	8.45	8.17	7.16	7.62	1.82	7.34
00:30:00	6.08	7.26	8.55	8.72	9.01	9.01	10.07	7.31	8.29
00:45:00	6.22	7.25	8.5	9.04	9.18	9.03	10.1	7.31	8.29
01:00:00	6.18	7.19	8.34	9.04	9.18	9	10.02	7.31	8.29
01:15:00	5.97	7.09	8.22	9.04	9.18	8.94	10.05	7.31	8.29
01:30:00	5.76	7.05	8.1	8.73	9.04	8.93	9.98	7.31	8.29
01:45:00	5.74	7.05	8.1	8.45	8.87	8.93	9.98	7.31	8.29
02:00:00	5.74	7.05	8.1	8.25	8.64	8.93	9.98	7.31	8.29
02:15:00	5.74	7.05	8.1	8.25	8.64	8.93	9.98	7.31	8.29
02:30:00	5.7	7.02	8.08	8.22	8.6	8.91	9.94	7.31	8.29

Hydraulic simulation results for solution F_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0002	0.0000	0.0133	0.0069	0.0021	0.0013	0.0013
00:30:00	0.0004	0.0001	0.0256	0.0243	0.0135	0.0052	0.0075
00:45:00	0.0006	0.0002	0.0284	0.0302	0.0293	0.0177	0.0161
01:00:00	0.0008	0.0003	0.0285	0.0317	0.0360	0.0312	0.0298
01:15:00	0.0009	0.0004	0.0284	0.0317	0.0358	0.0378	0.0406
01:30:00	0.0010	0.0006	0.0283	0.0314	0.0346	0.0385	0.0434
01:45:00	0.0010	0.0007	0.0283	0.0313	0.0343	0.0382	0.0433
02:00:00	0.0010	0.0008	0.0284	0.0313	0.0342	0.0380	0.0431
02:15:00	0.0010	0.0009	0.0285	0.0314	0.0342	0.0379	0.0430
02:30:00	0.0010	0.0010	0.0286	0.0315	0.0343	0.0380	0.0430

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.91	0	4.69	4.36	0	0	0
00:30:00	8.94	3.48	7.83	9.52	12.44	3.81	0.97

00:45:00	11.77	3.29	7.31	16.58	16.58	4.10	1.19
01:00:00	11.01	2.41	4.55	16.58	16.58	3.43	1.02
01:15:00	6.67	1.32	2.68	16.58	16.58	2.10	1.60
01:30:00	3.30	0.84	1.24	9.44	13.14	0.34	0.17
01:45:00	3.04	0.84	1.23	4.17	9.53	0.89	0.16
02:00:00	3.04	0.84	1.22	1.63	5.13	0.90	0.16
02:15:00	3.04	0.84	1.22	1.63	5.13	1.09	0.16
02:30:00	2.39	0.63	1.06	1.21	4.52	1.13	0.16

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.685	0.410	0	0	0
00:30:00	0.921	0.319	0.601	1.265	1.536	0.490	0.099
00:45:00	0.809	0.225	0.472	1.574	1.235	0.318	0.082
01:00:00	0.613	0.138	0.320	1.081	1.013	0.196	0.055
01:15:00	0.426	0.083	0.248	0.793	0.886	0.119	0.092
01:30:00	0.308	0.069	0.202	0.408	0.795	0.024	0.013
01:45:00	0.361	0.078	0.229	0.199	0.659	0.074	0.015
02:00:00	0.376	0.082	0.230	0.146	0.542	0.082	0.016
02:15:00	0.378	0.084	0.231	0.184	0.625	0.103	0.017
02:30:00	0.305	0.065	0.204	0.149	0.578	0.111	0.018

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.87	6.25	8.39	8.45	8.18	7.17	7.61	1.56	7.43
00:30:00	6.08	7.26	8.55	8.72	9.01	9.01	10.13	7.04	8.3
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.1	7.04	8.3
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.03	7.04	8.3
01:15:00	5.97	7.09	8.24	9.04	9.18	8.94	9.98	7.04	8.3
01:30:00	5.76	7.05	8.13	8.73	9.04	8.91	9.98	7.04	8.3
01:45:00	5.74	7.05	8.13	8.45	8.87	8.92	9.99	7.04	8.3
02:00:00	5.74	7.05	8.13	8.25	8.64	8.92	9.98	7.04	8.3
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.04	8.3
02:30:00	5.7	7.02	8.11	8.22	8.6	8.93	9.94	7.04	8.3

Hydraulic simulation results for solution G_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0002	0.0001	0.0002	0.0002	0.0003	0.0002	0.0002
00:30:00	0.0004	0.0002	0.0003	0.0004	0.0005	0.0003	0.0004
00:45:00	0.0005	0.0003	0.0004	0.0005	0.0007	0.0005	0.0005
01:00:00	0.0006	0.0004	0.0005	0.0006	0.0008	0.0007	0.0007
01:15:00	0.0007	0.0005	0.0006	0.0007	0.0009	0.0008	0.0009
01:30:00	0.0007	0.0006	0.0007	0.0008	0.0011	0.0010	0.0011
01:45:00	0.0007	0.0007	0.0008	0.0009	0.0011	0.0011	0.0012
02:00:00	0.0007	0.0008	0.0009	0.0010	0.0012	0.0012	0.0013
02:15:00	0.0007	0.0008	0.0009	0.0011	0.0013	0.0013	0.0015
02:30:00	0.0007	0.0008	0.0010	0.0012	0.0014	0.0014	0.0016

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.91	0	4.73	4.36	0	0	0
00:30:00	8.94	3.48	7.97	9.52	12.45	3.81	1.11
00:45:00	11.77	3.29	7.26	16.58	16.58	4.11	1.36
01:00:00	11.01	2.41	4.54	16.58	16.58	3.44	1.24
01:15:00	6.67	1.32	2.72	16.58	16.58	2.09	1.65
01:30:00	3.30	0.84	1.27	9.44	13.14	1.14	0.22
01:45:00	3.04	0.84	1.26	4.17	9.53	0.89	0.22
02:00:00	3.04	0.84	1.27	1.63	5.13	0.65	0.22
02:15:00	3.04	0.84	1.27	1.63	5.13	1.11	0.22
02:30:00	2.42	0.64	1.12	1.23	4.55	0.30	0.22

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.691	0.410	0	0	0
00:30:00	0.920	0.319	0.612	1.265	1.537	0.491	0.113
00:45:00	0.809	0.225	0.469	1.574	1.235	0.318	0.094
01:00:00	0.613	0.138	0.320	1.081	1.013	0.197	0.066
01:15:00	0.426	0.083	0.253	0.793	0.886	0.118	0.096
01:30:00	0.308	0.069	0.206	0.408	0.795	0.080	0.017
01:45:00	0.361	0.078	0.236	0.199	0.659	0.074	0.021

02:00:00	0.376	0.082	0.240	0.147	0.543	0.059	0.023
02:15:00	0.378	0.084	0.240	0.185	0.625	0.105	0.024
02:30:00	0.308	0.066	0.214	0.151	0.581	0.030	0.025

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.87	6.26	8.39	8.45	8.18	7.18	7.62	1.77	7.24
00:30:00	6.08	7.26	8.56	8.72	9.01	9.01	10.11	7.04	8.32
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.1	7.04	8.32
01:00:00	6.18	7.19	8.38	9.04	9.18	9	10.06	7.04	8.32
01:15:00	5.97	7.09	8.25	9.04	9.18	8.94	9.98	7.04	8.32
01:30:00	5.76	7.05	8.13	8.73	9.04	8.93	9.99	7.04	8.32
01:45:00	5.74	7.05	8.13	8.45	8.87	8.93	9.98	7.04	8.32
02:00:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.04	8.32
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.04	8.32
02:30:00	5.7	7.02	8.12	8.22	8.6	8.92	9.95	7.04	8.32

Hydraulic simulation results for solution H_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0001	0.0001	0.0001	0.0002	0.0002	0.0001	0.0001
00:30:00	0.0001	0.0001	0.0002	0.0003	0.0004	0.0003	0.0002
00:45:00	0.0001	0.0002	0.0002	0.0004	0.0005	0.0005	0.0003
01:00:00	0.0001	0.0002	0.0003	0.0004	0.0006	0.0006	0.0005
01:15:00	0.0001	0.0002	0.0003	0.0005	0.0007	0.0007	0.0006
01:30:00	0.0001	0.0002	0.0003	0.0005	0.0007	0.0007	0.0007
01:45:00	0.0001	0.0002	0.0004	0.0005	0.0007	0.0008	0.0008
02:00:00	0.0001	0.0003	0.0004	0.0005	0.0007	0.0008	0.0008
02:15:00	0.0001	0.0003	0.0004	0.0005	0.0007	0.0009	0.0009
02:30:00	0.0001	0.0003	0.0004	0.0006	0.0008	0.0009	0.0009

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.91	0	4.74	4.36	0	0	0
00:30:00	8.94	3.48	7.96	9.52	12.45	3.81	4.24

00:45:00	11.77	3.29	7.26	16.58	16.58	4.11	4.16
01:00:00	11.01	2.41	4.54	16.58	16.58	3.44	3.95
01:15:00	6.67	1.32	2.71	16.58	16.58	2.06	1.66
01:30:00	3.30	0.84	1.27	9.43	13.14	0.70	0.32
01:45:00	3.04	0.84	1.25	4.17	9.53	1.18	0.46
02:00:00	3.04	0.84	1.25	1.63	5.13	1.14	0.45
02:15:00	3.04	0.84	1.27	1.63	5.13	1.03	0.45
02:30:00	2.52	0.63	1.14	1.29	4.64	0.49	0.45

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.693	0.410	0	0	0
00:30:00	0.921	0.319	0.611	1.265	1.537	0.491	0.431
00:45:00	0.809	0.225	0.469	1.575	1.235	0.318	0.286
01:00:00	0.613	0.138	0.319	1.081	1.013	0.197	0.212
01:15:00	0.426	0.083	0.251	0.793	0.886	0.117	0.096
01:30:00	0.308	0.069	0.205	0.408	0.795	0.049	0.024
01:45:00	0.361	0.078	0.234	0.200	0.659	0.098	0.043
02:00:00	0.376	0.082	0.236	0.147	0.543	0.104	0.046
02:15:00	0.379	0.084	0.239	0.185	0.625	0.098	0.049
02:30:00	0.319	0.065	0.217	0.159	0.590	0.048	0.051

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.87	6.26	8.39	8.45	8.18	7.18	7.62	1.78	7.24
00:30:00	6.08	7.26	8.57	8.72	9.01	9.01	10.09	7.05	8.31
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.12	7.05	8.31
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.12	7.05	8.31
01:15:00	5.97	7.09	8.25	9.04	9.18	8.94	9.98	7.05	8.31
01:30:00	5.76	7.05	8.13	8.73	9.04	8.9	10.03	7.05	8.31
01:45:00	5.74	7.05	8.13	8.45	8.87	8.93	9.98	7.05	8.31
02:00:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.05	8.31
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.05	8.31
02:30:00	5.71	7.03	8.12	8.23	8.61	8.93	9.95	7.05	8.31

Hydraulic simulation results for solution S_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.26	0.67	4.50	7.72	7.72	7.72
00:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
00:45:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:15:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:45:00	3.26	3.26	0	5.67	7.72	7.72	7.72
02:00:00	3.26	3.26	0.50	2.26	7.72	7.72	7.72
02:15:00	3.05	3.26	0.78	2.47	7.72	7.72	7.72
02:30:00	3.04	3.26	0.92	2.56	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	1.09	0	0	0	0
00:45:00	1.45	0	0.50	11.67	0	0	0
01:00:00	0.78	0	0	10.53	0	0	0
01:15:00	0	0	0	8.05	0	0	0
01:30:00	0	0	0	0.55	0	0	0
01:45:00	0	0	0	0	0	0	0
02:00:00	0	0	0	0	0	0	0
02:15:00	0	0	0	0	0	0	0
02:30:00	0	0	0	0	0	0	0

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0.083	0	0	0	0
00:45:00	0.099	0	0.032	1.106	0	0	0
01:00:00	0.043	0	0	0.686	0	0	0
01:15:00	0	0	0	0.385	0	0	0
01:30:00	0	0	0	0.024	0	0	0
01:45:00	0	0	0	0	0	0	0

02:00:00	0	0	0	0	0	0	0
02:15:00	0	0	0	0	0	0	0
02:30:00	0	0	0	0	0	0	0

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.15	1.6	4.33	3.56	3.11	2.2	2.19	0	0
00:30:00	4.09	3.02	8.11	7.07	3.86	4.07	3.47	0	0
00:45:00	5.62	2.98	8.05	8.83	7.4	4.99	3.82	0	0.09
01:00:00	5.55	2.4	4.78	8.78	7.46	4.46	3.52	0	4.49
01:15:00	3.48	1.95	2.67	8.66	7.24	3.05	2.8	0	6.36
01:30:00	2.15	1.88	1.96	8.14	4.18	2.69	2.67	0	6.38
01:45:00	1.84	1.88	1.96	4.72	3.38	2.69	2.67	0	6.38
02:00:00	1.6	1.88	1.96	2.48	2.7	2.69	2.67	0	6.38
02:15:00	1.48	1.88	1.96	2.56	2.78	2.69	2.67	0	6.38
02:30:00	1.47	1.88	1.96	2.56	2.78	2.68	2.66	0	6.38

Hydraulic simulation results for solution T_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.26	3.03	2.61	7.72	7.29	6.87
00:30:00	3.26	3.26	0	0	7.72	7.72	7.72
00:45:00	3.26	3.26	0	0	7.72	7.72	7.72
01:00:00	3.26	3.26	0	0	7.72	7.72	7.72
01:15:00	3.26	3.26	0	0	7.72	7.72	7.72
01:30:00	3.26	3.26	0	0	7.72	7.72	7.72
01:45:00	3.26	3.26	0	0	7.72	7.72	7.72
02:00:00	3.26	3.26	2.87	2.86	7.72	7.72	7.72
02:15:00	3.05	3.26	2.56	2.57	7.72	7.72	7.72
02:30:00	3.04	3.26	2.59	2.59	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	4.35	0	0	0
00:30:00	0	0	3.04	9.53	0	3.82	1.29

00:45:00	1.50	0	2.43	16.58	0	4.10	0.88
01:00:00	0.82	0	0	16.58	0	3.43	0.91
01:15:00	0	0	0	16.58	0	1.87	1.66
01:30:00	0	0	0	9.43	0	1.09	0.22
01:45:00	0	0	0	4.17	0	1.01	0.40
02:00:00	0	0	0	1.63	0	1.00	0.88
02:15:00	0	0	0	1.63	0	1.00	0.86
02:30:00	0	0	0	1.60	0	0.69	0.31

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0.408	0	0	0
00:30:00	0	0	0.234	1.266	0	0.491	0.131
00:45:00	0.102	0	0.157	1.574	0	0.318	0.060
01:00:00	0.046	0	0	1.081	0	0.196	0.049
01:15:00	0	0	0	0.793	0	0.106	0.096
01:30:00	0	0	0	0.408	0	0.077	0.017
01:45:00	0	0	0	0.199	0	0.084	0.036
02:00:00	0	0	0	0.147	0	0.091	0.089
02:15:00	0	0	0	0.185	0	0.095	0.093
02:30:00	0	0	0	0.196	0	0.067	0.035

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.15	2.01	6.38	8.45	2.76	7.18	7.61	0	0
00:30:00	4.14	5.08	8.27	8.73	3.84	9.01	10.1	0	0
00:45:00	5.62	5.82	8.23	9.04	7.35	9.03	10.09	0	0.05
01:00:00	5.55	4.51	7.83	9.04	7.43	9	10.07	0	3.78
01:15:00	3.5	2.46	3.83	9.04	7.19	8.94	10.02	0	5.12
01:30:00	2.15	1.92	2.1	8.73	4.13	8.93	10.03	0	5.12
01:45:00	1.84	1.92	2.1	8.44	3.34	8.93	10.02	0	5.12
02:00:00	1.61	1.92	2.1	8.25	2.47	8.93	10.06	0	5.12
02:15:00	1.48	1.92	2.1	8.25	2.6	8.93	10.07	0	5.12
02:30:00	1.47	1.92	2.1	8.25	2.58	8.93	10.07	0	5.12

Hydraulic simulation results for solution U_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.26	3.26	6.97	6.50	4.60	3.15
00:30:00	3.26	3.26	0.44	7.72	7.72	7.72	7.72
00:45:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:15:00	3.26	3.26	0.06	7.72	7.72	7.72	7.72
01:30:00	3.26	3.26	0.32	7.72	7.72	7.72	7.72
01:45:00	3.26	3.26	1.54	7.72	7.72	7.72	7.72
02:00:00	3.26	3.26	3.26	4.99	5.20	5.68	5.98
02:15:00	3.05	3.26	3.26	4.89	4.89	4.89	4.89
02:30:00	3.04	3.26	3.26	4.89	4.89	4.89	4.89

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	3.64	2.07	12.41	3.82	3.31
00:45:00	1.71	0	3.02	13.34	16.58	4.10	3.45
01:00:00	1.02	0	0.38	12.18	16.58	3.44	2.91
01:15:00	0	0	0	9.67	16.58	1.87	1.66
01:30:00	0	0	0	2.05	13.14	1.00	0.92
01:45:00	0	0	0	0	9.53	1.00	0.84
02:00:00	0	0	0	0	5.13	1.00	0.94
02:15:00	0	0	0	0	5.13	1.00	0.88
02:30:00	0	0	0	0	5.10	0.71	0.37

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0.279	0.279	1.534	0.494	0.338
00:45:00	0.117	0	0.195	1.263	1.230	0.317	0.237
01:00:00	0.057	0	0.027	0.794	1.013	0.196	0.156
01:15:00	0	0	0	0.462	0.886	0.105	0.096
01:30:00	0	0	0	0.089	0.796	0.070	0.069
01:45:00	0	0	0	0	0.658	0.084	0.077

02:00:00	0	0	0	0	0.543	0.091	0.096
02:15:00	0	0	0	0	0.626	0.095	0.095
02:30:00	0	0	0	0	0.640	0.069	0.041

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.17	2.46	7.47	3	8.17	7.18	7.62	0	7.41
00:30:00	4.2	6.06	8.31	8.29	9.01	9.01	10.08	0.16	8.19
00:45:00	5.64	6.11	8.27	8.91	9.18	9.03	10.08	5.31	8.19
01:00:00	5.58	6.28	8.03	8.86	9.18	9	10.08	6.61	8.19
01:15:00	3.63	3.52	4.67	8.74	9.18	8.94	10.07	6.61	8.19
01:30:00	2.17	2.14	2.24	8.29	9.04	8.93	10.07	6.61	8.19
01:45:00	1.85	2.12	2.24	6	8.87	8.93	10.07	6.61	8.19
02:00:00	1.61	2.12	2.24	1.52	8.64	8.93	10.06	6.61	8.19
02:15:00	1.48	2.12	2.24	1.47	8.64	8.93	10.07	6.61	8.19
02:30:00	1.48	2.12	2.23	1.47	8.63	8.93	10.07	6.61	8.19

Hydraulic simulation results for solution V_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	3.26	3.26	2.847	2.56	1.25	0.13	0
00:30:00	3.26	3.26	3.26	3.26	3.25	3.23	3.21
00:45:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26
01:00:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26
01:15:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26
01:30:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26
01:45:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26
02:00:00	3.26	3.26	3.26	3.26	3.26	3.26	3.26
02:15:00	3.06	3.26	3.26	3.26	3.26	3.26	3.26
02:30:00	3.04	3.26	3.26	3.26	3.26	3.26	3.26

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	3.96	4.36	0	0	0
00:30:00	0	0	7.15	9.51	12.41	3.82	3.31

00:45:00	2.60	2.21	6.52	16.58	16.58	4.10	3.45
01:00:00	1.90	1.34	3.80	16.58	16.58	3.43	2.91
01:15:00	0	0.29	1.90	16.58	16.58	1.87	1.66
01:30:00	0	0	0.52	9.43	13.14	1.00	0.82
01:45:00	0	0	0.52	4.20	9.53	1.00	0.88
02:00:00	0	0	0.52	1.63	5.13	1.00	0.15
02:15:00	0	0	0.52	1.63	5.13	1.00	0.67
02:30:00	0	0	0.49	1.60	5.09	0.63	0.66

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.580	0.408	0	0	0
00:30:00	0	0	0.549	1.265	1.534	0.494	0.338
00:45:00	0.178	0.151	0.422	1.572	1.231	0.317	0.237
01:00:00	0.105	0.077	0.267	1.081	1.013	0.196	0.156
01:15:00	0	0.018	0.177	0.793	0.886	0.105	0.096
01:30:00	0	0	0.084	0.408	0.797	0.070	0.062
01:45:00	0	0	0.097	0.199	0.658	0.084	0.081
02:00:00	0	0	0.097	0.147	0.543	0.091	0.016
02:15:00	0	0	0.097	0.185	0.626	0.095	0.073
02:30:00	0	0	0.093	0.195	0.639	0.061	0.073

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	2.29	5.05	8.34	8.45	8.17	7.18	7.63	0	7.34
00:30:00	4.66	6.48	8.52	8.73	9.01	9.01	10.08	6.48	8.19
00:45:00	5.71	7.17	8.49	9.04	9.18	9.03	10.08	6.93	8.19
01:00:00	5.66	7.1	8.32	9.04	9.18	9	10.08	6.93	8.19
01:15:00	4.13	6.98	8.19	9.04	9.18	8.94	10.07	6.93	8.19
01:30:00	2.3	6.24	8.05	8.73	9.04	8.93	10.06	6.93	8.19
01:45:00	1.88	5.54	8.05	8.44	8.87	8.93	10.07	6.93	8.19
02:00:00	1.65	5.22	8.05	8.25	8.64	8.93	10.06	6.93	8.19
02:15:00	1.5	5.07	8.05	8.25	8.64	8.93	10.06	6.93	8.19
02:30:00	1.49	5	8.04	8.25	8.63	8.93	10.06	6.93	8.19

Hydraulic simulation results for solution W_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	1.02	0.43	1.44	1.24	0.97	0.17	0.01
00:30:00	1.05	0.99	2.22	2.19	2.14	1.92	1.79
00:45:00	1.07	1.06	2.34	2.34	2.34	2.32	2.31
01:00:00	1.06	1.06	2.34	2.35	2.36	2.36	2.36
01:15:00	1.04	1.05	2.32	2.33	2.34	2.35	2.35
01:30:00	1.02	1.03	2.29	2.29	2.31	2.32	2.32
01:45:00	1.02	1.02	2.27	2.27	2.28	2.29	2.29
02:00:00	1.02	1.02	2.27	2.27	2.28	2.28	2.28
02:15:00	1.02	1.02	2.27	2.27	2.28	2.28	2.28
02:30:00	1.02	1.02	2.27	2.27	2.28	2.28	2.28

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	3.88	0	3.46	4.36	0	0	0
00:30:00	7.87	3.47	6.62	9.51	12.40	3.82	3.31
00:45:00	10.71	3.29	6.00	16.58	16.58	4.10	3.45
01:00:00	9.95	2.41	3.28	16.58	16.58	3.43	2.91
01:15:00	5.63	1.31	1.39	16.58	16.58	1.87	1.66
01:30:00	2.31	0.84	0.01	9.43	13.13	1.00	0.91
01:45:00	2.02	0.84	0.01	4.20	9.52	1.00	0.58
02:00:00	2.02	0.84	0.01	1.63	5.12	1.00	0.56
02:15:00	2.02	0.84	0.01	1.63	5.12	1.00	1.40
02:30:00	2.01	0.84	0.01	1.62	5.11	0.92	0.25

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.426	0	0.507	0.408	0	0	0
00:30:00	0.812	0.319	0.508	1.266	1.533	0.494	0.338
00:45:00	0.735	0.225	0.388	1.572	1.231	0.317	0.237
01:00:00	0.553	0.138	0.231	1.081	1.013	0.196	0.156
01:15:00	0.360	0.082	0.129	0.793	0.886	0.105	0.096
01:30:00	0.216	0.069	0.002	0.408	0.796	0.070	0.068
01:45:00	0.241	0.078	0.002	0.199	0.657	0.084	0.053

02:00:00	0.250	0.082	0.002	0.147	0.542	0.091	0.058
02:15:00	0.252	0.084	0.002	0.185	0.625	0.095	0.151
02:30:00	0.251	0.085	0.002	0.198	0.641	0.089	0.028

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.8	6.26	8.3	8.45	8.17	7.18	7.62	1.7	7.42
00:30:00	6.03	7.26	8.49	8.73	9.01	9.01	10.08	6.93	8.19
00:45:00	6.17	7.25	8.46	9.04	9.18	9.03	10.08	6.93	8.19
01:00:00	6.13	7.19	8.29	9.04	9.18	9	10.08	6.93	8.19
01:15:00	5.91	7.09	8.14	9.04	9.18	8.94	10.07	6.93	8.19
01:30:00	5.69	7.05	7.96	8.73	9.04	8.93	10.06	6.93	8.19
01:45:00	5.67	7.05	7.96	8.44	8.87	8.93	10.06	6.93	8.19
02:00:00	5.67	7.05	7.96	8.25	8.64	8.93	10.06	6.93	8.19
02:15:00	5.67	7.05	7.96	8.25	8.64	8.93	10.07	6.93	8.19
02:30:00	5.66	7.05	7.96	8.25	8.64	8.93	10.06	6.93	8.19

Hydraulic simulation results for solution X_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0	0.04	0.27	0.21	0.05	0.02	0.00
00:30:00	0	0.10	0.38	0.37	0.35	0.29	0.20
00:45:00	0.0001	0.12	0.41	0.41	0.40	0.45	0.44
01:00:00	0.0001	0.12	0.42	0.42	0.42	0.47	0.47
01:15:00	0.0001	0.12	0.42	0.42	0.42	0.47	0.47
01:30:00	0.0001	0.12	0.41	0.42	0.42	0.47	0.47
01:45:00	0.0001	0.12	0.41	0.41	0.41	0.47	0.47
02:00:00	0.0001	0.12	0.41	0.41	0.41	0.47	0.47
02:15:00	0.0001	0.12	0.41	0.41	0.41	0.47	0.47
02:30:00	0.0002	0.12	0.41	0.41	0.41	0.47	0.47

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.91	0	4.43	4.36	0	0	0
00:30:00	8.94	3.36	7.56	9.52	12.44	3.76	0.81

00:45:00	11.77	3.17	7.01	16.58	16.58	4.06	0.87
01:00:00	11.01	2.29	4.26	16.58	16.58	3.38	0.78
01:15:00	6.67	1.20	2.38	16.58	16.58	2.05	1.65
01:30:00	3.30	0.72	0.97	9.44	13.14	1.15	0.05
01:45:00	3.04	0.72	0.97	4.17	9.53	0.91	0.04
02:00:00	3.04	0.72	0.96	1.63	5.13	0.25	0.04
02:15:00	3.04	0.72	0.97	1.63	5.13	0.76	0.04
02:30:00	2.34	0.51	0.79	1.18	4.48	0.90	0.04

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.647	0.410	0	0	0
00:30:00	0.920	0.307	0.580	1.265	1.536	0.484	0.082
00:45:00	0.809	0.217	0.453	1.574	1.234	0.314	0.060
01:00:00	0.613	0.131	0.300	1.081	1.013	0.193	0.042
01:15:00	0.426	0.075	0.221	0.793	0.886	0.116	0.096
01:30:00	0.308	0.059	0.158	0.408	0.795	0.081	0.004
01:45:00	0.361	0.067	0.182	0.199	0.659	0.076	0.004
02:00:00	0.376	0.071	0.181	0.147	0.543	0.023	0.004
02:15:00	0.379	0.072	0.182	0.185	0.625	0.072	0.005
02:30:00	0.299	0.052	0.152	0.145	0.573	0.088	0.005

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.87	6.23	8.38	8.45	8.18	7.02	7.62	1.32	7.2
00:30:00	6.08	7.26	8.55	8.72	9.01	9.01	10.14	7.02	8.31
00:45:00	6.22	7.24	8.51	9.04	9.18	9.02	10.1	7.02	8.31
01:00:00	6.18	7.18	8.35	9.04	9.18	9	10.03	7.02	8.31
01:15:00	5.97	7.08	8.23	9.04	9.18	8.94	9.98	7.02	8.31
01:30:00	5.76	7.03	8.1	8.73	9.04	8.94	9.97	7.02	8.31
01:45:00	5.74	7.03	8.1	8.45	8.87	8.92	9.99	7.02	8.31
02:00:00	5.74	7.03	8.1	8.25	8.64	8.91	9.98	7.02	8.31
02:15:00	5.74	7.03	8.1	8.25	8.64	8.93	9.98	7.02	8.31
02:30:00	5.7	7	8.09	8.22	8.6	8.93	9.94	7.02	8.31

Hydraulic simulation results for solution Y_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0729	0.0073	0.0011	0.0004	0.0004	0.0002	0.0001
00:30:00	0.1033	0.0606	0.0219	0.0098	0.0015	0.0005	0.0003
00:45:00	0.1077	0.0990	0.0741	0.0595	0.0307	0.0036	0.0008
01:00:00	0.1046	0.1076	0.1077	0.1002	0.0841	0.0372	0.0166
01:15:00	0.1028	0.1057	0.1109	0.1116	0.1141	0.0975	0.0774
01:30:00	0.1011	0.1038	0.1081	0.1093	0.1125	0.1129	0.1115
01:45:00	0.1003	0.1024	0.1065	0.1071	0.1087	0.1104	0.1114
02:00:00	0.1002	0.1019	0.1057	0.1060	0.1072	0.1083	0.1090
02:15:00	0.1002	0.1017	0.1054	0.1056	0.1065	0.1071	0.1076
02:30:00	0.1002	0.1017	0.1053	0.1055	0.1063	0.1066	0.1069

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.81	0	4.70	4.36	0	0	0
00:30:00	8.84	3.48	7.86	9.52	12.45	3.81	1.26
00:45:00	11.66	3.29	7.33	16.58	16.58	4.11	1.31
01:00:00	10.90	2.41	4.57	16.58	16.58	3.44	1.19
01:15:00	6.57	1.32	2.72	16.58	16.58	2.10	0.61
01:30:00	3.21	0.84	1.27	9.44	13.14	1.12	0.21
01:45:00	2.94	0.84	1.26	4.17	9.53	0.90	0.21
02:00:00	2.94	0.84	1.25	1.63	5.13	1.16	0.21
02:15:00	2.94	0.84	1.25	1.63	5.13	1.10	0.21
02:30:00	2.32	0.78	1.10	1.22	4.54	0.75	0.21

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.531	0	0.687	0.410	0	0	0
00:30:00	0.910	0.319	0.603	1.265	1.536	0.491	0.128
00:45:00	0.802	0.225	0.473	1.574	1.235	0.318	0.091
01:00:00	0.607	0.138	0.322	1.081	1.013	0.197	0.064
01:15:00	0.420	0.083	0.252	0.793	0.886	0.119	0.035
01:30:00	0.299	0.069	0.206	0.408	0.795	0.079	0.016
01:45:00	0.349	0.078	0.235	0.199	0.659	0.075	0.019

02:00:00	0.363	0.082	0.235	0.147	0.543	0.106	0.021
02:15:00	0.366	0.084	0.236	0.185	0.625	0.104	0.022
02:30:00	0.295	0.080	0.211	0.151	0.580	0.073	0.023

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.86	6.25	8.39	8.45	8.18	7.18	7.63	1.55	7.42
00:30:00	6.08	7.26	8.56	8.72	9.01	9.01	10.1	7.03	8.3
00:45:00	6.21	7.25	8.53	9.04	9.18	9.03	10.1	7.03	8.3
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.05	7.03	8.3
01:15:00	5.96	7.09	8.25	9.04	9.18	8.94	10.05	7.03	8.3
01:30:00	5.76	7.05	8.13	8.73	9.04	8.93	9.98	7.03	8.3
01:45:00	5.74	7.05	8.13	8.45	8.87	8.92	9.98	7.03	8.3
02:00:00	5.74	7.05	8.13	8.25	8.64	8.94	9.98	7.03	8.3
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.03	8.3
02:30:00	5.69	7.04	8.12	8.22	8.6	8.92	9.94	7.03	8.3

Hydraulic simulation results for solution Z_{T10}

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.0002	0.0001	0.0002	0.0002	0.0002	0.0002	0.0001
00:30:00	0.0004	0.0002	0.0003	0.0004	0.0004	0.0003	0.0002
00:45:00	0.0006	0.0003	0.0004	0.0005	0.0006	0.0005	0.0004
01:00:00	0.0007	0.0004	0.0005	0.0006	0.0007	0.0006	0.0005
01:15:00	0.0008	0.0005	0.0006	0.0007	0.0008	0.0008	0.0006
01:30:00	0.0008	0.0007	0.0007	0.0008	0.0009	0.0009	0.0008
01:45:00	0.0008	0.0007	0.0008	0.0009	0.001	0.001	0.0009
02:00:00	0.0008	0.0008	0.0009	0.001	0.0011	0.0011	0.001
02:15:00	0.0008	0.0009	0.001	0.0011	0.0012	0.0012	0.0011
02:30:00	0.0008	0.0009	0.001	0.0012	0.0013	0.0013	0.0012

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.91	0	4.73	4.36	0	0	0
00:30:00	8.94	3.48	7.97	9.52	12.45	3.81	1.32

00:45:00	11.77	3.29	7.28	16.58	16.58	4.11	1.37
01:00:00	11.01	2.41	4.58	16.58	16.58	3.44	1.27
01:15:00	6.67	1.32	2.71	16.58	16.58	2.09	2.22
01:30:00	3.30	0.84	1.27	9.44	13.14	0.90	0.48
01:45:00	3.04	0.84	1.26	4.17	9.53	1.08	0.49
02:00:00	3.04	0.84	1.26	1.63	5.13	1.10	0.49
02:15:00	3.04	0.84	1.25	1.63	5.13	1.13	0.49
02:30:00	2.42	0.64	1.12	1.23	4.55	0.31	0.49

Pollution loads at CSO chambers

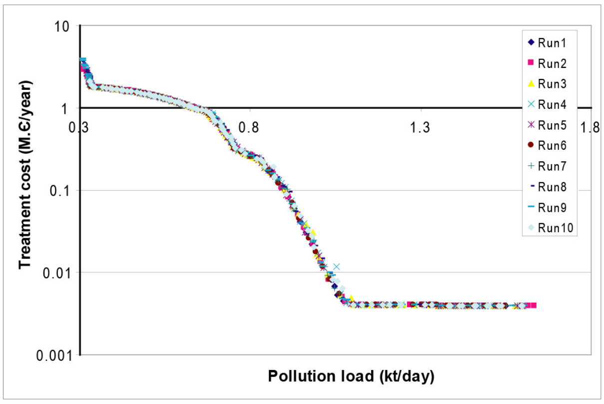
Time	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.542	0	0.691	0.410	0	0	0
00:30:00	0.920	0.319	0.612	1.265	1.537	0.491	0.134
00:45:00	0.809	0.225	0.470	1.574	1.235	0.318	0.094
01:00:00	0.613	0.138	0.323	1.081	1.013	0.197	0.068
01:15:00	0.426	0.083	0.252	0.793	0.886	0.118	0.129
01:30:00	0.308	0.069	0.206	0.408	0.795	0.063	0.036
01:45:00	0.361	0.078	0.236	0.199	0.659	0.090	0.045
02:00:00	0.376	0.082	0.237	0.147	0.543	0.100	0.049
02:15:00	0.378	0.084	0.237	0.185	0.626	0.107	0.052
02:30:00	0.308	0.066	0.213	0.151	0.581	0.030	0.054

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.87	6.26	8.39	8.45	8.18	7.18	7.62	1.76	7.24
00:30:00	6.08	7.26	8.56	8.72	9.01	9.01	10.06	7.04	8.32
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.1	7.04	8.32
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.04	7.04	8.32
01:15:00	5.97	7.09	8.25	9.04	9.18	8.94	10.08	7.04	8.32
01:30:00	5.76	7.05	8.13	8.73	9.04	8.92	10.09	7.04	8.32
01:45:00	5.74	7.05	8.13	8.45	8.87	8.93	10.09	7.04	8.32
02:00:00	5.74	7.05	8.13	8.25	8.64	8.93	10.09	7.04	8.32
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	10.09	7.04	8.32
02:30:00	5.7	7.02	8.12	8.22	8.6	8.92	10.05	7.04	8.32

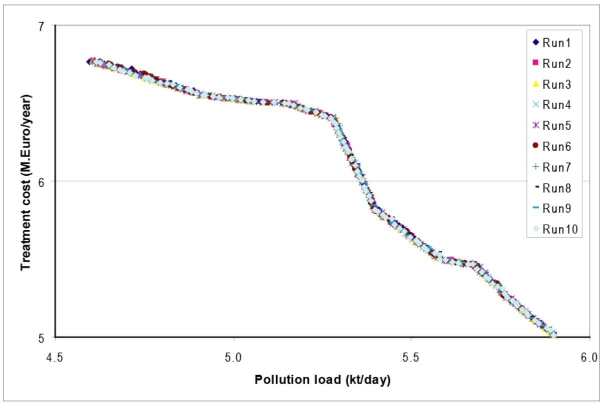
APPENDIX D – RANDOM RUNS AND SELECTED SOLUTIONS FROM DYNAMIC OPTIMIZATION

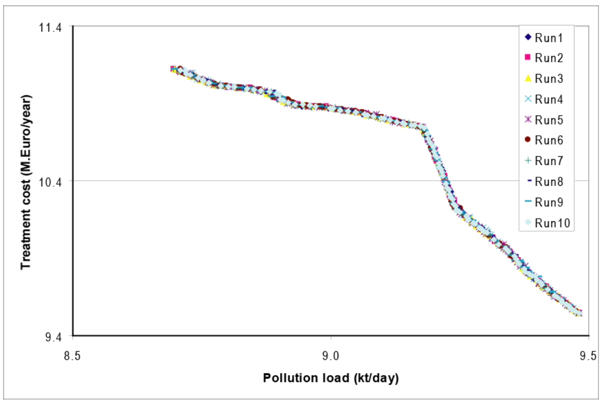
D.1 Random runs for NSGA II constraint handling approach for single storm



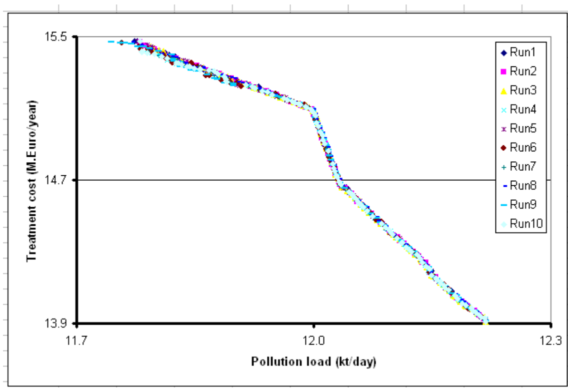
At 15 minutes

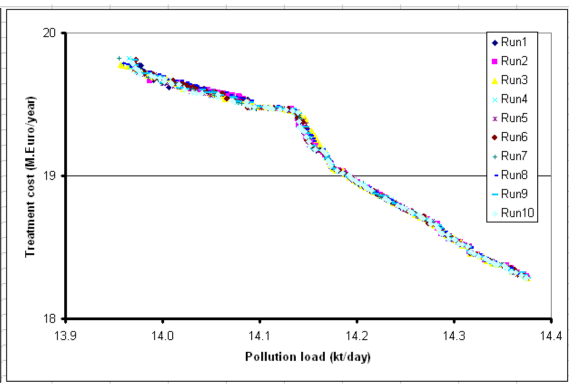
Minimum pollution load Pareto optimal fronts





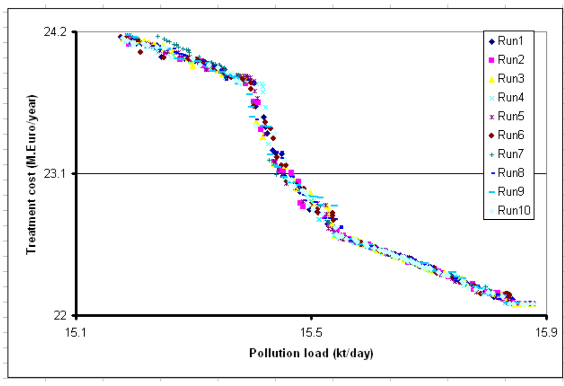
At 30 minutes   At 45 minutes

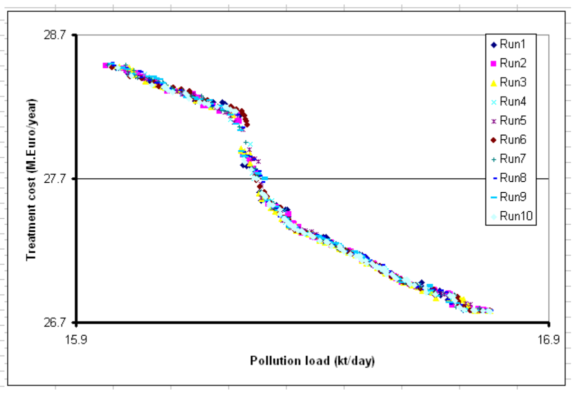




At 1 hour

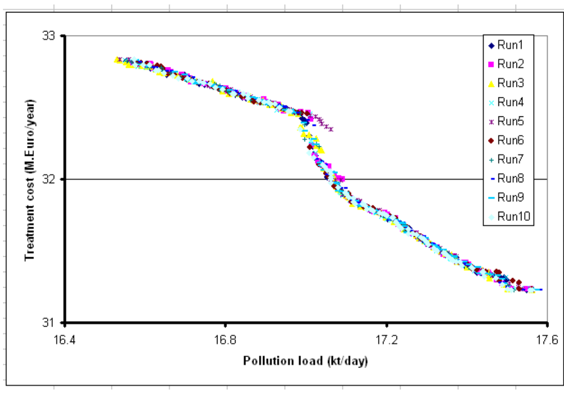
At 1 hour & 15 minutes

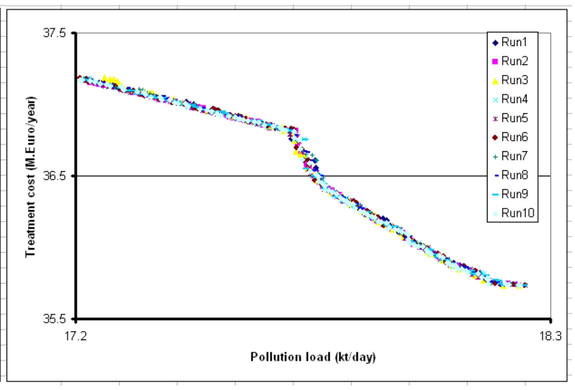




At 1 hour & 30 minutes

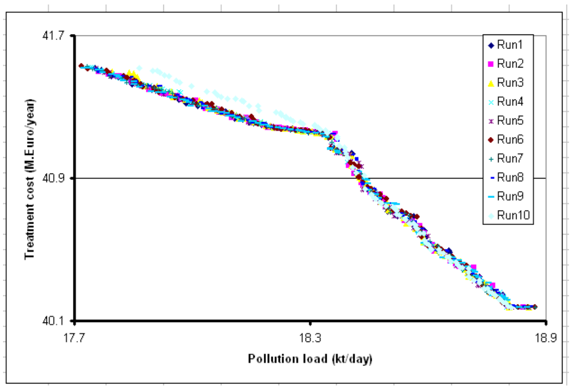
At 1 hour & 45 minutes





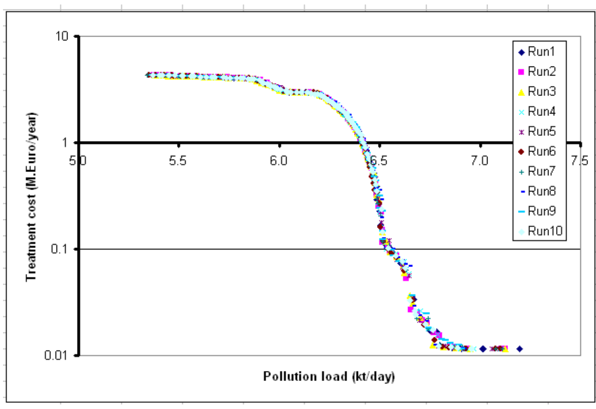
At 2 hours

At 2 hours & 15 minutes

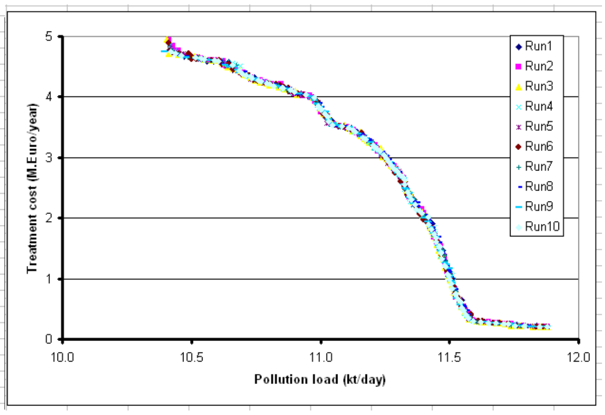


At 2 hours & 30 minutes

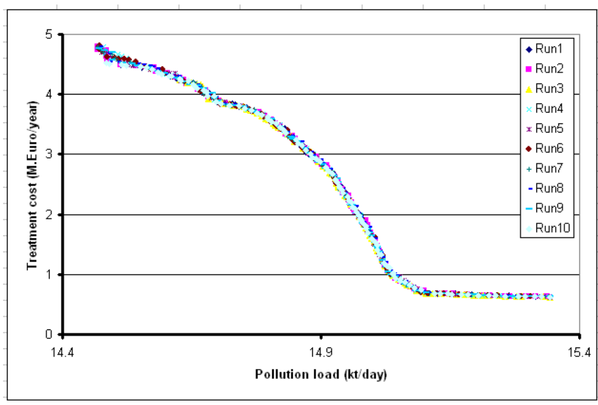
Minimum cost Pareto optimal fronts



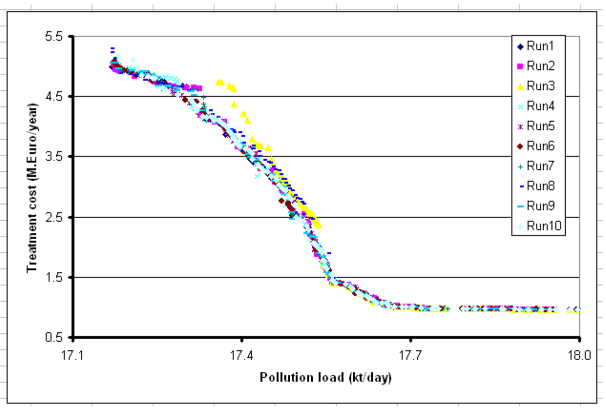
At 30 minutes



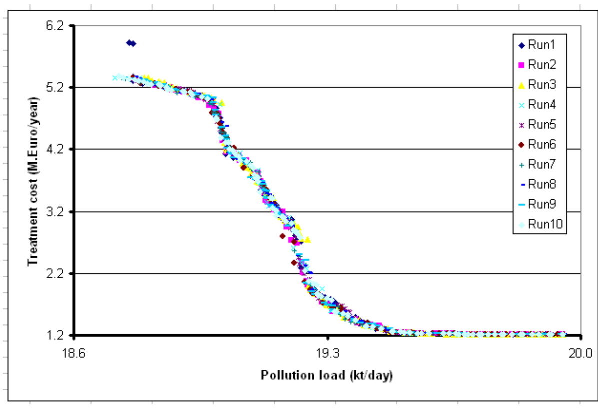
At 45 minutes



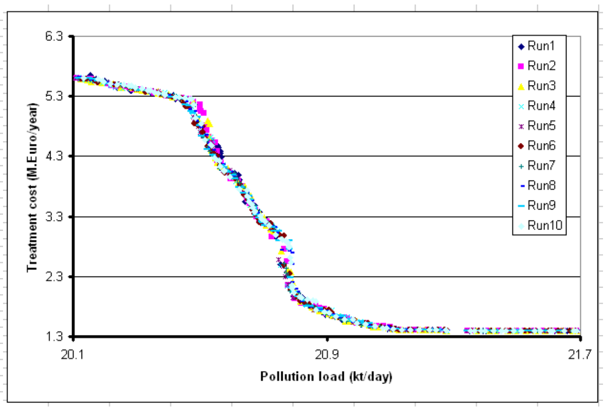
At 1 hour



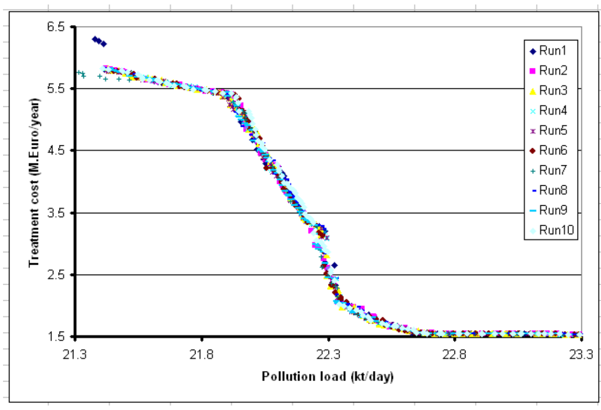
At 1 hour & 15 minutes



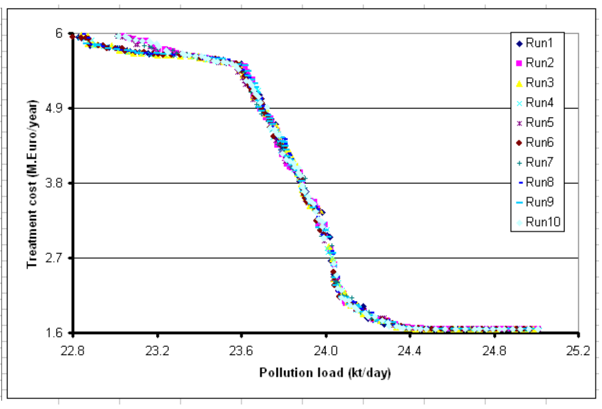
At 1 hour & 30 minutes



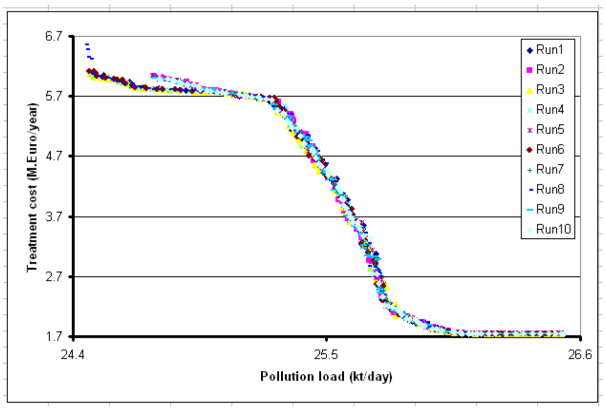
At 1 hour & 45 minutes



At 2 hours

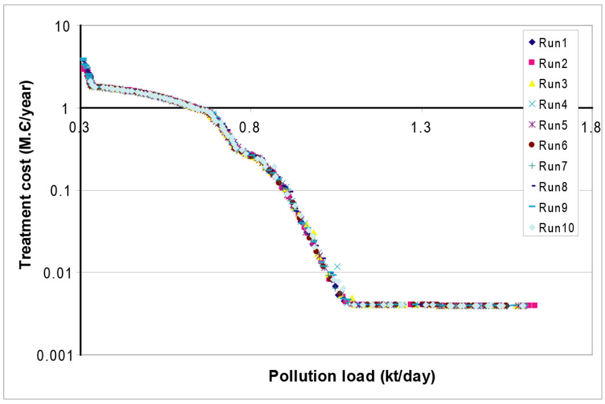


At 2 hours & 15 minutes



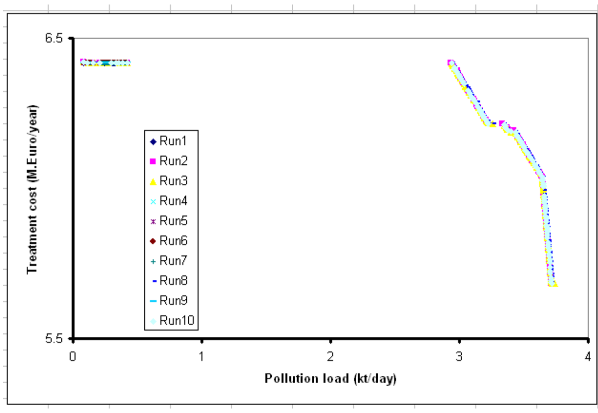
At 2 hours & 30 minutes

D.2 Random runs for SWMM constraint handling approach for single storm

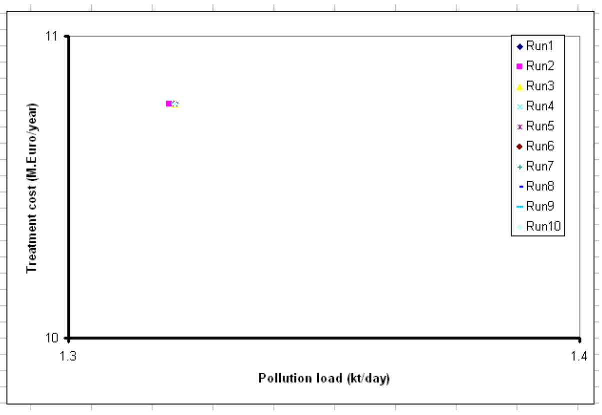


At 15 minutes

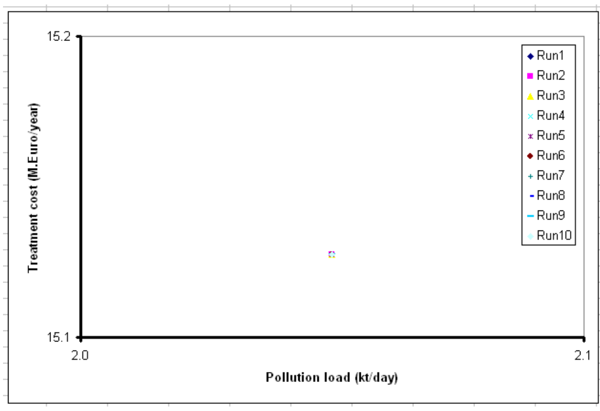
Minimum pollution load Pareto optimal fronts



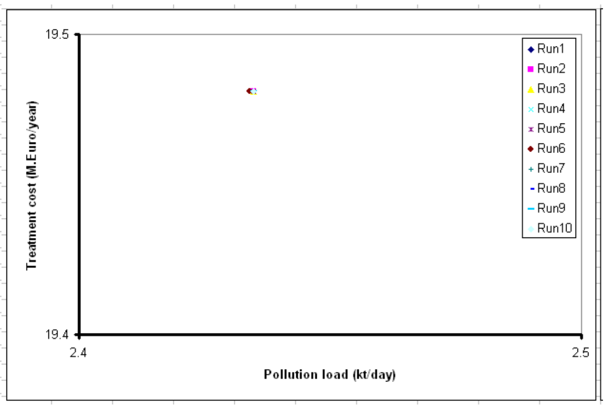
At 30 minutes



At 45 minutes

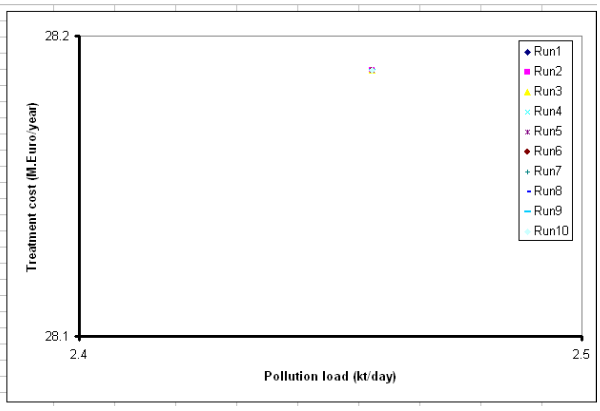


At 1 hour

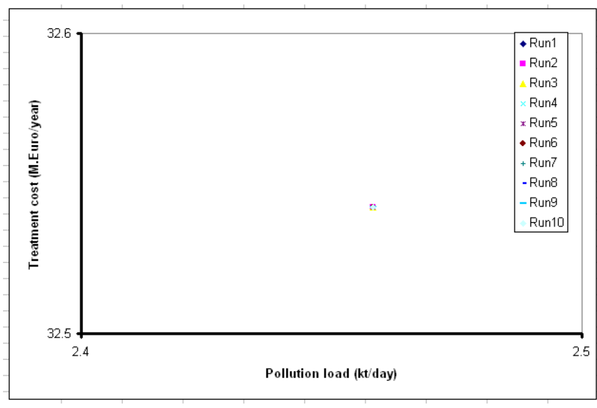


At 1 hour & 15 minutes

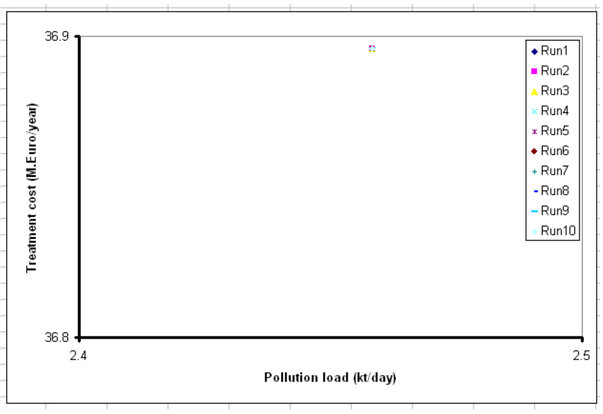
At 1 hour & 30 minutes



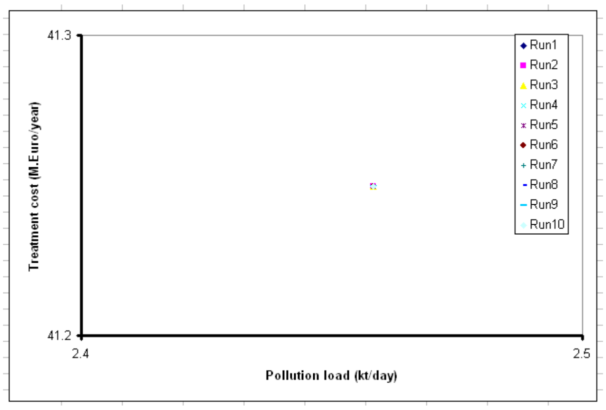
At 1 hour & 45 minutes



At 2 hours

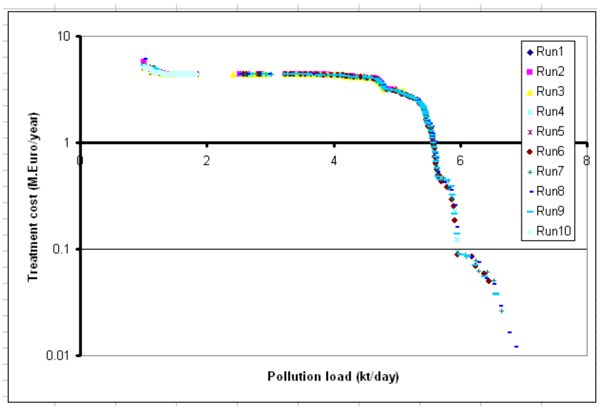


At 2 hours & 15 minutes

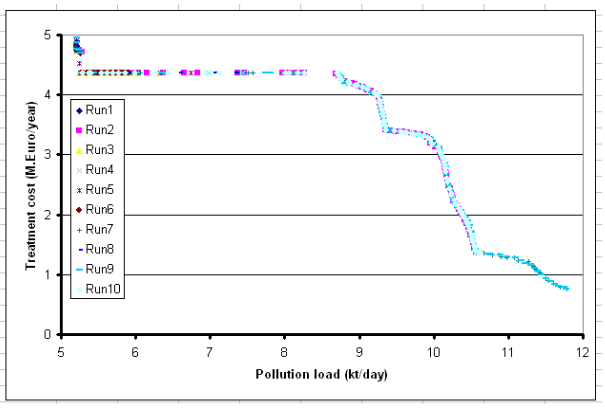


At 2 hours & 30 minutes

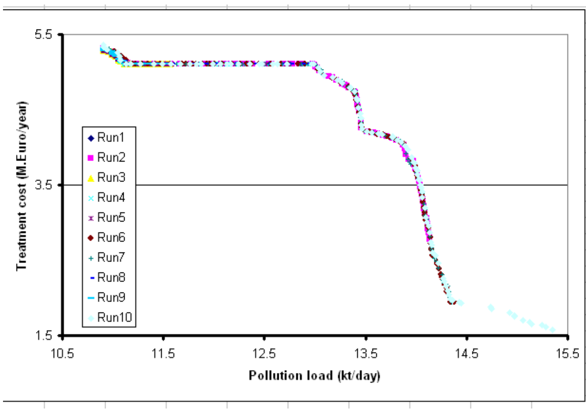
Minimum cost Pareto optimal fronts



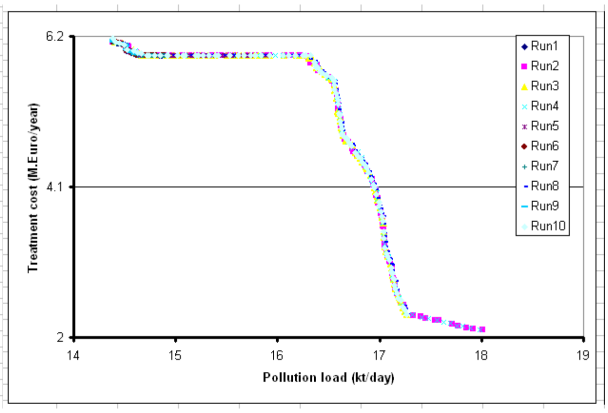
At 30 minutes



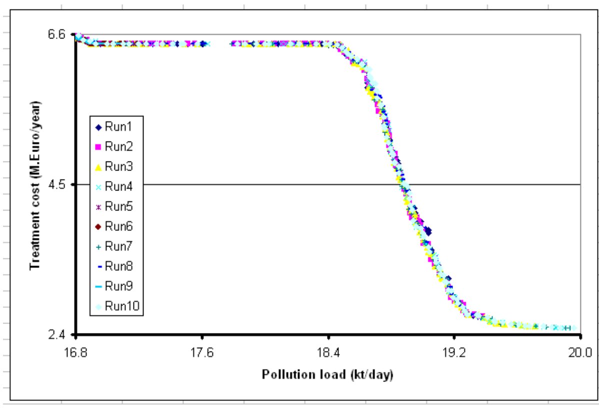
At 45 minutes



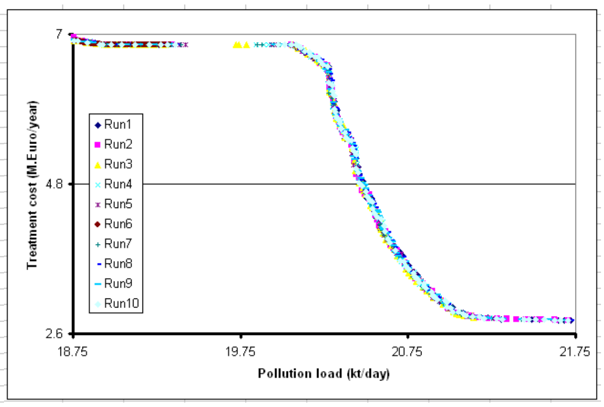
At 1 hour



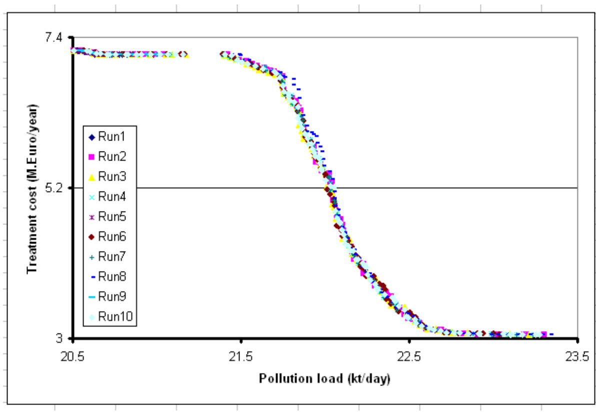
At 1 hour & 15 minutes



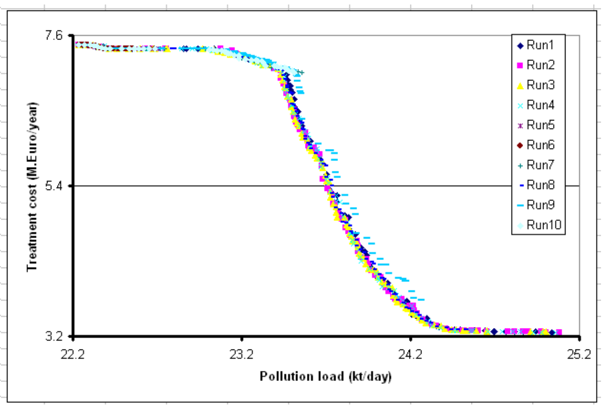
At 1 hour & 30 minutes



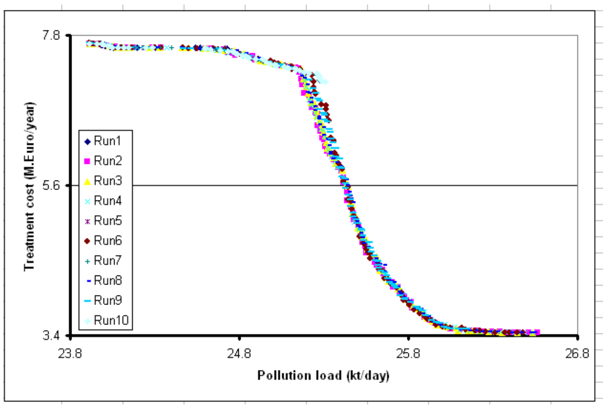
At 1 hour & 45 minutes



At 2 hours

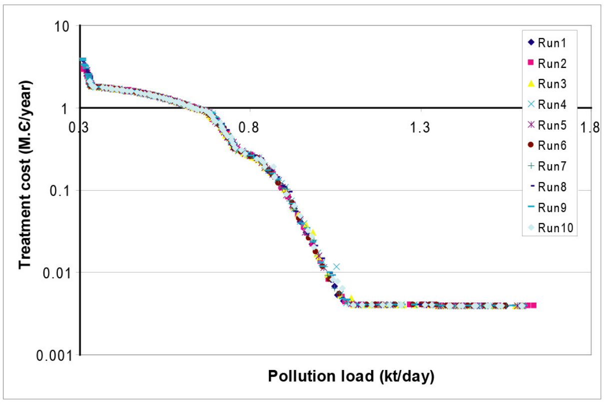


At 2 hours & 15 minutes



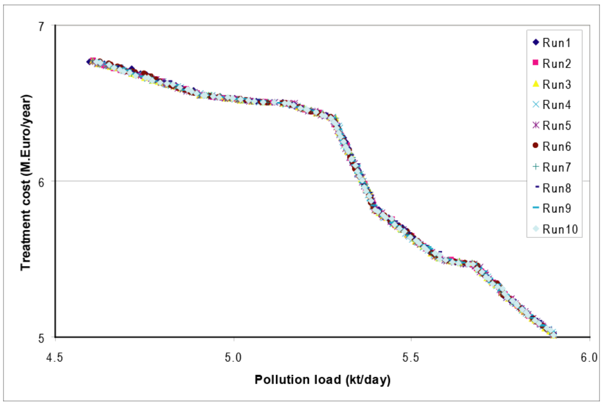
At 2 hours & 30 minutes

D.3 Random runs for NSGA II constraint handling approach for two consecutive storms

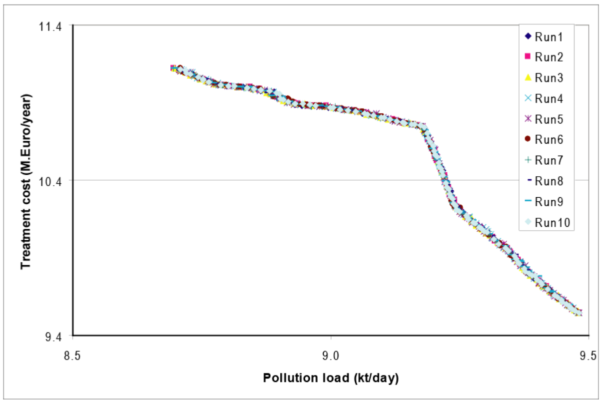


At 15 minutes

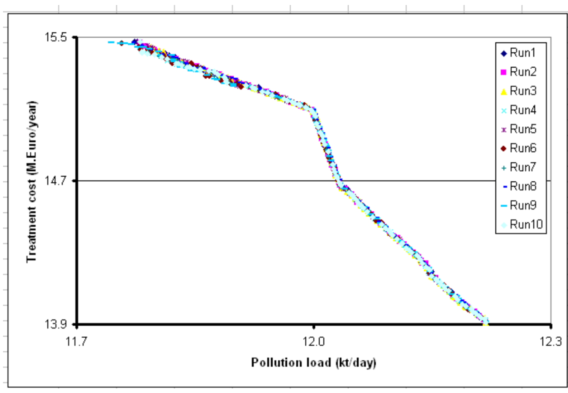
Minimum pollution load Pareto optimal fronts



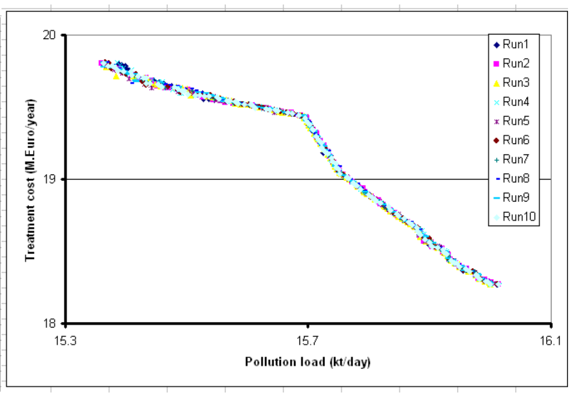
At 30 minutes



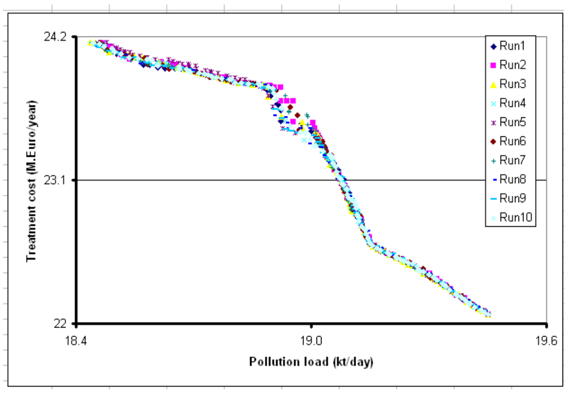
At 45 minutes



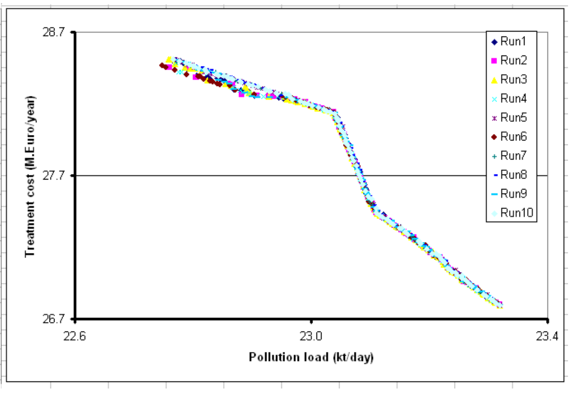
At 1 hour



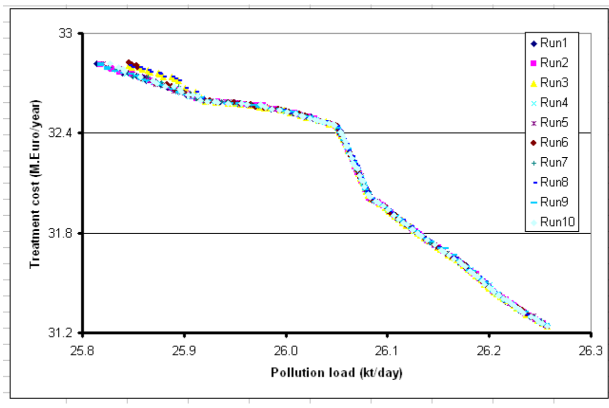
At 1 hour & 15 minutes



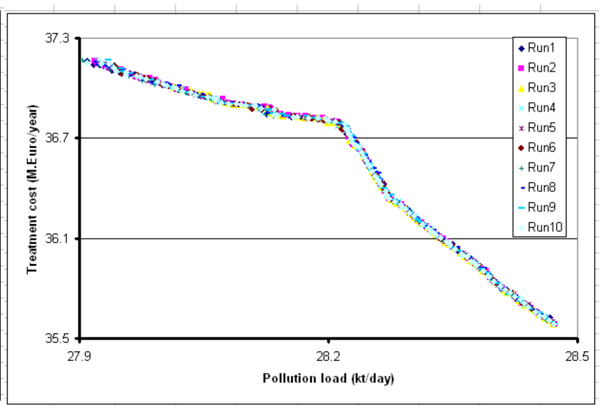
At 1 hour & 30 minutes



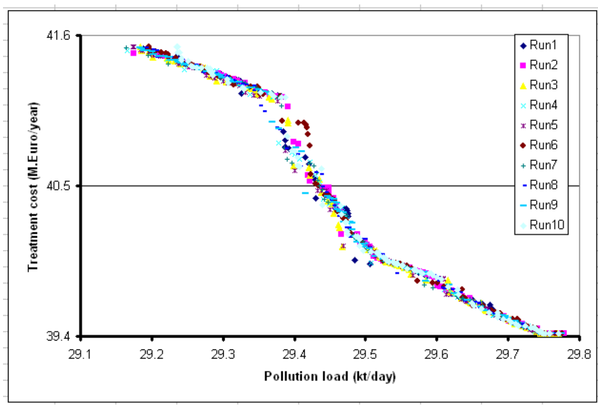
At 1 hour & 45 minutes



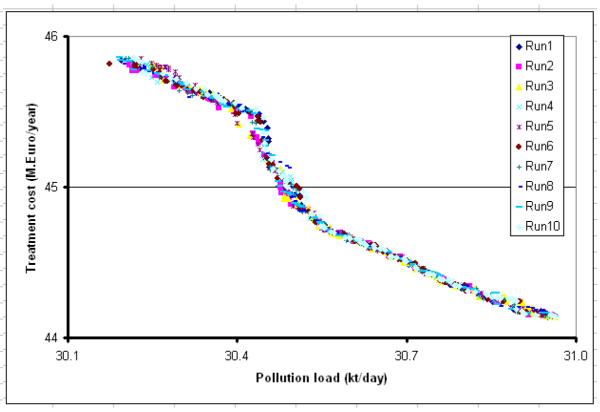
At 2 hours



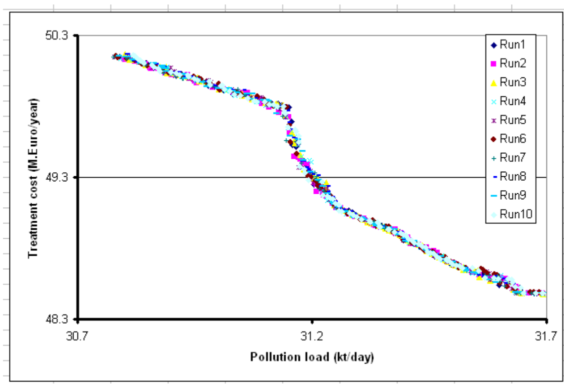
At 2 hours & 15 minutes



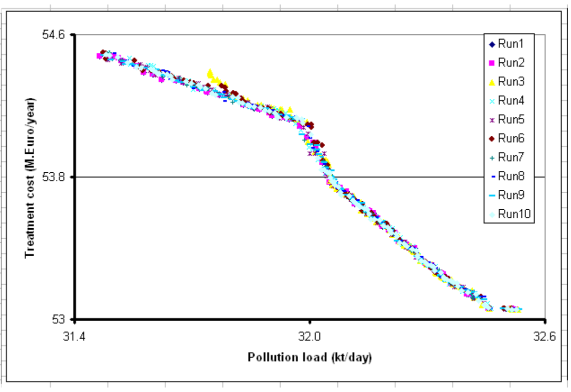
At 2 hours & 30 minutes



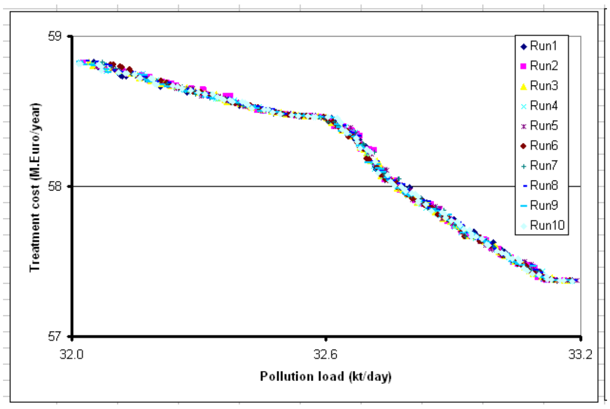
At 2 hours & 45 minutes



At 3 hours

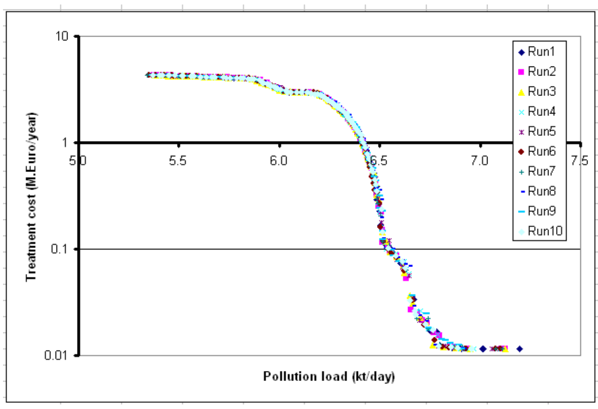


At 3 hours & 15 minutes

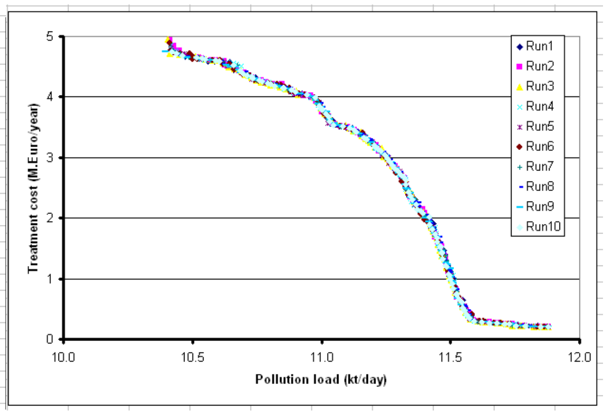


At 3 hours & 30 minutes

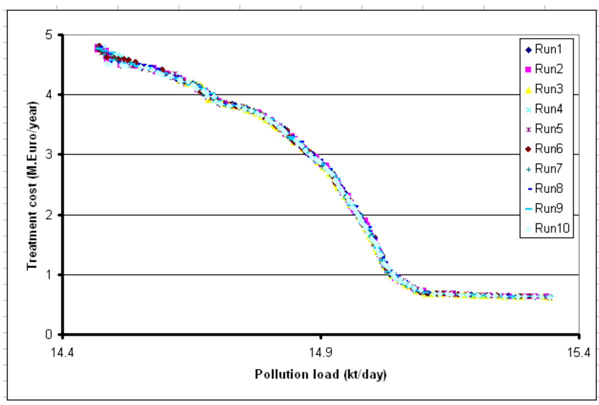
Minimum cost Pareto optimal fronts



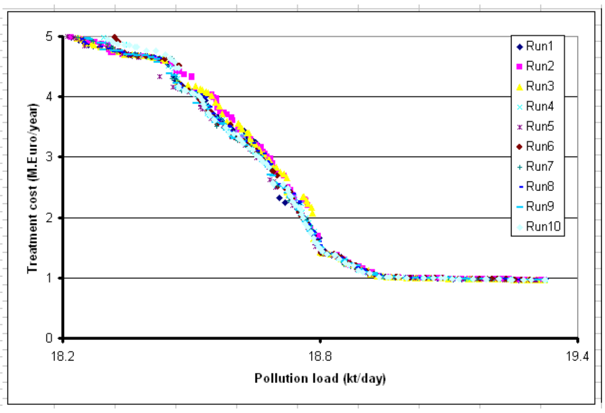
At 30 minutes



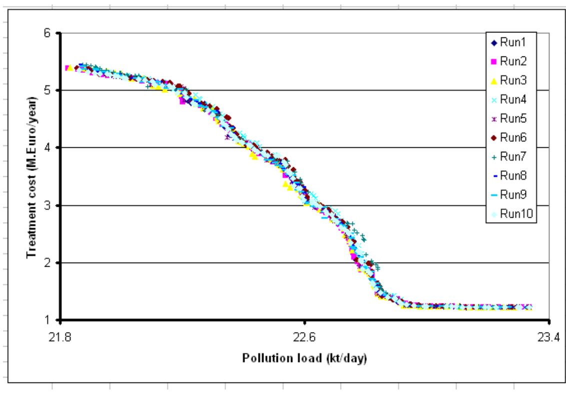
At 45 minutes



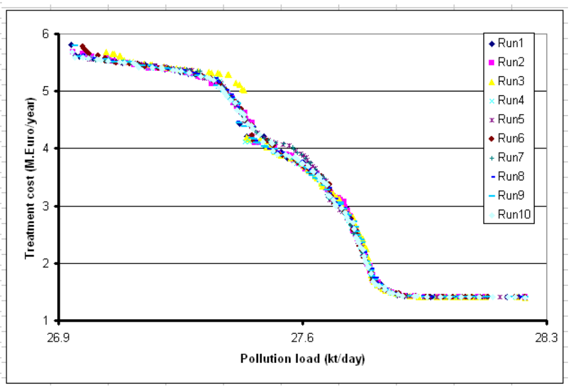
At 1 hour



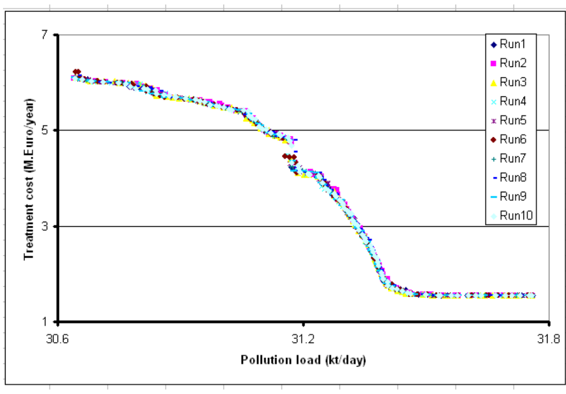
At 1 hour & 15 minutes



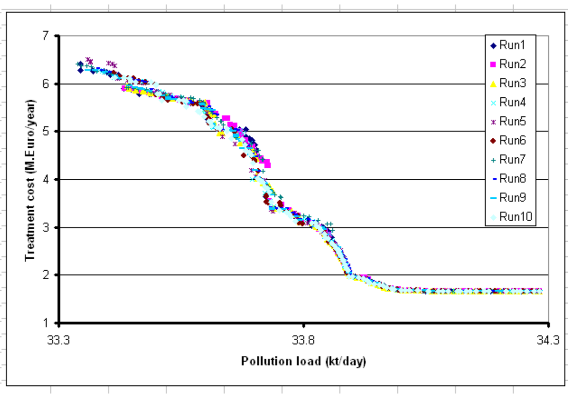
At 1 hour & 30 minutes



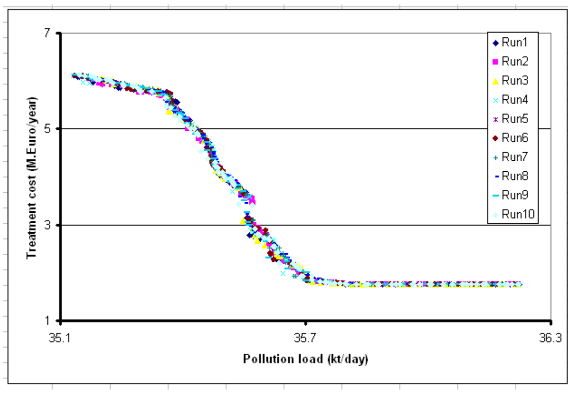
At 1 hour & 45 minutes



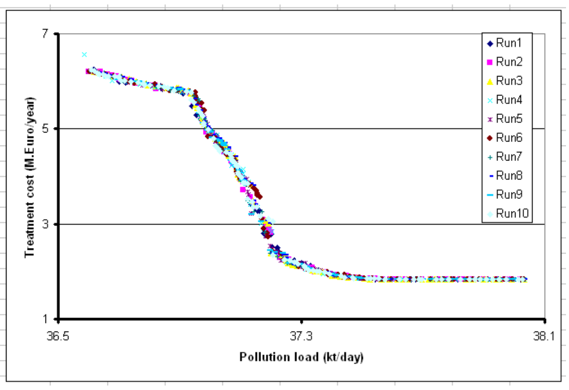
At 2 hours



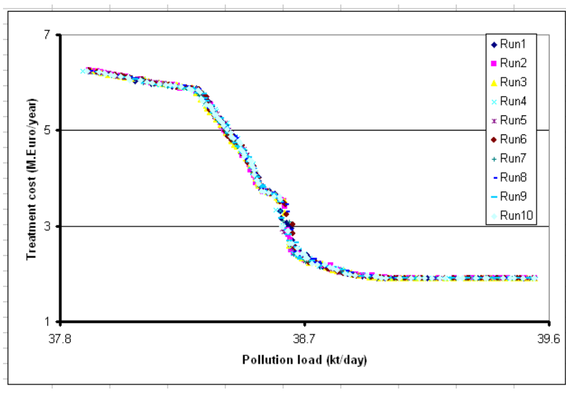
At 2 hours & 15 minutes



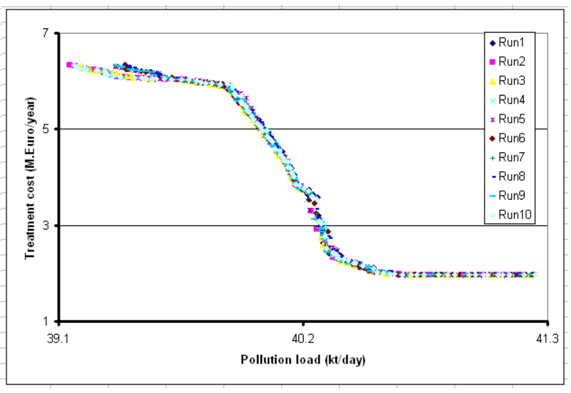
At 2 hours & 30 minutes



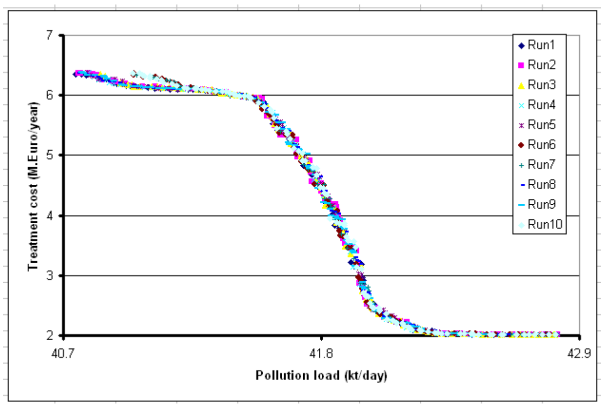
At 2 hours & 45 minutes



At 3 hours

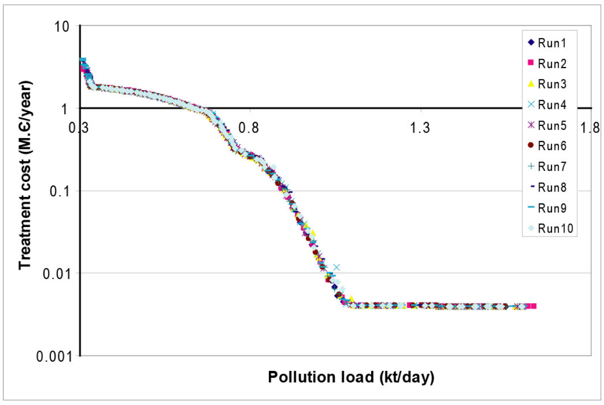


At 3 hours & 15 minutes



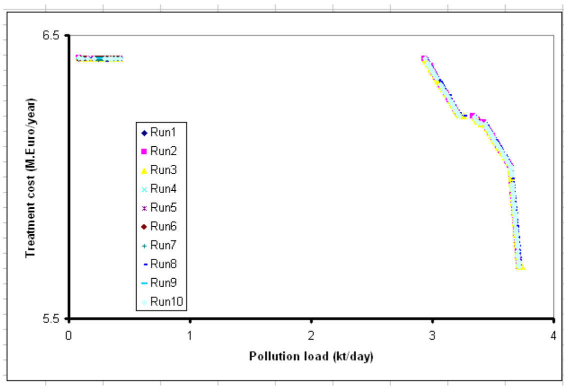
At 3 hours & 30 minutes

D.4 Random runs for SWMM constraint handling approach for two consecutive storms

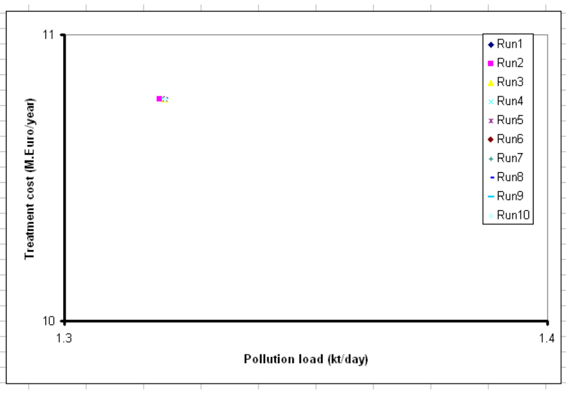


At 15 minutes

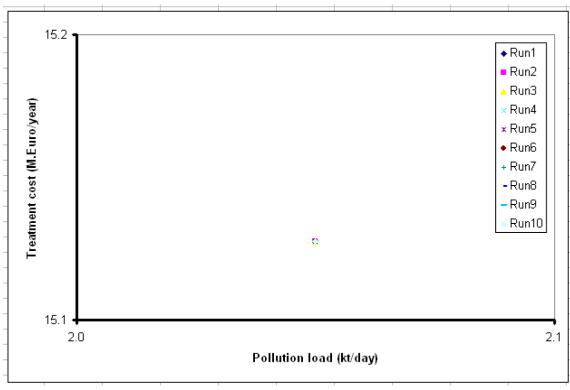
Minimum pollution load Pareto optimal fronts



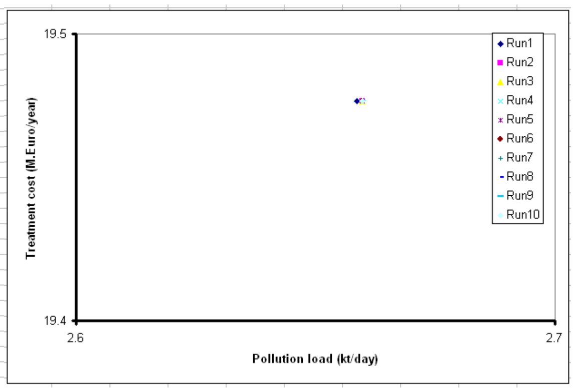
At 30 minutes



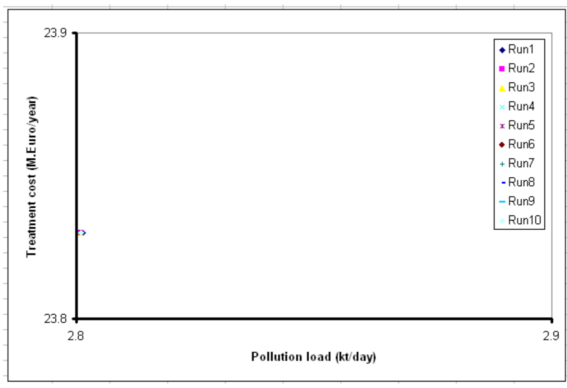
At 45 minutes



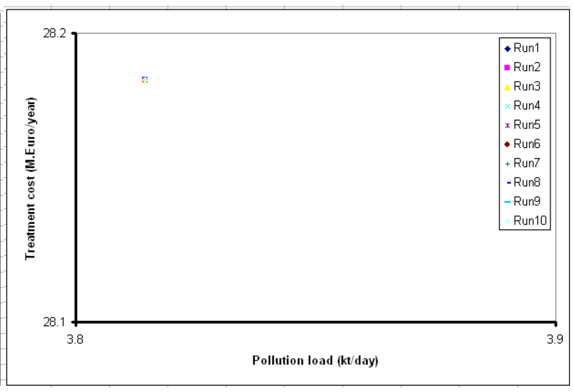
At 1 hour



At 1 hour & 15 minutes

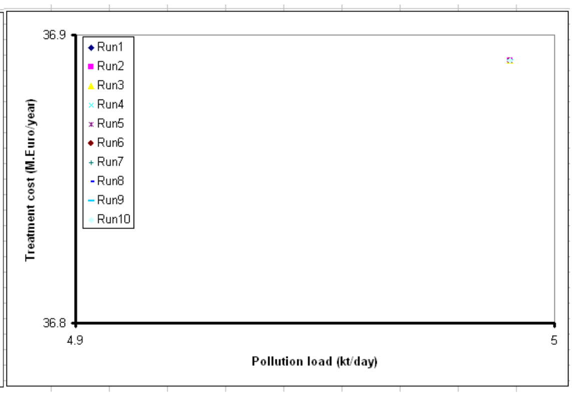


At 1 hour & 30 minutes



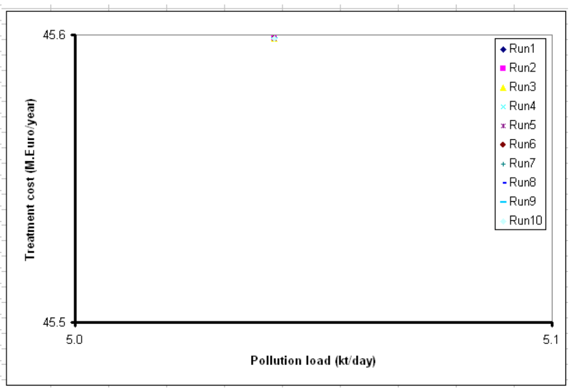
At 1 hour & 45 minutes

At 2 hours

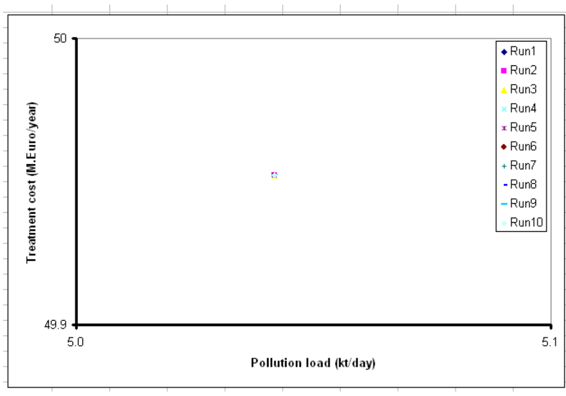


At 2 hours & 15 minutes

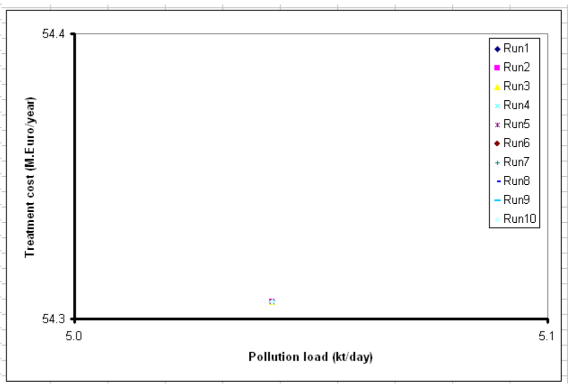
At 2 hours & 30 minutes



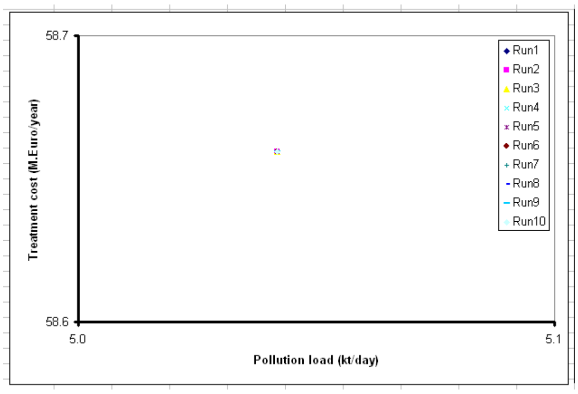
At 2 hours & 45 minutes



At 3 hours

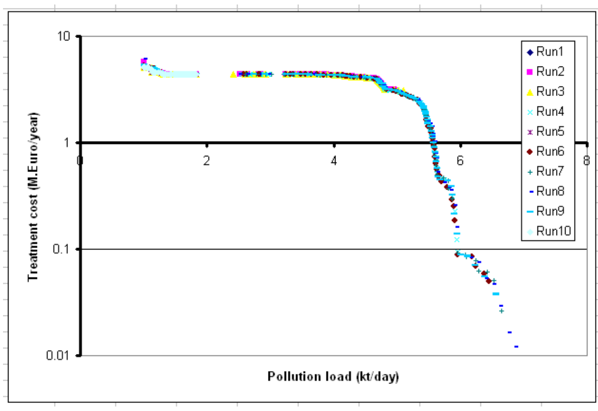


At 3 hours & 15 minutes

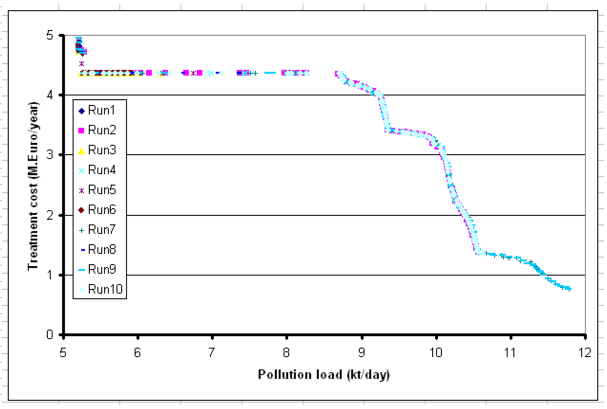


At 3 hours & 30 minutes

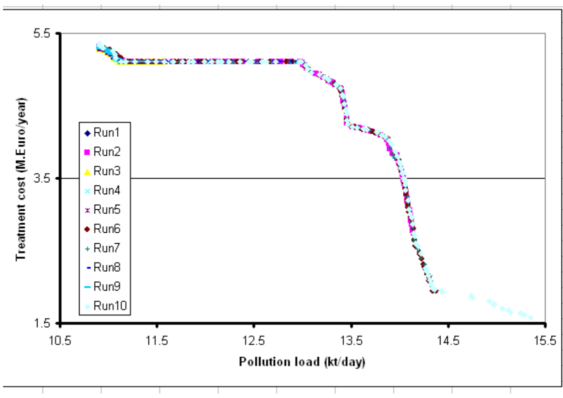
Minimum cost Pareto optimal fronts



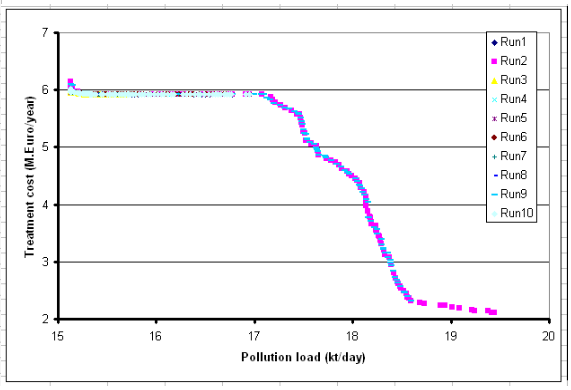
At 30 minutes



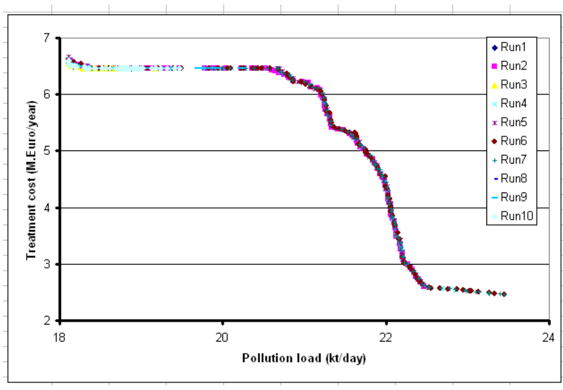
At 45 minutes



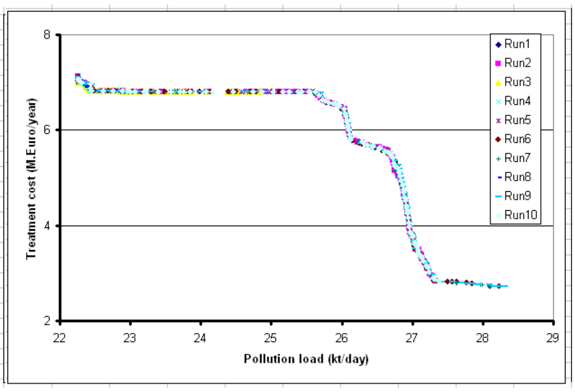
At 1 hour



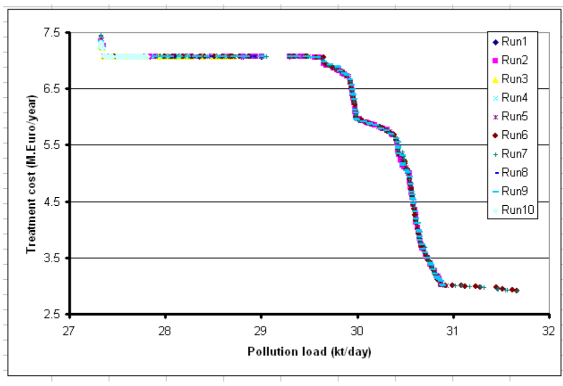
At 1 hour & 15 minutes



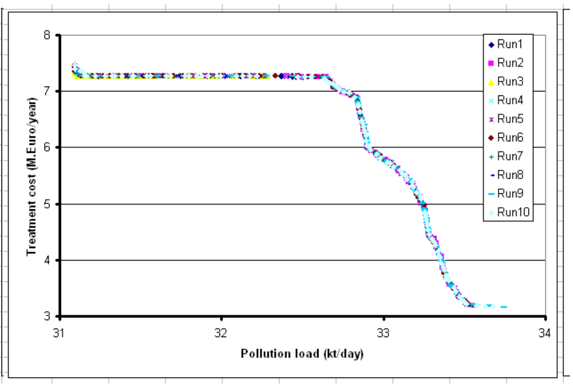
At 1 hour & 30 minutes



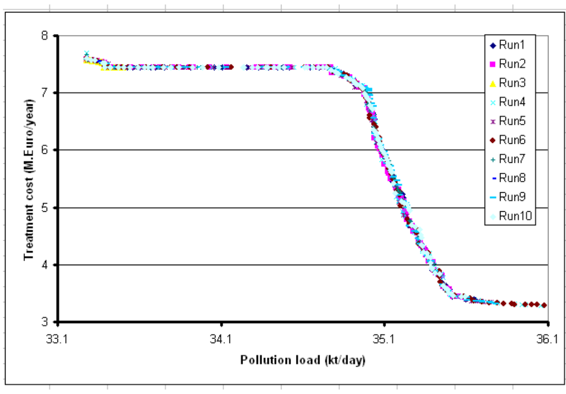
At 1 hour & 45 minutes



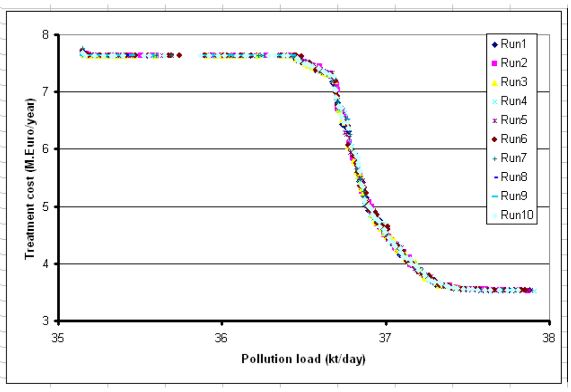
At 2 hours



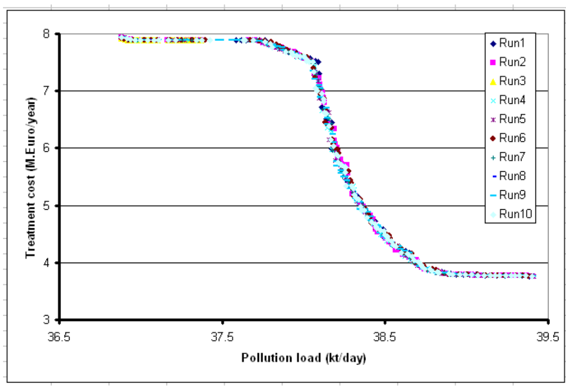
At 2 hours & 15 minutes



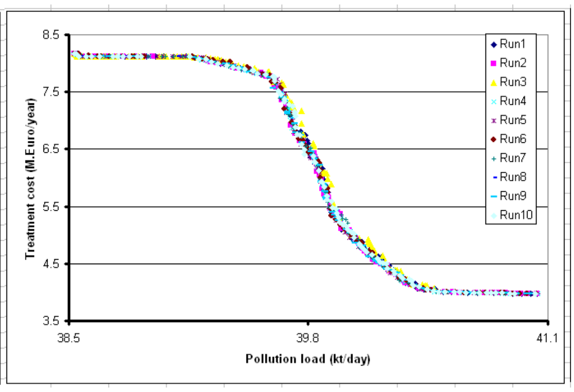
At 2 hours & 30 minutes



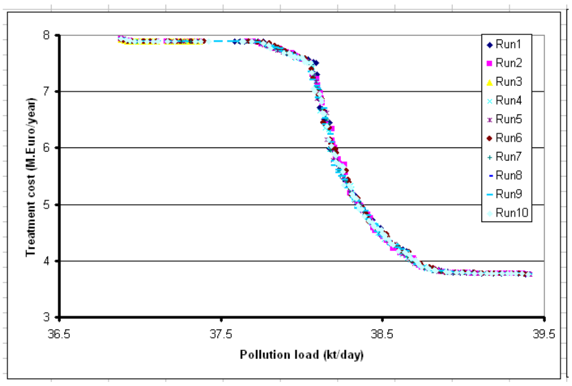
At 2 hours & 45 minutes



At 3 hours

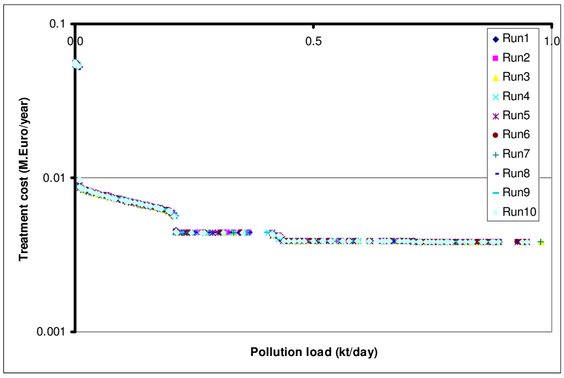


At 3 hours & 15 minutes



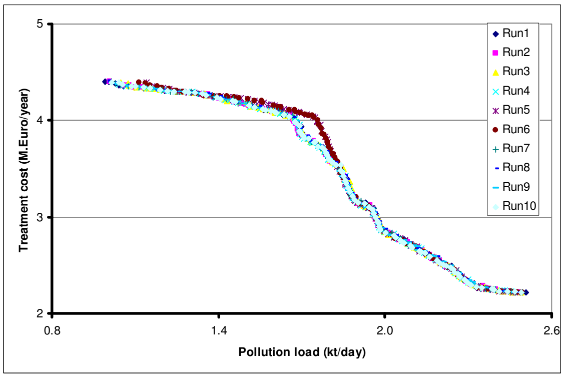
At 3 hours & 30 minutes

D.5 Random runs for NSGA II constraint handling approach for migrating downstream storms

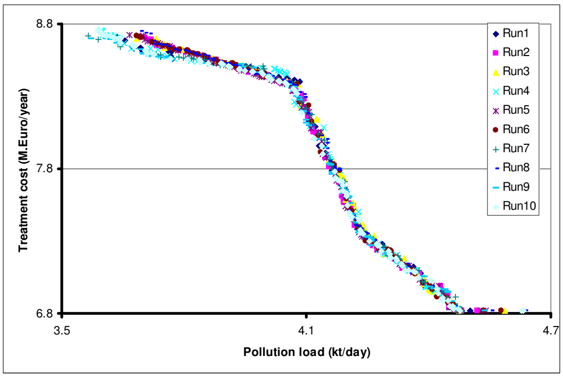


At 15 minutes

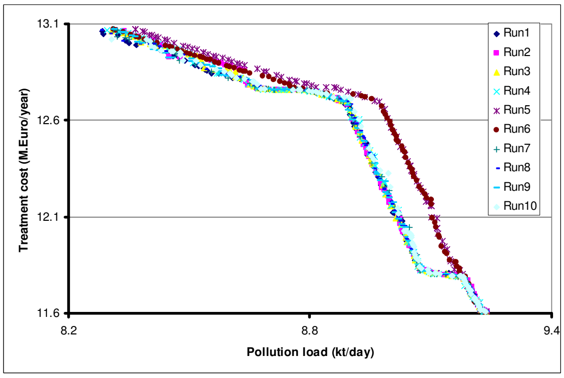
Minimum pollution load Pareto optimal fronts



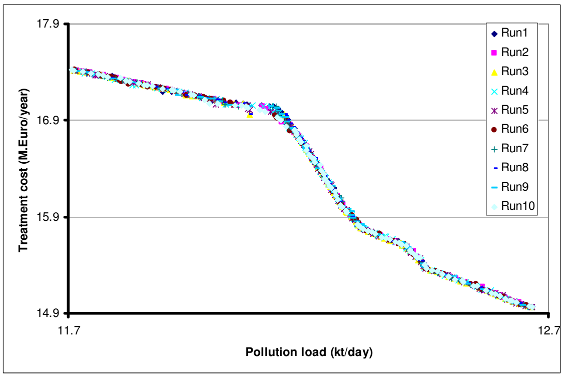
At 30 minutes



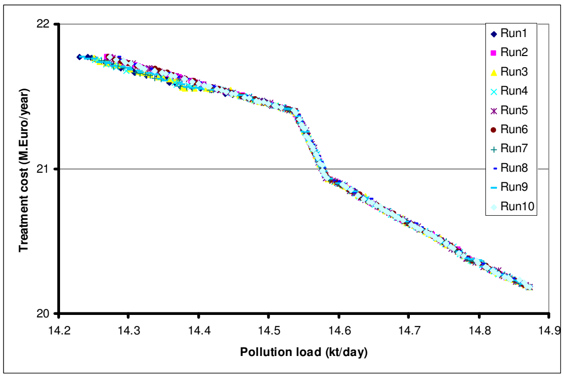
At 45 minutes



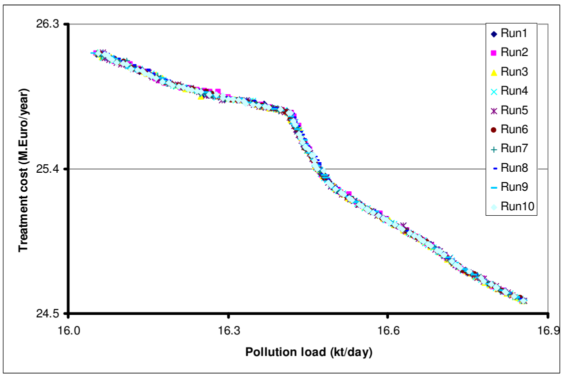
At 1 hour



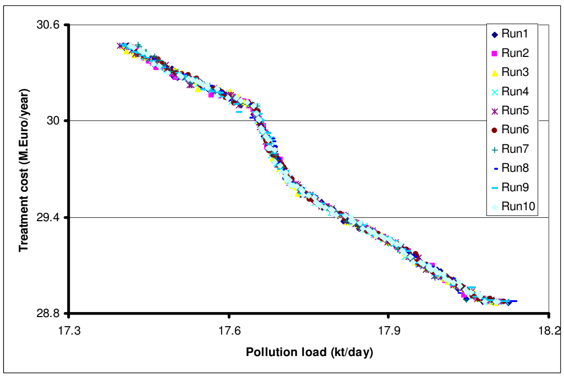
At 1 hour & 15 minutes



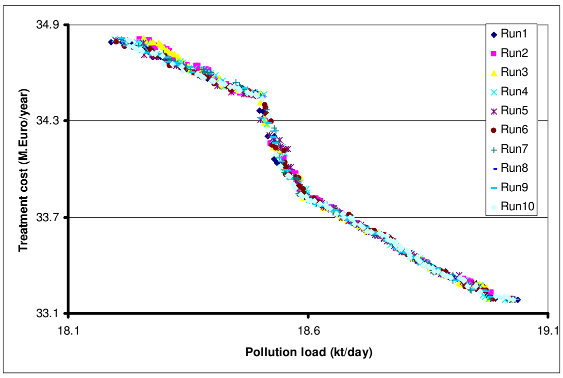
At 1 hour & 30 minutes



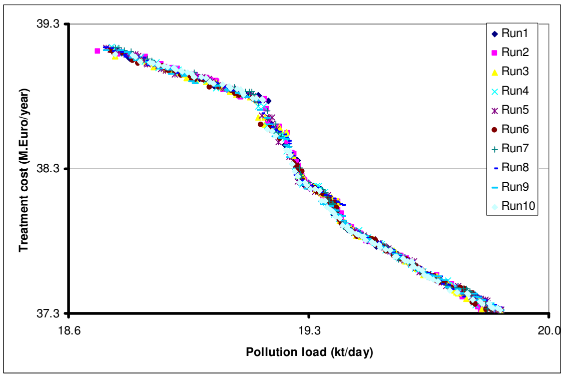
At 1 hour & 45 minutes



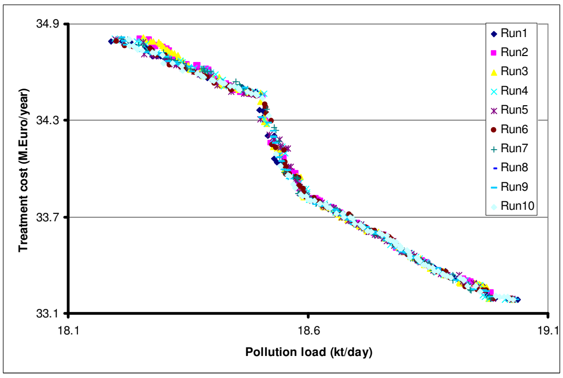
At 2 hours



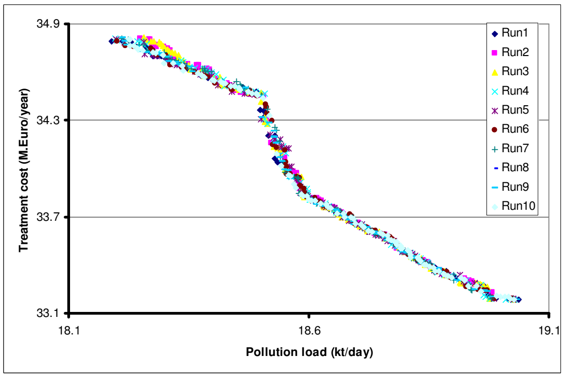
At 2 hours & 15 minutes



At 2 hours & 30 minutes

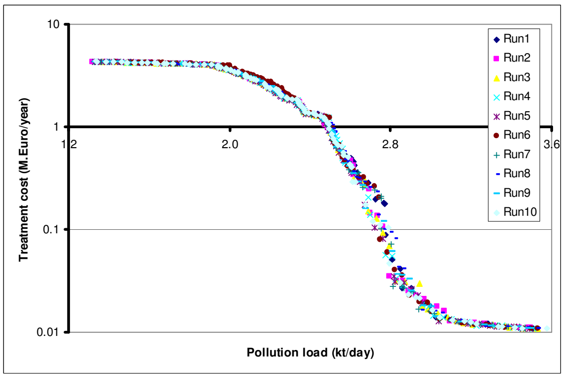


At 2 hours & 45 minutes

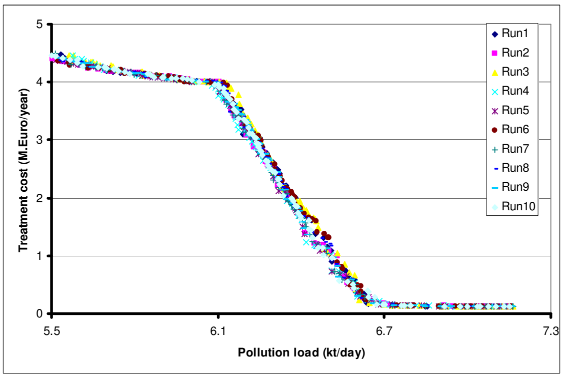


At 3 hours

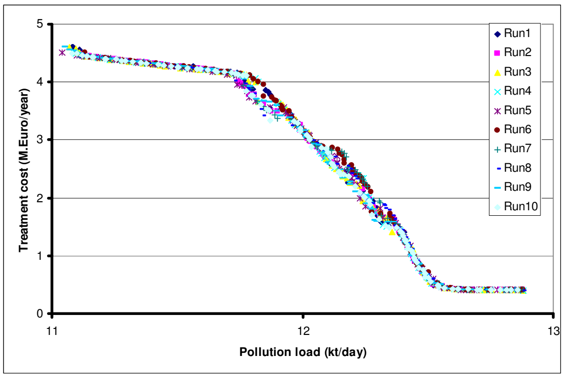
Minimum cost Pareto optimal fronts



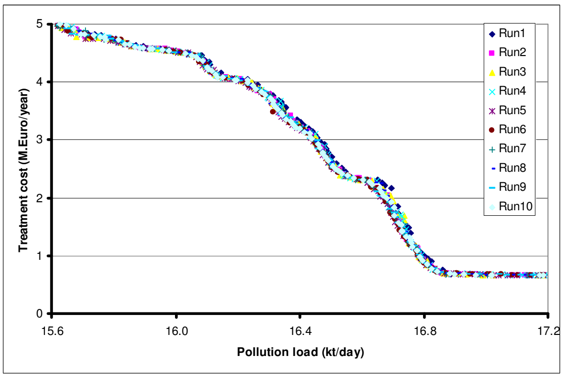
At 30 minutes



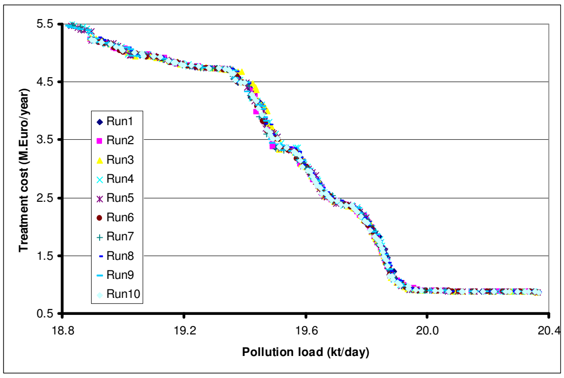
At 45 minutes



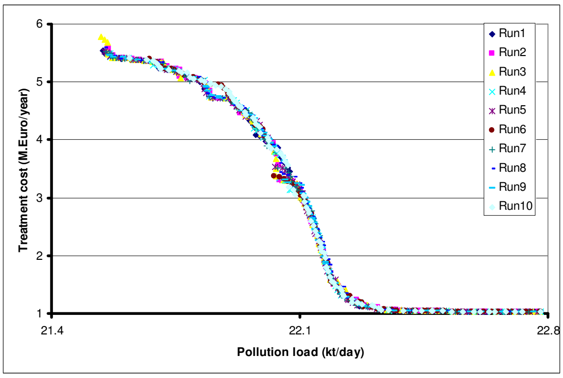
At 1 hour



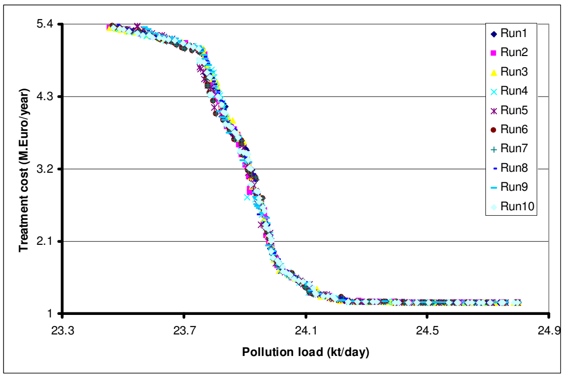
At 1 hour & 15 minutes



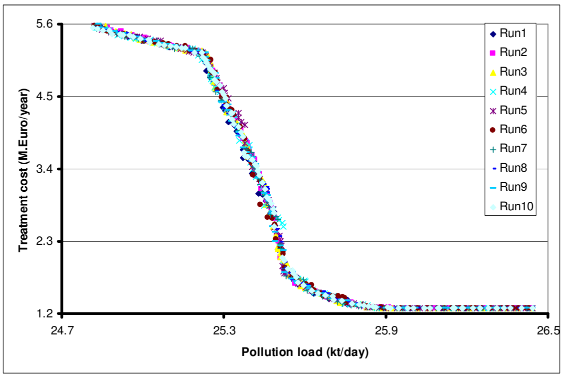
At 1 hour & 30 minutes



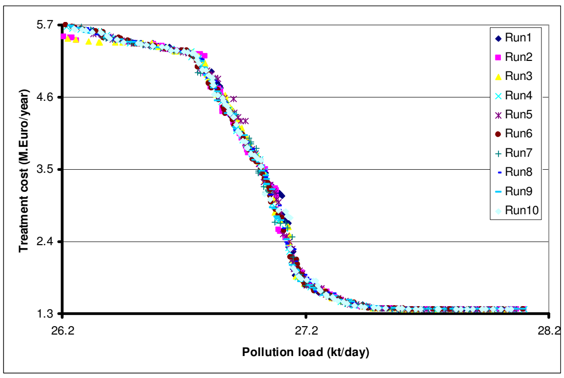
At 1 hour & 45 minutes



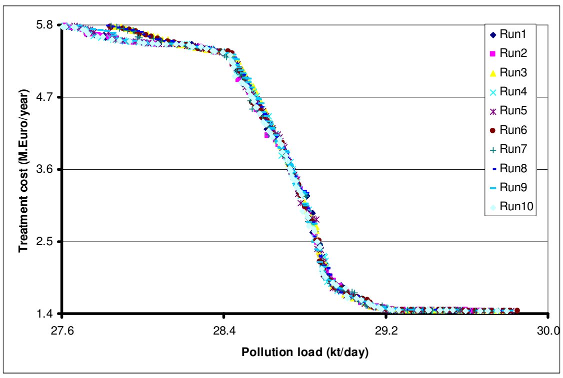
At 2 hours



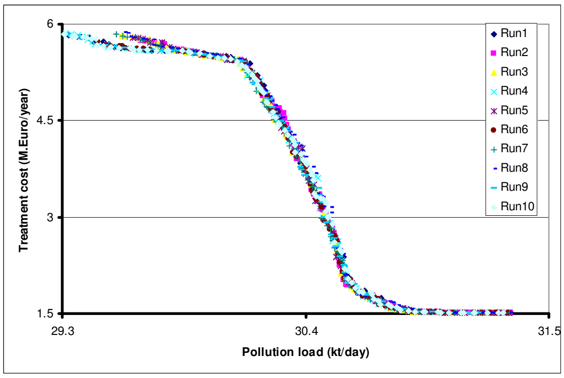
At 2 hours & 15 minutes



At 2 hours & 30 minutes

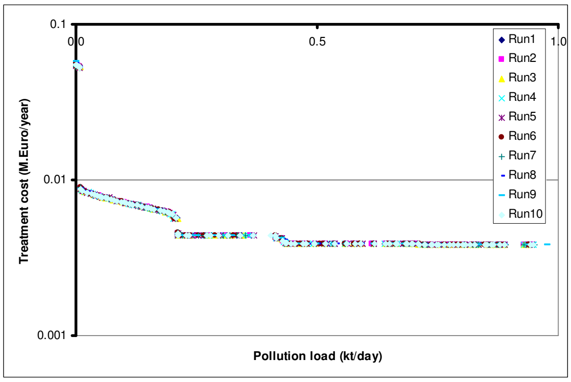


At 2 hours & 45 minutes



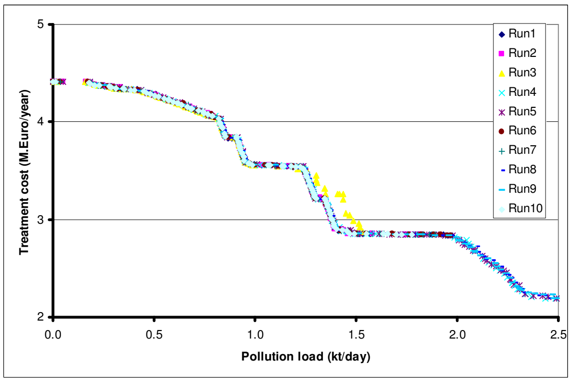
At 3 hours

D.6 Random runs for SWMM constraint handling approach for migrating downstream storms



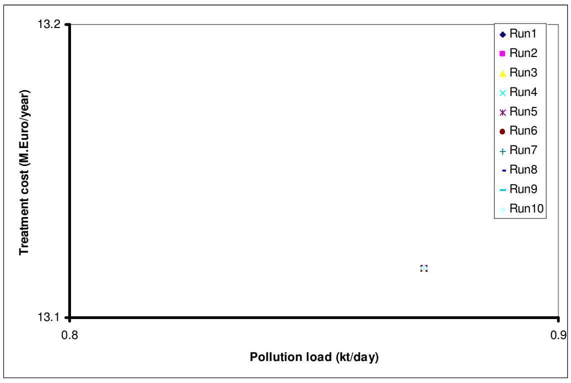
At 15 minutes

Minimum pollution load Pareto optimal fronts

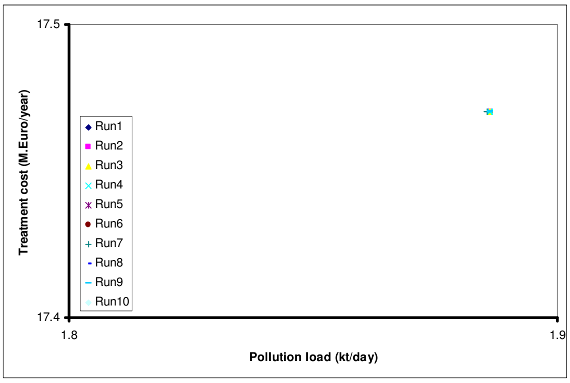


At 30 minutes

At 45 minutes

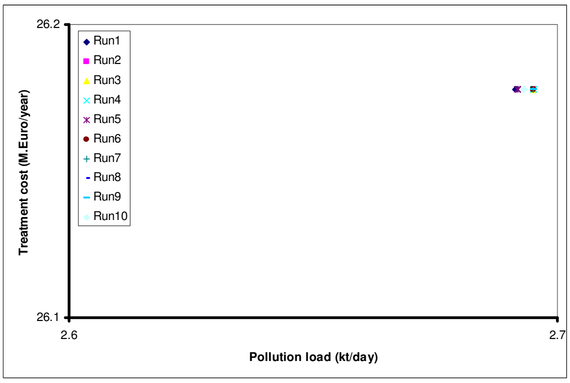


At 1 hour



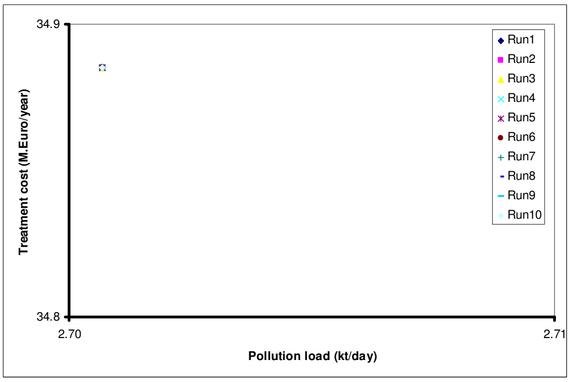
At 1 hour & 15 minutes

At 1 hour & 30 minutes



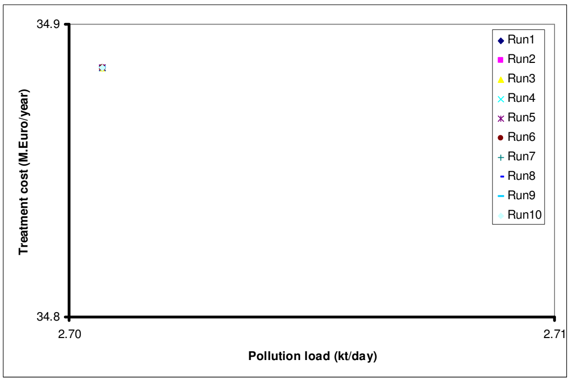
At 1 hour & 45 minutes

At 2 hours

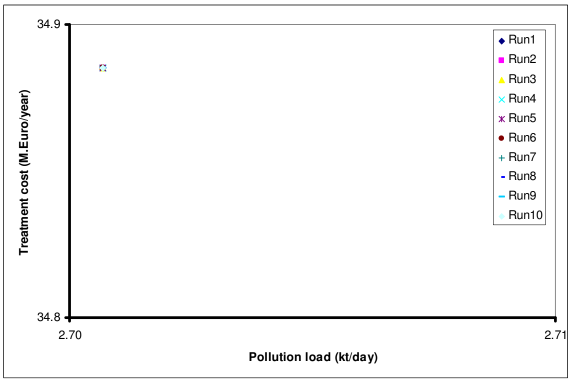


At 2 hours & 15 minutes

At 2 hours & 30 minutes

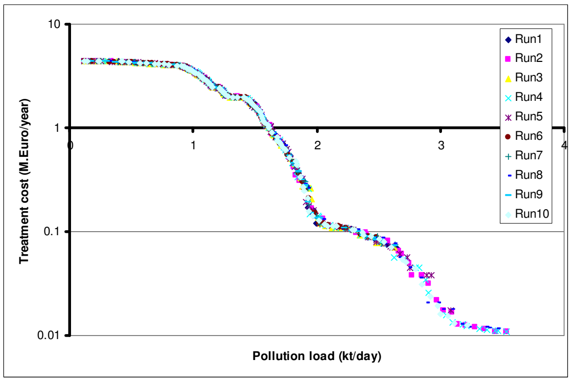


At 2 hours & 45 minutes

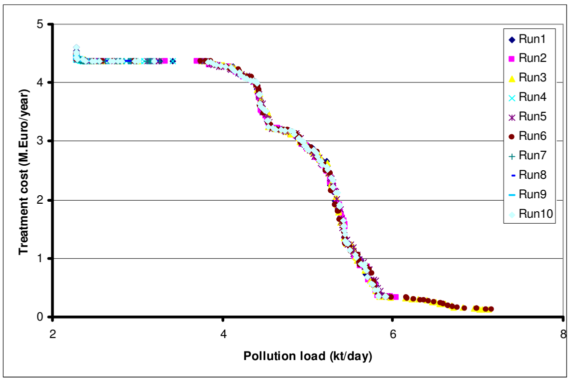


At 3 hours

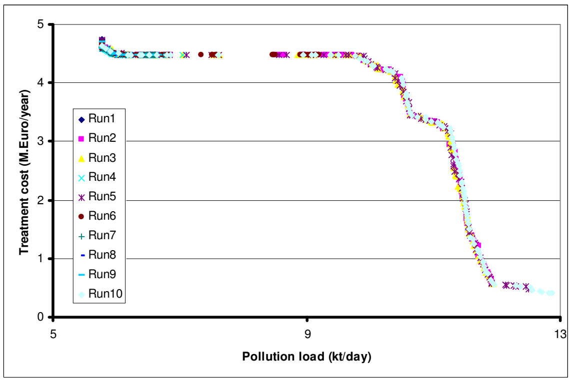
Minimum cost Pareto optimal fronts



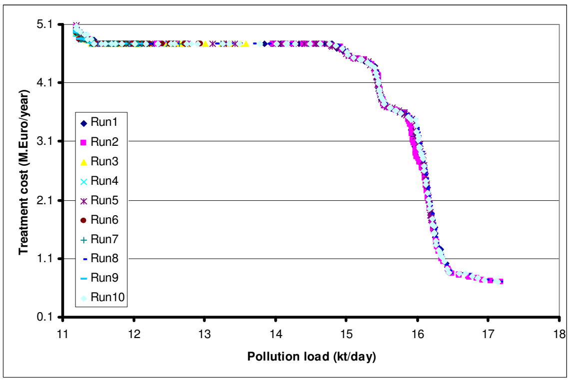
At 30 minutes



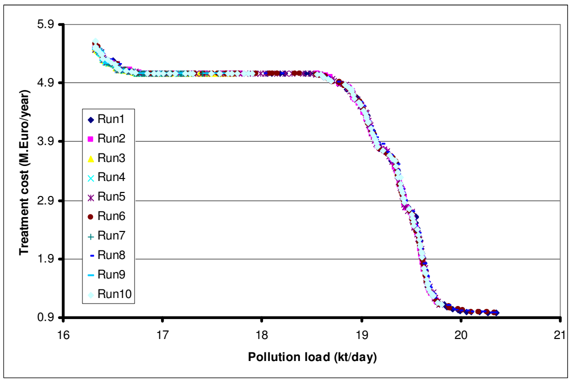
At 45 minutes



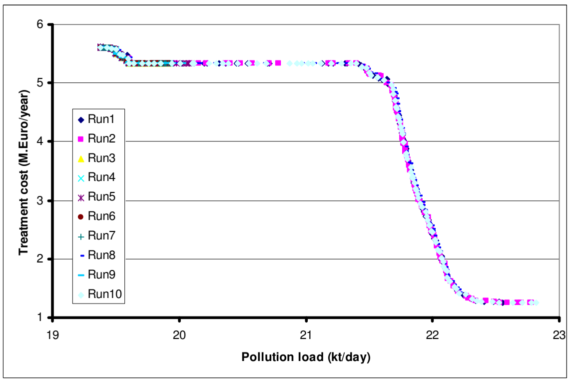
At 1 hour



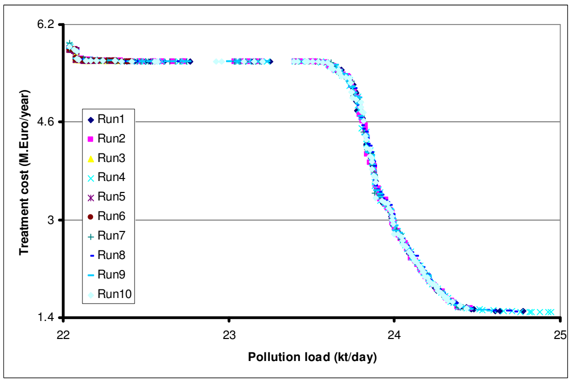
At 1 hour & 15 minutes



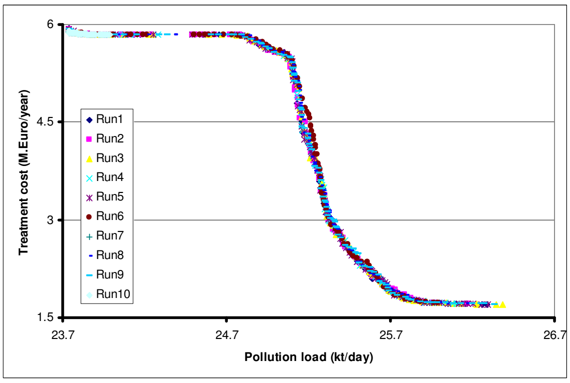
At 1 hour & 30 minutes



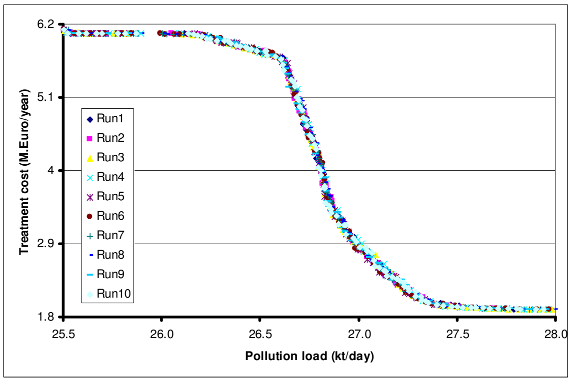
At 1 hour & 45 minutes



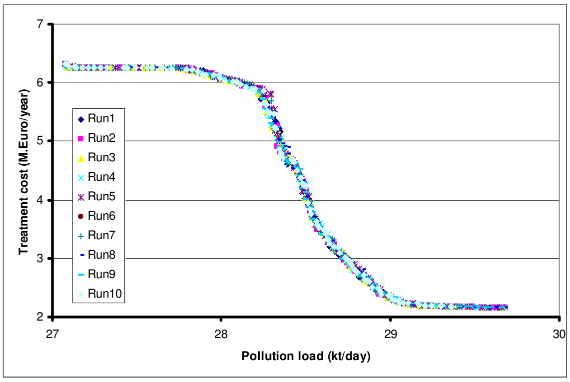
At 2 hours



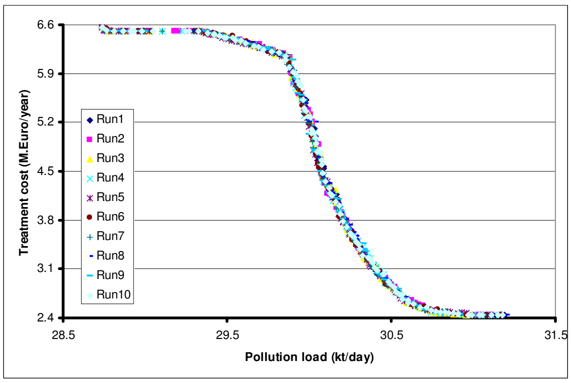
At 2 hours & 15 minutes



At 2 hours & 30 minutes

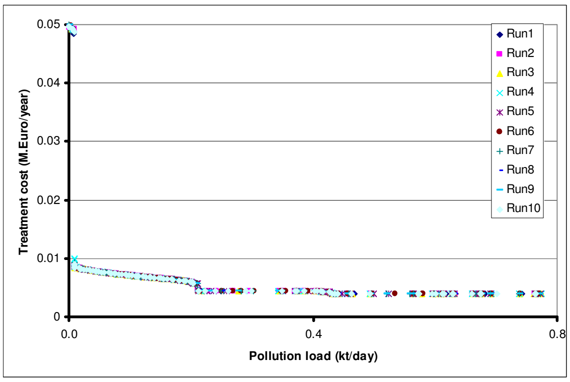


At 2 hours & 45 minutes



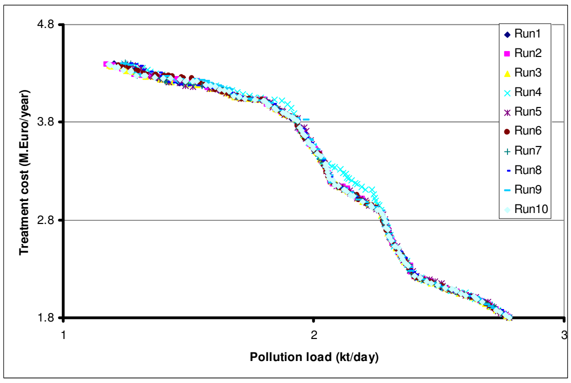
At 3 hours

D.7 Random runs for NSGA II constraint handling approach for migrating upstream storms

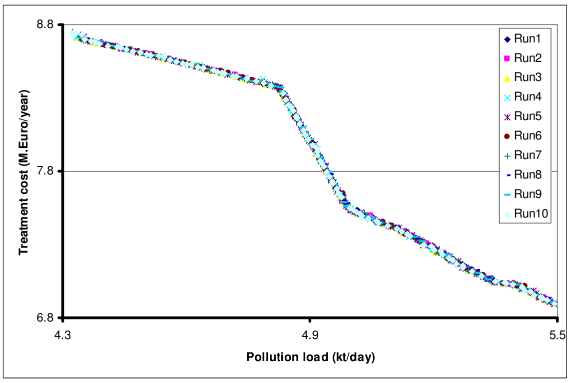


At 15 minutes

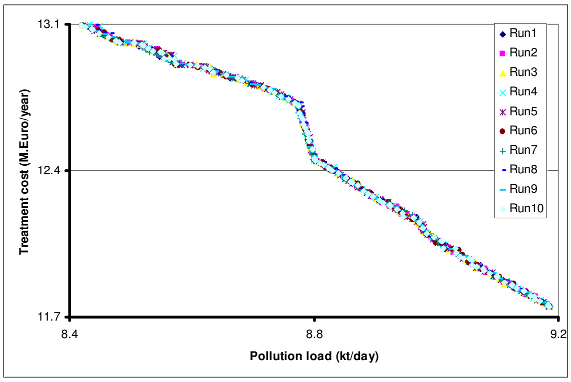
Minimum pollution load Pareto optimal fronts



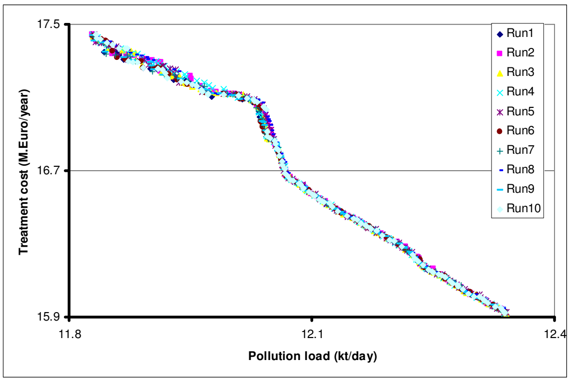
At 30 minutes



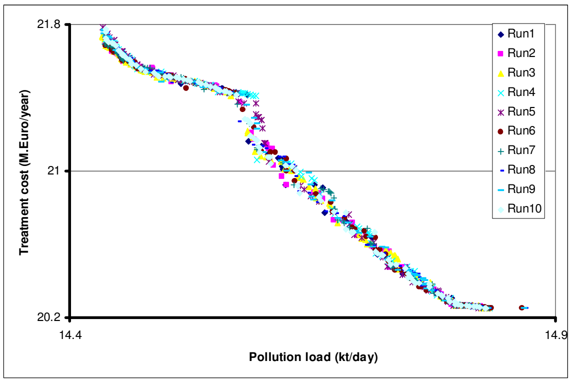
At 45 minutes



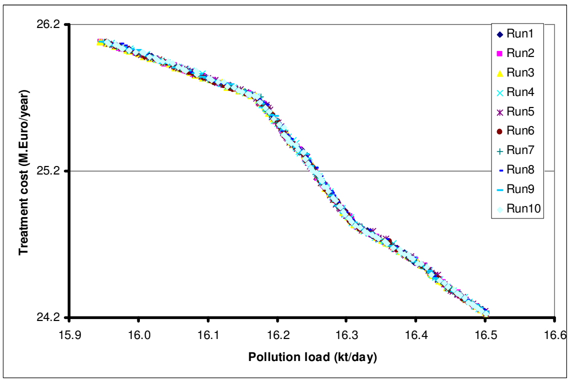
At 1 hour



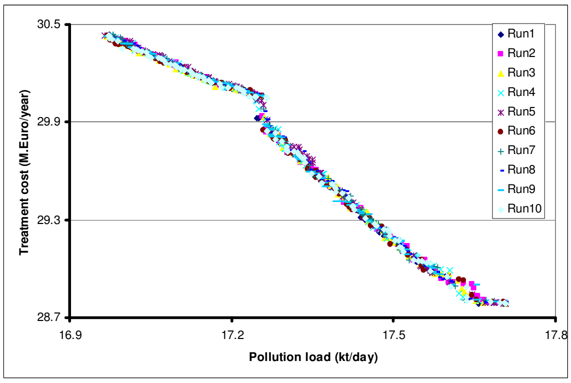
At 1 hour & 15 minutes



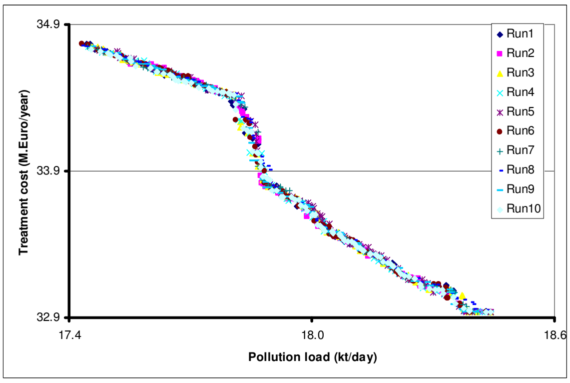
At 1 hour & 30 minutes



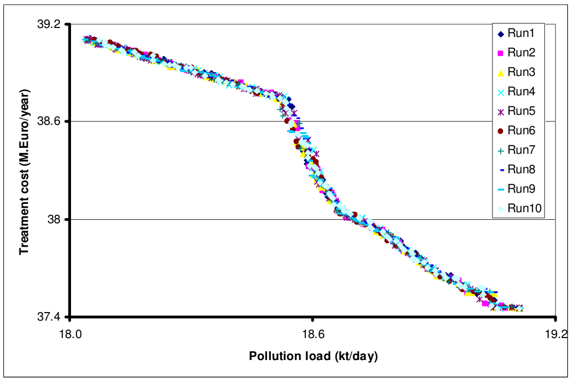
At 1 hour & 45 minutes



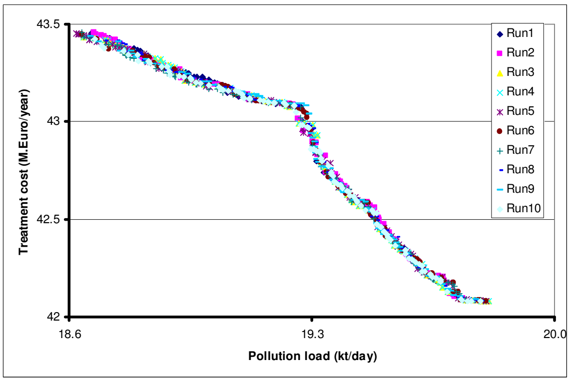
At 2 hours



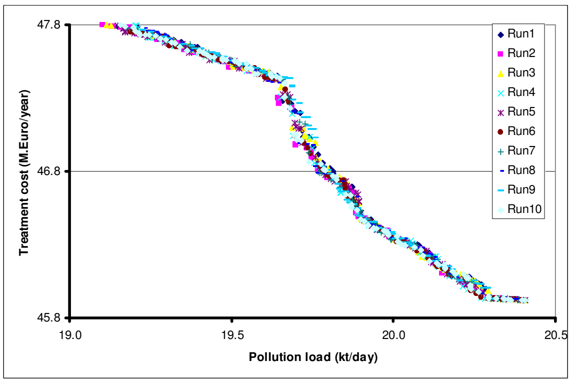
At 2 hours & 15 minutes



At 2 hours & 30 minutes

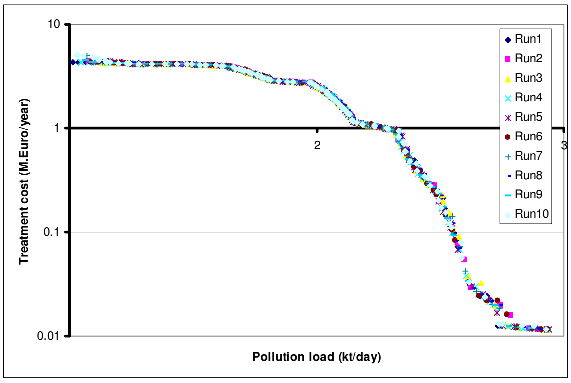


At 2 hours & 45 minutes

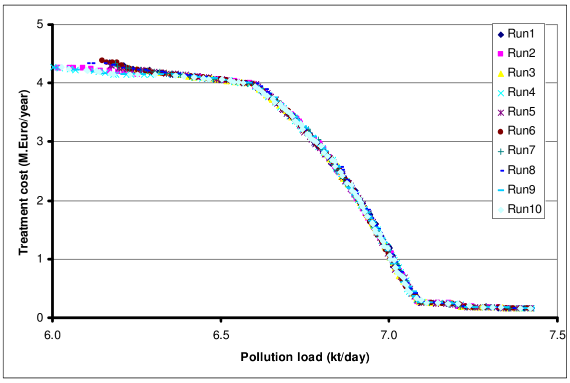


At 3 hours

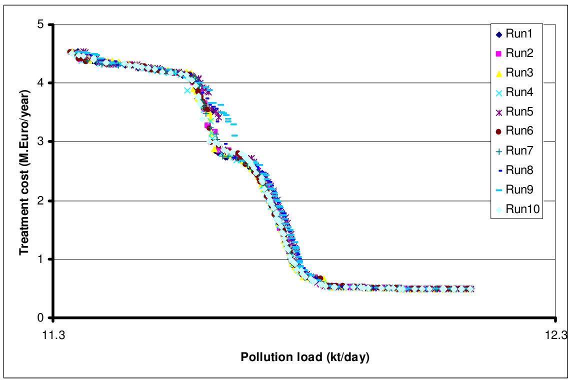
Minimum cost Pareto optimal fronts



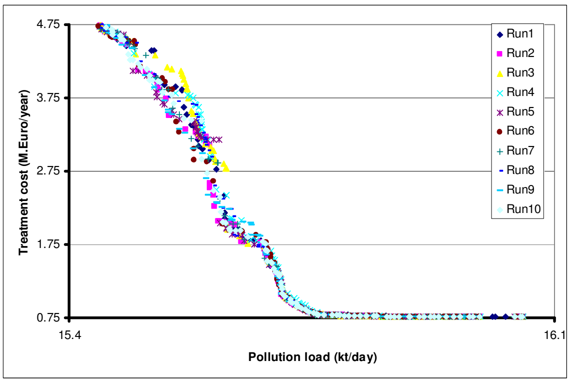
At 30 minutes



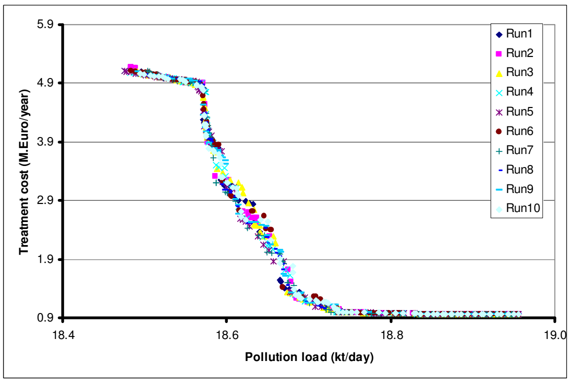
At 45 minutes



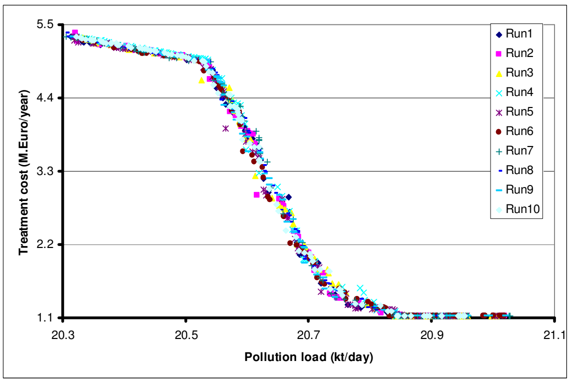
At 1 hour



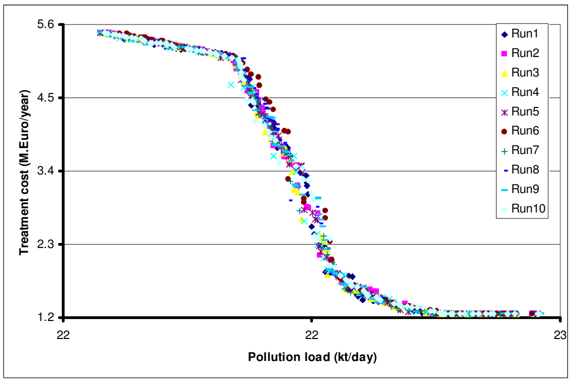
At 1 hour & 15 minutes



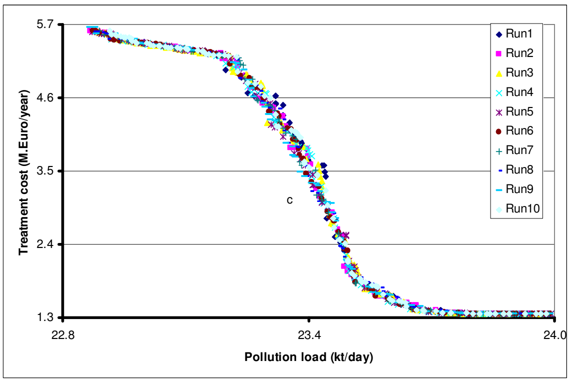
At 1 hour & 30 minutes



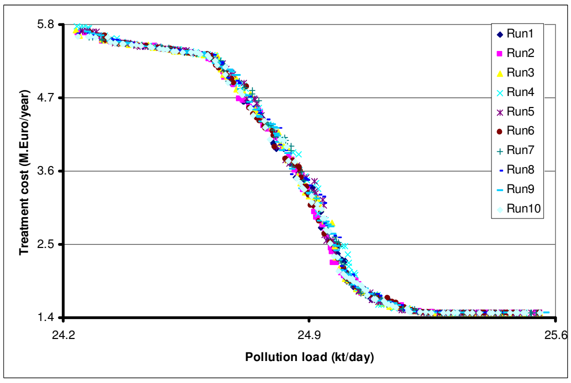
At 1 hour & 45 minutes



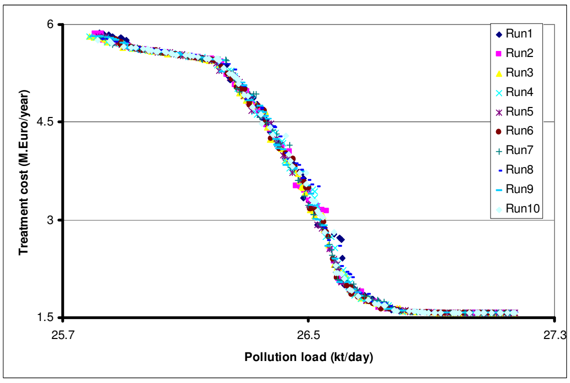
At 2 hours



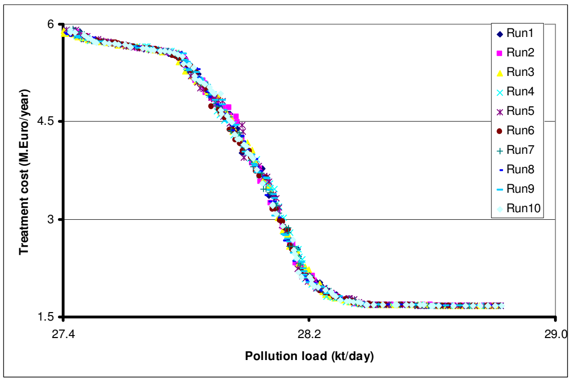
At 2 hours & 15 minutes



At 2 hours & 30 minutes

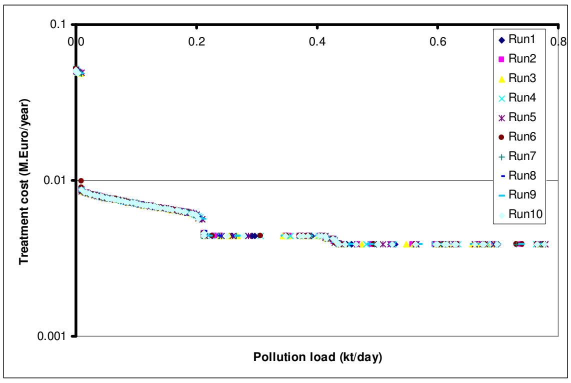


At 2 hours & 45 minutes



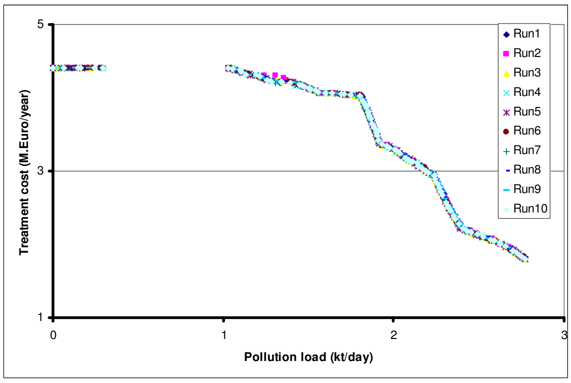
At 3 hours

D.8 Random runs for SWMM constraint handling approach for migrating upstream storms

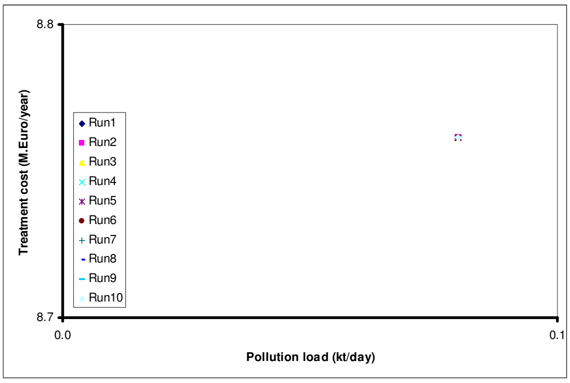


At 15 minutes

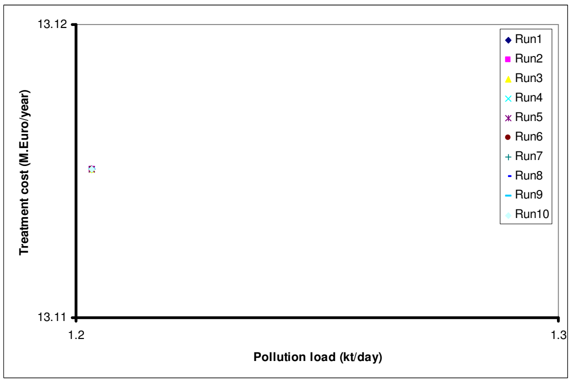
Minimum pollution load Pareto optimal fronts



At 30 minutes

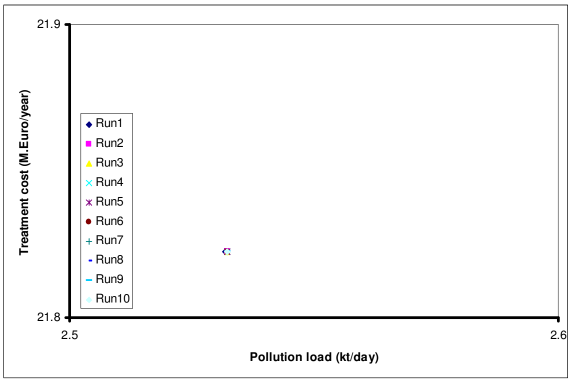


At 45 minutes



At 1 hour

At 1 hour & 15 minutes

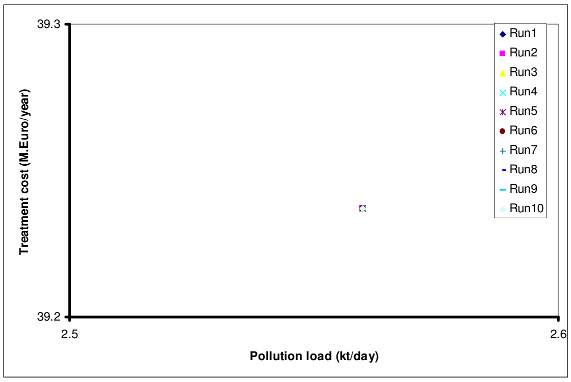


At 1 hour & 30 minutes

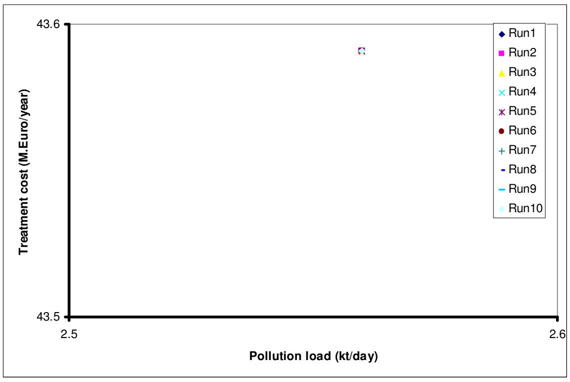
At 1 hour & 45 minutes

At 2 hours

At 2 hours & 15 minutes



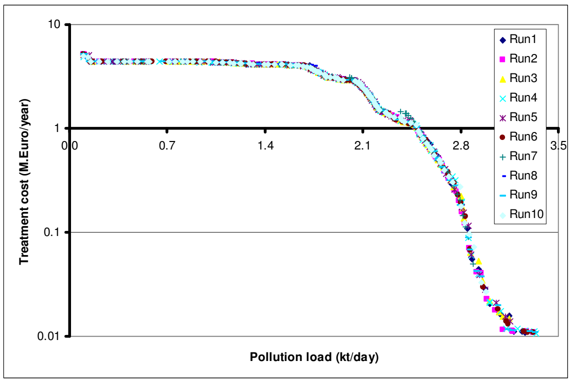
At 2 hours & 30 minutes



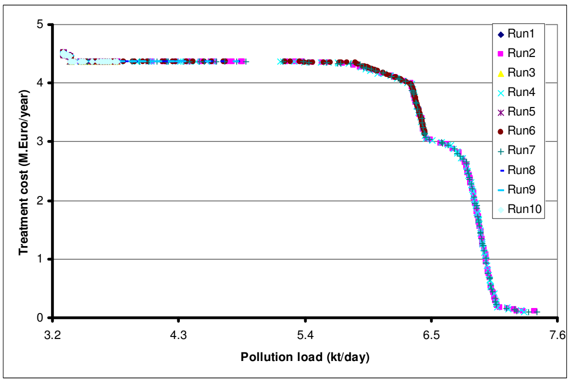
At 2 hours & 45 minutes

At 3 hours

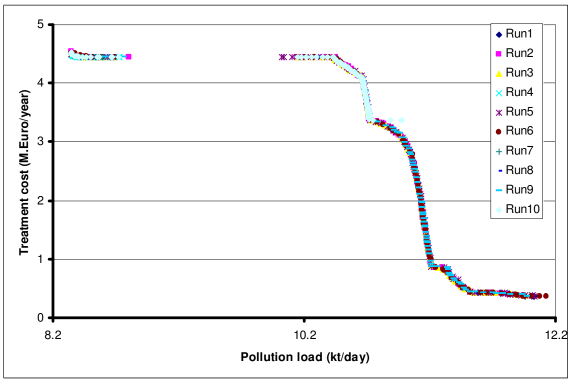
Minimum cost Pareto optimal fronts



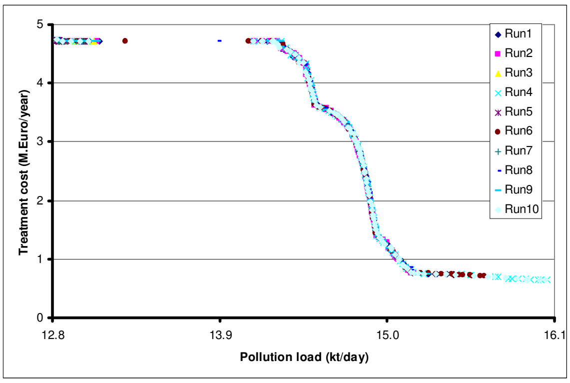
At 30 minutes



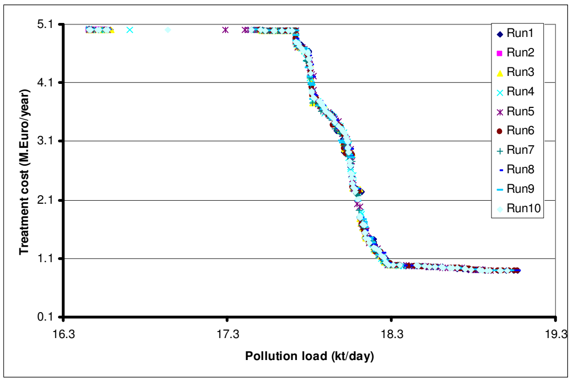
At 45 minutes



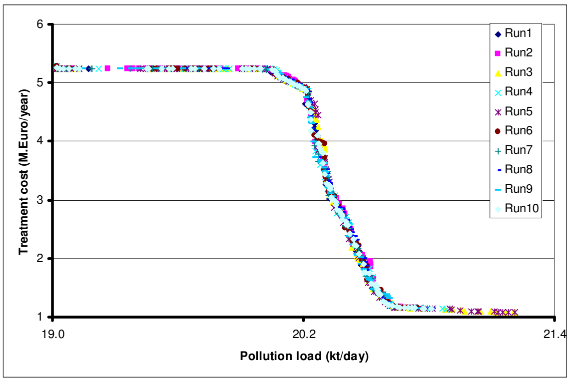
At 1 hour



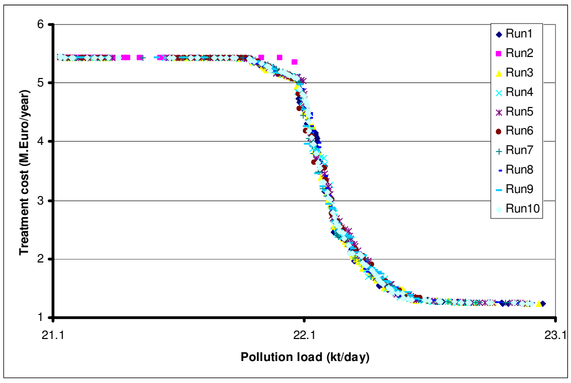
At 1 hour & 15 minutes



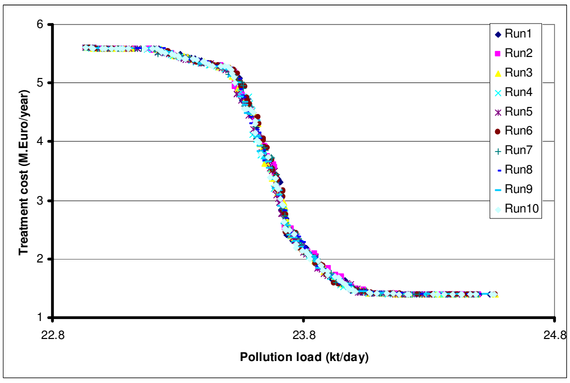
At 1 hour & 30 minutes



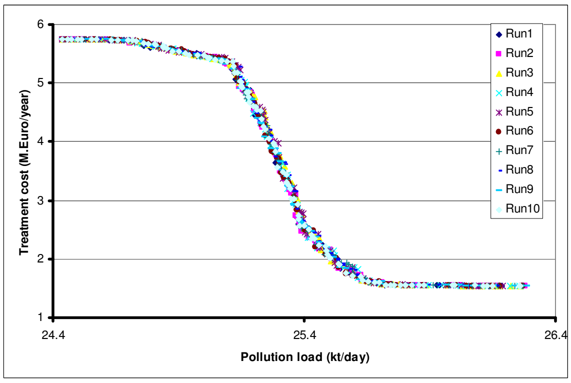
At 1 hour & 45 minutes



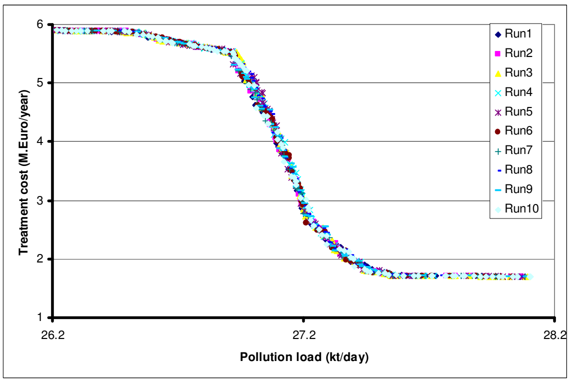
At 2 hours



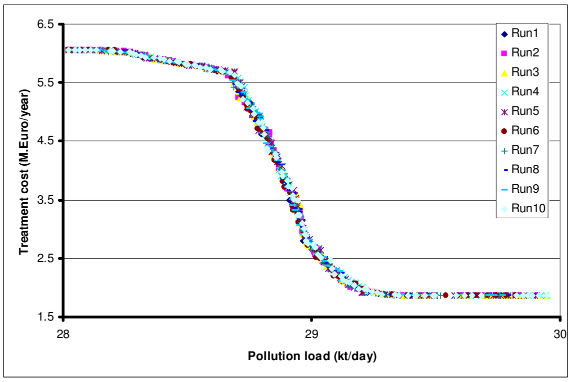
At 2 hours & 15 minutes



At 2 hours & 30 minutes



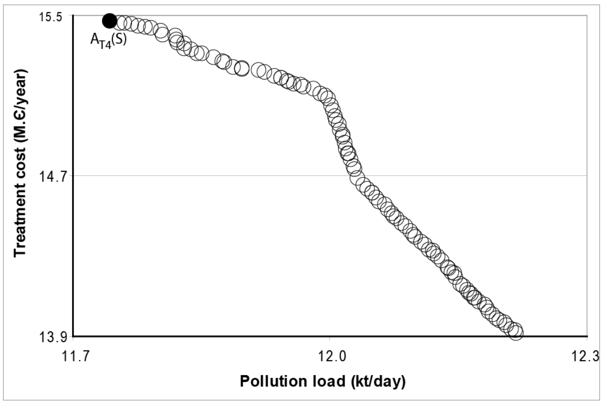
At 2 hours & 45 minutes



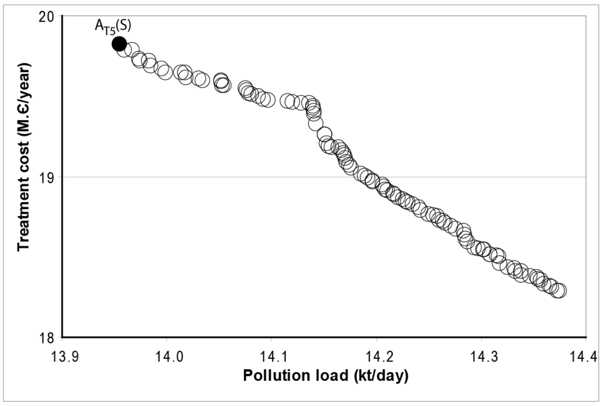
At 3 hours

D.9 Selected solutions from NSGA II constraint handling approach for single storm

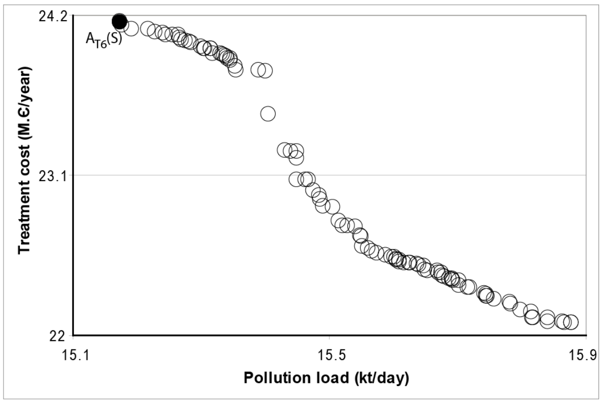
Minimum pollution load solutions



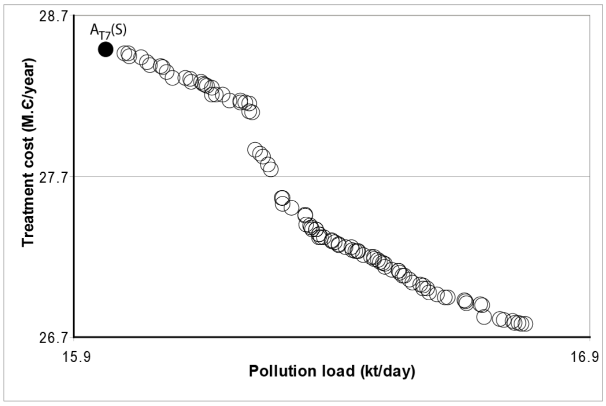
At 1 hour



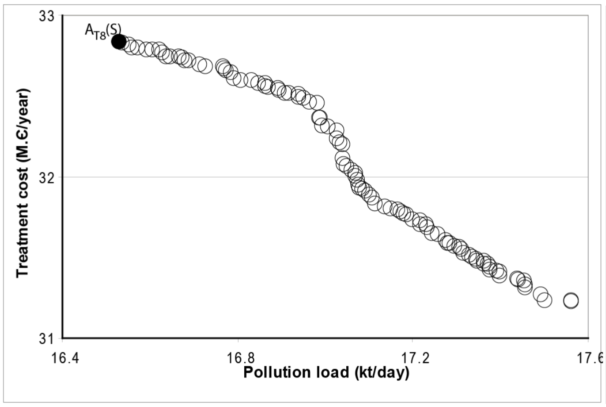
At 1 hour & 15 minutes



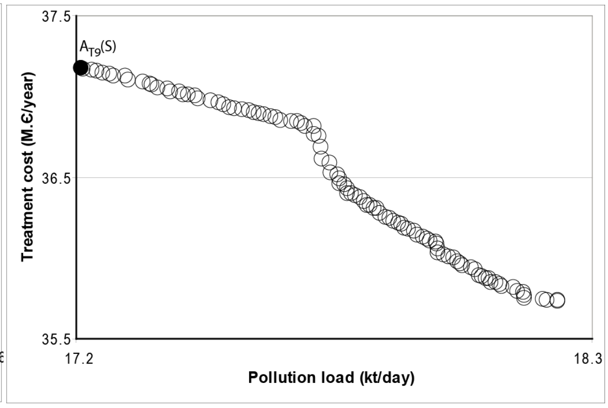
At 1 hour & 30 minutes



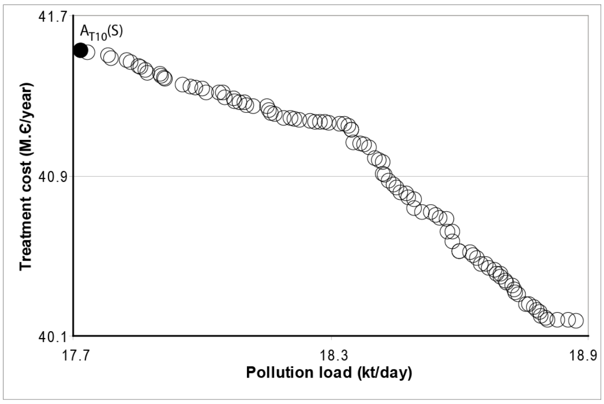
At 1 hour & 45 minutes



At 2 hours

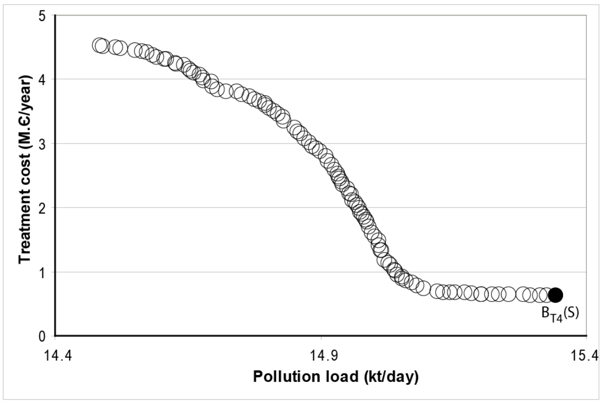


At 2 hours & 15 minutes

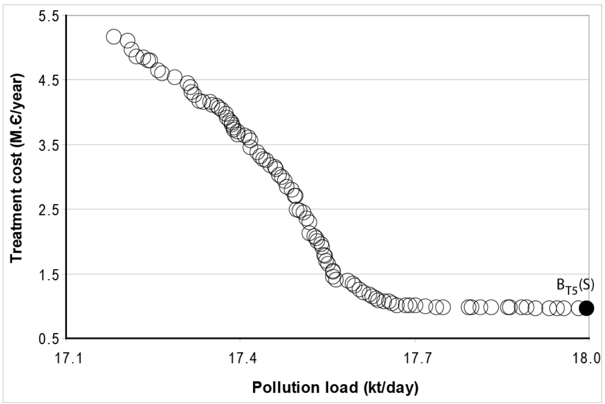


At 2 hours & 30 minutes

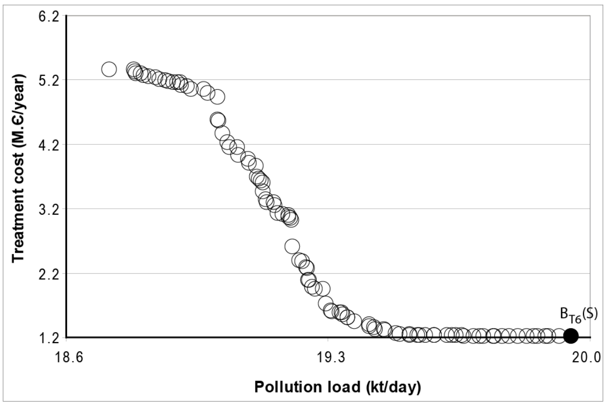
Minimum cost solutions



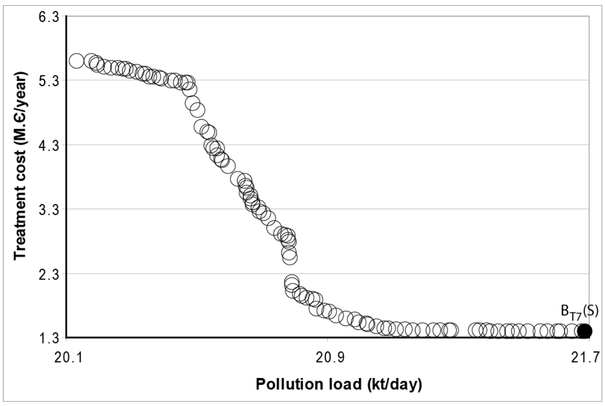
At 1 hour



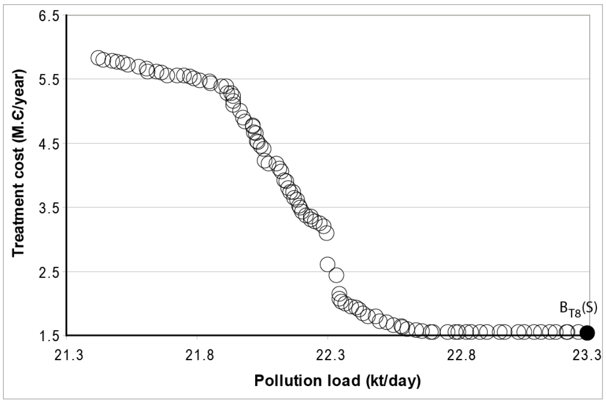
At 1 hour & 15 minutes



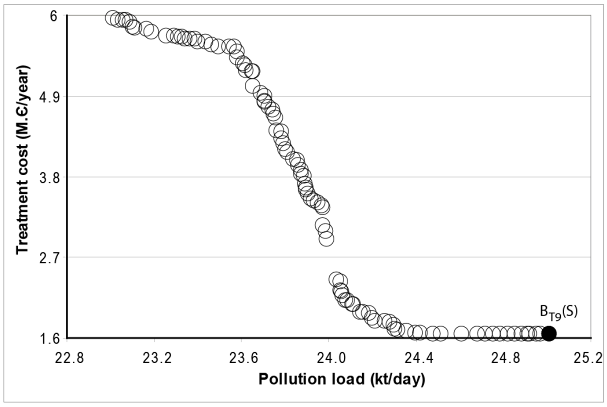
At 1 hour & 30 minutes



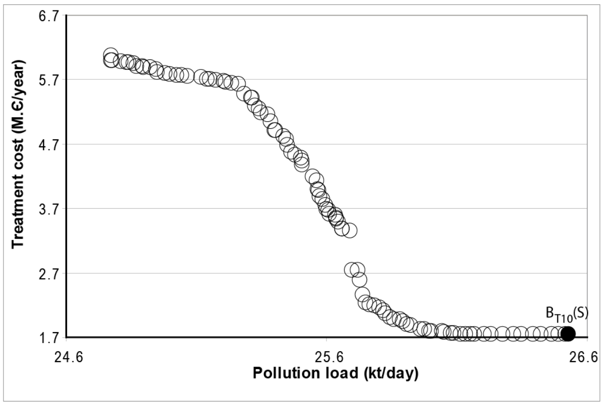
At 1 hour & 45 minutes



At 2 hours



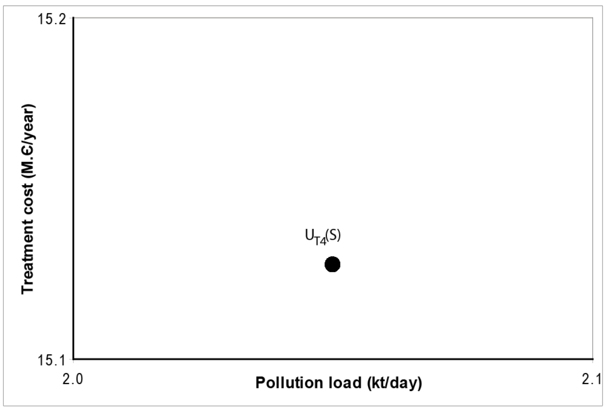
At 2 hours & 15 minutes



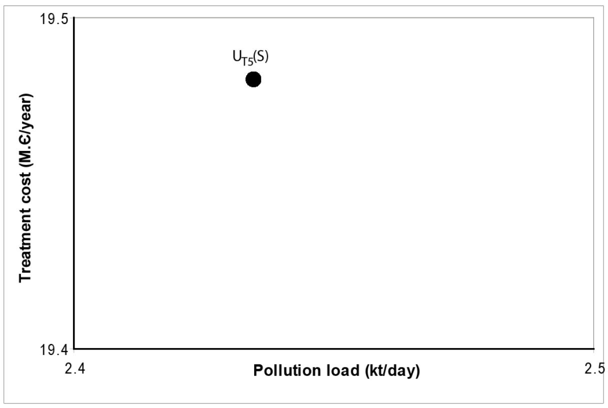
At 2 hours & 30 minutes

D.10 Selected solutions from SWMM constraint handling approach for single storm

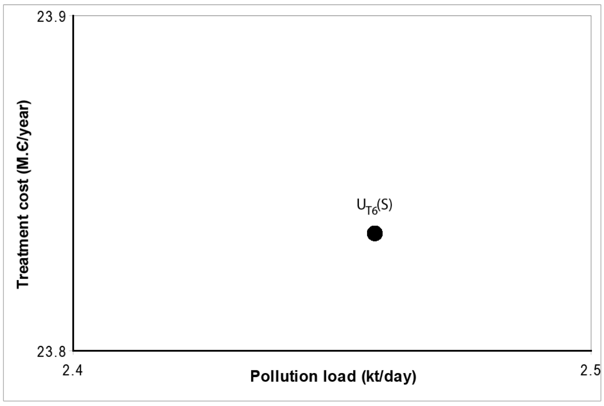
Minimum pollution load solutions



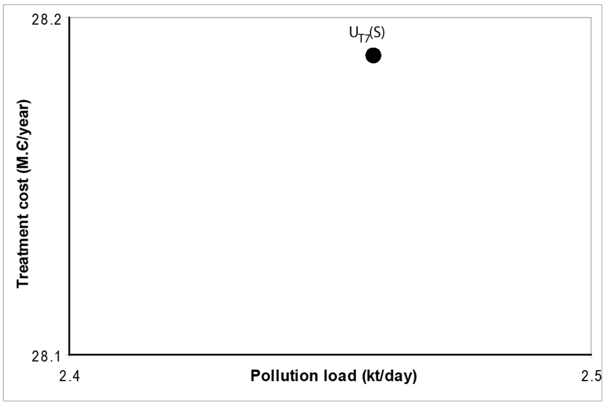
At 1 hour



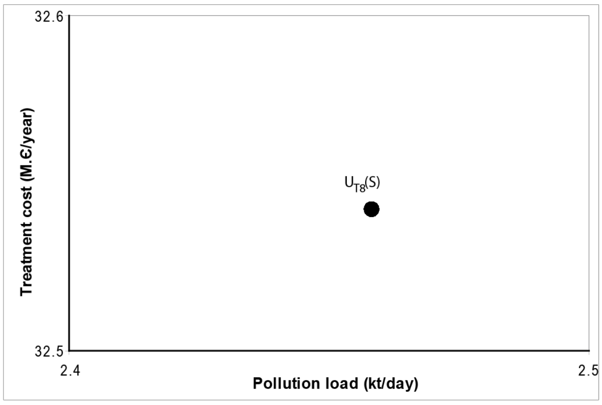
At 1 hour & 15 minutes



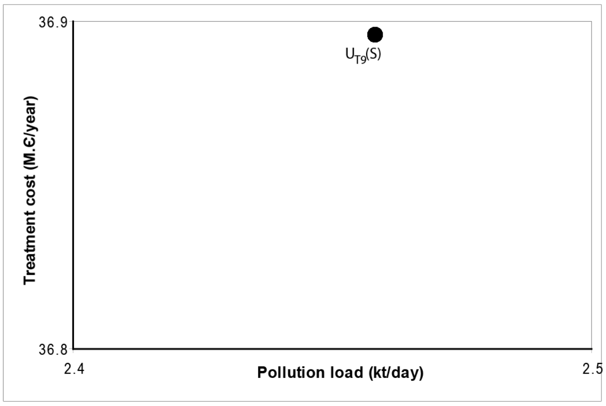
At 1 hour & 30 minutes



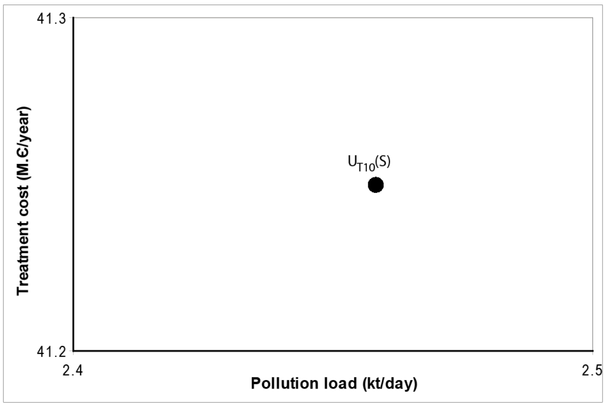
At 1 hour & 45 minutes



At 2 hours

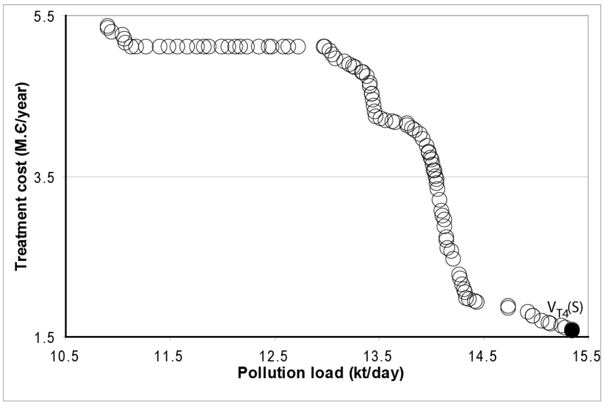


At 2 hours & 15 minutes

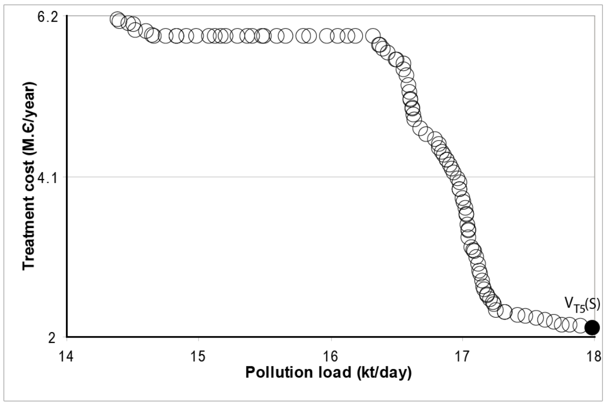


At 2 hours & 30 minutes

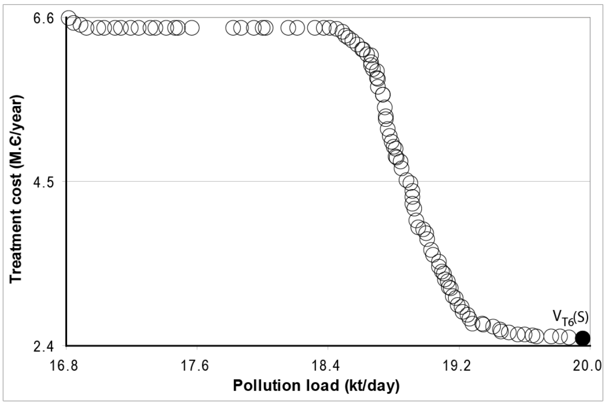
Minimum cost solutions



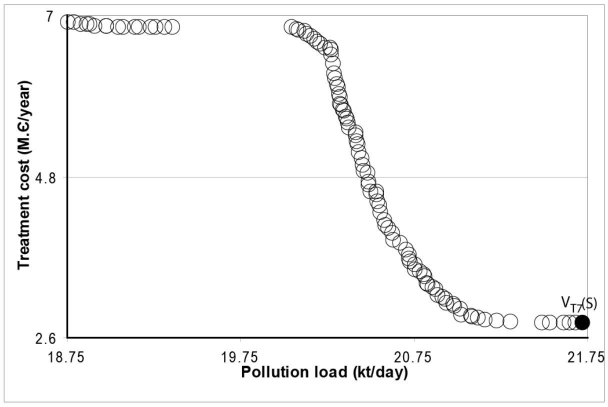
At 1 hour



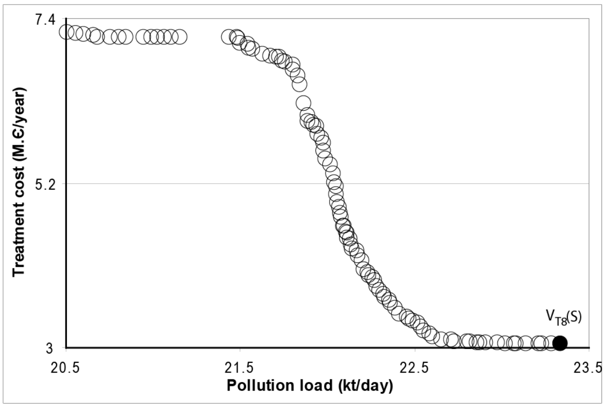
At 1 hour & 15 minutes



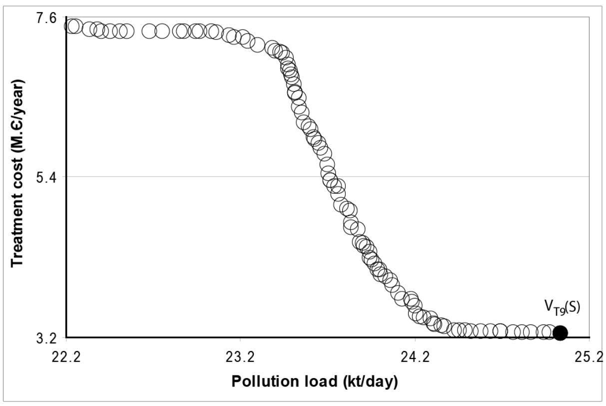
At 1 hour & 30 minutes



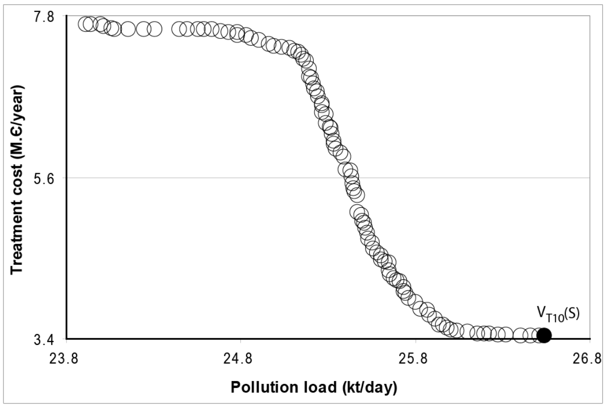
At 1 hour & 45 minutes



At 2 hours



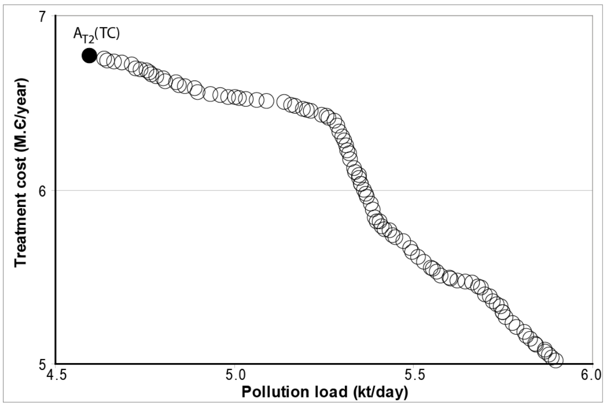
At 2 hours & 15 minutes



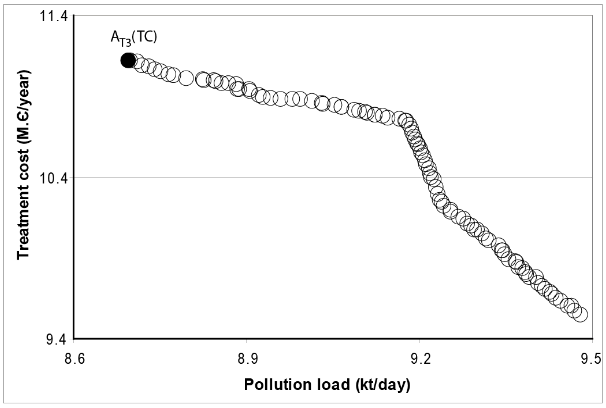
At 2 hours & 30 minutes

D.11 Selected solutions from NSGA II constraint handling approach for two consecutive storms

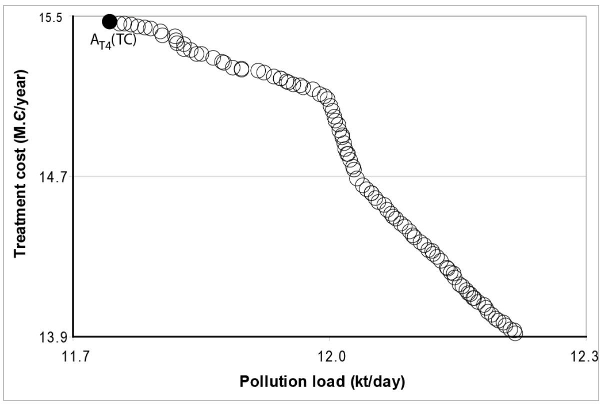
Minimum pollution load solutions



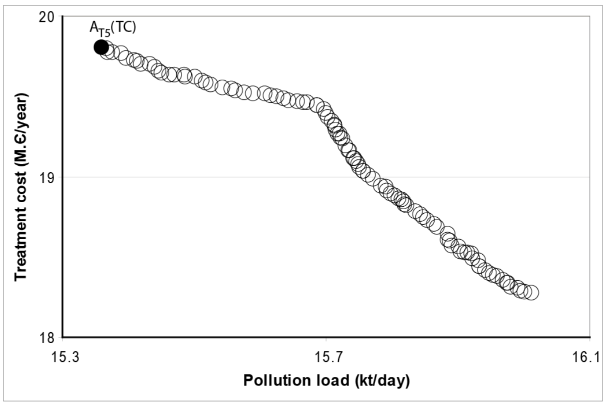
At 30 minutes



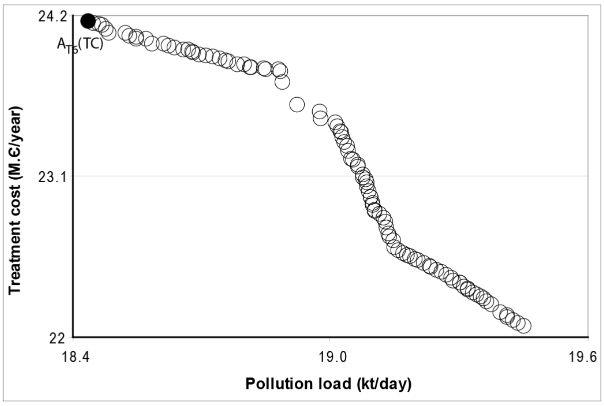
At 45 minutes



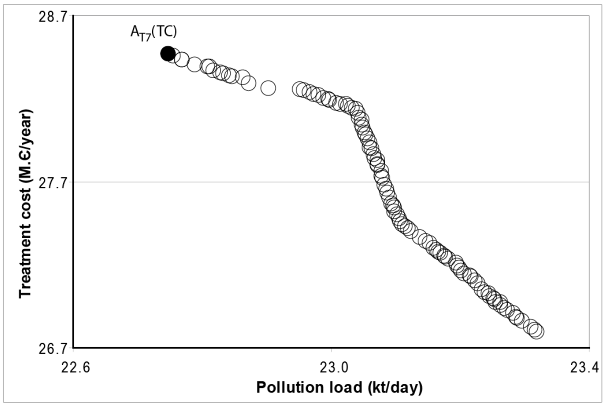
At 1 hour



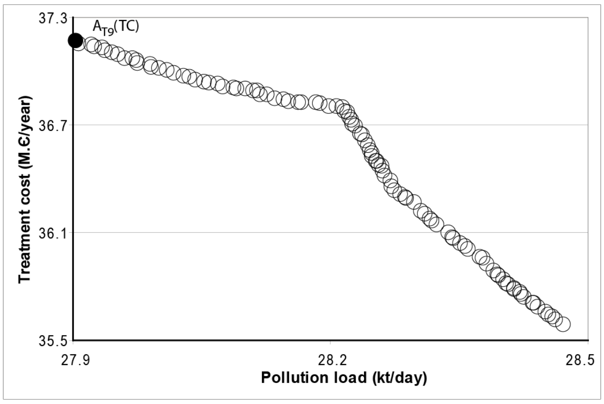
At 1 hour & 15 minutes



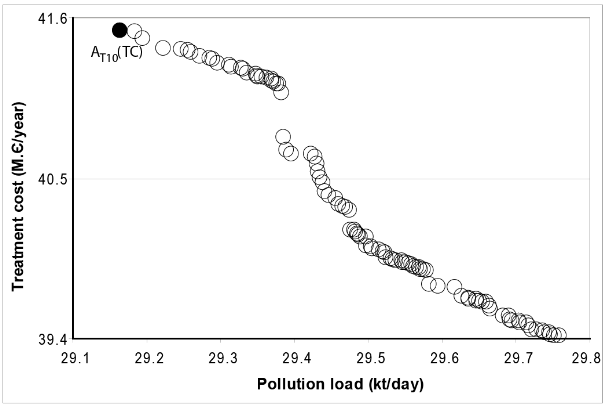
At 1 hour & 30 minutes



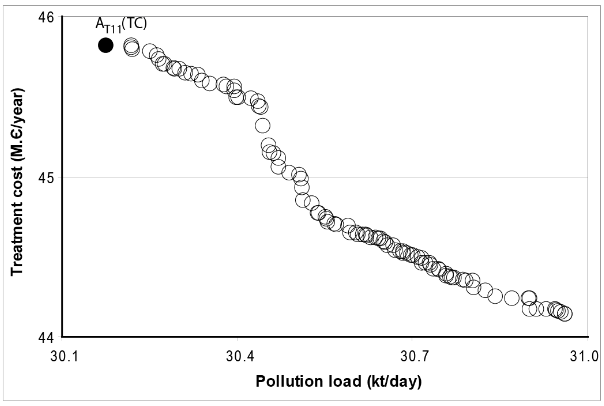
At 1 hour & 45 minutes



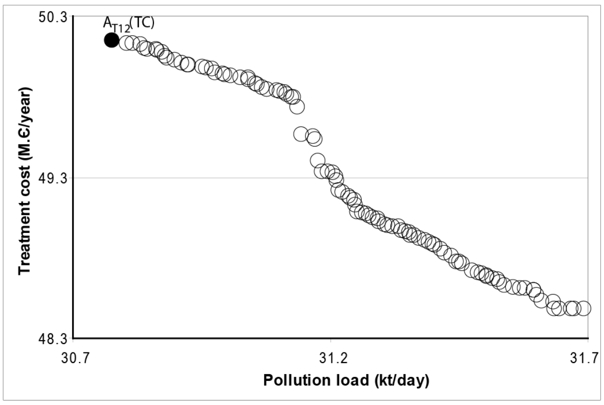
At 2 hours &15 minutes



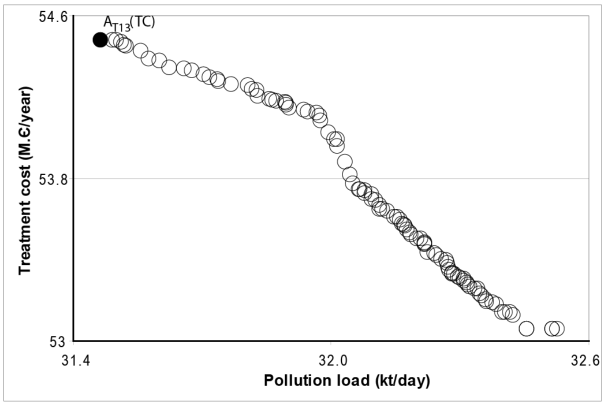
At 2 hours & 30 minutes



At 2 hours & 45 minutes

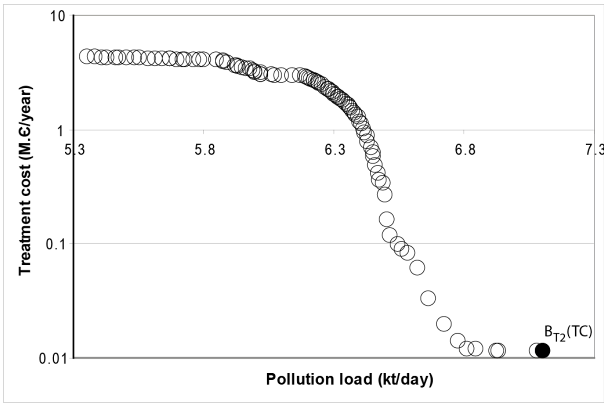


At 3 hours

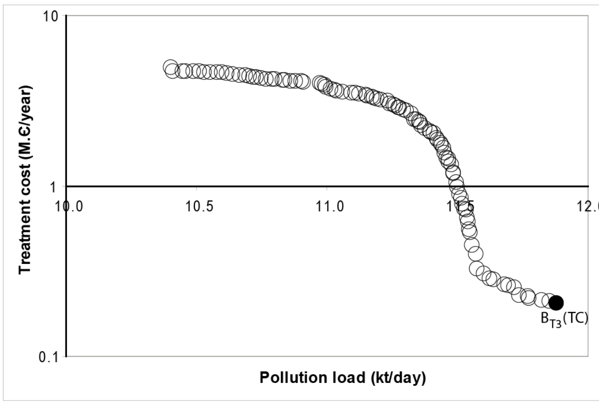


At 3 hours & 15 minutes

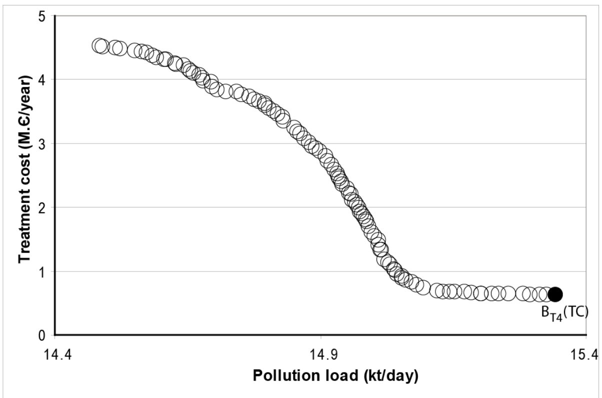
Minimum cost solutions



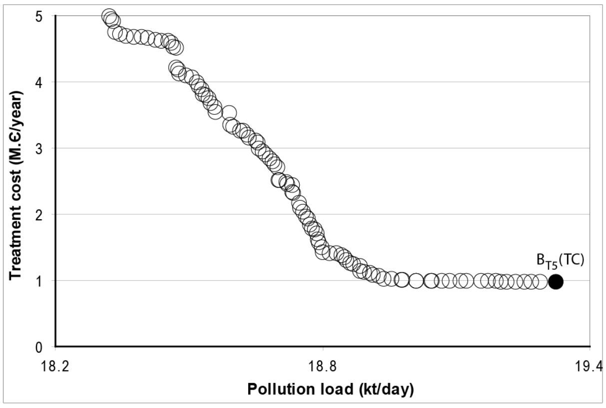
At 30 minutes



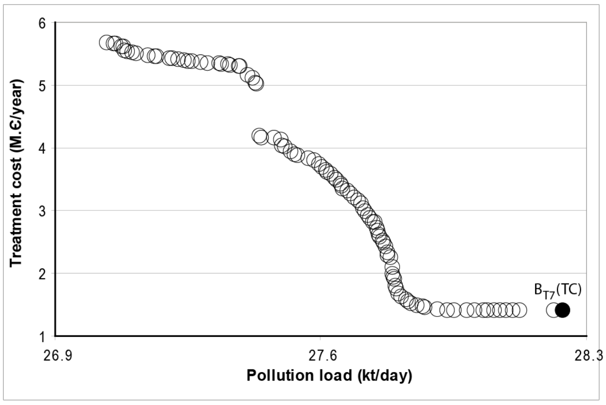
At 45 minutes



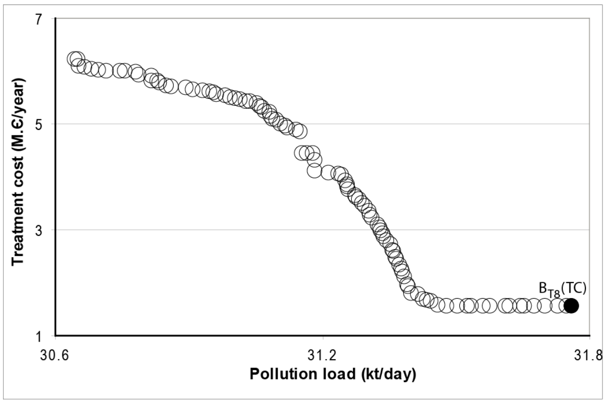
At 1 hour



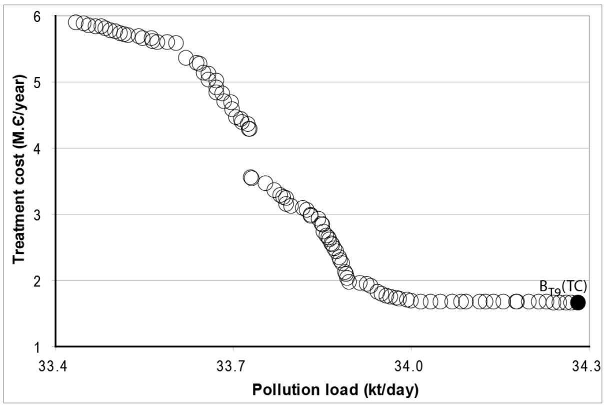
At 1 hour & 15 minutes



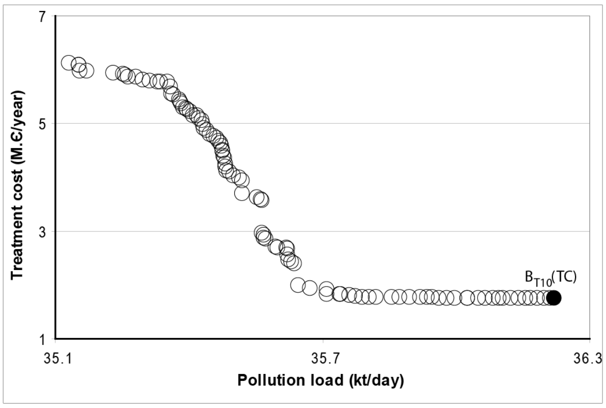
At 1 hour & 45 minutes



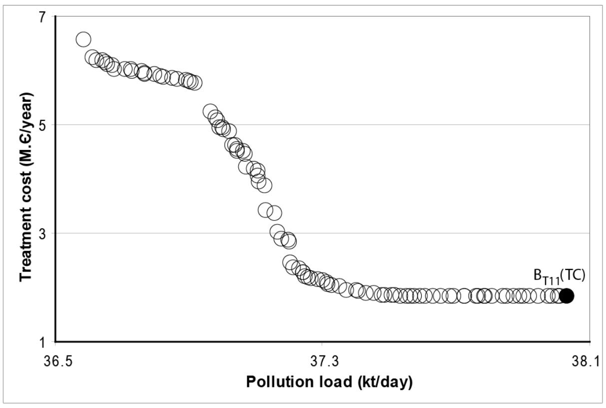
At 2 hours



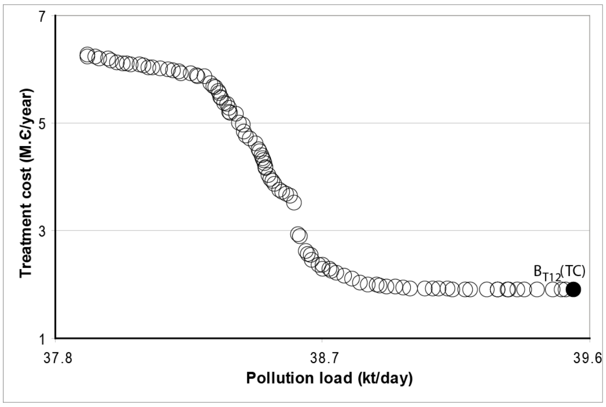
At 2 hours & 15 minutes



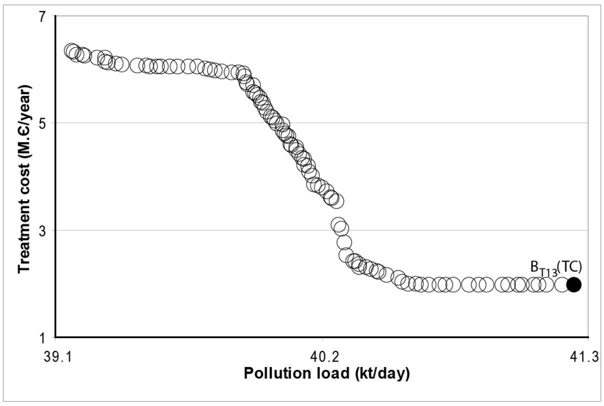
At 2 hours & 30 minutes



At 2 hours & 45 minutes



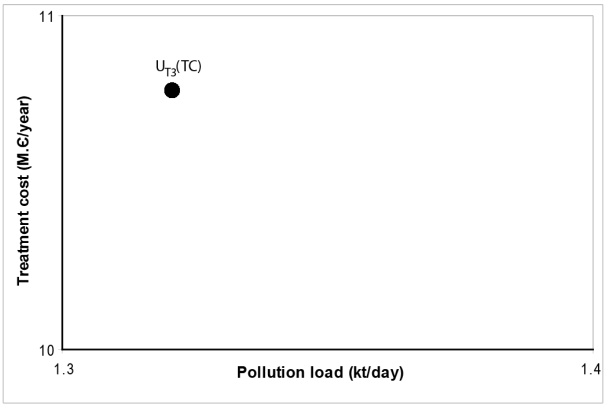
At 3 hours



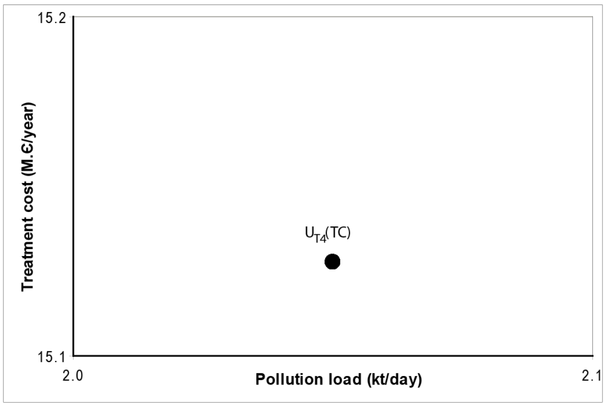
At 3 hours & 15 minutes

D.12 Selected solutions from SWMM constraint handling approach for two consecutive storms

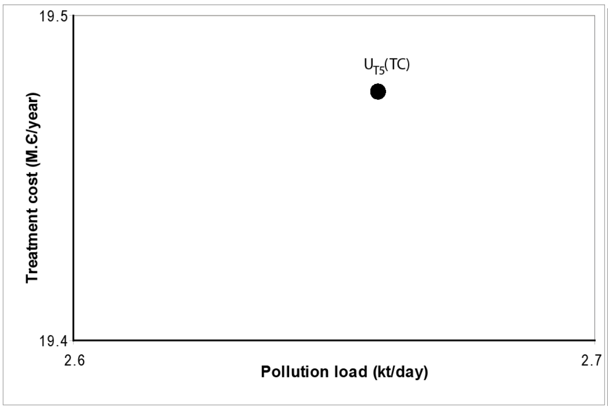
Minimum pollution load solutions



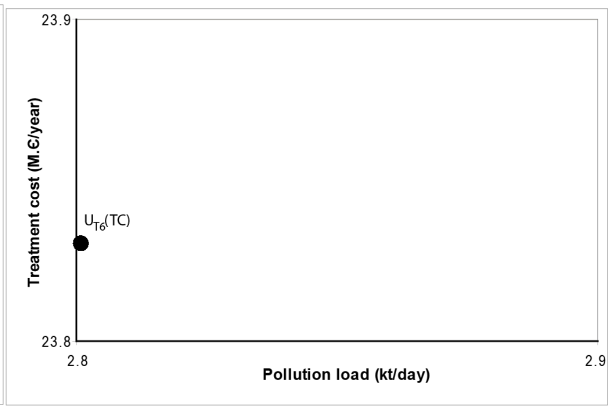
At 45 minutes



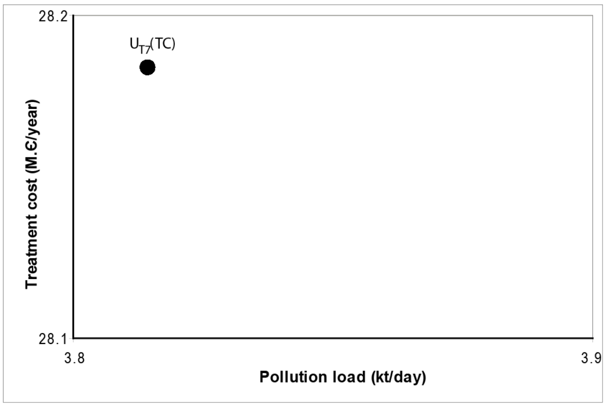
At 1 hour



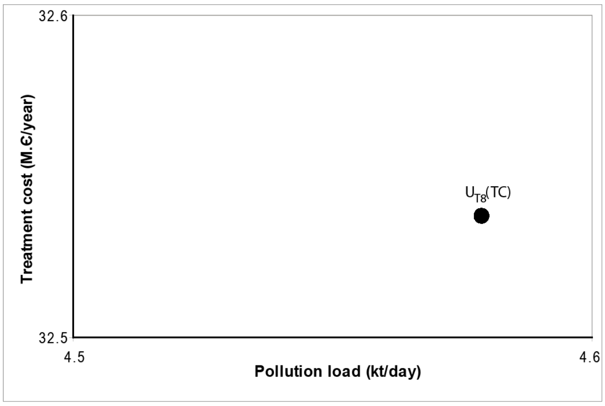
At 1 hour & 15 minutes



At 1 hour & 30 minutes

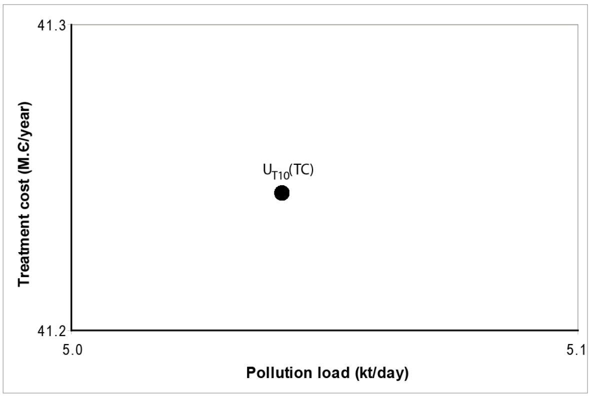


At 1 hour & 45 minutes

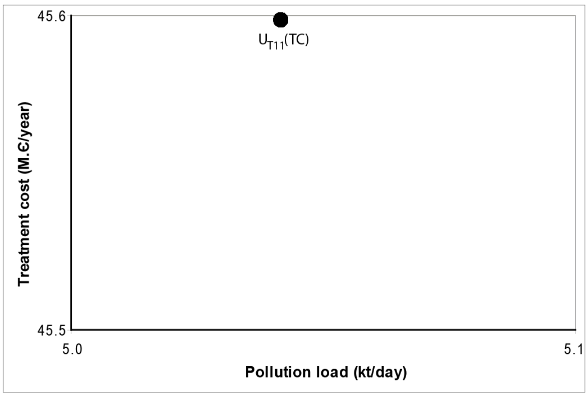


At 2 hours

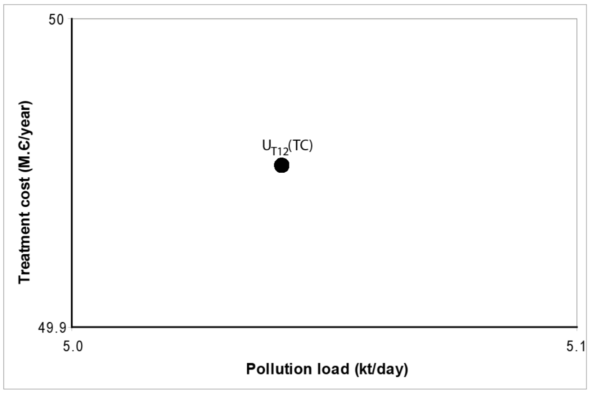
At 2 hours & 15 minutes



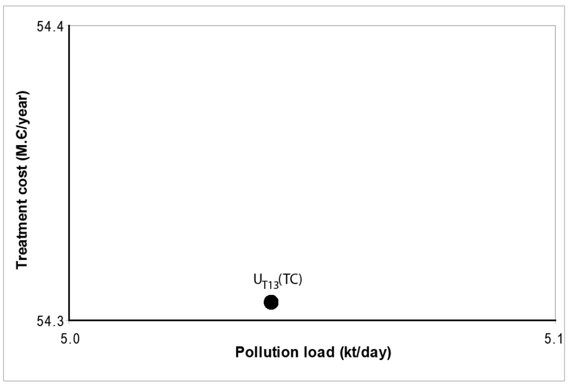
At 2 hours & 30 minutes



At 2 hours & 45 minutes

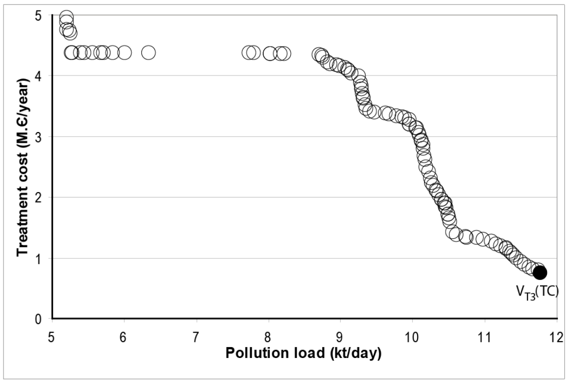


At 3 hours

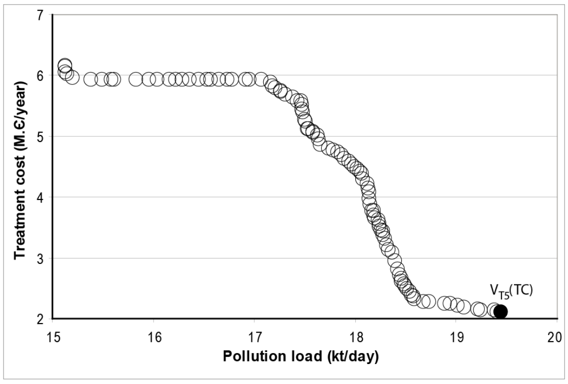


At 3 hours &15 minutes

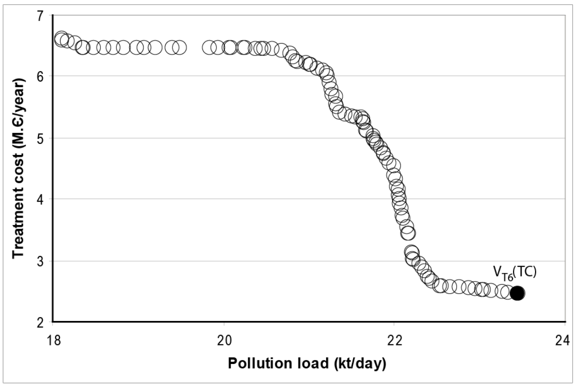
Minimum cost solutions



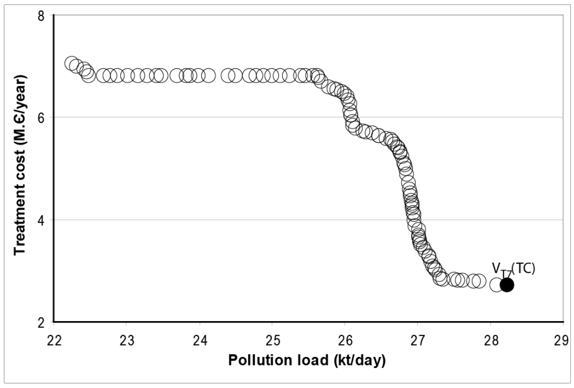
At 45 minutes



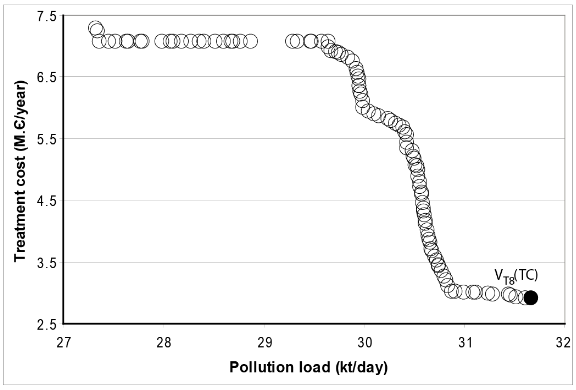
At 1 hour & 15 minutes



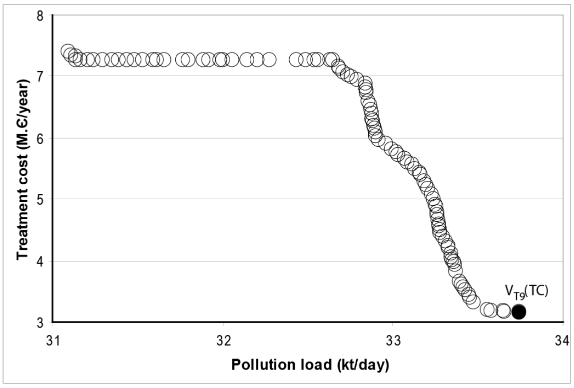
At 1 hour & 30 minutes



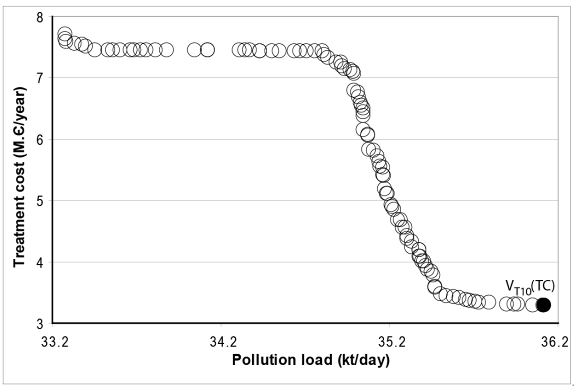
At 1 hour & 45 minutes



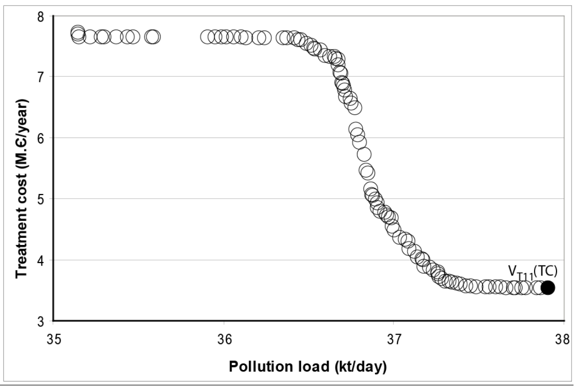
At 2 hours



At 2 hours & 15 minutes



At 2hours & 30 minutes



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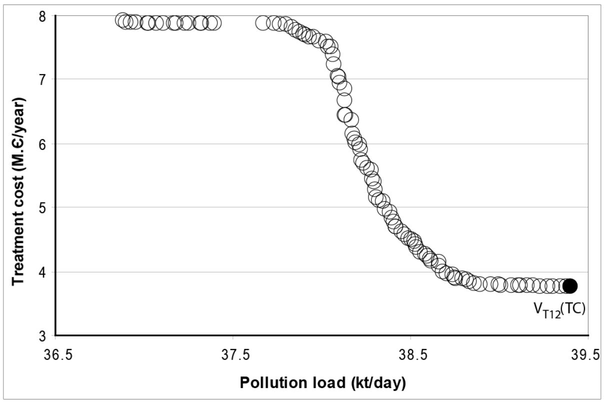
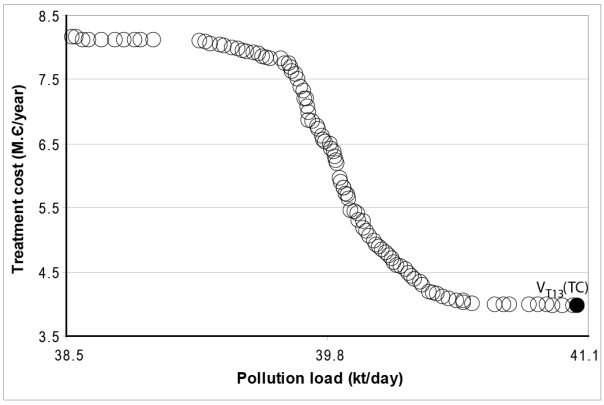
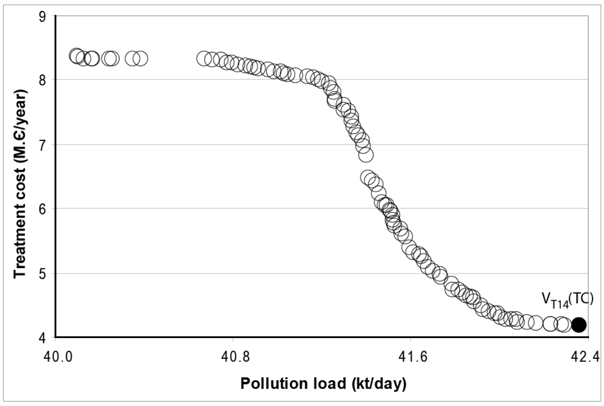


At 3 hours



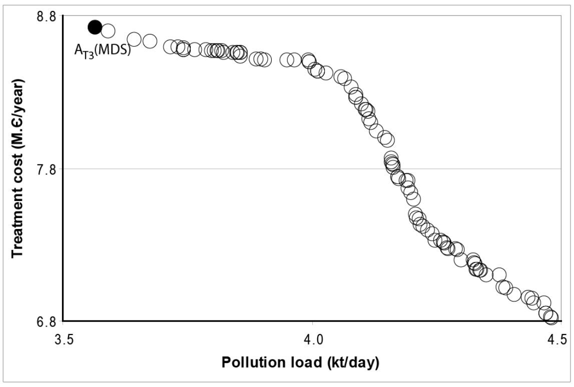
At 3 hours & 15 minutes



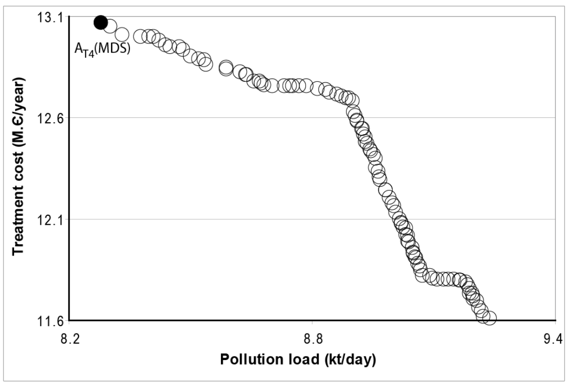
At 3 hours & 30 minutes

D.13 Selected solutions from NSGA II constraint handling approach for migrating downstream storms

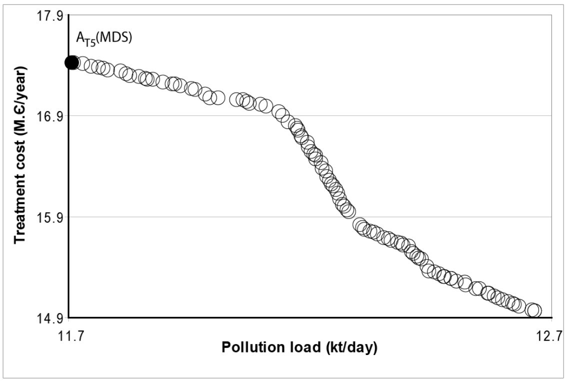
Minimum pollution load solutions



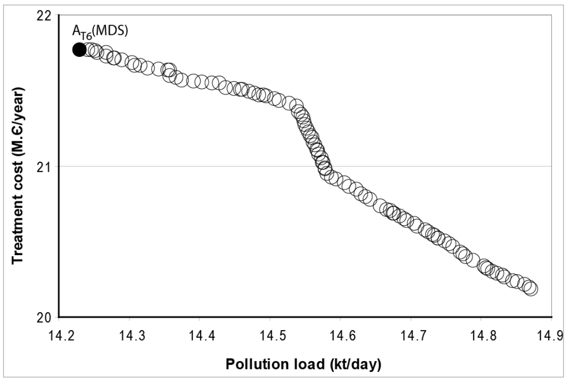
At 45 minutes



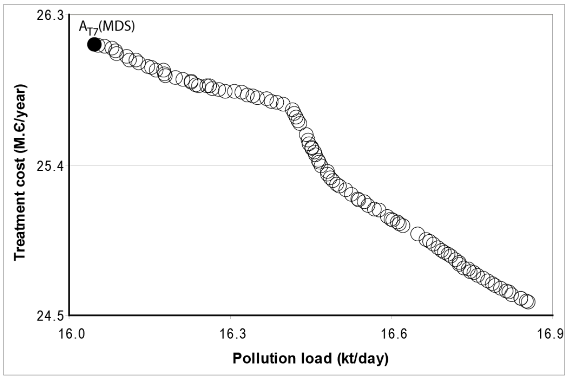
At 1 hour



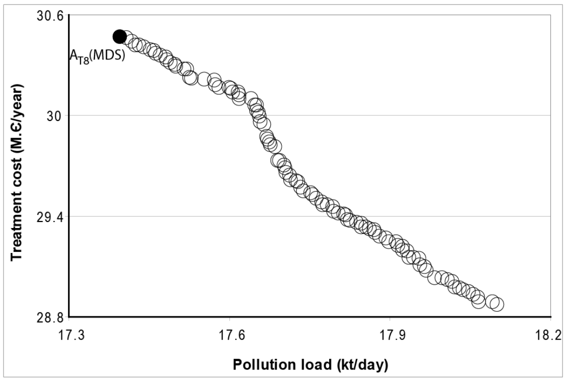
At 1 hour & 15 minutes



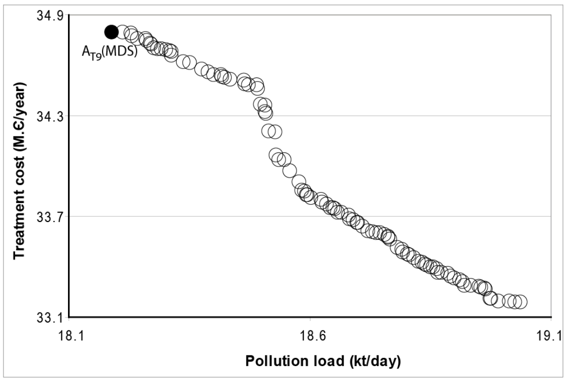
At 1 hour & 30 minutes



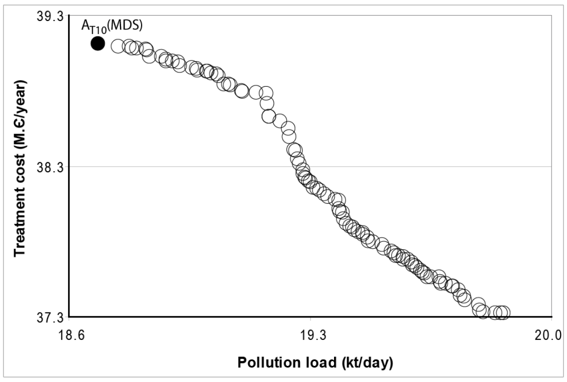
At 1 hour & 45 minutes



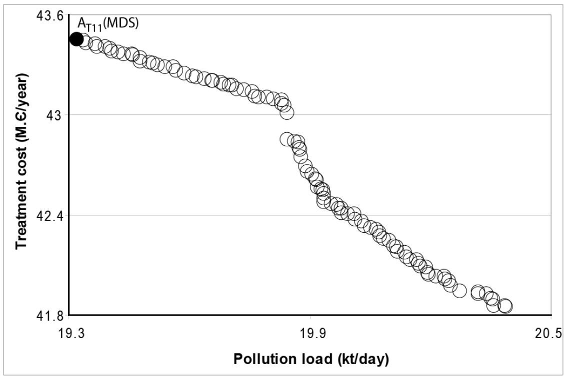
At 2 hours



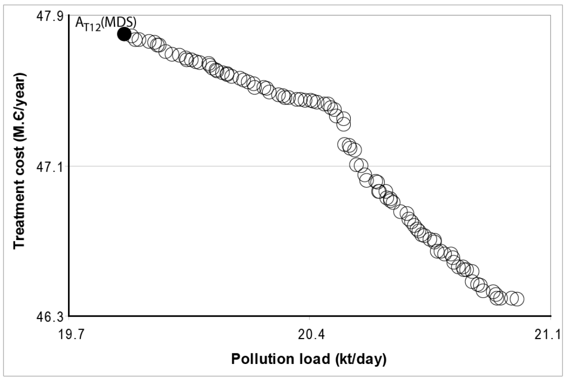
At 2 hours & 15 minutes



At 2 hours & 30 minutes

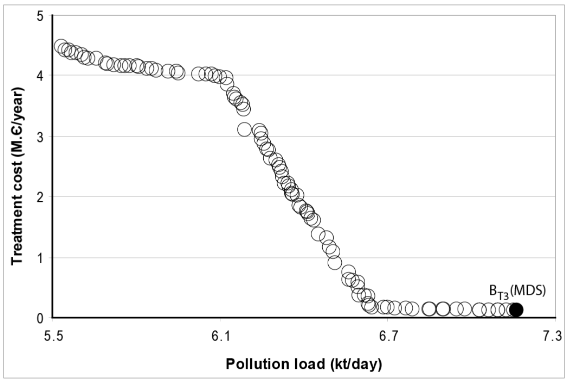


At 2 hours & 45 minutes

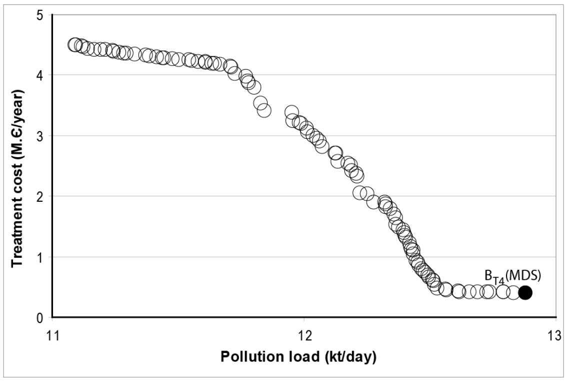


At 3 hours

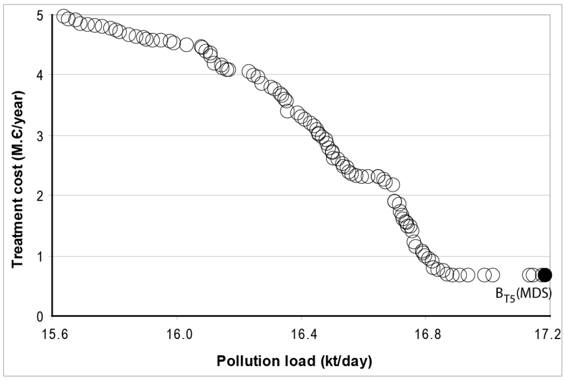
Minimum cost solutions



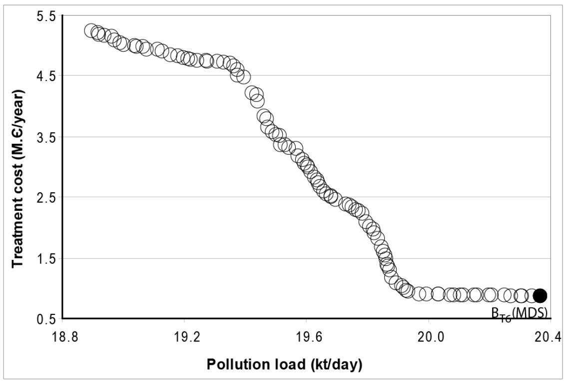
At 45 minutes



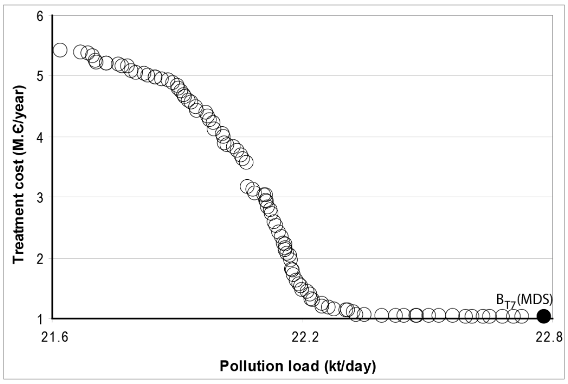
At 1 hour



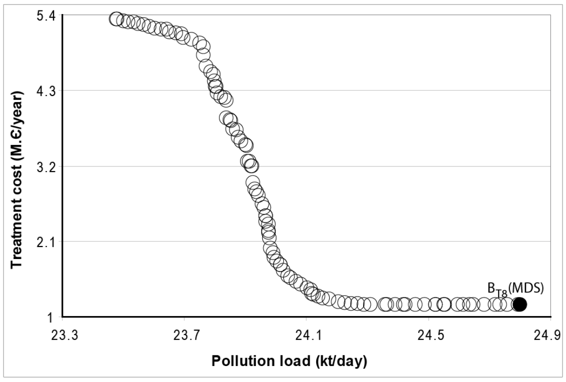
At 1 hour & 15 minutes



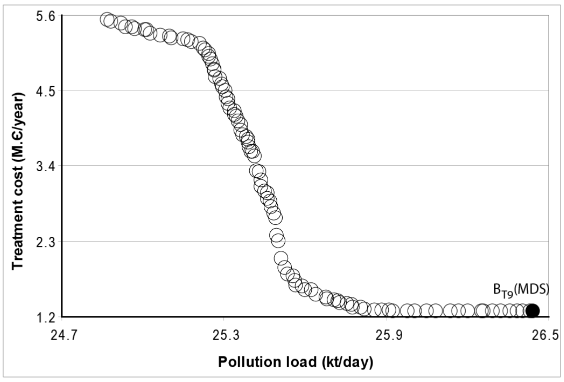
At 1 hour & 30 minutes



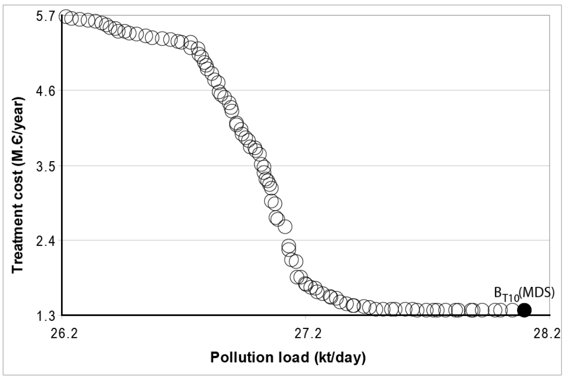
At 1 hour & 45 minutes



At 2 hours



At 2 hours & 15 minutes



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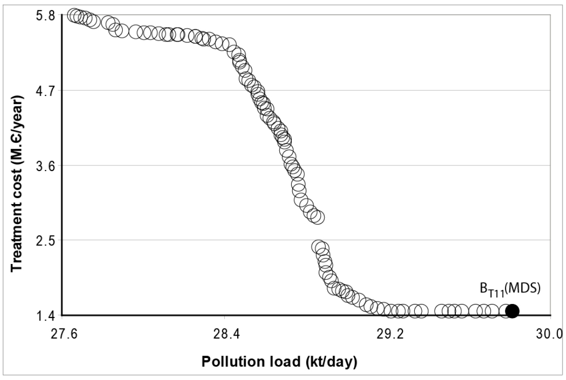
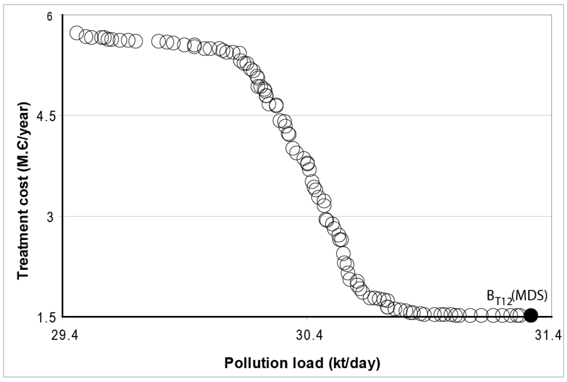


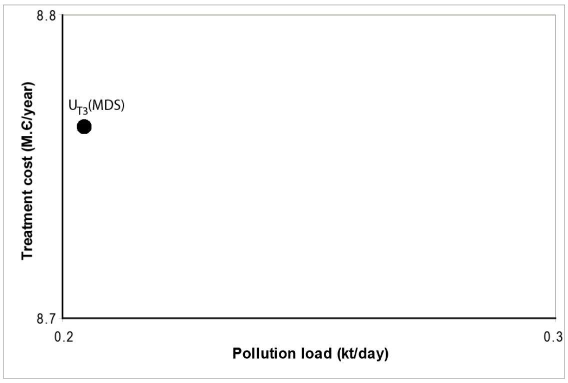
2 hours & 45 minutes



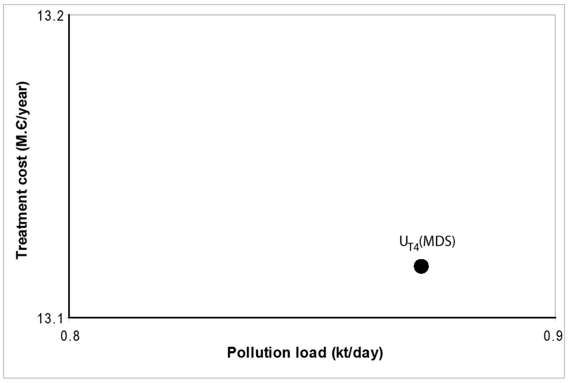
3 hours

D.14 Selected solutions from SWMM constraint handling approach for migrating downstream storms

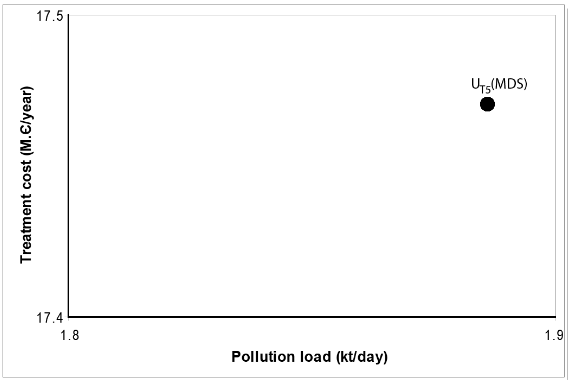
Minimum pollution load solutions



At 45 minutes

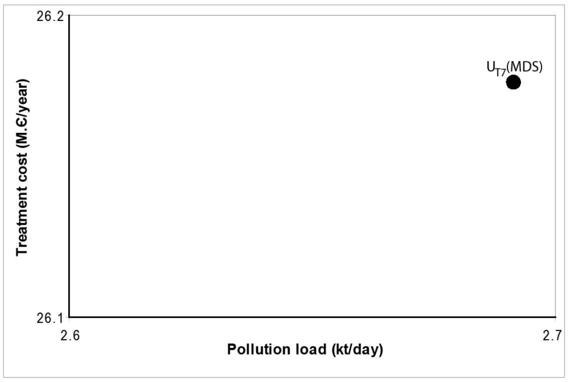


At 1 hour

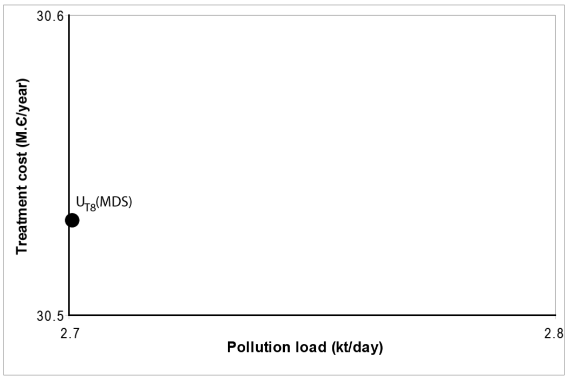


At 1 hour & 15 minutes

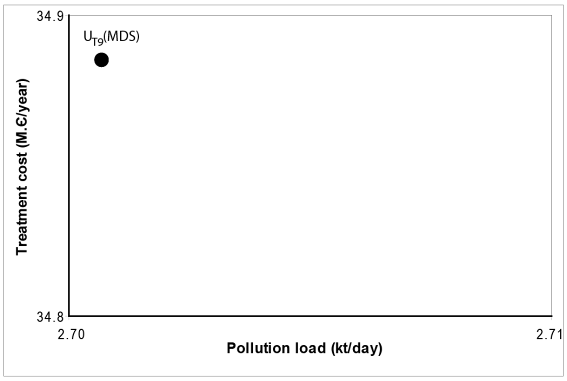
At 1 hour & 30 minutes



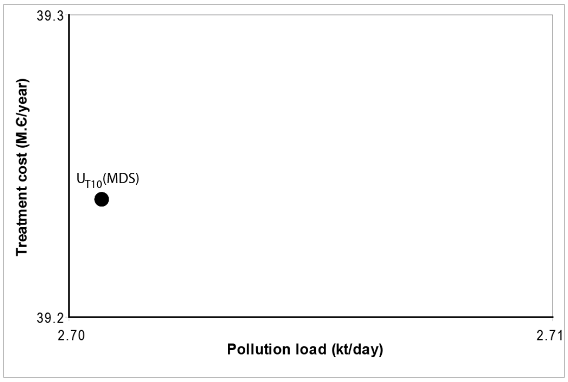
At 1 hour & 45 minutes



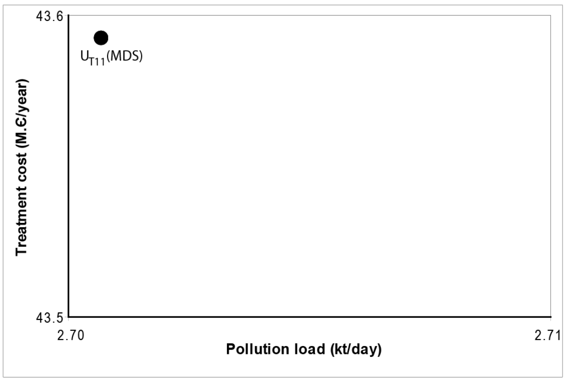
At 2 hours



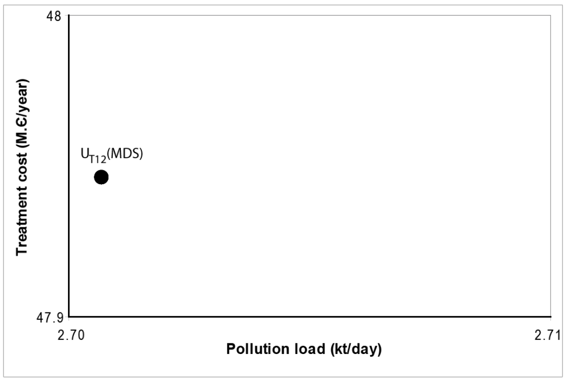
At 2 hours & 15 minutes



At 2 hours & 30 minutes

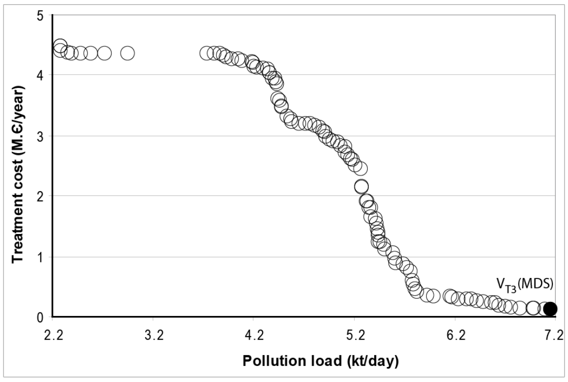


2 hours & 45 minutes



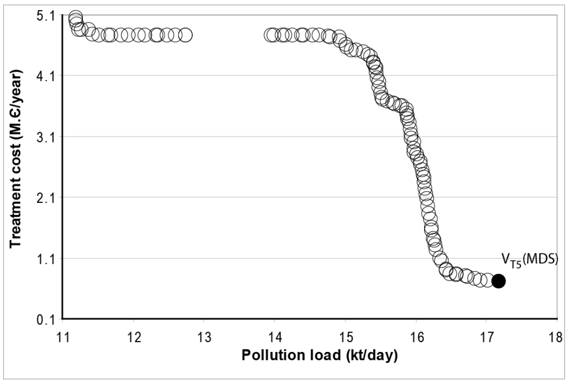
3 hours

Minimum cost solutions

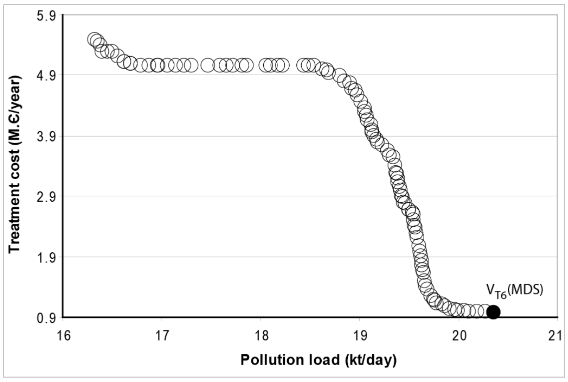


At 45 minutes

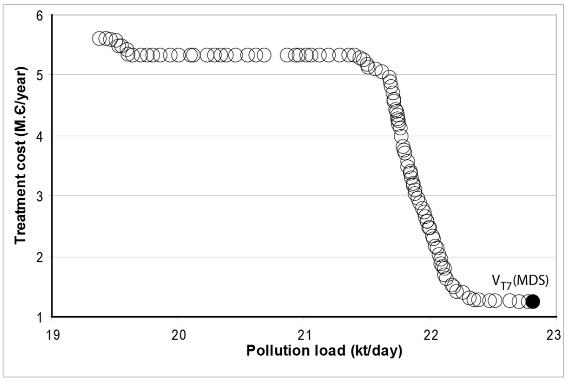
At 1 hour



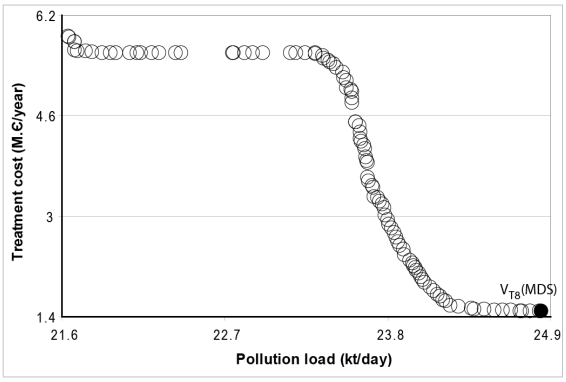
At 1 hour & 15 minutes



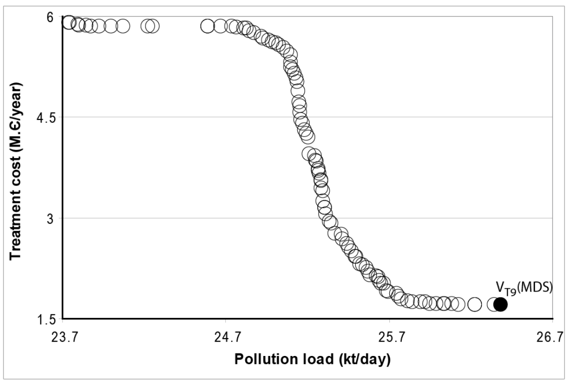
At 1 hour & 30 minutes



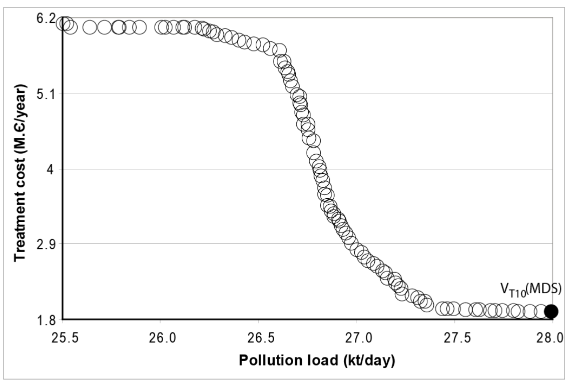
At 1 hour & 45 minutes



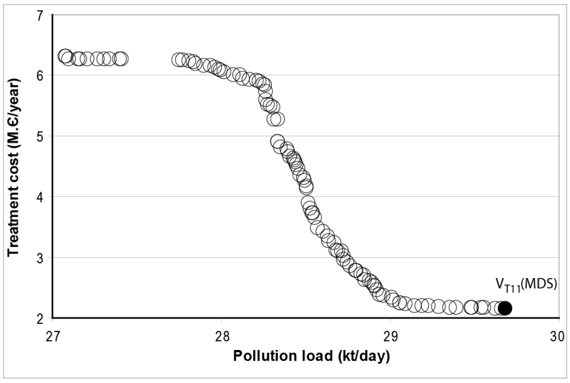
At 2 hours



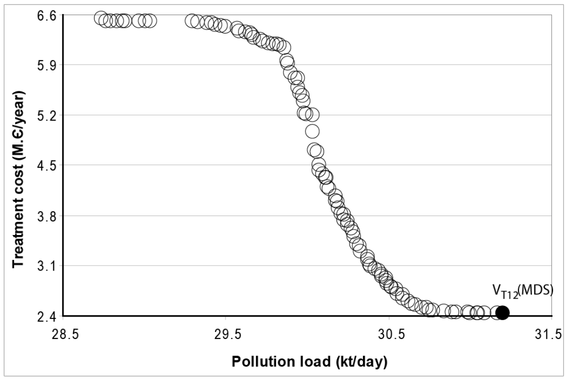
At 2 hours & 15 minutes



At 2 hours & 30 minutes



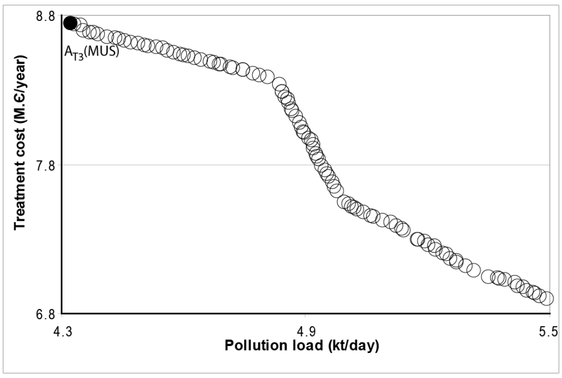
At 2 hours & 45 minutes



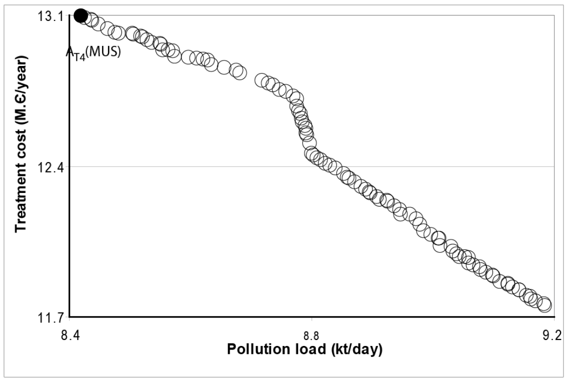
At 3 hours

D.15 Selected solutions from NSGA II constraint handling approach for migrating upstream storms

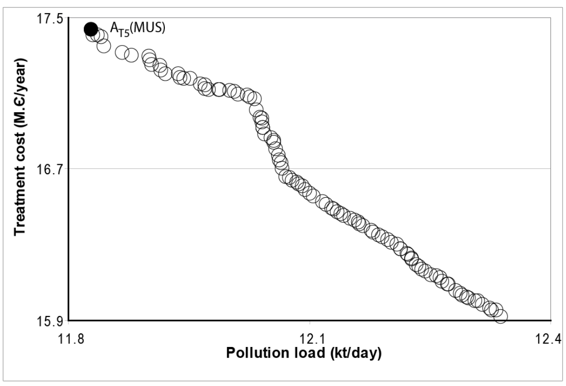
Minimum pollution load solutions



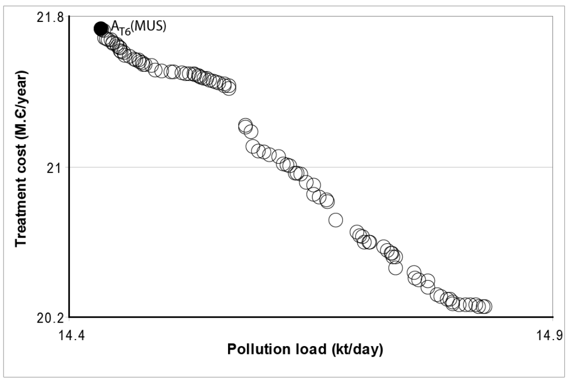
At 45 minutes



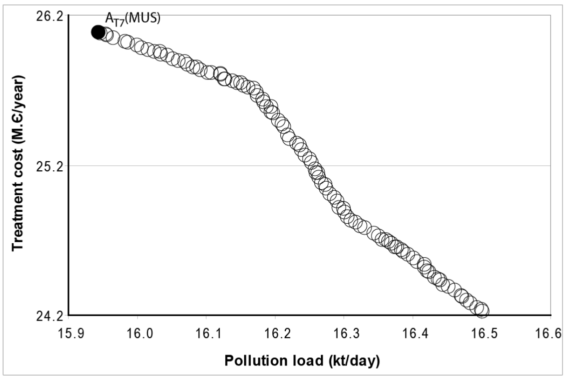
At 1 hour



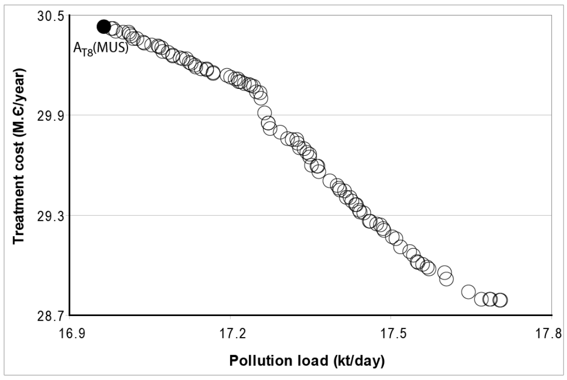
At 1 hour & 15 minutes



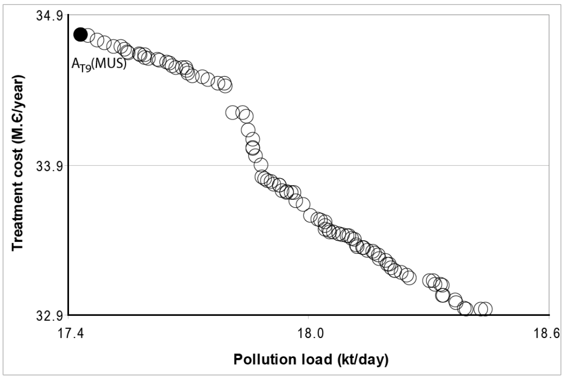
At 1 hour & 30 minutes



At 1 hour & 45 minutes



At 2 hours



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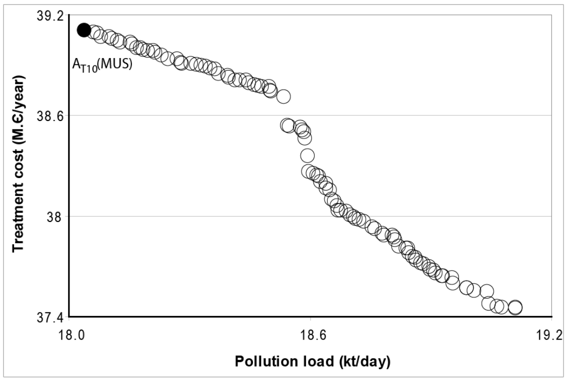

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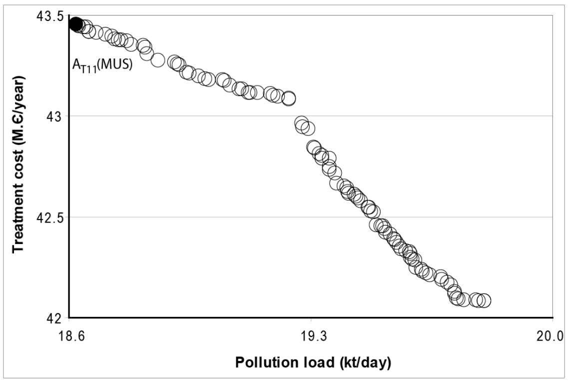
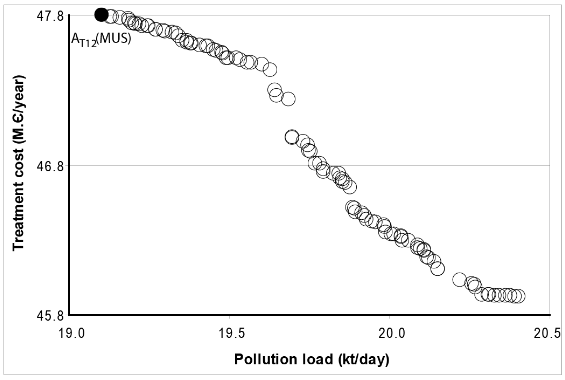
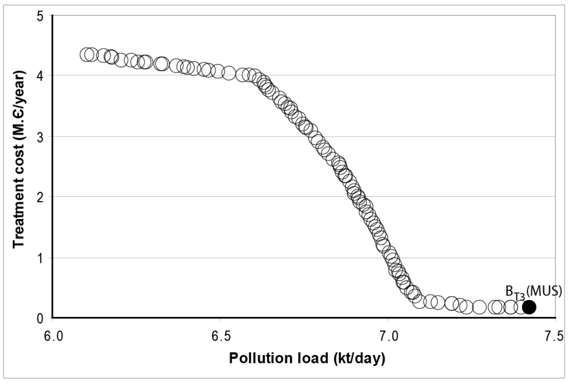


At 2 hours & 45 minutes

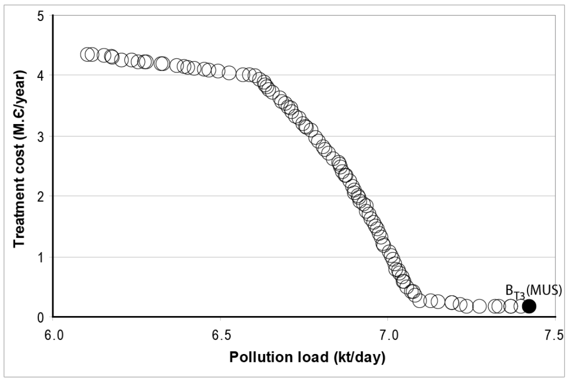


At 3 hours

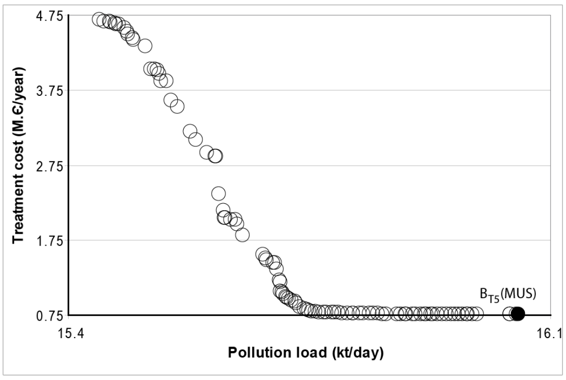
Minimum cost solutions



At 45 minutes



At 1 hour



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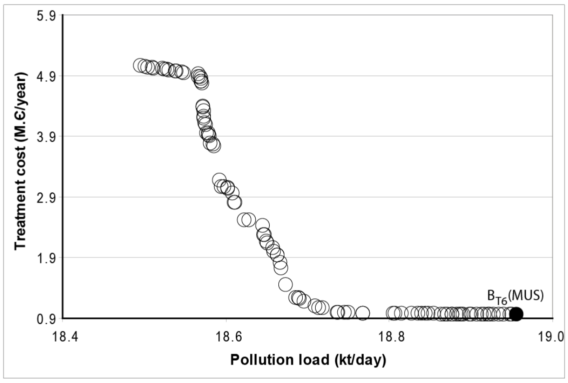

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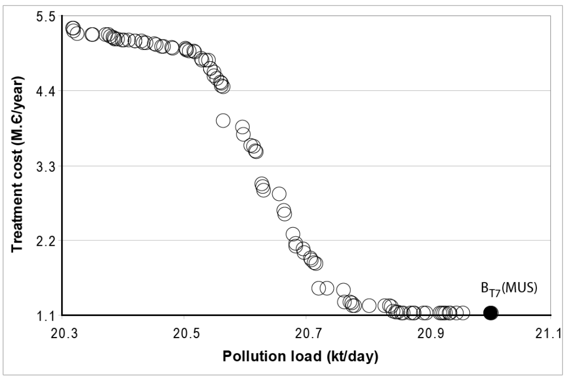

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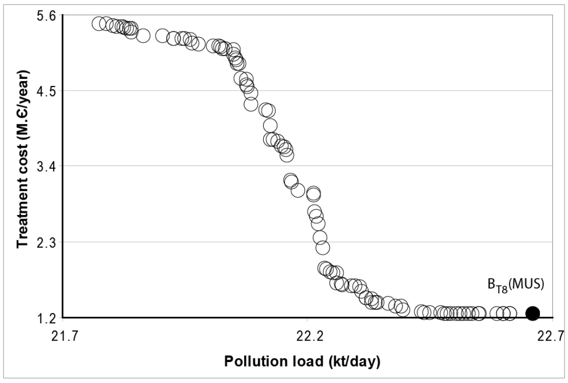
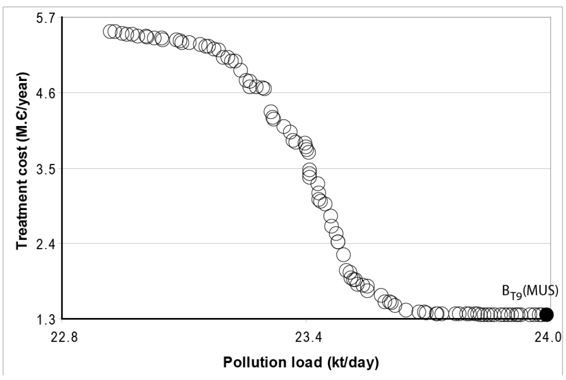
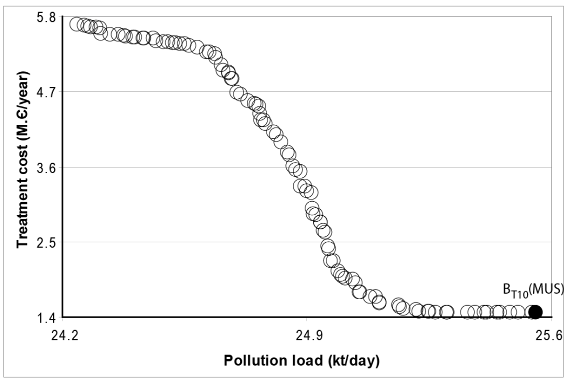
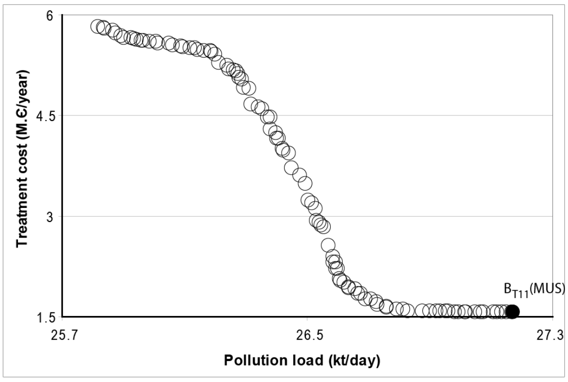


At 2 hours

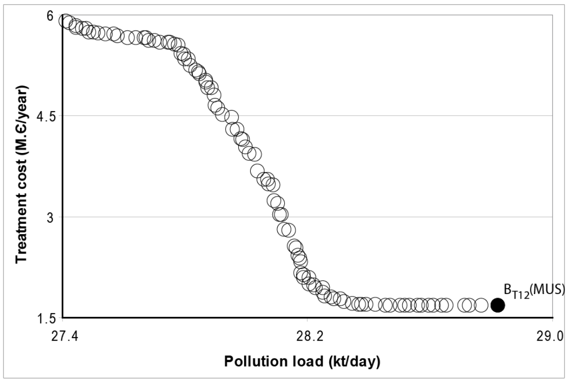




At 2 hours & 15 minutes At 2 hours & 30 minutes



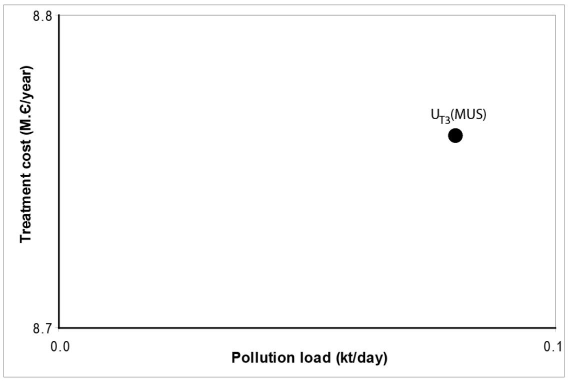
At 2 hours & 45 minutes



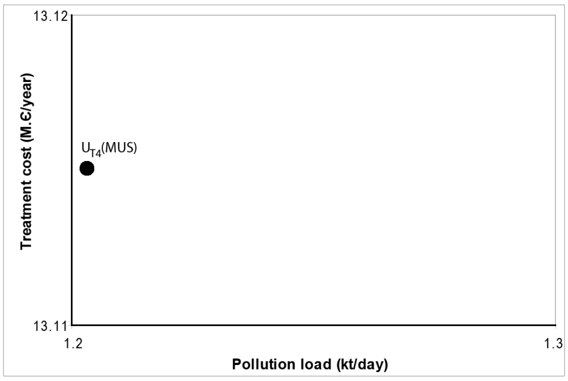
At 3 hours

D.16 Selected solutions from SWMM constraint handling approach for migrating upstream storms

Minimum pollution load solutions



At 45 minutes



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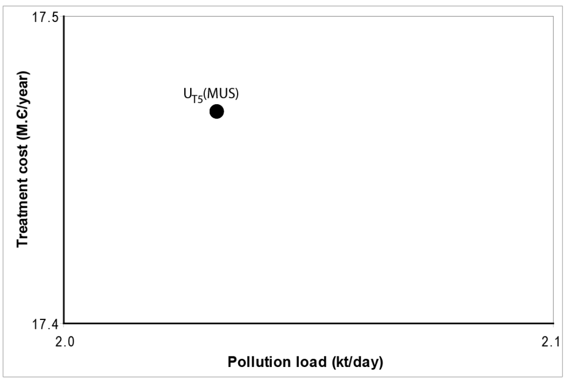
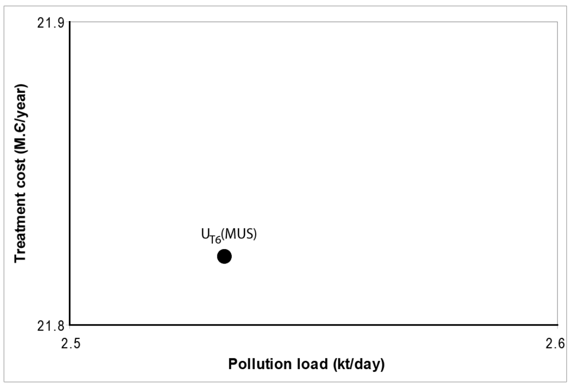


At 1 hour & 15 minutes



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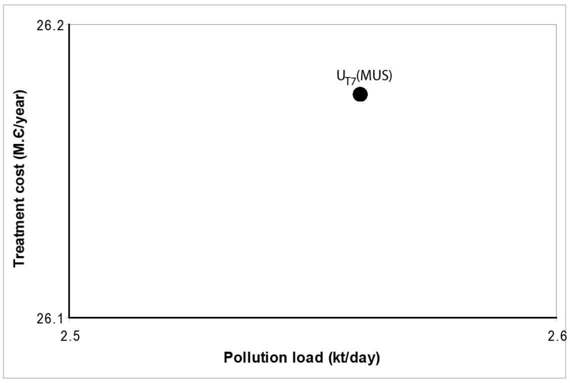
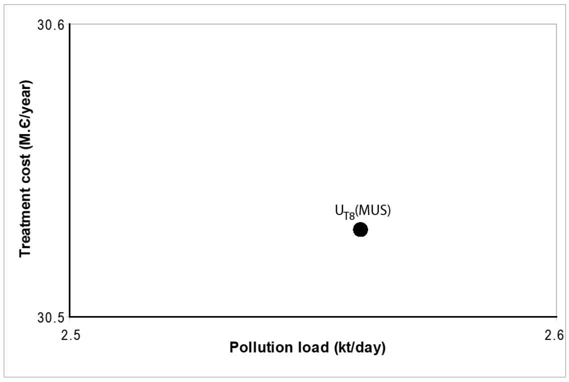
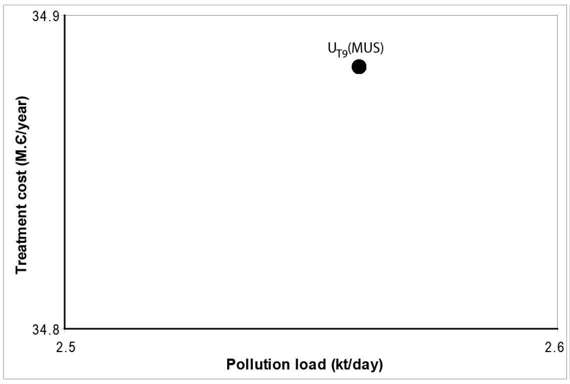


At 1 hour & 45 minutes



At 2 hours



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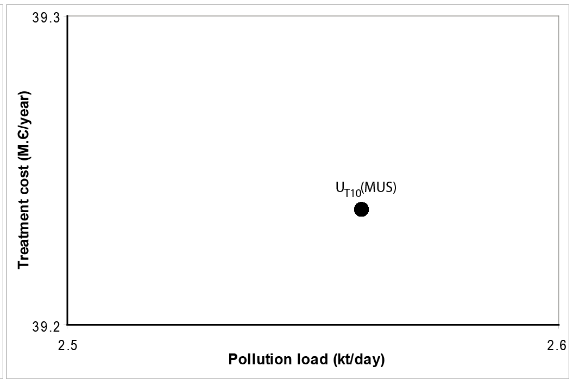

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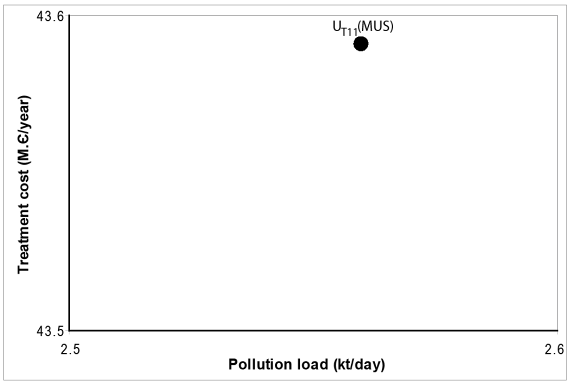

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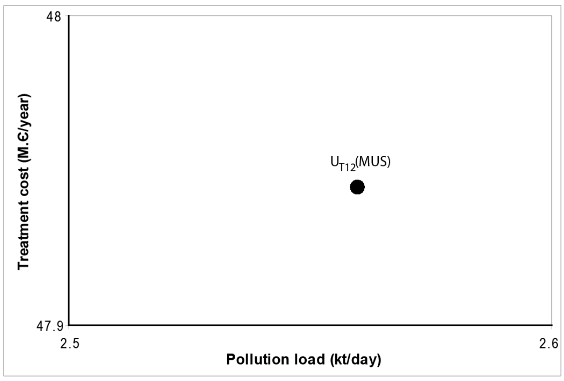
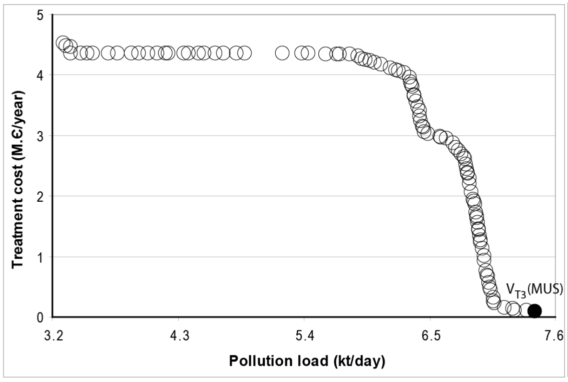


At 3 hours

Minimum cost solutions



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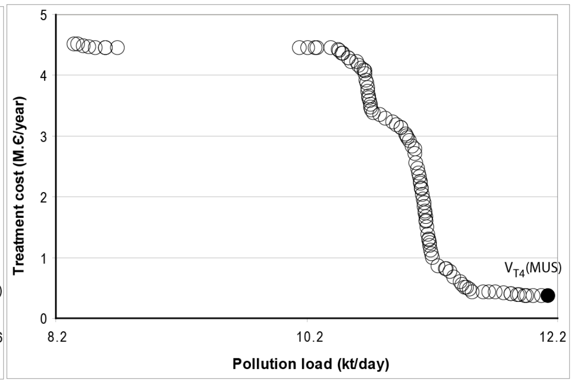

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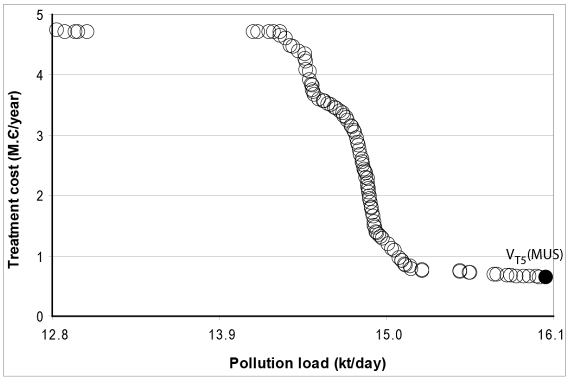

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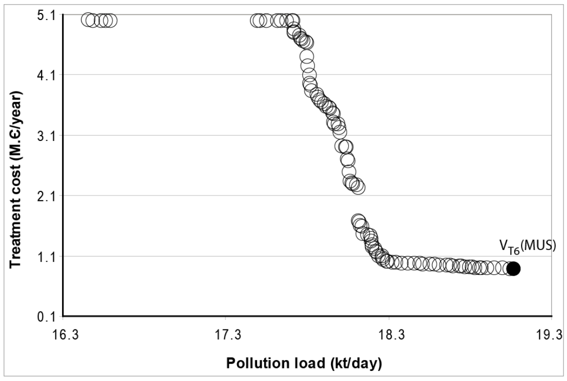
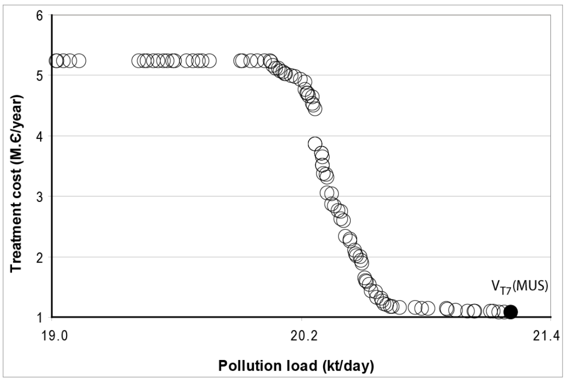
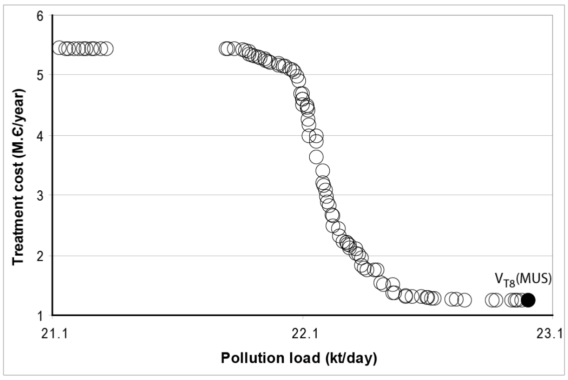


At 1 hour & 30 minutes



At 1 hour &45 minutes



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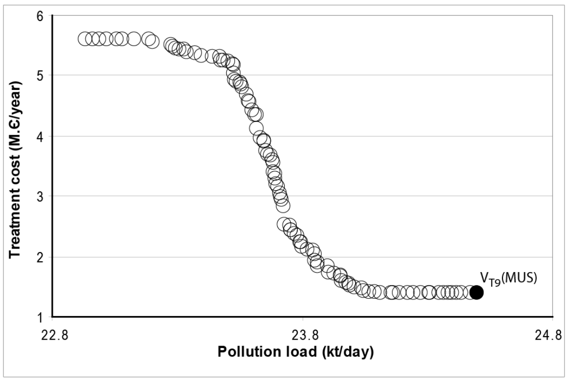
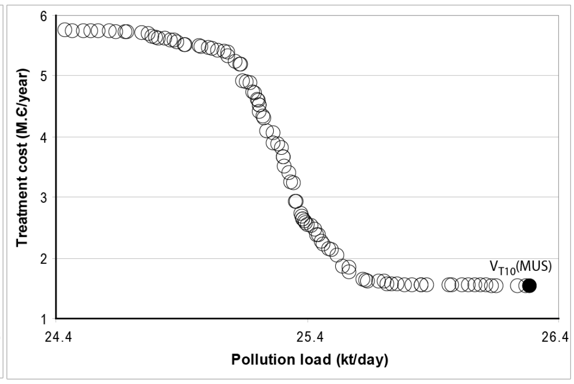
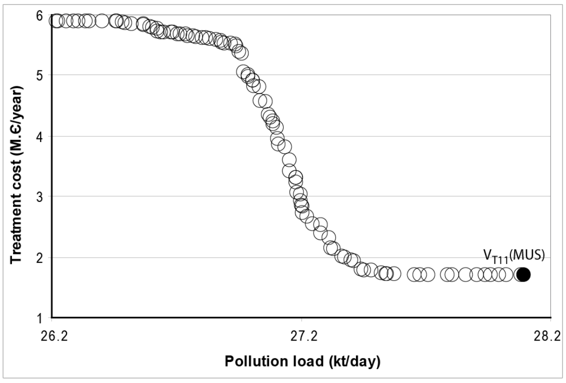
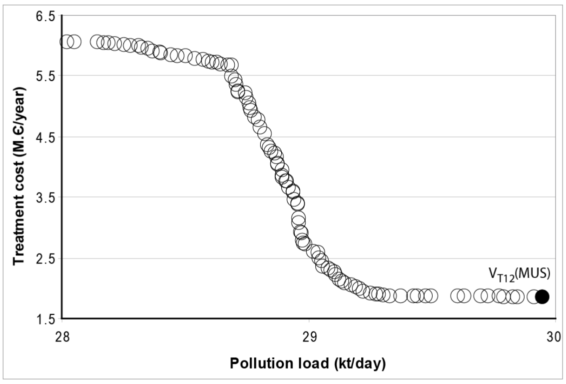




At 2 hours & 15 minutes | At 2 hours & 30 minutes





At 2 hours & 45 minutes | At 3 hours

APPENDIX E – HYDRAULIC SIMULATION RESULTS FOR DYNAMIC OPTIMIZATION

E.1 Single storm condition

Minimum cost approach under NSGA II constraint handling

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.569	0	4.741	4.361	0	0	0
00:30:00	8.872	3.475	7.965	9.516	12.443	3.814	3.611
00:45:00	11.767	3.290	7.263	16.579	16.579	4.105	2.034
01:00:00	11.008	2.407	4.537	16.579	16.579	3.438	1.917
01:15:00	6.668	1.316	2.707	16.579	16.579	2.059	1.666
01:30:00	3.304	0.840	1.265	9.434	13.138	1.098	1.214
01:45:00	3.039	0.840	1.251	4.172	9.528	1.122	1.232
02:00:00	3.038	0.840	1.268	1.630	5.130	1.029	1.231
02:15:00	3.038	0.840	1.270	1.630	5.130	0.769	1.231
02:30:00	2.516	0.632	1.129	1.294	4.644	0.702	0.901

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.504	0	0.692	0.692	0	0	0
00:30:00	0.913	0.318	0.611	0.611	1.536	0.491	0.367
00:45:00	0.809	0.225	0.469	0.469	1.235	0.318	0.140
01:00:00	0.613	0.138	0.319	0.319	1.013	0.197	0.103
01:15:00	0.426	0.083	0.251	0.251	0.886	0.117	0.096
01:30:00	0.308	0.069	0.205	0.205	0.795	0.077	0.092
01:45:00	0.361	0.078	0.234	0.234	0.659	0.094	0.113
02:00:00	0.375	0.082	0.239	0.239	0.543	0.093	0.126
02:15:00	0.378	0.084	0.239	0.239	0.625	0.073	0.133
02:30:00	0.319	0.065	0.215	0.215	0.591	0.069	0.101

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.84	6.26	8.39	8.45	8.18	7.18	7.62	1.79	7.24
00:30:00	6.08	7.26	8.57	8.72	9.01	9.01	10.09	7.05	8.31
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.11	7.05	8.31
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.04	7.05	8.31
01:15:00	5.97	7.09	8.25	9.04	9.18	8.94	9.98	7.05	8.31
01:30:00	5.76	7.05	8.13	8.73	9.04	8.93	9.98	7.05	8.31
01:45:00	5.74	7.05	8.13	8.45	8.87	8.93	9.98	7.05	8.31
02:00:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.05	8.31
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.05	8.31
02:30:00	5.71	7.03	8.12	8.23	8.61	8.91	9.99	7.05	8.31

Minimum pollution load approach under SWMM constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.74	1.65	3.26	5.33	4.61	2.69	1.42
00:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
00:45:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:15:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:45:00	3.26	3.26	0	5.67	7.72	7.72	7.72
02:00:00	3.26	3.26	0.50	2.27	7.72	7.72	7.72
02:15:00	2.87	3.26	1.96	3.46	7.72	7.72	7.72
02:30:00	3.26	3.26	1.39	2.89	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	1.087	0	0	0	0
00:45:00	1.453	0	0.500	11.667	0	0	0
01:00:00	0.781	0	0	10.527	0	0	0
01:15:00	0	0	0	8.049	0	0	0
01:30:00	0	0	0	0.548	0	0	0

01:45:00	0	0	0	0	0	0	0
02:00:00	0	0	0	0	0	0	0
02:15:00	0	0	0	0	0	0	0
02:30:00	0	0	0	0	0	0	0

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0.083	0	0	0	0
00:45:00	0.099	0	0.032	1.106	0	0	0
01:00:00	0.043	0	0	0.686	0	0	0
01:15:00	0	0	0	0.385	0	0	0
01:30:00	0	0	0	0.024	0	0	0
01:45:00	0	0	0	0	0	0	0
02:00:00	0	0	0	0	0	0	0
02:15:00	0	0	0	0	0	0	0
02:30:00	0	0	0	0	0	0	0

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.25	6.25	7.83	7.96	8.15	7.15	7.61	1.72	7.43
00:30:00	4.32	4.12	8.11	7.1	3.75	3.77	5.63	1.96	7.56
00:45:00	5.62	2.84	8.05	8.83	6.92	4.09	3.81	1.96	7.56
01:00:00	5.55	3.76	4.82	8.78	7.21	8.37	3.52	1.96	7.56
01:15:00	3.57	2.15	3	8.66	8.03	2.9	2.96	1.96	8.26
01:30:00	3.44	1.89	2.29	8.14	3.99	2.74	9.47	1.96	8.26
01:45:00	2.37	1.89	1.97	4.72	3.81	6.37	2.67	1.96	8.26
02:00:00	1.86	1.88	3.35	2.48	2.7	2.69	2.67	1.96	8.26
02:15:00	3.28	3.5	1.98	2.5	7.4	4.76	2.67	1.96	8.26
02:30:00	1.84	1.88	3.36	2.54	2.76	2.68	2.66	1.96	8.26

Minimum cost approach under SWMM constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.39	0.10	0.01	0.0	0.0	0.0	0.0

00:30:00	0.78	0.68	0.44	0.31	0.09	0.0	0.0
00:45:00	0.05	0.23	0.37	0.42	0.51	0.43	0.30
01:00:00	0.01	0.07	0.13	0.16	0.22	0.31	0.35
01:15:00	0.09	0.07	0.08	0.09	0.11	0.16	0.18
01:30:00	0.02	0.05	0.06	0.07	0.08	0.10	0.11
01:45:00	0.02	0.03	0.04	0.05	0.06	0.07	0.08
02:00:00	0.02	0.03	0.03	0.04	0.04	0.05	0.06
02:15:00	0.01	0.02	0.03	0.03	0.03	0.04	0.04
02:30:00	0.03	0.02	0.03	0.03	0.03	0.03	0.04

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.404	0.0	4.741	4.361	0.0	0.0	0.0
00:30:00	8.505	3.472	7.966	9.516	12.448	3.814	2.066
00:45:00	11.765	3.288	7.305	16.579	16.579	4.102	3.583
01:00:00	10.953	2.406	4.564	16.579	16.579	3.438	4.010
01:15:00	6.603	1.311	2.693	16.579	16.579	1.885	1.719
01:30:00	3.297	0.839	1.265	9.435	13.138	1.195	1.272
01:45:00	3.020	0.836	1.260	4.172	9.528	1.408	1.275
02:00:00	3.018	0.837	1.268	1.630	5.130	1.278	1.275
02:15:00	3.022	0.836	1.266	1.630	5.130	0.686	1.275
02:30:00	2.468	0.791	1.121	1.282	4.627	0.722	0.933

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.486	0.0	0.692	0.410	0.0	0.0	0.0
00:30:00	0.875	0.318	0.612	1.265	1.537	0.491	0.210
00:45:00	0.809	0.225	0.472	1.575	1.235	0.318	0.246
01:00:00	0.610	0.138	0.321	1.081	1.013	0.197	0.215
01:15:00	0.422	0.082	0.250	0.793	0.886	0.107	0.099
01:30:00	0.308	0.069	0.205	0.408	0.795	0.084	0.096
01:45:00	0.358	0.078	0.235	0.200	0.659	0.118	0.118
02:00:00	0.373	0.082	0.239	0.147	0.543	0.116	0.130
02:15:00	0.376	0.084	0.239	0.185	0.625	0.065	0.138
02:30:00	0.313	0.081	0.214	0.158	0.589	0.071	0.105

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.83	6.26	8.39	8.45	8.18	7.18	7.62	1.78	7.24
00:30:00	6.06	7.26	8.57	8.72	9.01	9.01	10.05	7	8.31
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.08	7	8.31
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.11	7	8.31
01:15:00	5.96	7.09	8.25	9.04	9.18	8.94	9.99	7	8.31
01:30:00	5.76	7.05	8.13	8.73	9.04	8.93	9.98	7	8.31
01:45:00	5.74	7.05	8.13	8.45	8.87	8.94	9.98	7	8.31
02:00:00	5.74	7.05	8.13	8.25	8.64	8.94	9.98	7	8.31
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7	8.31
02:30:00	5.7	7.04	8.12	8.23	8.61	8.91	9.99	7	8.31

E.2 Two consecutive storms

Minimum pollution load approach under NSGA II constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.72	1.65	3.24	5.82	4.87	3.01	1.80
00:30:00	1.73	3.24	3.19	5.15	5.42	7.67	7.56
00:45:00	3.20	3.16	3.21	3.20	3.15	4.09	7.69
01:00:00	3.24	3.25	3.24	3.30	3.29	5.62	7.69
01:15:00	1.19	1.42	3.25	3.34	3.54	5.87	7.64
01:30:00	1.40	3.20	3.07	3.03	3.52	5.76	7.71
01:45:00	3.13	3.19	3.07	3.04	2.95	4.99	7.27
02:00:00	3.12	3.15	3.15	3.15	3.16	5.46	7.65
02:15:00	1.15	1.38	3.25	3.33	3.52	5.60	7.71
02:30:00	1.90	1.84	3.00	3.20	7.52	7.36	7.59
02:45:00	2.13	2.11	3.25	3.27	6.76	7.25	7.27
03:00:00	3.04	2.99	3.26	3.24	6.66	7.48	7.44
03:15:00	2.92	3.01	3.15	3.19	6.76	7.52	7.52
03:30:00	1.50	1.68	3.20	3.26	6.87	7.40	7.51

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	2.685	0	0	0	0
00:30:00	7.186	0.000	7.920	7.506	12.059	0	3.244
00:45:00	8.593	3.290	7.173	16.579	16.579	3.062	0
01:00:00	7.804	2.411	4.390	16.579	16.579	1.103	1.009
01:15:00	5.491	1.405	2.351	16.579	16.579	0	0
01:30:00	6.508	0.022	7.958	9.397	12.513	0	0
01:45:00	8.907	3.305	7.431	16.579	16.579	2.019	0.347
02:00:00	8.177	3.177	4.435	16.579	16.579	1.532	0.245
02:15:00	5.791	1.884	1.280	16.579	16.579	0.263	0.002
02:30:00	2.025	0.945	0.103	10.266	10.248	1.046	0.916
02:45:00	0.912	0.840	0.093	6.777	8.198	0.386	1.389
03:00:00	0.006	0.831	0.900	1.614	2.508	0.125	1.466
03:15:00	0.137	0.764	0.163	1.587	1.561	0.498	1.523
03:30:00	1.501	0.797	0	1.588	1.639	0.095	0.889

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0.393	0	0	0	0
00:30:00	0.745	0	0.633	0.996	1.489	0	0.333
00:45:00	0.599	0.239	0.537	1.575	1.233	0.241	0
01:00:00	0.508	0.173	0.419	1.181	1.030	0.083	0.065
01:15:00	0.433	0.120	0.280	1.083	1.000	0	0
01:30:00	0.573	0.002	0.671	0.816	0.993	0	0
01:45:00	0.543	0.214	0.496	1.542	1.356	0.140	0.024
02:00:00	0.435	0.162	0.292	1.091	0.971	0.073	0.013
02:15:00	0.346	0.101	0.107	0.722	0.831	0.013	0
02:30:00	0.176	0.066	0.015	0.380	0.549	0.066	0.061
02:45:00	0.107	0.074	0.017	0.271	0.504	0.031	0.120
03:00:00	0.001	0.080	0.170	0.120	0.228	0.011	0.145
03:15:00	0.017	0.076	0.031	0.172	0.186	0.047	0.161
03:30:00	0.188	0.081	0	0.191	0.205	0.009	0.098

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.2	6.25	8.25	5.5	8.17	7.13	7.6	1.72	7.4

00:30:00	5.99	6.85	8.56	8.63	8.99	8.82	10.08	6.86	8.19
00:45:00	6.07	7.25	8.52	9.04	9.18	8.98	9.43	6.96	8.19
01:00:00	6.03	7.19	8.36	9.04	9.18	8.93	10.07	6.96	8.19
01:15:00	5.9	7.1	8.22	9.04	9.18	8.85	10.06	6.96	8.19
01:30:00	5.95	6.92	8.56	8.73	9.01	8.16	9.03	6.96	8.19
01:45:00	6.08	7.25	8.54	9.04	9.18	8.95	10.07	6.96	8.19
02:00:00	6.05	7.24	8.37	9.04	9.18	8.94	10.06	6.96	8.19
02:15:00	5.92	7.14	8.13	9.04	9.18	8.92	10.05	6.96	8.19
02:30:00	5.67	7.06	7.98	8.77	8.91	8.93	10.06	6.96	8.19
02:45:00	5.56	7.05	7.98	8.59	8.81	8.92	10.06	6.96	8.19
03:00:00	5.43	7.05	8.1	8.25	8.46	8.92	10.06	6.96	8.19
03:15:00	5.46	7.04	7.94	8.25	8.39	8.92	10.07	6.96	8.19
03:00:00	5.62	7.04	1.39	8.25	8.39	8.92	10.07	6.96	8.19

Minimum cost approach under NSGA II constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.39	0.10	0.01	0	0	0	0
00:30:00	0.13	0.19	0.16	0.13	0.04	0	0
00:45:00	0.02	0.08	0.13	0.14	0.15	0.09	0.04
01:00:00	0.01	0.03	0.06	0.07	0.10	0.12	0.13
01:15:00	0	0.02	0.03	0.04	0.05	0.08	0.09
01:30:00	0	0.01	0.02	0.02	0.03	0.05	0.06
01:45:00	0	0.01	0.01	0.01	0.02	0.03	0.04
02:00:00	0	0.01	0.01	0.01	0.01	0.02	0.03
02:15:00	0	0	0.01	0.01	0.01	0.01	0.02
02:30:00	0	0	0	0.01	0.01	0.01	0.01
02:45:00	0	0	0	0	0.01	0.01	0.01
03:00:00	0	0	0	0	0.01	0.01	0.01
03:15:00	0.01	0	0	0	0.00	0.01	0.01
03:30:00	0.01	0.01	0	0	0	0.01	0.01

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.511	0	4.704	4.366	0.002	0	0
00:30:00	8.868	3.476	7.945	9.515	12.444	3.814	2.028
00:45:00	11.767	3.289	7.273	16.579	16.579	4.104	2.287

01:00:00	11.007	2.407	4.537	16.579	16.579	3.438	2.190
01:15:00	6.667	1.415	3.806	16.579	16.579	2.002	1.664
01:30:00	7.992	2.454	7.961	9.433	13.137	3.175	2.468
01:45:00	12.027	3.411	7.441	16.579	16.579	4.116	3.983
02:00:00	11.202	3.183	5.081	16.579	16.579	3.727	3.544
02:15:00	6.908	1.890	2.940	16.579	16.579	2.159	1.987
02:30:00	3.904	0.948	1.324	10.266	14.750	1.066	1.511
02:45:00	3.038	0.840	1.262	6.771	11.648	0.769	1.144
03:00:00	3.034	0.840	1.257	1.629	5.885	1.305	1.140
03:15:00	3.031	0.840	1.257	1.630	5.130	0.852	1.140
03:30:00	2.552	0.796	1.148	1.319	4.680	0.514	0.834

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.500	0	0.688	0.410	0	0	0
00:30:00	0.919	0.320	0.636	1.262	1.534	0.494	0.207
00:45:00	0.822	0.239	0.545	1.578	1.237	0.323	0.172
01:00:00	0.717	0.173	0.434	1.180	1.030	0.260	0.141
01:15:00	0.525	0.121	0.452	1.083	1.001	0.195	0.124
01:30:00	0.701	0.214	0.671	0.819	1.042	0.348	0.216
01:45:00	0.734	0.221	0.496	1.540	1.353	0.287	0.271
02:00:00	0.597	0.162	0.335	1.091	0.972	0.178	0.186
02:15:00	0.413	0.101	0.247	0.722	0.831	0.105	0.105
02:30:00	0.338	0.066	0.194	0.380	0.791	0.068	0.101
02:45:00	0.354	0.074	0.234	0.271	0.717	0.061	0.099
03:00:00	0.374	0.080	0.237	0.122	0.535	0.116	0.113
03:15:00	0.377	0.084	0.237	0.176	0.610	0.080	0.121
03:30:00	0.323	0.081	0.219	0.159	0.592	0.050	0.093

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.84	6.25	8.39	8.45	8.18	7.15	7.59	1.45	7.22
00:30:00	6.08	7.26	8.56	8.72	9.01	9.01	10.12	7	8.29
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.11	7	8.29
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.04	7	8.29
01:15:00	5.97	7.1	8.33	9.04	9.18	8.95	9.98	7	8.29
01:30:00	6.03	7.19	8.56	8.73	9.04	8.99	10.01	7	8.29
01:45:00	6.23	7.26	8.54	9.04	9.18	9.03	10.12	7	8.29

02:00:00	6.19	7.24	8.4	9.04	9.18	9.01	10.1	7	8.29
02:15:00	5.98	7.15	8.27	9.04	9.18	8.95	9.99	7	8.29
02:30:00	5.8	7.06	8.14	8.78	9.11	8.93	10	7	8.29
02:45:00	5.74	7.05	8.13	8.6	8.97	8.93	9.99	7	8.29
03:00:00	5.74	7.05	8.13	8.25	8.68	8.94	9.98	7	8.29
03:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7	8.29
03:00:00	5.71	7.04	8.12	8.23	8.61	8.93	9.99	7	8.29

Minimum pollution load approach under SWMM constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.72	1.64	3.26	5.34	4.58	2.68	1.41
00:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
00:45:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:15:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:45:00	3.26	3.26	0	7.72	7.72	7.72	7.72
02:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72
02:15:00	3.26	3.26	0	7.72	7.72	7.72	7.72
02:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
02:45:00	3.26	3.26	0	7.19	7.72	7.72	7.72
03:00:00	3.26	3.26	0	1.88	7.72	7.72	7.72
03:15:00	3.10	3.26	0.99	2.61	7.72	7.72	7.72
03:30:00	3.04	3.26	1.01	2.63	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	1.087	0	0	0	0
00:45:00	1.453	0	0.500	11.667	0	0	0
01:00:00	0.781	0	0	10.527	0	0	0
01:15:00	0	0	0	8.028	0	0	0.002
01:30:00	0	0	1.124	0.548	0	0	0
01:45:00	1.674	0	0.626	9.368	0	0	0
02:00:00	0.933	0	0	10.873	0	0	0

02:15:00	0	0	0	9.474	0	0	0
02:30:00	0	0	0	1.375	0	0	0
02:45:00	0	0	0	0	0	0	0
03:00:00	0	0	0	0	0	0	0
03:15:00	0	0	0	0	0	0	0
03:30:00	0	0	0	0	0	0	0

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0.087	0	0	0	0
00:45:00	0.101	0	0.037	1.108	0	0	0
01:00:00	0.051	0	0	0.749	0	0	0
01:15:00	0	0	0	0.525	0	0	0
01:30:00	0	0	0.095	0.048	0	0	0
01:45:00	0.102	0	0.042	0.870	0	0	0
02:00:00	0.050	0	0	0.715	0	0	0
02:15:00	0	0	0	0.412	0	0	0
02:30:00	0	0	0	0.051	0	0	0
02:45:00	0	0	0	0	0	0	0
03:00:00	0	0	0	0	0	0	0
03:15:00	0	0	0	0	0	0	0
03:30:00	0	0	0	0	0	0	0

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.19	6.25	7.8	7.92	8.16	7.13	7.59	1.72	7.4
00:30:00	4.3	4.11	8.11	7.08	3.75	3.76	5.63	1.96	7.54
00:45:00	5.62	2.85	8.05	8.83	6.9	4.1	3.81	1.96	7.54
01:00:00	5.55	3.79	4.81	8.78	7.11	8.6	3.52	1.96	7.54
01:15:00	3.51	1.97	7.2	8.66	6.75	2.91	9.13	1.96	7.54
01:30:00	3.85	2.29	8.12	8.14	6.54	4.69	3.95	1.96	8.23
01:45:00	5.64	2.93	8.06	8.71	5.34	4.13	3.88	1.96	8.23
02:00:00	5.57	2.77	5.97	8.8	7.38	3.85	3.7	1.96	8.23
02:15:00	3.97	2.1	7.26	8.73	6.87	2.91	2.91	1.96	8.23
02:30:00	5.36	2	1.97	8.23	4.42	2.69	2.7	1.96	8.23
02:45:00	1.93	1.96	2.18	7.02	3.62	2.69	2.67	1.96	8.23
03:00:00	1.68	1.88	2.06	2.81	2.98	2.69	2.67	1.96	8.23

03:15:00	1.49	1.88	2.34	2.59	2.84	4.24	6.33	1.96	8.23
03:00:00	1.47	1.88	1.96	2.56	2.77	2.68	2.66	1.96	8.23

Minimum cost approach under SWMM constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.39	0.10	0.01	0	0	0	0
00:30:00	0.78	0.68	0.43	0.31	0.09	0	0
00:45:00	0.05	0.23	0.37	0.42	0.51	0.42	0.29
01:00:00	0.01	0.07	0.13	0.16	0.22	0.31	0.35
01:15:00	0.01	0.03	0.06	0.07	0.10	0.15	0.18
01:30:00	0.02	0.02	0.03	0.04	0.05	0.08	0.10
01:45:00	0.03	0.03	0.03	0.03	0.04	0.05	0.06
02:00:00	0.06	0.04	0.03	0.03	0.03	0.04	0.04
02:15:00	0.06	0.08	0.06	0.05	0.04	0.03	0.03
02:30:00	0.01	0.05	0.07	0.06	0.06	0.05	0.04
02:45:00	0.02	0.03	0.04	0.05	0.06	0.06	0.05
03:00:00	0.07	0.04	0.04	0.04	0.04	0.05	0.05
03:15:00	0.02	0.04	0.04	0.04	0.04	0.04	0.04
03:30:00	0.02	0.03	0.04	0.04	0.04	0.04	0.04

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.472	0	4.704	4.366	0.002	0	0
00:30:00	8.465	3.472	7.945	9.515	12.444	3.814	2.199
00:45:00	11.766	3.288	7.289	16.579	16.579	4.094	3.661
01:00:00	11.006	2.406	4.549	16.579	16.579	3.438	2.928
01:15:00	6.657	1.413	3.808	16.579	16.579	2.003	1.665
01:30:00	7.964	2.453	7.963	9.433	13.137	3.175	2.468
01:45:00	11.983	3.409	7.448	16.579	16.579	4.116	2.110
02:00:00	11.141	3.176	5.046	16.579	16.579	3.726	2.131
02:15:00	6.872	1.862	2.937	16.579	16.579	2.158	2.965
02:30:00	3.898	0.947	1.323	10.266	14.750	1.002	0.708
02:45:00	3.004	0.840	1.261	6.771	11.648	1.150	0.541
03:00:00	2.980	0.839	1.266	1.629	5.886	1.165	0.540
03:15:00	3.029	0.834	1.257	1.630	5.130	0.745	0.540

03:30:00	2.533	0.794	1.147	1.314	4.672	0.697	0.540

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.496	0	0.688	0.410	0	0	0
00:30:00	0.877	0.319	0.636	1.261	1.534	0.494	0.225
00:45:00	0.822	0.239	0.546	1.579	1.237	0.322	0.275
01:00:00	0.717	0.173	0.435	1.180	1.030	0.260	0.189
01:15:00	0.524	0.121	0.453	1.083	1.001	0.195	0.124
01:30:00	0.699	0.214	0.671	0.819	1.042	0.348	0.216
01:45:00	0.732	0.221	0.497	1.540	1.353	0.287	0.144
02:00:00	0.594	0.162	0.332	1.091	0.972	0.178	0.112
02:15:00	0.410	0.100	0.246	0.722	0.831	0.105	0.156
02:30:00	0.338	0.066	0.194	0.380	0.791	0.064	0.047
02:45:00	0.350	0.074	0.234	0.271	0.717	0.092	0.047
03:00:00	0.367	0.080	0.239	0.122	0.535	0.103	0.053
03:15:00	0.377	0.083	0.237	0.176	0.610	0.070	0.057
03:30:00	0.321	0.081	0.219	0.159	0.591	0.068	0.060

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.84	6.25	8.39	8.45	8.18	7.15	7.59	1.45	7.22
00:30:00	6.05	7.26	8.56	8.72	9.01	9.01	10.13	6.99	8.29
00:45:00	6.22	7.25	8.53	9.04	9.18	9.03	10.08	6.99	8.29
01:00:00	6.18	7.19	8.37	9.04	9.18	9	10.14	6.99	8.29
01:15:00	5.96	7.1	8.33	9.04	9.18	8.95	9.98	6.99	8.29
01:30:00	6.03	7.19	8.56	8.73	9.04	8.99	10.01	6.99	8.29
01:45:00	6.23	7.26	8.54	9.04	9.18	9.03	10.12	6.99	8.29
02:00:00	6.19	7.24	8.4	9.04	9.18	9.01	10.1	6.99	8.29
02:15:00	5.98	7.14	8.27	9.04	9.18	8.95	10.09	6.99	8.29
02:30:00	5.8	7.06	8.14	8.78	9.11	8.94	9.99	6.99	8.29
02:45:00	5.74	7.05	8.13	8.6	8.97	8.93	9.98	6.99	8.29
03:00:00	5.74	7.05	8.13	8.25	8.68	8.94	9.98	6.99	8.29
03:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	6.99	8.29
03:00:00	5.71	7.04	8.12	8.23	8.61	8.91	9.95	6.99	8.29

E.3 Migrating downstream storms

Minimum pollution load approach under NSGA II constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.758	1.6361	1.7495	1.3931	0.9376	0.1421	0.0058
00:30:00	0.8721	1.1497	3.2384	3.5482	7.4106	7.6769	7.6498
00:45:00	2.2737	3.1734	2.882	7.2975	6.8621	7.4976	7.3444
01:00:00	0.487	0.808	1.256	2.0019	4.6397	7.0764	7.6745
01:15:00	3.1701	3.079	3.2396	3.0357	2.5779	4.8336	7.7155
01:30:00	3.2247	3.2554	3.2599	3.27	3.2528	5.3659	7.6693
01:45:00	3.0359	3.0659	3.2201	3.2312	3.2545	5.4023	7.6721
02:00:00	2.5599	2.6149	3.2407	3.4768	5.2968	6.4555	7.6488
02:15:00	1.6968	1.803	3.2309	3.2703	6.402	7.2559	7.4571
02:30:00	2.3409	2.2902	3.2506	3.1495	6.9815	7.4039	7.336
02:45:00	2.9235	2.8815	3.1243	3.13	7.6821	7.5561	7.5776
03:00:00	1.5269	1.696	3.227	4.6144	7.2908	7.5375	7.6588

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	8.001	0	0	1.058	0	0	0
00:45:00	9.514	2.487	7.935	0	9.171	0	1.250
01:00:00	10.371	3.214	7.288	12.713	13.399	0	0.373
01:15:00	3.506	1.963	5.073	16.579	16.579	1.473	0
01:30:00	0.223	1.060	3.465	16.579	16.579	1.722	1.046
01:45:00	0.005	0.829	1.420	13.256	16.579	0.184	0.151
02:00:00	0.484	0.830	0.594	8.828	11.162	0.114	0.129
02:15:00	1.328	0.823	0	3.203	4.268	0.316	0.110
02:30:00	0.708	0.838	0	1.626	1.254	0.521	0.174
02:45:00	0.141	0.836	0.134	1.512	0.558	0.995	1.450
03:00:00	1.479	0.804	0	0.301	2.521	0.579	0.559

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7

00:15:00	0	0	0	0	0	0	0
00:30:00	0.830	0	0	0.134	0	0	0
00:45:00	0.657	0.214	0.743	0	0.944	0	0.145
01:00:00	0.582	0.212	0.511	1.742	1.615	0	0.038
01:15:00	0.225	0.117	0.346	1.454	1.153	0.140	0
01:30:00	0.021	0.074	0.276	1.027	0.994	0.113	0.068
01:45:00	0.001	0.072	0.186	0.635	0.916	0.011	0.008
02:00:00	0.060	0.079	0.109	0.373	0.692	0.008	0.008
02:15:00	0.165	0.082	0	0.189	0.343	0.025	0.009
02:30:00	0.088	0.085	0	0.164	0.144	0.046	0.017
02:45:00	0.018	0.085	0.025	0.177	0.069	0.094	0.151
03:00:00	0.185	0.082	0	0.037	0.317	0.056	0.061

Wastewater depths at CSO chambers and storage tanks

Time	Water depths (m)								
(hr:min:sec)	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.12	5.9	7.85	8.04	7.93	5.33	5.12	0	4.96
00:30:00	6.03	6.84	7.33	8.18	8.08	8.88	10.02	6.89	8.09
00:45:00	6.11	7.19	8.56	7.12	8.85	8.87	10.07	6.91	8.19
01:00:00	6.14	7.25	8.53	8.88	9.05	8.8	10.05	6.91	8.19
01:15:00	5.78	7.15	8.4	9.04	9.18	8.94	9.92	6.91	8.19
01:30:00	5.48	7.07	8.3	9.04	9.18	8.94	10.07	6.91	8.19
01:45:00	5.42	7.05	8.14	8.9	9.18	8.92	10.06	6.91	8.19
02:00:00	5.52	7.05	8.05	8.7	8.95	8.92	10.06	6.91	8.19
02:15:00	5.6	7.05	1.55	8.37	8.58	8.92	10.06	6.91	8.19
02:30:00	5.54	7.05	7.86	8.25	8.36	8.92	10.06	6.91	8.19
02:45:00	5.47	7.05	7.92	8.24	8.29	8.93	10.06	6.91	8.19
03:00:00	5.62	7.04	1.41	8.11	8.46	8.93	10.06	6.91	8.19

Minimum cost approach under NSGA II constraint handling

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)						
(hr:min:sec)	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.35	0.08	0	0	0	0	0
00:30:00	0.03	0.11	0.11	0.09	0.03	0	0
00:45:00	0.02	0.05	0.08	0.08	0.09	0.04	0.02
01:00:00	0.01	0.02	0.04	0.05	0.07	0.08	0.07
01:15:00	0.01	0.02	0.02	0.03	0.04	0.06	0.06

01:30:00	0.01	0.01	0.02	0.02	0.03	0.04	0.04
01:45:00	0	0.01	0.01	0.01	0.02	0.03	0.03
02:00:00	0	0.01	0.01	0.01	0.01	0.02	0.02
02:15:00	0	0	0.01	0.01	0.01	0.01	0.02
02:30:00	0	0	0.01	0.01	0.01	0.01	0.01
02:45:00	0	0	0	0	0.01	0.01	0.01
03:00:00	0.05	0.02	0.01	0	0.01	0.01	0.01

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.735	0	1.199	1.605	0	0	0
00:30:00	8.935	1.220	3.439	1.630	5.130	1.159	0.981
00:45:00	11.760	3.483	7.909	5.615	9.184	1.463	1.477
01:00:00	11.003	3.240	7.557	13.291	15.774	2.862	2.410
01:15:00	6.662	1.925	5.498	16.579	16.579	4.224	3.994
01:30:00	3.301	1.054	3.519	16.579	16.579	3.880	3.692
01:45:00	3.039	0.840	1.574	13.267	16.579	2.315	1.516
02:00:00	3.037	0.840	1.260	9.043	12.940	1.080	1.225
02:15:00	3.038	0.840	1.266	3.218	7.208	1.013	1.135
02:30:00	3.039	0.840	1.262	1.630	5.130	1.012	1.136
02:45:00	3.021	0.838	1.266	1.630	5.130	1.010	1.136
03:00:00	2.501	0.643	1.147	1.320	4.682	0.726	0.831

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.533	0	0.235	0.203	0	0	0
00:30:00	0.926	0.112	0.442	0.206	0.647	0.115	0.115
00:45:00	0.814	0.299	0.743	0.514	0.949	0.138	0.171
01:00:00	0.618	0.214	0.530	1.819	1.898	0.397	0.246
01:15:00	0.427	0.115	0.374	1.455	1.156	0.402	0.376
01:30:00	0.309	0.073	0.281	1.027	0.994	0.255	0.242
01:45:00	0.361	0.073	0.207	0.636	0.917	0.134	0.084
02:00:00	0.375	0.080	0.232	0.382	0.802	0.071	0.078
02:15:00	0.378	0.083	0.238	0.190	0.578	0.082	0.094
02:30:00	0.379	0.085	0.238	0.164	0.585	0.090	0.109
02:45:00	0.377	0.085	0.239	0.191	0.634	0.095	0.119
03:00:00	0.317	0.066	0.219	0.164	0.597	0.071	0.091

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.85	5.92	8.12	8.24	7.94	5.33	5.12	0	4.96
00:30:00	6.08	7.05	8.31	8.25	8.64	8.93	9.99	7.18	8.3
00:45:00	6.22	7.26	8.56	8.53	8.86	8.93	10	7.23	8.3
01:00:00	6.18	7.25	8.54	8.9	9.15	8.97	10.07	7.23	8.3
01:15:00	5.97	7.15	8.43	9.04	9.18	9.03	10.08	7.23	8.3
01:30:00	5.76	7.07	8.3	9.04	9.18	9.02	10.12	7.23	8.3
01:45:00	5.74	7.05	8.16	8.91	9.18	8.96	10.09	7.23	8.3
02:00:00	5.74	7.05	8.13	8.71	9.03	8.93	9.99	7.23	8.3
02:15:00	5.74	7.05	8.13	8.38	8.75	8.93	9.98	7.23	8.3
02:30:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.23	8.3
02:45:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.23	8.3
03:00:00	5.71	7.03	8.12	8.23	8.61	8.91	9.99	7.23	8.3

Minimum pollution load approach under SWMM constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	2.70	1.59	1.73	1.49	0.94	0.14	0.01
00:30:00	3.26	3.26	0.91	2.62	7.72	7.72	7.72
00:45:00	3.26	3.26	0	5.21	7.72	7.72	7.72
01:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:15:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:45:00	2.60	3.26	0	7.72	7.72	7.72	7.72
02:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72
02:15:00	3.26	3.26	0	3.42	7.72	7.72	7.72
02:30:00	2.88	3.26	1.60	3.10	7.72	7.72	7.72
02:45:00	3.26	3.26	1.37	2.88	7.72	7.72	7.72
03:00:00	2.86	3.26	1.95	3.54	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0	0	0	0	0

00:45:00	1.454	0	1.117	0	0	0	0
01:00:00	0.781	0	0.735	4.169	0	0	0
01:15:00	0	0	0	11.565	0	0	0
01:30:00	0	0	0	9.838	0	0	0
01:45:00	0	0	0	4.099	0	0	0
02:00:00	0	0	0	0.218	0	0	0
02:15:00	0	0	0	0	0	0	0
02:30:00	0	0	0	0	0	0	0
02:45:00	0	0	0	0	0	0	0
03:00:00	0	0	0	0	0	0	0

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0	0	0	0	0
00:45:00	0.100	0	0.104	0	0	0	0
01:00:00	0.044	0	0.051	0.573	0	0	0
01:15:00	0	0	0	1.014	0	0	0
01:30:00	0	0	0	0.609	0	0	0
01:45:00	0	0	0	0.196	0	0	0
02:00:00	0	0	0	0.009	0	0	0
02:15:00	0	0	0	0	0	0	0
02:30:00	0	0	0	0	0	0	0
02:45:00	0	0	0	0	0	0	0
03:00:00	0	0	0	0	0	0	0

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.24	5.84	7.82	7.98	7.89	5.32	5.12	0	4.96
00:30:00	5.12	2.85	7.15	2.82	2.71	4.92	10	0	5.15
00:45:00	5.62	4.92	8.12	4.68	3.39	2.8	2.7	0	5.15
01:00:00	5.55	6.04	8.08	8.44	7.74	3.41	2.93	2.87	7.1
01:15:00	3.53	2.12	6.39	8.83	7.34	4.31	3.93	2.93	7.34
01:30:00	2.65	2.14	7.03	8.75	7.6	8.51	4.37	2.93	7.82
01:45:00	5.1	2.53	5.39	8.44	6.2	3.11	3.72	2.93	7.82
02:00:00	2.86	6.1	2.33	8.1	3.95	5.93	2.83	3.99	7.82
02:15:00	1.89	1.88	1.96	3.27	3.07	2.68	3.45	4.07	7.82
02:30:00	3.29	3.49	1.98	2.54	7.89	4.77	2.67	4.07	7.95

02:45:00	1.86	1.88	3.34	2.55	2.78	2.69	2.67	4.07	7.95
03:00:00	3.29	3.51	1.97	2.5	7.48	4.79	2.67	4.07	7.95

Minimum cost approach under SWMM constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.35	0.08	0	0	0	0	0
00:30:00	0.04	0.11	0.11	0.09	0.03	0	0
00:45:00	0.02	0.05	0.08	0.08	0.09	0.04	0.02
01:00:00	0.06	0.06	0.06	0.06	0.07	0.08	0.07
01:15:00	0.07	0.07	0.06	0.06	0.06	0.07	0.07
01:30:00	0.04	0.05	0.06	0.06	0.06	0.07	0.07
01:45:00	0.01	0.03	0.04	0.05	0.06	0.06	0.06
02:00:00	0.01	0.03	0.03	0.04	0.04	0.05	0.06
02:15:00	0.14	0.07	0.04	0.04	0.04	0.04	0.04
02:30:00	0.05	0.08	0.08	0.07	0.06	0.05	0.04
02:45:00	0.01	0.04	0.06	0.07	0.07	0.06	0.06
03:00:00	0.01	0.03	0.04	0.05	0.06	0.07	0.07

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	4.732	0	1.198	1.605	0	0	0
00:30:00	8.930	1.218	3.439	1.630	5.130	1.159	0.981
00:45:00	11.732	3.479	7.909	5.615	9.184	1.457	1.477
01:00:00	10.938	3.228	7.557	13.291	15.774	2.862	2.410
01:15:00	6.610	1.924	5.497	16.579	16.579	4.223	4.012
01:30:00	3.285	1.054	3.519	16.579	16.579	3.880	3.685
01:45:00	3.038	0.833	1.573	13.267	16.579	2.315	2.442
02:00:00	3.000	0.821	1.257	9.043	12.939	1.089	1.204
02:15:00	2.917	0.829	1.254	3.218	7.207	0.808	0.523
02:30:00	3.009	0.839	1.255	1.630	5.130	1.043	0.566
02:45:00	3.035	0.837	1.257	1.630	5.130	1.012	0.564
03:00:00	2.549	0.639	1.139	1.320	4.681	0.681	0.564

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.533	0	0.235	0.203	0	0	0
00:30:00	0.926	0.112	0.442	0.206	0.647	0.115	0.115
00:45:00	0.813	0.299	0.743	0.514	0.949	0.138	0.171
01:00:00	0.614	0.213	0.530	1.819	1.898	0.397	0.246
01:15:00	0.424	0.115	0.374	1.455	1.156	0.402	0.377
01:30:00	0.307	0.073	0.281	1.027	0.994	0.255	0.242
01:45:00	0.360	0.073	0.207	0.636	0.917	0.134	0.135
02:00:00	0.371	0.078	0.231	0.382	0.802	0.072	0.076
02:15:00	0.363	0.082	0.236	0.190	0.578	0.065	0.043
02:30:00	0.375	0.085	0.237	0.164	0.585	0.093	0.055
02:45:00	0.379	0.085	0.237	0.191	0.634	0.095	0.059
03:00:00	0.323	0.066	0.217	0.164	0.597	0.066	0.062

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.85	5.92	8.12	8.24	7.94	5.33	5.12	0	4.96
00:30:00	6.08	7.05	8.31	8.25	8.64	8.93	9.99	7.18	8.3
00:45:00	6.22	7.26	8.56	8.53	8.86	8.93	10	7.23	8.3
01:00:00	6.18	7.25	8.54	8.9	9.15	8.97	10.07	7.23	8.3
01:15:00	5.96	7.15	8.43	9.04	9.18	9.03	10.08	7.23	8.3
01:30:00	5.76	7.07	8.3	9.04	9.18	9.02	10.12	7.23	8.3
01:45:00	5.74	7.05	8.16	8.91	9.18	8.96	10.07	7.23	8.3
02:00:00	5.74	7.05	8.13	8.71	9.03	8.93	9.99	7.23	8.3
02:15:00	5.74	7.05	8.13	8.38	8.75	8.93	9.99	7.23	8.3
02:30:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.23	8.3
02:45:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.23	8.3
03:00:00	5.71	7.03	8.12	8.23	8.61	8.93	9.95	7.23	8.3

E.4 Migrating upstream storms

Minimum pollution load approach under NSGA II constraint handling

Flow rates through interceptor sewer sections

Time	Interceptor sewer flow rates (m/s)

(hr:min:sec)	C1	C2	C3	C4	C5	C6	C7
00:15:00	1.86	1.06	1.41	1.39	0.82	0.13	0.01
00:30:00	1.74	1.74	3.19	5.59	5.34	6.48	7.61
00:45:00	2.77	3.23	3.08	5.22	7.70	7.38	7.67
01:00:00	3.24	3.23	3.25	3.26	3.30	4.26	7.71
01:15:00	3.25	3.26	3.25	3.25	3.25	5.66	7.58
01:30:00	2.06	2.17	3.26	3.32	3.44	5.83	7.11
01:45:00	2.21	2.21	3.24	2.73	7.06	7.71	7.69
02:00:00	1.55	1.82	3.22	3.26	6.05	7.15	7.65
02:15:00	2.19	2.15	3.14	2.99	6.96	7.57	7.53
02:30:00	2.99	3.00	3.24	3.24	7.68	7.64	7.57
02:45:00	1.49	1.64	3.18	3.26	6.38	7.54	7.68
03:00:00	2.32	2.28	3.22	3.30	7.64	7.71	7.62

Combined sewer overflow rates at CSO chambers

Time	Combined sewer overflows (m^3/s)						
(hr:min:sec)	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	1.299	0	0.038	0	7.339	0	1.665
00:45:00	0.271	0	7.714	5.607	8.551	4.227	1.110
01:00:00	2.907	3.150	7.641	16.579	16.579	3.078	0
01:15:00	8.867	3.301	5.731	16.579	16.579	0	0
01:30:00	9.294	2.667	2.910	16.579	16.579	0	0
01:45:00	6.340	1.514	0.582	9.531	9.727	0	0
02:00:00	3.034	0.639	0	5.921	7.838	0	0
02:15:00	0.858	0.838	0	1.621	1.283	0.526	0.249
02:30:00	0.061	0.775	0.114	1.593	0.634	0.267	0.145
02:45:00	1.539	0.838	0	1.603	2.194	0.512	0.289
03:00:00	0.698	0.804	0	1.519	0.614	0.087	0.141

Pollution loads at CSO chambers

Time	Pollution loads (kt/day)						
(hr:min:sec)	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0.162	0	0.006	0	0.849	0	0.179
00:45:00	0.034	0	0.677	0.748	1.077	0.384	0.081
01:00:00	0.337	0.301	0.516	1.586	1.289	0.191	0
01:15:00	0.723	0.226	0.366	1.081	1.009	0	0
01:30:00	0.583	0.147	0.215	0.786	0.880	0	0
01:45:00	0.363	0.089	0.068	0.403	0.565	0	0
02:00:00	0.230	0.050	0.0	0.255	0.509	0	0

02:15:00	0.096	0.077	0.0	0.134	0.128	0.049	0.029
02:30:00	0.008	0.076	0.021	0.177	0.077	0.026	0.017
02:45:00	0.191	0.084	0.0	0.194	0.275	0.050	0.034
03:00:00	0.087	0.082	0.0	0.189	0.077	0.009	0.016

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.27	5.49	7.91	8.01	7.94	5.59	6.77	0	4.96
00:30:00	5.6	6.13	7.93	8	8.76	8.8	10.07	4.05	8.19
00:45:00	5.48	6.88	8.55	8.53	8.83	9.03	10.07	6.92	8.19
01:00:00	5.73	7.24	8.55	9.04	9.18	8.98	9.35	6.92	8.19
01:15:00	6.08	7.25	8.44	9.04	9.18	8.92	10.04	6.92	8.19
01:30:00	6.1	7.21	8.26	9.04	9.18	6.51	1.56	6.92	8.19
01:45:00	5.95	7.11	8.03	8.73	8.88	8.87	6.71	6.92	8.19
02:00:00	5.74	7.02	1.37	8.54	8.79	8.92	9.69	6.92	8.19
02:15:00	5.56	7.05	7.67	8.25	8.36	8.92	10.06	6.92	8.19
02:30:00	5.45	7.04	7.91	8.25	8.3	8.92	10.06	6.92	8.19
02:45:00	5.62	7.05	1.39	8.25	8.44	8.92	10.06	6.92	8.19
03:00:00	5.54	7.04	7.48	8.24	8.29	8.92	10.06	6.92	8.19

Minimum cost approach under NSGA II constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.39	0.11	0.01	0	0	0	0
00:30:00	0.04	0.12	0.13	0.12	0.04	0	0
00:45:00	0.01	0.05	0.08	0.09	0.10	0.06	0.03
01:00:00	0	0.02	0.04	0.05	0.07	0.09	0.09
01:15:00	0	0.01	0.02	0.03	0.04	0.06	0.07
01:30:00	0	0.01	0.01	0.02	0.02	0.04	0.04
01:45:00	0.04	0.01	0.01	0.01	0.02	0.02	0.03
02:00:00	0.01	0.02	0.01	0.01	0.01	0.02	0.02
02:15:00	0.01	0.02	0.02	0.02	0.02	0.02	0.02
02:30:00	0	0.01	0.01	0.02	0.02	0.02	0.02
02:45:00	0.01	0.01	0.01	0.01	0.01	0.02	0.02
03:00:00	0.00	0.01	0.01	0.01	0.01	0.01	0.01

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	2.815	0	1.197	1.605	0	0	0
00:30:00	3.038	0	2.815	3.827	7.372	2.513	0.724
00:45:00	3.04	1.616	7.641	7.705	11.117	4.291	0.971
01:00:00	6.181	3.173	7.654	16.579	16.579	3.958	0.805
01:15:00	12.116	3.333	5.89	16.579	16.579	2.441	1.9
01:30:00	11.342	2.668	3.855	16.579	16.579	1.379	0.093
01:45:00	8.452	1.514	1.806	9.535	13.538	0.85	0.084
02:00:00	4.54	0.84	1.275	5.922	10.49	0.781	0.084
02:15:00	3.04	0.838	1.271	1.63	5.135	1.04	0.084
02:30:00	3.037	0.84	1.25	1.63	5.13	1.481	0.084
02:45:00	3.032	0.84	1.247	1.63	5.130	1.169	0.084
03:00:00	3.004	0.837	1.273	1.607	5.097	1.661	0.062

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.351	0	0.234	0.203	0	0	0
00:30:00	0.379	0	0.463	0.409	0.853	0.364	0.078
00:45:00	0.380	0.152	0.672	1.028	1.399	0.387	0.071
01:00:00	0.719	0.302	0.517	1.590	1.291	0.246	0.044
01:15:00	0.991	0.228	0.376	1.081	1.009	0.130	0.107
01:30:00	0.712	0.147	0.285	0.786	0.880	0.084	0.007
01:45:00	0.484	0.089	0.212	0.403	0.785	0.066	0.008
02:00:00	0.345	0.066	0.231	0.255	0.683	0.068	0.008
02:15:00	0.337	0.077	0.239	0.135	0.509	0.096	0.009
02:30:00	0.371	0.082	0.236	0.181	0.618	0.142	0.009
02:45:00	0.377	0.084	0.235	0.197	0.641	0.114	0.010
03:00:00	0.375	0.085	0.240	0.200	0.642	0.164	0.007

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.73	5.48	8.12	8.24	7.94	5.61	6.79	0	4.96
00:30:00	5.74	6.14	8.26	8.42	8.76	8.96	10.01	4.17	8.3
00:45:00	5.74	7.12	8.56	8.64	8.95	9.03	10.1	6.95	8.3

01:00:00	5.94	7.25	8.54	9.04	9.18	9.02	10.1	6.95	8.3
01:15:00	6.23	7.25	8.45	9.04	9.18	8.96	9.99	6.95	8.3
01:30:00	6.2	7.2	8.33	9.04	9.18	8.93	10	6.95	8.3
01:45:00	6.06	7.11	8.18	8.73	9.05	8.93	9.98	6.95	8.3
02:00:00	5.84	7.05	8.13	8.55	8.91	8.93	9.98	6.95	8.3
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	6.95	8.3
02:30:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	6.95	8.3
02:45:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	6.95	8.3
03:00:00	5.74	7.05	8.13	8.25	8.64	8.94	9.98	6.95	8.3

Minimum pollution load approach under SWMM constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	1.86	1.07	1.42	1.40	0.83	0.13	0.01
00:30:00	3.26	3.26	0	3.12	7.72	7.72	7.72
00:45:00	3.26	3.26	0	6.98	7.72	7.72	7.72
01:00:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:15:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:30:00	3.26	3.26	0	7.72	7.72	7.72	7.72
01:45:00	3.26	3.26	0	7.72	7.72	7.72	7.72
02:00:00	3.26	3.26	0	6.98	7.72	7.72	7.72
02:15:00	3.26	3.26	0	1.94	7.72	7.72	7.72
02:30:00	2.90	3.26	1.94	3.45	7.72	7.72	7.72
02:45:00	3.26	3.26	1.39	2.89	7.72	7.72	7.72
03:00:00	2.87	3.26	1.95	3.54	7.72	7.72	7.72

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	CSO (m^3/s)						
	Tank1	Tank2	Tank3	Tank4	Tank5	Tank6	Tank7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0	0	0	0	0
00:45:00	0	0	0.914	0	0	0	0
01:00:00	0	0	0.861	11.137	0	0	0
01:15:00	1.712	0	0	10.595	0	0	0
01:30:00	1.068	0	0	9.154	0	0	0
01:45:00	0	0	0	0.664	0	0	0
02:00:00	0	0	0	0	0	0	0

02:15:00	0	0	0	0	0	0	0
02:30:00	0	0	0	0	0	0	0
02:45:00	0	0	0	0	0	0	0
03:00:00	0	0	0	0	0	0	0

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0	0	0	0	0	0	0
00:30:00	0	0	0	0	0	0	0
00:45:00	0	0	0.080	0	0	0	0
01:00:00	0	0	0.058	1.065	0	0	0
01:15:00	0.138	0	0	0.690	0	0	0
01:30:00	0.067	0	0	0.434	0	0	0
01:45:00	0	0	0	0.028	0	0	0
02:00:00	0	0	0	0	0	0	0
02:15:00	0	0	0	0	0	0	0
02:30:00	0	0	0	0	0	0	0
02:45:00	0	0	0	0	0	0	0
03:00:00	0	0	0	0	0	0	0

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.25	5.43	7.88	7.98	7.93	5.58	6.77	0	4.96
00:30:00	2.86	1.89	6.89	5.09	7.53	6.47	10.04	0	6.59
00:45:00	1.88	3.38	8.1	7.17	4.44	8.67	5.7	0	6.67
01:00:00	3.01	2.68	8.09	8.81	6.13	3.96	3.63	0	6.67
01:15:00	5.64	3.04	7.08	8.78	7.23	3.1	2.89	0	7.12
01:30:00	5.58	2.49	7.54	8.72	7.36	2.79	3.09	0	7.5
01:45:00	4.9	1.98	2.18	8.15	6.54	8.06	2.67	0	8.26
02:00:00	2.67	1.88	2.06	5.9	3.47	2.68	2.67	0	8.26
02:15:00	2.02	1.88	3.35	2.61	2.83	2.69	2.67	0	8.26
02:30:00	3.33	3.5	1.97	2.5	7.44	4.76	2.67	0	8.26
02:45:00	1.87	1.88	3.35	2.54	2.77	2.69	2.67	0	8.26
03:00:00	3.29	3.51	1.96	2.49	7.48	4.79	2.67	0	8.26

Minimum cost approach under SWMM constraint handling

Flow rates through interceptor sewer sections

Time (hr:min:sec)	Interceptor sewer flow rates (m/s)						
	C1	C2	C3	C4	C5	C6	C7
00:15:00	0.31	0.07	0	0	0	0	0
00:30:00	0.04	0.10	0.09	0.07	0.02	0	0
00:45:00	0.05	0.06	0.08	0.08	0.08	0.03	0.01
01:00:00	0.03	0.04	0.06	0.06	0.08	0.08	0.06
01:15:00	0.01	0.03	0.04	0.04	0.05	0.07	0.07
01:30:00	0.01	0.02	0.03	0.03	0.04	0.05	0.05
01:45:00	0.02	0.02	0.02	0.02	0.03	0.03	0.04
02:00:00	0.05	0.03	0.02	0.02	0.02	0.03	0.03
02:15:00	0.02	0.04	0.03	0.03	0.03	0.03	0.03
02:30:00	0.01	0.02	0.03	0.03	0.03	0.03	0.03
02:45:00	0.01	0.02	0.02	0.02	0.03	0.03	0.03
03:00:00	0.04	0.02	0.02	0.02	0.02	0.03	0.03

Combined sewer overflow rates at CSO chambers

Time (hr:min:sec)	Combined sewer overflows (m^3/s)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	2.857	0	1.197	1.605	0	0	0
00:30:00	3.033	0	2.815	3.827	7.372	2.513	0.724
00:45:00	2.990	1.612	7.641	7.705	11.117	4.291	0.971
01:00:00	6.161	3.173	7.654	16.579	16.579	3.958	0.819
01:15:00	12.111	3.332	5.890	16.579	16.579	2.441	1.961
01:30:00	11.331	2.668	3.855	16.579	16.579	1.379	1.710
01:45:00	8.477	1.514	1.806	9.535	13.538	0.244	1.614
02:00:00	4.492	0.833	1.275	5.922	10.490	0.901	1.616
02:15:00	3.031	0.837	1.274	1.630	5.133	0.847	1.616
02:30:00	3.032	0.840	1.255	1.630	5.130	1.200	1.616
02:45:00	3.032	0.839	1.279	1.630	5.130	0.842	1.616
03:00:00	2.953	0.824	1.257	1.607	5.097	1.147	1.616

Pollution loads at CSO chambers

Time (hr:min:sec)	Pollution loads (kt/day)						
	T1	T2	T3	T4	T5	T6	T7
00:15:00	0.356	0	0.234	0.203	0	0	0
00:30:00	0.378	0	0.463	0.409	0.853	0.364	0.078
00:45:00	0.374	0.152	0.672	1.028	1.399	0.387	0.071

01:00:00	0.716	0.302	0.517	1.590	1.291	0.246	0.045
01:15:00	0.991	0.228	0.376	1.081	1.009	0.130	0.111
01:30:00	0.712	0.147	0.285	0.786	0.880	0.084	0.123
01:45:00	0.486	0.089	0.212	0.403	0.785	0.019	0.145
02:00:00	0.341	0.065	0.231	0.255	0.683	0.079	0.163
02:15:00	0.336	0.076	0.240	0.135	0.509	0.079	0.173
02:30:00	0.370	0.082	0.237	0.181	0.618	0.115	0.179
02:45:00	0.377	0.084	0.241	0.197	0.641	0.082	0.183
03:00:00	0.369	0.084	0.237	0.200	0.642	0.113	0.186

Wastewater depths at CSO chambers and storage tanks

Time (hr:min:sec)	Water depths (m)								
	T1	T2	T3	T4	T5	T6	T7	T8	T9
00:15:00	5.73	5.5	8.12	8.24	7.94	5.61	6.79	0	4.96
00:30:00	5.74	6.14	8.26	8.42	8.76	8.96	10.01	4.25	8.3
00:45:00	5.74	7.12	8.56	8.64	8.95	9.03	10.1	7.11	8.3
01:00:00	5.94	7.25	8.54	9.04	9.18	9.02	10.11	7.11	8.3
01:15:00	6.23	7.25	8.45	9.04	9.18	8.96	9.99	7.11	8.3
01:30:00	6.2	7.2	8.33	9.04	9.18	8.93	9.99	7.11	8.3
01:45:00	6.06	7.11	8.18	8.73	9.05	8.94	9.98	7.11	8.3
02:00:00	5.84	7.05	8.13	8.55	8.91	8.93	9.98	7.11	8.3
02:15:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.11	8.3
02:30:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.11	8.3
02:45:00	5.74	7.05	8.13	8.25	8.64	8.93	9.98	7.11	8.3
03:00:00	5.74	7.05	8.13	8.25	8.64	8.92	9.98	7.11	8.3

APPENDIX F – PUBLICATIONS

F.1 Optimal control of combined sewer systems using SWMM 5.0

(In the *Transactions on The Built Environment,* 122, pp 87 – 96)

Presented at Urban Water 2012 – 1st International Conference on the Design, Construction, Maintenance, Monitoring and Control of Urban Water Systems, New Forest, United Kingdom, 25th – 27th April 2012

Optimal control of combined sewer systems using SWMM 5.0

U.S. Rathnayake & T.T. Tanyimboh
Department of Civil Engineering, University of Strathclyde, Glasgow, United Kingdom.

Abstract

Combined sewer networks carry wastewater and stormwater together. Capacity limitation of these sewer networks results in combined sewer overflows (CSOs) during high-intensity storms. Untreated CSOs when directly discharged to the nearby natural water bodies cause many environmental problems. Controlling urban wastewater systems is one possible way of addressing the environmental issues from CSOs. However, controlling urban sewer systems optimally is still a challenge, when considering the receiving water quality effects. In this study, a multi-objective optimization approach was formulated considering the pollution load to the receiving water from CSOs and the cost of the wastewater treatment. The optimization model was tested using an interceptor sewer system. The results demonstrate the benefits of the multi-objective optimization approach and its potential to establish the key properties of a range of control strategies through an analysis of the various trade-offs involved.

Keywords: combined sewer overflows, effluent quality index, evolutionary computing, genetic algorithm, multi-objective optimization, combined sewer systems.

1 Introduction

Combined sewer networks carry dry weather flow (DWF) and stormwater together. Capacity limitation of these sewer networks results in combined sewer overflows (CSOs) during high-intensity storms. Untreated CSOs when directly discharged to the nearby natural water bodies cause many environmental problems. Controlling existing urban sewer networks is one possible way of addressing the issues in urban wastewater systems. However, it is still a challenge, when considering the receiving water quality effects.

Most of the literature on controlling combined sewer systems is based on volumetric measures [1, 2, 3]. These include the optimal storage controls to utilize the temporary storage in sewer networks to provide more detention time. However, they failed to address the issue of water quality in both combined sewers and receiving waters. In addition, economic measures, such as treatment cost at downstream wastewater treatment plant, in general are not considered. Furthermore, most of the previous work was based on simplified hydraulic models [4]. Complexity of the problem is the main issue in developing a holistic approach.

However, Rathnayake and Tanyimboh [5] have proposed a multi-objective optimization approach in controlling urban wastewater systems based on receiving water quality due to CSOs and the wastewater treatment cost. More importantly that approach considers the results from full hydraulic simulations.

This paper presents an improvement to the multi-objective optimization approach discussed in Rathnayake and Tanyimboh [5]. Deb's binary tournament selection technique [6] was used to handle the constraints in Rathnayake and Tanyimboh [5], whereas a different constraint handling technique was used to handle the constraints in this paper. A detailed explanation about this constraint handling technique can be found in section 5.

A multi-objective optimization approach was developed, considering the pollution load to the receiving water from CSOs and the wastewater treatment cost. Simulation results from a full hydraulic model, including water quality routing were used in the optimization. The performance of the multi-objective optimization model was tested on a simple interceptor sewer system. Results from full hydraulic simulations were presented for different optimal controlling settings.

2 Pollution load evaluation

Effluent quality index (*EQI*) is formulated to calculate the pollution load in a water body as a single variable. Five important water quality parameters, total suspended solids (*TSS*), chemical oxygen demand (*COD*), five-day biochemical oxygen demand (*BOD*), total Kjeldahl nitrogen (*TKN*) and nitrates/nitrites (*NOX*) are accumulated together in forming this single measure. A detailed explanation of this *EQI* can be found in Mussati *et al.* [7] and Rathnayake and Tanyimboh [8].

3 Wastewater treatment cost

It is the common practice to have a treatment plant with an overall capacity of 6×DWF. However, the full treatment capacity is further limited to 3×DWF and the surplus flow is temporarily stored in equalization tanks which have the same role as primary sedimentation tanks. In a case where the total flow is more than 6×DWF, the storm tanks fill completely and overflow to the nearby natural water. Based on various cost models from literature, a generic cost function based on the treated water volume was adopted. The treatment cost, C_T (€/year) is described as

$$C_T = \begin{cases} A \times {q_T}^{0.659}, & q_T \leq 3 \times DWF \qquad (1a) \\ B + \frac{2}{3} C, & 3 \times DWF \leq q_T \leq 6 \times DWF \qquad (1b) \\ B + \frac{2}{3} D, & q_T > 6 \times DWF \qquad (1c) \end{cases}$$

where,

$$A = 916.862 \times 86400^{0.659} \qquad (2)$$

$$B = 916.862 \times \left(3 \times DWF\right)^{0.659} \qquad (3)$$

$$C = 1.69 \times \left(q_T - 3 \times DWF\right) + 11376 \qquad (4)$$

$$D = \left(1.69 \times 3 \times DWF\right) + 11376 \qquad (5)$$

where q_T (m^3/s) is the treated wastewater volume flow rate.

Total treatment cost, including personnel, energy, maintenance, waste and other costs, when the wastewater flow rate is less than or equal to 3×DWF is given by Hernandez-Sancho *et al.* [9]. However, the additional cost, including storage cost, should be included, when the flow rate is more than 3×DWF. Eqns (1b & 1c) give the wastewater treatment cost at the treatment plant and the operational and maintenance cost for storage tanks when the flow rate is in between 3×DWF and 6×DWF and when more than 6×DWF respectively. More details on the development of Eqn (1) can be found in Rathnayake and Tanyimboh [5].

4 Problem formulation and solution

Typical configurations for an interceptor sewer and a CSO chamber are shown in the Figure 1. The first objective function was formulated to minimize the pollution load to receiving water through the CSOs. *EQI*, which gives the pollution load, was used to formulate this objective function.

$$Minimize \quad F_1 = \sum_{i=1}^{n} P_i \qquad (6)$$

where n and P_i are the number of interceptor nodes or CSO chamber points and the pollution load to the receiving water from the i^{th} CSO chamber respectively.

The second objective function was formulated to minimize the wastewater treatment cost at downstream treatment plant.

$$Minimize \;\; F_2 = C_T \tag{7}$$

where C_T is the treatment cost at the wastewater treatment plant. Referring to Figure 1, the continuity equations are described as

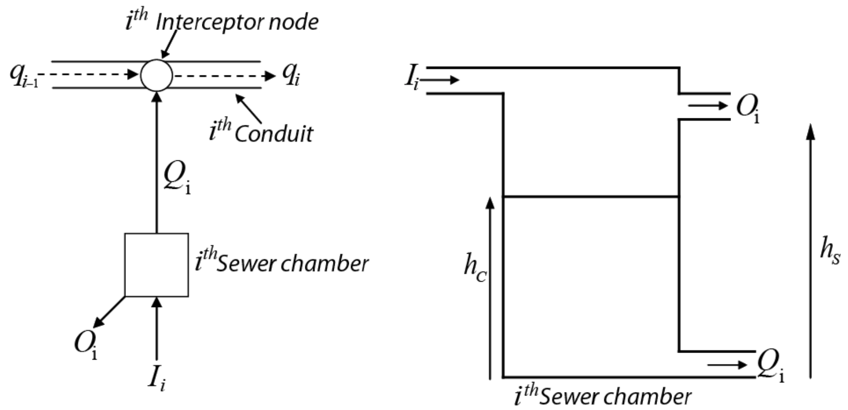
$$Q_i + q_{i-1} - q_i = 0 \tag{8}$$

$$A_C \frac{\Delta h_C}{\Delta t} = I_i - Q_i \qquad ; h_C < h_S \tag{9}$$

$$A_C \frac{\Delta h_C}{\Delta t} = I_i - Q_i - O_i ; \qquad h_C > h_s \tag{10}$$

$$0 \le q_i \le q_{\max,i} \tag{11}$$

where A_C is the surface area of the CSO chamber and $q_{max,i}$ is the maximum flow rate at i^{th} conduit.



I_i – Catchment inflow to node i
Q_i – Flow from i^{th} sewer chamber to interceptor node i
q_i – Through flow in interceptor sewer at node i
O_i – Combined sewer over flow discharge at node i
h_C – Water level in sewer chamber
h_S – Spill level of sewer chamber

Figure 1: Schematic diagram of sewer chamber.

5 Solution to the multi-objective optimization approach

The hydraulic model, U.S. EPA SWMM 5.0 [10] and the multi-objective optimization module, NSGA II [6] were coupled using "C" programming language. NSGA II has already been successfully applied to many practical optimization problems in various disciplines. SWMM 5.0 is a dynamic rainfall-runoff simulation model and is primarily used to simulate urban and sub-urban areas.

It is assumed here that wastewater flow from CSO chamber to the interceptor sewer is controlled using an orifice at the bottom of the CSO chamber. Orifice openings were initially generated randomly. Hence, the decision variables (q_i) of the optimization approach were indirectly generated. Next, a full hydraulic simulation, including water quality routing was carried out using SWMM 5.0 the results of which were used to calculate the pollution load F_1 and the wastewater treatment cost F_2.

Mass balance and conservation of energy were automatically satisfied by the hydraulic model. Maximum flow rates allowed through conduits were formulated inside the hydraulic model. SWMM 5.0 conduit features in defining the maximum flow rates were used in formulating the maximum flow rates allowed through conduits as shown in the eqn (11). By contrast, Deb's binary tournament selection technique was used to handle constraints in Rathnayake and Tanyimboh [5]. The maximum flow rates allowed through conduits were externally satisfied by this constraint handling approach. A detailed explanation about Deb's constraint handling technique can be found in Deb *et al.* [6] and Rathnayake and Tanyimboh [5]

6 Case study

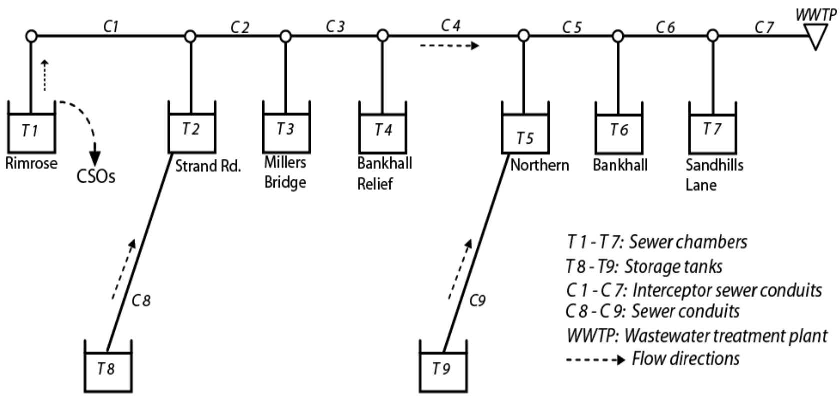


Figure 2: Interceptor sewer system.

Developed multi-objective optimization model was applied to a simplified interceptor system. A description of this interceptor sewer system can be found in Thomas [11]. The interceptor sewer system was modified, for this study. The CSO chambers *T1* to *T7* are described in Thomas [11]. Two storage tanks (*T8* and *T9*) were introduced at upper catchments of Strand Rd. and Northern. Figure 2 shows the modified interceptor sewer system. Maximum flow rates allowed through *C1*, *C2* and *C3* are 3.26 m^3/s and that of *C4*, *C5*, *C6* and *C7* are 7.72 m^3/s. The diameter for *C1* to *C3* is 1.66 m and that of *C4* to *C7* is 2.44 m. Depths of the CSO chambers (*T1* to *T7*) and storage tanks (*T8* and *T9*) are 6.42, 7.91, 8.95, 9.04, 9.18, 9.47, 10.26, 8.00 and 9.00 m respectively. Storage tanks *T8* and *T9* are generic and the details of the flow control in these tanks are not discussed in this paper.

Diurnal effects of the DWF were not considered in this study. Therefore, average flow rates of DWFs were fed to the *T1, T3, T4, T6, T7* CSO chambers and *T8, T9* storage tanks. More details on the storm run-off flow hydrographs can be found in Thomas [11]. Five different land-uses, including residential, industrial, commercial, agricultural and mid urban were assumed when generating the pollutographs for five different water quality constituents [12]. Shapes of the pollutographs of five different water constituents (*TSS*, *COD*, *BOD*, *NOX*, *and TKN*) were reviewed from the literature [13, 14, 15, 16]. More details of these pollutographs can be found in Rathnayake and Tanyimboh [8].

A basic real-coded NSGA II program was used in this study. The optimization process was done with a population of 100, 100 generations and a crossover probability of 1. The simulated binary crossover uses a probability distribution around two parents to create two children solutions [17]. The distribution indices for crossover and mutation operators were kept at 20 [6]. The distribution indices for crossover and mutation are parameters that define the shape of the probability distribution for the simulated binary crossover and polynomial mutation respectively [17].

Many optimization runs with different random seeds were conducted. Different mutation probabilities were tried in different runs. The reason for selecting different mutation probabilities was to compare the performance of the mutation probabilities for this optimization problem. Polynomial mutation, described in Deb *et al.* [6], was used for this optimization approach. The polynomial mutation operator creates a new value for the decision variable, which is near the vicinity of the original value using a probability distribution.

Routing time-step in SWMM 5.0 was kept at 30 seconds, and the results were obtained at 15 minutes. Then, the NSGA II optimization module was run using the obtained results. Each GA run took about 9 minutes on a Pentium 4 desktop personal computer with a Core 2 Duo processor and 4 GB of RAM.

7 Results and discussion

The best Pareto optimal front was achieved with a mutation probability of 0.4 over the entire population of solutions, i.e. when 40% of randomly selected decision variables from the whole population were muted. Pareto optimal front for 0.4 mutation rate is shown in Figure 3.

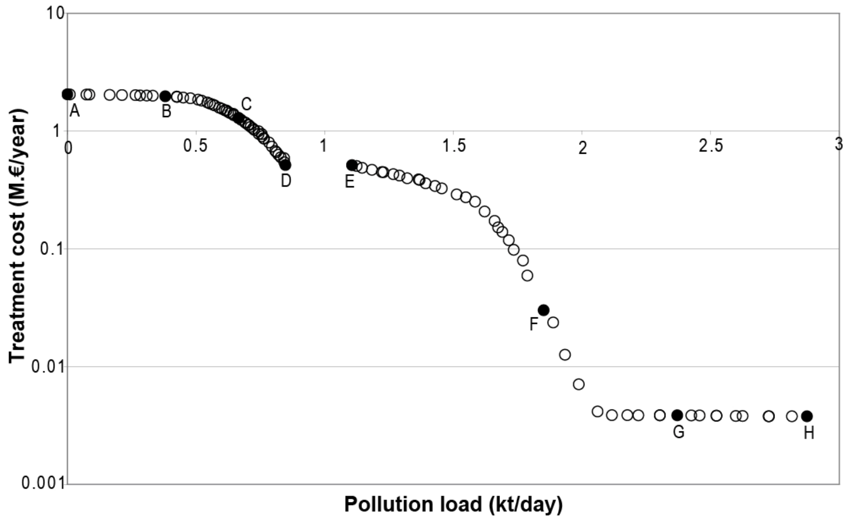


Figure 3: Best Pareto optimal front achieved

Solutions A to H (Figure 3) were selected for further assessment. Results from full hydraulic simulations for these solutions are presented in the following tables.

Table 1: Flow rates through the interceptor sewer sections at t = 15 minutes

Solution	Interceptor sewer flow rates (m^3/s)						
	C1	*C2*	*C3*	*C4*	*C5*	*C6*	*C7*
A	2.70	1.61	3.26	5.37	4.57	2.67	1.40
B	2.70	1.61	3.26	5.01	4.45	2.56	1.32
C	2.69	1.60	3.26	4.16	3.74	1.86	0.69
D	2.70	1.60	3.26	3.19	2.73	0.91	0.17
E	2.70	1.60	3.26	3.14	2.71	0.89	0.17
F	2.70	1.55	1.78	1.62	0.87	0.10	0
G	2.50	1.46	0.43	0.12	0	0	0
H	0.03	0	0	0	0	0	0

Table 2: Combined sewer overflow rates at CSO chambers at t = 15 minutes

Solution	Combined sewer overflows (m^3/s)								
	T1	*T2*	*T3*	*T4*	*T5*	*T6*	*T7*	*T8*	*T9*
A	0	0	0	0	0	0	0	0	0
B	0	0	0.01	1.84	0	0	0	0	0
C	0	0	0.01	3.34	0	0	0	0	0
D	0	0	0	4.36	0	0	0	0	0
E	0	0	0.97	4.35	0	0	0	0	0
F	0	0	3.79	4.36	0	0	0	0	0
G	1.62	0	4.74	4.36	0	0	0	0	0
H	4.86	0	4.74	4.36	0	0	0	0	0

Table 3: Pollution loads at CSO chambers and storage tanks at t = 15 minutes

Solution	Pollution loads (kt/day)								
	T1	*T2*	*T3*	*T4*	*T5*	*T6*	*T7*	*T8*	*T9*
A	0	0	0	0	0	0	0	0	0
B	0	0	0	0.38	0	0	0	0	0
C	0	0	0	0.66	0	0	0	0	0
D	0	0	0	0.85	0	0	0	0	0
E	0	0	0.26	0.85	0	0	0	0	0
F	0	0	1.00	0.85	0	0	0	0	0
G	0.27	0	1.25	0.85	0	0	0	0	0
H	0.77	0	1.25	0.85	0	0	0	0	0

Table 4: Wastewater depths at CSO chambers and storage tanks at t = 15 minutes

Solution	Wastewater depths (m)								
	T1	*T2*	*T3*	*T4*	*T5*	*T6*	*T7*	*T8*	*T9*
A	5.37	0	7.87	8.01	0.02	7.18	7.63	6.47	7.84
B	5.37	0	7.95	8.28	0.02	7.18	7.63	6.47	7.84
C	5.40	0	7.93	8.38	0.02	7.18	7.63	6.47	7.84
D	5.40	0	7.91	8.45	0.02	7.18	7.63	6.47	7.84
E	5.40	0	8.10	8.45	0.02	7.18	7.63	6.47	7.84
F	5.41	0	8.32	8.45	0.02	7.18	7.63	6.47	7.88
G	5.63	0	8.39	8.45	0.02	7.18	7.62	6.53	7.80
H	5.86	0	8.39	8.45	0.02	7.18	7.62	6.53	7.80

As stated above the flow rates through the conduits of the interceptor sewer were constrained. It can be clearly seen in Table 1 that the flow rates through these conduits are less than or equal to the maximum allowed flow rate for all the tabulated cases. Furthermore, Table 2 shows the CSO rates for Solutions A to H. Solution A that corresponds to the minimum pollution load to receiving water has smaller CSO rates than Solution H that corresponds to the minimum wastewater treatment cost. Table 3 shows the pollution load to the receiving water from CSO chambers and the storage tanks for Solutions A to H. Solution A has the minimum pollution load to the receiving water, whereas the Solution H has the maximum pollution load. Table 4 shows the wastewater depths at CSO chambers and storage tanks for Solutions A to H. It can be seen in Table 4 that the storage tanks (*T8* and *T9*) store wastewater in order to prevent CSOs at downstream *T2* and *T5* CSO chambers.

In comparison with the results presented in Rathnayake and Tanyimboh [5], the results obtained here show a better optimal control of urban wastewater systems. Solution A that corresponds to the minimum pollution load to receiving water gives 0 kt/day pollution load whereas it is 5.57 kt/day in Rathnayake and Tanyimboh [5]. Similarly Solution H corresponds to the minimum wastewater treatment cost gives 3776 €/year cost whereas it is 3859 €/year in Rathnayake and Tanyimboh [5].

8 Conclusions

Hydraulic simulation results show that the developed multi-objective optimization approach produces feasible solutions. The presented multi-objective optimization approach shows a considerable improvement in controlling urban wastewater systems compared to the previous work by the same authors. However, the proposed model gives the optimal CSO control settings where a single set of static control settings is used throughout the 15 minutes storm duration. Further research is requited in developing an extended period dynamic optimization procedure for the full duration of the storm. The discussed constraint handling approach and the obtained results will be used to develop the dynamic optimization approach.

Acknowledgement

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F.2 Integrated Optimal Control of Urban Sewer Systems

Presented at *IWA WCE 2012 – World Congress on Water, Climate and Energy,* Dublin, Ireland.

World Congress on Water, Climate and Energy

Integrated optimal control of urban wastewater systems

Upaka S. Rathnayake[1] and Tiku T. Tanyimboh[1]
Upaka S. Rathnayake[2]
[1&2] Department of Civil Engineering, University of Strathclyde, Glasgow, United Kingdom

Abstract

Sewer networks are designed to collect and transport wastewater to treatment plants. However, during wet weather periods stormwater runoff flows into these sewers and combined sewer overflows (CSOs) occur. Damage to the nearby natural waters from these CSOs is noticeable. This is because of the high pollution concentrations in CSOs. Controlling urban wastewater systems is one possible way of addressing the environmental issues from CSOs. Therefore, this research explores the development of a holistic framework that is intended to be used for the multi-objective optimization of urban wastewater systems, considering water quality in both sewers and receiving waters and the economics of wastewater treatment. Dry weather flows (DWFs) and stormwater runoff water quality compositions were considered. Temporal and spatial variations of the stormwater runoff were incorporated using pollutographs for different land-uses.

Keywords: Combined sewer overflows, Effluent quality index, Land-use, Multi-objective optimization, Pollutographs, Urban wastewater systems

Introduction

Sewer networks are designed to gather and transport wastewater to treatment plants. However, during wet weather periods stormwater runoff flows into these sewers and combined sewer overflows (CSOs) occur. This is due to the limited capacity of sewers. CSOs are a concerned environmental burden for most of the urban cities. Untreated CSOs when directly discharged to the nearby natural water bodies cause many environmental problems. This is because of the increased pollution levels at natural water bodies. Though combined sewers are no longer constructed because of the growing environmental concerns, the existing ones still operate in many cities all around the world. At the same time, these sewers have to bear more dry weather flows (DWFs) because of the ongoing urbanization in most of the cities. In addition, more stormwater volumes, compared to earlier, flow into the existing combined sewers in some cities. This is because of the increasing rainfall, caused due to the global warming.

Most of the previous literature on controlling combined sewer systems is based on volumetric measures (Beraud et al., 2010, Darsono et al., 2007 and Cembrano et al., 2004). These basically include optimal storage controls to utilize the temporary storage in sewer networks to provide more retention time. Therefore, these previous studies aimed at minimizing CSOs. However, they have failed to address the issue of water quality in both combined sewers and receiving waters. In addition, economic measures, such as treatment cost at the downstream wastewater treatment plant, were not considered. Furthermore, most of the previous studies were based on simplified hydraulic models and some followed single objective approaches. Complexity of the problem is the main issue in developing a holistic approach.

This research aims at addressing the identified gaps as stated above. A multi-objective optimization approach is being developed, considering flows and water quality in combined sewer flows and economic aspects of the wastewater treatment. Dry weather flow and stormwater runoff water quality compositions were considered. Temporal and spatial variations of the stormwater runoff were incorporated using pollutographs for different land-uses.

Pollution load evaluation
Effluent quality index (*EQI*) is formulated to evaluate the pollution load in a water body as a single variable. Five important water quality parameters, total suspended solids (*TSS*), chemical oxygen demand (*COD*), five-day biochemical oxygen demand (*BOD*), total Kjeldahl nitrogen (*TKN*) and nitrates/nitrites (*NOX*) are accumulated together in forming this single measure.

EQI was originally used as a performance index. However, it is found as a sensitivity index in previous literature. Furthermore, many researchers have identified it, as a better index to express the quality of the wastewater and the pollution load to receiving water bodies. Therefore, this index can be used in representing the damage to the receiving waters from the CSOs.

Effluent quality index is described as:

$$EQI = \frac{1}{1000\left(t_f - t_0\right)} \int_{t_0}^{t_f} \left(2C_{TSS} + 1C_{COD} + 2C_{BOD} + 20C_{NOX} + 20C_{TKN}\right) Q_e\left(t\right) dt \qquad (1)$$

where $Q_e(t)$, t_f, and t_0 are the flow rate, final and initial time respectively. C_{TSS}, C_{COD}, C_{NOX}, C_{BOD} and C_{TKN} are the concentrations of total suspended solids, chemical oxygen demand, nitrates and nitrites, five-day biochemical oxygen demand and total Kjeldahl nitrogen, respectively. Concentrations of these five water quality parameters are weighted sum over one complete year. The numerical values in front of these concentrations represent the weighting factors. These weighting factors are applied to denote the contribution of each water quality parameter (Mussati et al., 2002). These factors are based on the Flandes' effluent quality formula for calculating fines (Vanrolleghem et al., 1996).

Wastewater treatment cost
The funding available for maintenance and operation of wastewater treatment plants is limited. Therefore, authorities always want to minimize the maintenance and treatment cost at treatment plants. Maintenance and treatment costs are usually expressed as a percentage of design and construction cost of a particular wastewater treatment plant. However, there are few empirical formulae to express these costs, based on the treated wastewater volume.

It is a usual practice to have a treatment plant with an overall capacity of 6*DWF. However, the full treatment capacity is further limited to 3*DWF and the rest of the flow is temporarily stored in equalization tanks. Therefore, the proposed cost formulae should be able to address both wastewater treatment cost and the storage cost. Referring to various cost models from the previous literature, a cost function, based on the treated water volume, was proposed.

The treatment cost, C_t (Euro/year) is described as:

$$C_t = \begin{cases} A * V_t^{0.659}, & V_t \le 3 * DWF \quad (2a) \\ B + (2/3)C, & 6 * DWF \ge V_t > 3 * DWF \quad (2b) \end{cases}$$

where,

$$A = 916.862 * (86400)^{0.659} \quad (3)$$

$$B = 916.862 * (3 * DWF)^{0.659} \quad (4)$$

$$C = 1.69 * (V_t - 3 * DWF) + 11376 \quad (5)$$

where V_t (m^3/s) is the treated wastewater volume at time *t*.

Total treatment cost, when the wastewater flow rate is less than or equal to 3*DWF is given by Hernandez-Sancho et al. (2008). This includes the costs for personnel, energy, maintenance, waste and other costs. However, the additional cost, including storage cost, should be included, when the flow rate is more than 3*DWF. Excess wastewater above full treatment capacity is usually transferred to equalization tanks. An equalization tank plays the same role as a primary sedimentation tank. Therefore, the operational and maintenance cost of an equalization tank is assumed to be the same as a primary sedimentation tank. Equation (5) gives the annual operational and maintenance cost for a primary sedimentation tank based on the volume flow rate (United Nations, 2003). In addition, this includes the operation and maintenance costs of sludge pumps. Numerical value 2/3 in equation (2b) is used as a typical conversion rate for Euro to US$.

Water quality in combined sewer flows

Concentrations of water quality constituents of sewer flow are necessary in calculating the pollution load from CSOs. Compositions of the DWF and stormwater runoff should be considered in evaluating the pollution load. Three pollutant concentration levels of DWF can be found from Metcalf and Eddy (1991). Concentration levels of five water quality constituents, which are used to calculate the *EQI* are tabulated in Table 1. These concentration values show the typical composition of untreated domestic wastewater.

Table 1. Pollutant composition of DWF

Water quality constituent	Concentration level		
	Weak	Medium	Strong
TSS (mg/L)	100	220	350
COD (mg/L)	250	500	1000
BOD (mg/L)	110	220	400
TKN(mg/L)	12	25	50
NOX (mg/L)	20	40	85

It is reasonable to assume the composition of the DWF is the same for different catchments. However, the composition of the stormwater runoff is different from one land-use to another. Furthermore, the land-use patterns are different from a catchment to another. Duncan (1999) gives a detailed overview about the composition of the stormwater runoff for different land-uses. Table 2 presents the composition of stormwater runoff for five different land-uses. In addition to the different pollution compositions for different land-uses, the temporal variations of the water quality constituents in stormwater runoff are significant. Pollutographs represent these concentration variations with time. However, the shapes of the pollutographs of different water quality constituents are different to each other. These shapes were reviewed from the previous literature (Li et al., 2007, Qin et al., 2010, Morris et al., 1998 and Yusop et al., 2005).

Table 2. Pollutant composition of stormwater runoff

Land-use	TSS (mg/L)	COD (mg/L)	BOD (mg/L)	TKN (mg/L)	NOX (mg/L)
Residential	50 - 400	35 - 175	8.0 - 25	1.2 - 5.5	1.2 - 5.5
Industrial	45 - 500	70 - 410	7.0 - 25	1.2 - 4.2	1.2 - 4.2
Commercial	50 - 350	30 - 220	9.5 - 22	1.1 - 3.5	1.1 - 3.5
Agricultural	65 - 550	12 - 85	1.0 - 10	1.5 - 9.5	1.5 - 9.5
Mid urban	35 - 850	25 - 75	4.0 - 12	1.5 - 7.5	1.5 - 7.5

Case study

The interceptor sewer system in Thomas (2000) was modified as presented in the following paragraphs. Longitudinal section of this interceptor sewer is shown in Fig. 1.

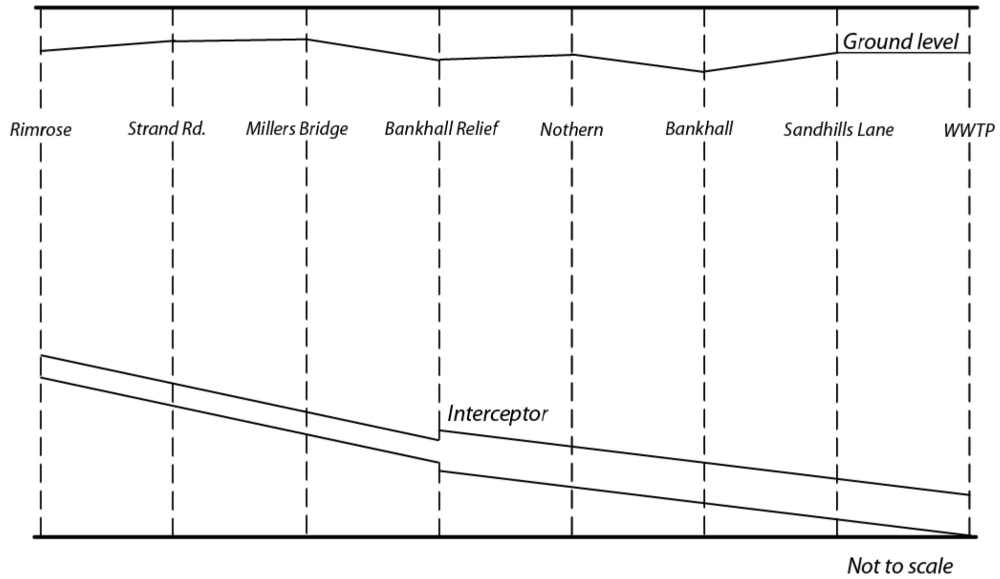

Fig. 1. Longitudinal section of the interceptor sewer.

It is a common practice to have stormwater storage tanks in upper catchments. Therefore, in addition to the CSO chambers described in Thomas (2000), two additional storage tanks (T8 and T9) were introduced to upper catchments of Strand Rd. and Nothern. These two storage tanks were placed 10 km away from the corresponding CSO chambers. Fig. 2 gives a detailed graphical view of the modified interceptor sewer system.

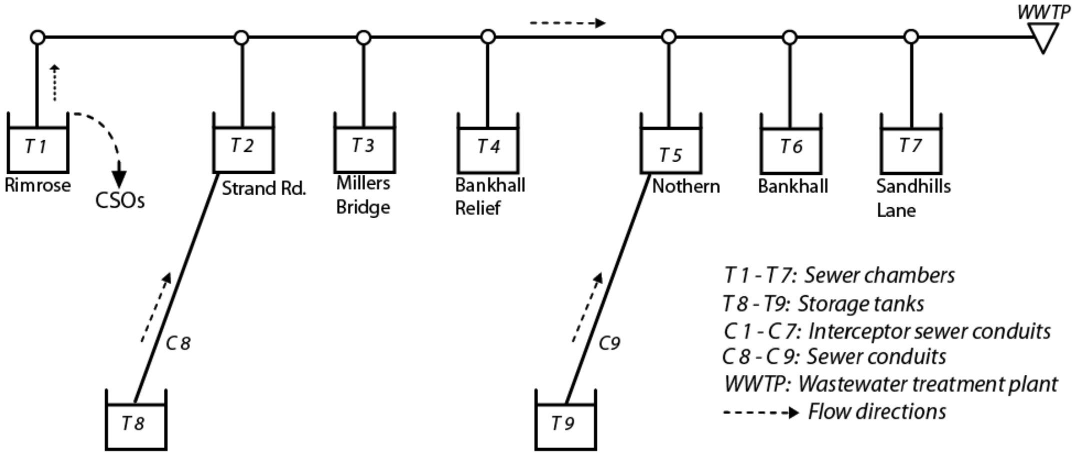

Fig. 2. Modified interceptor sewer system.

Geometrical information of the interceptor sewer system is presented in Tables 3 and 4. Inflows to sewer system from DWFs are presented in Table 3. In addition to the DWF, constant fixed inflows (Thomas, 2000) were fed into the sewer system. Even though, DWFs have diurnal effects, they were not considered in this study. Average DWF and fixed inflow rates were fed into the T1, T3, T4, T6, T7 CSO chambers and T8, T9 storage tanks. DWFs and fixed inflows of Strand Rd. and Nothern catchments were assumed to flow to the T8 and T9 storage tanks respectively, during the storm conditions. In addition to the DWF and fixed inflows, inflows from a single storm were fed into these inflow locations. Details of the flow hydrographs from stormwater runoff can be found in Thomas (2000).

Table 3. Geometrical information for interceptor and inflows

Interceptor point	Invert elevation (m)	Sewer diameter (m)	Length of sewers (m)	Fixed inflow (m^3/s)	DWF (m^3/s)
Rimrose (T1)	4.075	1.66	895	1.24	0.3
Strand Rd. (T2)	2.882	1.66	740	0	0
Millers Bridge (T3)	1.895	1.66	465	0.97	0.04
Bankhall Relief (T4)	1.275	2.44	19	0.69	0.14
Nothern (T5)	1.256	2.44	710	0	0
Bankhall (T6)	0.546	2.44	350	0.29	0.11
Sandhills Lane (T7)	0.196	2.44	196	0.31	0.09
T8	4.0	1.66	10000	0.25	0.09
T9	2.0	2.44	10000	2.13	0.50

Table 4. Geometrical information for CSO chambers

Interceptor point	Chamber area (m^2)	Chamber height (m)	Orifice height (m)	Orifice width (m)
T1	282.82	6.42	1.45	1.25
T2	136.03	7.91	0.625	1.70
T3	50.31	8.95	0.625	1.50
T4	169.78	9.04	0.625	2.08
T5	328.24	9.18	1.45	2.65
T6	167.06	9.47	0.625	1.80
T7	147.95	10.26	0.625	1.65
T8	136.03	9.0	NA	NA
T9	328.24	10.0	NA	NA

Different land-uses were hypothetically assigned to the above seven catchments. Flow rates of the average DWFs were considered, when assigning these land-uses to the respective catchments. It was assumed that higher DWF rates are conveyed to the sewer networks from residential land-use. Therefore, Rimrose and Upper Nothern catchments were assigned as the residential areas. Furthermore, agricultural land-use was assumed to convey the lowest DWFs. Therefore, Millers Bridge catchment was assigned as an agricultural area. These land-use patterns and assigned catchments based on the DWF rates are described in Table 5.

Table 5. Assumed land-use patterns of catchments

Catchment	Land-use pattern
Rimrose / Upper Nothern	Residential
Upper Strand Rd. / Sandhills Lane	Commercial
Millers Bridge	Agricultural
Bankhall Relief	Industrial
Bankhall	Mid Urban

Results and discussion

Different pollutographs for *TSS*, *COD*, *BOD*, *TKN* and *NOX* were developed for every catchment. Concentrations from DWFs and stormwater runoff were summed together in generating these pollutographs. However, pollutographs for T8 and T9 are only with concentrations of stormwater runoff. DWF concentrations from the catchments of Strand Rd. and Nothern were directly fed to the T2 and T5 CSO chambers. A medium concentration level of water quality constituents in DWF, as stated in Table 1, was assumed for this example. Fig. 3 – 8 present few examples of developed pollutographs for the different land-uses.

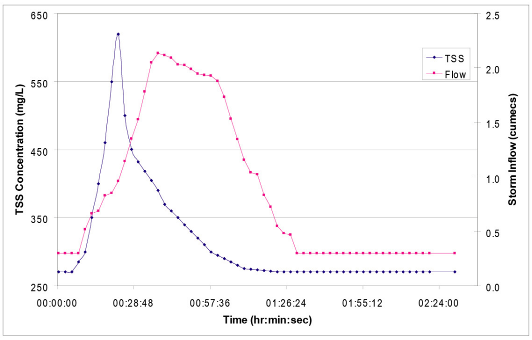

Fig. 3. TSS pollutograph – T1

Fig. 4. TKN pollutograph – T6.

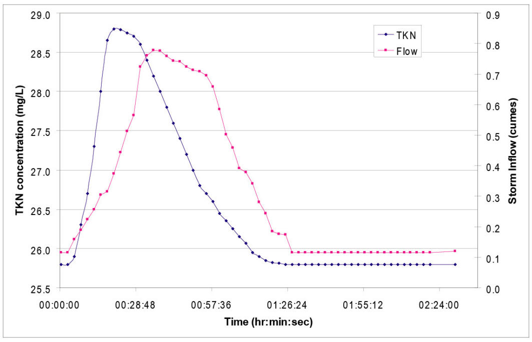

Fig. 4. TKN pollutograph – T6.

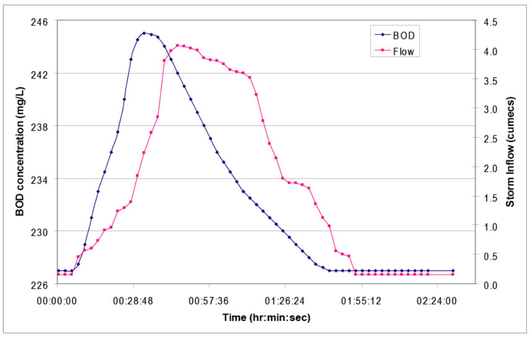

Fig. 5. BOD pollutograph – T4.

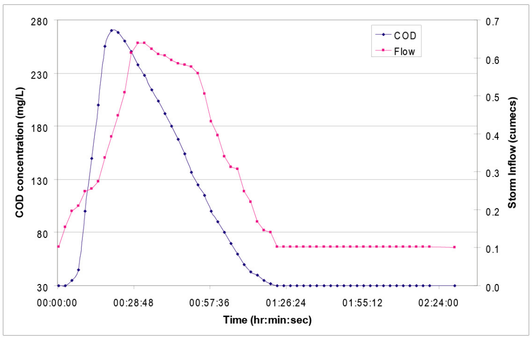

Fig. 6. COD pollutograph – T8.

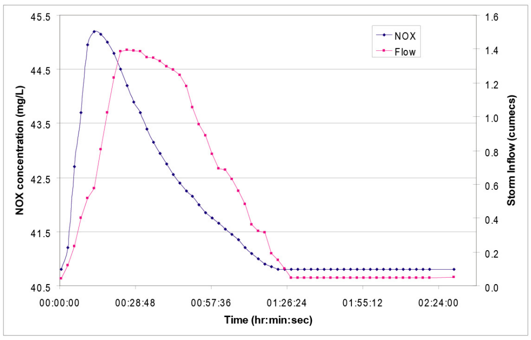

Fig. 7. NOX pollutograph – T3.

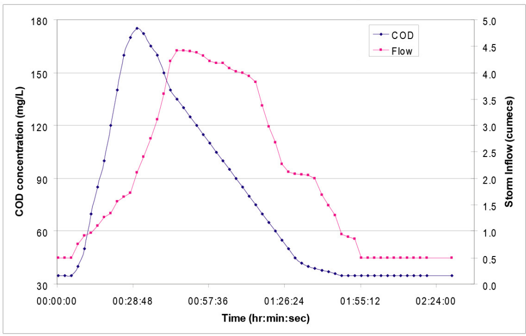

Fig. 8. COD pollutograph – T9.

It can be clearly seen that the peaks of all pollutographs occur before the corresponding peaks of the stormwater runoff. This is because of the first flush phenomenon. First flush is the initial surface runoff from a storm. During the first flush, stormwater runoff has higher pollution concentration levels compared to the remainder of the storm. The higher concentration levels are notable for surface runoff after a dry period. In addition, the differences of the shapes of the pollutographs are clearly visualized from Fig. 3 – 8. Falling limb of TSS pollutograph shows a sudden drop, whereas others show a mild drop after the first flush

Concentrations of different water quality constituents were found in CSOs from the hydraulic simulations. They are based on the inputted pollutographs. These concentrations were used to calculate the pollution load from the CSOs at different CSO locations. The calculated pollution loads and the wastewater treatment cost are being used to develop a multi-objective optimization solution approach. Solutions of this multi-objective optimization problem are expected to give the optimal control settings to control the urban wastewater systems based on the receiving water qualities at CSO locations and the wastewater treatment cost.

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F.3 Multi-objective optimization of urban wastewater systems

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HIC 2012, Hamburg, GERMANY

MULTI-OBJECTIVE OPTIMIZATION OF URBAN WASTEWATER SYSTEMS

UPAKA S. RATHNAYAKE AND TIKU T. TANYIMBOH

Department of Civil Engineering, University of Strathclyde Glasgow. John Anderson Building, 107 Rottenrow, Glasgow G4 0NG, United Kingdom

Combined sewer overflows (CSOs) are one of the environmental problems in many cities. Damage to the natural environment by these CSOs is considerable. Controlling urban wastewater systems is one possible way of addressing the environmental issues from CSOs. However, controlling urban sewer systems optimally is still a challenge, when considering the receiving water quality effects. In this study, a multi-objective optimization approach was formulated considering the pollution load to the receiving water from CSOs and the cost of the wastewater treatment. The optimization model was tested using an interceptor sewer system. Results from the study show some promising findings.

INTRODUCTION

Combined sewer overflows (CSOs) are an environmental burden for most of the urban cities. The damage to the nearby natural waters from these CSOs is noticeable. Therefore, sewer systems managers have to introduce control decisions to control the existing sewer systems.

Previous researchers have used advanced optimization techniques, such as genetic algorithms to find optimal solutions in urban wastewater systems [1]. However, these studies have failed to address the issue of water quality in both combined sewers and receiving waters. In addition, economic measures, such as cost at the downstream wastewater treatment plant, were not considered. Due to the complexity of the problem, some studies were carried out with simplified hydraulic models [5].

In this study, a multi-objective optimization approach was developed, considering the pollution load to the receiving water from CSOs and the wastewater treatment cost. More importantly, a full hydraulic simulation was carried out, instead of considering the simplified hydraulic models.

POLLUTION LOAD EVALUATION

Effluent quality index (*EQI*) is formulated to evaluate the pollution load in a water body as a single variable. Five important water quality parameters, total suspended solids (*TSS*), chemical oxygen demand (*COD*), five-day biochemical oxygen demand (*BOD*), total Kjeldahl nitrogen (*TKN*) and nitrates/nitrites (*NOX*) are accumulated together in forming this single measure. Many researchers have identified it as an index to express the quality of the wastewater and the pollution load to receiving water bodies. Effluent Quality Index (kg/day) is described as

$$EQI = \frac{1}{1000\left(t_f - t_0\right)} \int_{t_0}^{t_f} \left(2C_{TSS} + C_{COD} + 2C_{BOD} + 20C_{NOX} + 20C_{TKN}\right) Q_e\left(t\right) dt \tag{1}$$

where $Q_e(t)$, t_f, and t_0 are the flow rate, final and initial time respectively. C_{TSS}, C_{COD}, C_{NOX}, C_{BOD} and C_{TKN} are the concentrations of total suspended solids, chemical oxygen demand, nitrates and nitrites, five-day biochemical oxygen demand and total Kjeldahl nitrogen, respectively. Concentrations of these five water quality parameters are weighted sum over one complete year. The numerical values in front of these concentrations represent the weighting factors. These weighting factors are applied to denote the contribution of each water quality parameter [7]. These factors are based on the Flandes effluent quality formula for calculating fines [11].

WASTEWATER TREATMENT COST

The funding availability for maintenance and operation of wastewater treatment plants is limited. Therefore, authorities always want to minimize the maintenance and treatment cost at treatment plants.

It is a usual practice to have a treatment plant with an overall capacity of 6×DWF. However, the full treatment capacity is further limited to 3×DWF and the surplus flow is temporary stored in equalization tanks which have the same role as primary sedimentation tanks. In a case where the total flow is more than 6×DWF, the storm tanks fill completely and overflow to the nearby natural water. Therefore, the cost function should be able to address both wastewater treatment cost and the storage cost. Based on various cost models from the literature, a generic cost function based on the treated water volume was adopted. The treatment cost, C (€/year) is described as

$$C = \begin{cases} 916.862 \times \left(86400 \times V\right)^{0.659}, & V \leq 3 \times DWF \quad \text{(2a)} \\ 916.862 \times \left(3 \times DWF\right)^{0.659} + \frac{2}{3}\left(1.69\left(V - 3 \times DWF\right) + 11376\right), & 6 \times DWF \geq V \geq 3 \times DWF \quad \text{(2b)} \\ 916.862\left(3 \times DWF\right)^{0.659} + \frac{2}{3}\left(\left(1.69 \times 3 \times DWF\right) + 11376\right), & V > 6 \times DWF \quad \text{(2c)} \end{cases}$$

where V (m^3/s) is the treated wastewater volume flow rate.

Total treatment cost, including personnel, energy, maintenance, waste and other costs, when the wastewater flow rate is less than or equal to 3×DWF is given by Hernandez-Sancho *et al.* [4] . However, the additional cost, including storage cost, should be included, when the flow rate is more than 3×DWF. Operational and maintenance cost of an equalization tank is assumed to be the same as a primary sedimentation tank. Equation (2b) gives the total wastewater treatment cost and the operational and maintenance cost for a primary sedimentation tank when the flow rate is in between 3×DWF to 6×DWF [10]. Equation (2c) gives the total wastewater treatment cost and the storm tank storage operational and maintenance cost when the flow rate is more than 6×DWF. Numerical value 2/3 in Equations (2b & 2c) is used as a typical conversion rate for € to US$.

PROBLEM FORMULATION AND SOLUTION

Schematics of a typical interceptor sewer and a CSO chamber are shown in the Figure 1. Inflows from catchments' DWF and stormwater runoffs (I_i) are introduced to CSO chambers.

The first objective function was formulated to minimize the pollution load to receiving water through the CSOs. *EQI*, which gives the pollution load, was used to formulate this objective function.

$$Minimize \quad F_1 = \sum_{i=1}^{n} P_i \tag{3}$$

where n and P_i are the number of interceptor nodes or CSO chamber points and the pollution load to the receiving water from the i^{th} CSO chamber respectively. P_i can be expressed as

$$P_i = EQI_i \tag{4}$$

where EQI_i is the effluent quality index at node i (Equation 1).

The second objective function was formulated to minimize the cost of wastewater treatment at the downstream treatment plant (Equation 2).

$$Minimize \quad F_2 = C \tag{5}$$

where C is the treatment cost at the wastewater treatment plant.

With reference to Figure 1, the continuity equations are

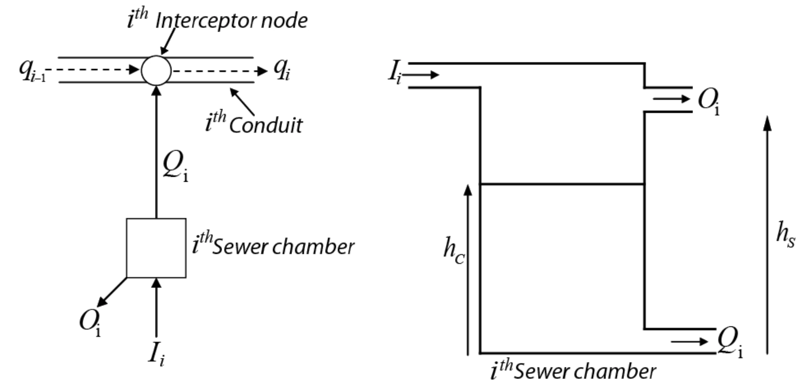
$$Q_i + q_{i-1} - q_i = 0 \tag{6}$$

$$A_C \frac{\Delta h_C}{\Delta t} = I_i - Q_i \quad ; \; h_C < h_S \tag{7}$$

$$A_C \frac{\Delta h_C}{\Delta t} = I_i - Q_i - O_i \quad ; \; h_C > h_s \tag{8}$$

$$0 \le q_i \le q_{\max,i} \tag{9}$$

where A_C is the surface area of the CSO chamber and $q_{max,i}$ is the maximum flow rate at i^{th} conduit.



I_i – Catchment inflow to node i
Q_i – Flow from i^{th} sewer chamber to interceptor node i
q_i – Through flow in interceptor sewer at node i
Oi – Combined sewer over flow discharge at node i
h_C – Water level in sewer chamber
h_S – Spill level of sewer chamber

Figure 1. Schematic diagram of sewer chamber

U.S. EPA SWMM 5.0 [8], the hydraulic model was linked with NSGA II [2] using C programming language. NSGA II, a multi-objective optimization module has already been successfully applied to many practical optimization problems in various disciplines, including urban wastewater systems.

It is assumed here that wastewater flow from CSO chamber to the interceptor sewer is controlled using an orifice at the bottom of the CSO chamber. Orifice openings were initially generated randomly. Hence, the decision variables (q_i) of the optimization approach were indirectly generated. Next, a full hydraulic simulation, including water quality routing was carried out using SWMM 5.0 the results of which were used to calculate the pollution load F_1 and the wastewater treatment cost F_2.

The mass balance and the conservation of energy were automatically satisfied by the hydraulic model. However, the flow rates in interceptor sewer, described at Equation 9, were externally satisfied by the multi-objective optimization module using a tournament constraint handling approach [2]. It uses the binary tournament selection, where two potential solutions are picked at random from the population and the better solution is selected. These two prospective solutions can be either feasible or infeasible based on the constraints. This can lead to three situations as follows:

1. Both solutions are feasible;
2. One is feasible and the other is not; and
3. Both are infeasible.

Solution 1 is deemed to be better than Solution 2 if one of the following conditions is true:

1. Solution 1 is feasible and Solution 2 is infeasible;
2. Both solutions are feasible and Solution 1 dominates Solution 2; and
3. Both are infeasible, but Solution 1 has a lower overall constraint violation

CASE STUDY

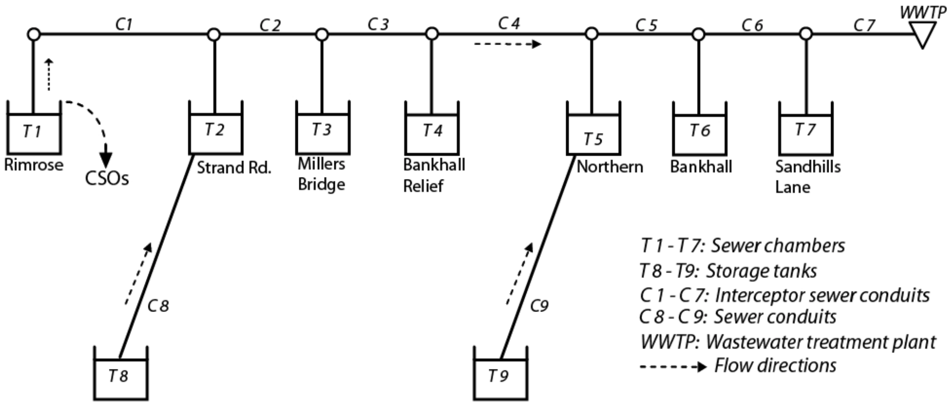


Figure 2. Interceptor sewer system

Developed multi-objective optimization model was applied to a simplified interceptor system. A description of this interceptor sewer system can be found in Thomas [9]. This interceptor sewer system was modified, for this study. The CSO chambers *T1* to *T7* are described in Thomas [9]. Two storage tanks (*T8* and *T9*) were introduced at upper catchments of Strand Rd. and Northern. Figure 2 shows the modified interceptor sewer system. Maximum flow rates allowed through *C1*, *C2* and *C3* are 3.26 m^3/s and that of *C4*, *C5*, *C6* and *C7* are 7.72 m^3/s. The diameters for *C1* to *C3* are 1.66 m and that of *C4* to *C7* are 2.44 m. Depth of the CSO chambers (*T1* to *T7*) and storage tanks (*T8* and *T*9) are 6.42, 7.91, 8.95, 9.04, 9.18, 9.47, 10.26, 8.00 and 9.00 m respectively. Storage tanks *T8* and *T9* are generic and the details of the flow control in these tanks are not discussed in this paper.

Without considering the diurnal effects of the DWF, average flow rates were fed to the *T1, T3, T4, T6, T7* CSO chambers and *T8, T9* storage tanks. More details on the storm run-off flow hydrographs can be found in Thomas [9]. Five different land-uses, including residential, industrial, commercial, agricultural and mid urban were assumed when generating the pollutographs for five different water quality constituents [3]. Shapes of the pollutographs of five different water constituents (*TSS*, *COD*, *BOD*, *NOX*, *and TKN*) were reviewed from the literature.

A basic real-coded NSGA II program was used in this study. The optimization process was done with a population of 100, 100 generations and a simulated binary crossover probability of 1. Many optimization runs with different random seeds were conducted. Different mutation probabilities were tried in different runs. The reason for selecting different mutation probabilities was to compare the performance of the mutation probabilities for this optimization problem. Polynomial mutation, described in Deb *et al.* [2], was used for this optimization approach. The polynomial mutation operator creates a new value for the decision variable, which is near the vicinity of the original value using a probability distribution.

Routing time-step in SWMM 5.0 was kept at 30 seconds, and the results were obtained at 15 minutes. Then, the NSGA II optimization module was run using the obtained results. Figure 3 shows the Pareto optimal fronts for some of the mutation probabilities tested. Each

GA run took about 8 minutes on a Pentium 4 desktop personal computer with a Core 2 Duo processor and 4 GB of RAM.

RESULTS AND DISCUSSION

The best Pareto optimal front was achieved with a mutation probability of 0.6 over the entire population of solutions. Pareto optimal front for 0.6 mutation rate from Figure 3 is shown in Figure 4. Solutions A to H (Figure 4) were selected for further assessment. Results from full hydraulic simulations for these solutions are presented in the following tables.

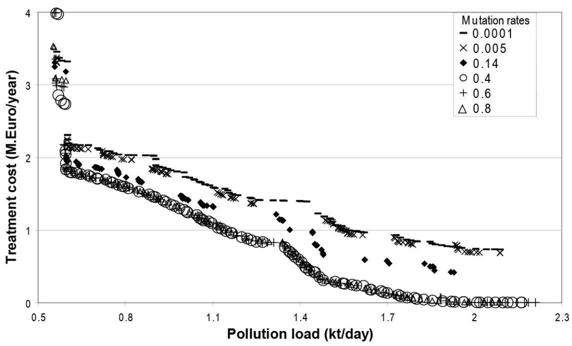


Figure 3. Pareto optimal fronts for different mutation rates

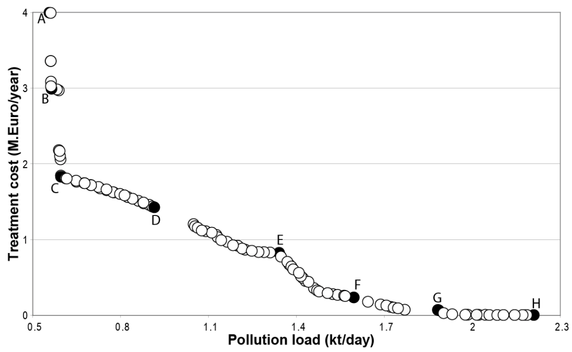


Figure 4. Pareto optimal front for 0.6 mutation rate

As stated above the flow rates through the sections of the interceptor sewer were constrained. It can be clearly seen in Table 1 that the flow rates through these conduits are less than or equal to the maximum allowed flow rate for all the tabulated cases. Furthermore, Table 2 shows the CSO rates for Solutions A to H. Solution A that corresponds to the minimum pollution load to receiving water has smaller CSO rates than Solution H that corresponds to the minimum wastewater treatment cost. Table 3 shows the wastewater depths at CSO chambers and storage tanks for Solutions A to H. It can be seen in Table 3 that the storage tanks (*T8* and *T9*) store wastewater in order to prevent CSOs at downstream *T2* and *T5* CSO chambers.

Table 1. Flow rates through the interceptor sewer sections at t = 15 minutes

Solution	Interceptor sewer flow rates (m^3/s)						
	C1	*C2*	*C3*	*C4*	*C5*	*C6*	*C7*
A	2.68	1.60	3.25	6.71	5.86	5.24	4.05
B	2.68	1.60	3.25	6.62	5.71	3.77	2.48
C	2.68	1.60	3.26	5.03	4.16	2.36	1.19
D	2.70	1.60	2.09	3.94	3.21	1.77	0.81
E	2.66	1.63	0.61	2.41	1.82	0.89	0.35
F	2.70	1.61	2.82	2.48	1.85	0.43	0.05
G	2.68	1.51	1.34	1.51	0.86	0.15	0.01
H	2.59	1.54	0.47	0.14	0.00	0.00	0.00

Table 2. Combined sewer overflow rates at CSO chambers at t = 15 minutes

Solution	Combined sewer overflows (m^3/s)								
	T1	*T2*	*T3*	*T4*	*T5*	*T6*	*T7*	*T8*	*T9*
A	0	0	2.10	0	0	0	0	0	0
B	0	0	2.13	0	0	0	0	0	0
C	0	0	2.25	0	0	0	0	0	0
D	0	0	3.46	0	0	0	0	0	0
E	0	0	4.75	0.42	0	0	0	0	0
F	0	0	2.82	4.36	0	0	0	0	0
G	0.01	0	4.09	4.08	0	0	0	0	0
H	0.64	0	4.74	4.36	0	0	0	0	0

Table 3. Wastewater depths at CSO chambers and storage tanks at t = 15 minutes

Solution	Wastewater depths (m)								
	T1	*T2*	*T3*	*T4*	*T5*	*T6*	*T7*	*T8*	*T9*
A	5.41	0	8.2	2.91	0.02	1.81	7.63	6.47	7.84
B	5.42	0	8.21	3.07	0.02	7.18	7.63	6.47	7.84
C	5.41	0	8.22	7.93	0.02	7.18	7.62	6.47	7.84
D	5.36	0	8.3	8.03	0.02	7.18	7.63	6.47	7.84
E	5.37	0	8.39	8.13	0.02	7.18	7.63	6.47	7.84
F	5.39	0	8.26	8.45	0.02	7.18	7.63	6.47	7.88
G	5.42	0	8.34	8.43	0.02	7.18	7.63	6.47	7.88
H	5.52	0	8.39	8.45	0.02	7.18	7.62	6.53	7.8

The proposed model gives the optimal CSO control settings where a single set of static control settings is used throughout the 15 minute storm duration. The ultimate objective of this research is to develop an extended period dynamic optimization procedure for the full duration of the storm. The model development is still in progress and these initial results will be used to make improvements.

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