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Evolutionary Multi-Objective Optimal Control of Combined Sewer Overflows

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Abstract

This paper presents a novel multi-objective evolutionary optimization approach for the active control of intermittent unsatisfactory discharges from combined sewer systems. The procedure proposed considers the unsteady flows and water quality in the sewers together with the wastewater treatment costs. The distinction between the portion of wastewater that receives full secondary treatment and the overall capacity of the wastewater treatment works (including storm overflow tanks) is addressed. Temporal and spatial variations in the concentrations of the primary contaminants are incorporated also. The formulation is different from previous approaches in the literature in that in addition to the wastewater treatment cost we consider *at once* the relative polluting effects of the various primary contaminants in wastewater. This is achieved by incorporating a measure of the overall pollution called the effluent quality index. The differences between two diametrically opposed control objectives are illustrated, i.e. the minimization of the pollution of the receiving water or, alternatively, the minimization of the wastewater treatment cost. Results are included for a realistic interceptor sewer system that show that the combination of a multi-objective genetic algorithm and a stormwater management model is effective. The genetic algorithm achieved consistently the frontier optimal control settings that, in turn, revealed the trade-offs between the wastewater treatment cost and pollution of the receiving water.

Keywords: Optimal control, combined sewer system, effluent quality index, integrated wastewater management, water pollution control, wastewater treatment cost

1 INTRODUCTION

Combined sewer systems carry both sanitary wastewater and stormwater. However, the capacity of sewers is limited and this results in occasional flooding of some areas and intermittent unsatisfactory discharges to the receiving water known as combined sewer overflows (Rodriguez *et al.* 2012). Even though new combined sewer systems are no longer constructed because of greater environmental awareness and more effective regulation in recent years, the existing networks still operate in many cities. Therefore, controlling the operation of the existing systems is self-evidently a possible way of alleviating the problem of pollution due to the intermittent unsatisfactory discharges from combined sewer systems. The causes, adverse consequences and mitigation measures of pluvial flooding are discussed in Susnik *et al.* (2014). A review of the optimization of the design of sewer systems is available in Karovic and Mays (2014). The description of a combined sewer overflow chamber and its modelling is available in Chen *et al.* (2013). Details on the effectiveness of best management practices are available in Kaini *et al.* (2012). Best management practices are measures that contribute to improve or safeguard the condition of receiving water bodies.

Most of the previous approaches in the literature utilised control measures for combined sewer overflows based on the volume of wastewater discharged from the sewer system without considering water quality (Beraud *et al.* 2010, Cembrano *et al.* 2004, Darsono *et al.* 2007, Joseph-Duran *et al.* 2014). However, Lau *et al.* (2002) showed that, in isolation, the frequency and volume of discharges cannot be considered as accurate indicators of the receiving water quality because the biochemical oxygen demand, ammonia and dissolved oxygen concentrations lacked a strong relationship with the frequency and volume of discharges. Similarly, Rauch *et al.* (1998) concluded that a reduction in the volume of the wastewater discharges does not necessarily improve water quality in the receiving water. These findings have raised doubts about the benefits of purely volumetric approaches to the control of intermittent unsatisfactory discharges or combined sewer overflows (CSOs).

On the other hand, Fu *et al.* (2008 and 2010) addressed the water quality in the receiving water by treating several water quality indices (e.g. biochemical oxygen demand and the concentration of dissolved oxygen) as separate objectives to be optimized. However, the approach leads to complications because on the whole the water quality measures are highly interrelated. Also, while the effects of combined sewer discharges on the receiving water have been considered previously (Fu *et al.* 2010, Lacour and Schutze 2010, Petruck *et al.* 1998, Vanrolleghem *et al.* 2005, Weinreich *et al.* 1997) the associated trade-offs were not addressed directly. For example, the wastewater treatment costs corresponding to alternative control settings were not considered (Darsono and Labade 2007, Fu *et al.* 2010). A major disadvantage of single-objective optimization (Cembrano *et al.* 2004, Darsono *et al.* 2007, Rauch *et al.* 1999b) is that it provides only one optimal solution to the problem. Single objective optimization methods can be useful if the trade-off between the objectives is not important. The multi-objective approach, on the other hand, considers the integrity of all the objectives and, therefore, reduces somewhat the requirement to consider the relative importance of each objective beforehand. In other words, all objectives are considered together to obtain the Pareto-optimal solutions in a single run of the optimization procedure.

Furthermore, some previous studies used simplified hydraulic models (Meirlaen *et al.* 2002, Vanrolleghen and Meirlaen 2002, Vanrolleghem *et al.* 2005) due to the complexity of the problem and the fact that accurate hydraulic models are computationally very demanding and time consuming to use. Self-evidently, simplified hydraulic models sacrifice accuracy to varying degrees and, in any case, the development of such models can be challenging, expensive and time consuming (Vanrolleghem *et al.* 2005). Therefore, we used the fully dynamic rainfall-runoff hydraulic simulation model SWMM 5.0 (Rossman 2009) that also accounts for the water quality in the wastewater collection network. SWMM 5.0 is a simulation model for urban and sub-urban areas in which gradually varying unsteady flows are routed using the mass conservation and momentum equations. The equations are solved using the finite difference method to obtain the flow rates in the sewers and water levels at the junctions. Also, for water quality routing, the sewers and tanks are modelled as continuously stirred tank reactors in which mass conservation is used to calculate the concentrations of the chemical species leaving a conduit or tank at the end of each flow routing time step (Rossman 2006).

We developed a multi-objective optimization approach that considers the unsteady flows and water quality in the sewer system together with the resulting variations in the cost of treating the fraction of the wastewater that reaches the wastewater treatment works. The optimization model accounts for both full secondary treatment and the partial treatment provided by storm overflow tanks. The aims were to minimize the pollution of the receiving water and the wastewater treatment cost. The temporal and spatial variations in the concentrations of suspended solids and other primary contaminants were considered in the optimization model. Therefore, unlike previous approaches, we considered *at once* the combined effects of the various pollutants by means of the effluent quality index that is a measure of the rate of pollution. Due to the complexity of the problem, only the combined sewer system was considered in the optimization model. The additional challenges that the dynamic response of the receiving water introduces have not been addressed yet in the proposed model. We used the stormwater management model SWMM 5.0 (Rossman 2009) for the hydraulic simulations. Two contrasting control strategies were considered i.e. minimization of the pollution load to the receiving water or, alternatively, minimization of the wastewater treatment cost. The overall aim was to investigate the potential benefits of active system control and the effectiveness of the proposed approach. Initial results that are based on a realistic interceptor sewer system are included for demonstration purposes and to help identify possible priority areas for future improvement.

2 MODEL FORMULATION AND SOLUTION

Two opposing aims were considered in the multi-objective optimization model. The first was to minimize the pollution load to the receiving water while the second was to minimize the cost of treating the wastewater at the wastewater treatment plant downstream. Minimization of the pollution load to the receiving water aims primarily to manage the pollution load from the combined sewer overflows by spatial and temporal control of the flows in the entire interceptor sewer system. This is done by taking advantage of any spare capacity in the sewerage system including the sewers, the wastewater treatment works and any on-line or off-line storage. Inevitably this leads to variations in the flow regime at the wastewater treatment works. To account for this we included the wastewater treatment cost in the optimization model as an objective that is to be minimized.

Figure 1 shows an interceptor sewer system. For the i^{th} overflow chamber at time t, $h_{i,t}$ is the water level; Hs_i is the *s*pill level; $I_{i,t}$ is the inflow rate from the catchment; $O_{i,t}$ is the rate of discharge to the receiving water; $q_{i,t}$ is the volumetric through-flow rate in the sewer; and $Q_{i,t}$ is the volumetric flow rate from the combined sewer overflow chamber to the interceptor sewer. The constraints that address the continuity of flow are as follows.

$$0 \le q_{i,t} \le q_{\max,i} \tag{1}$$

$$Q_{i,t} + q_{i-1,t} - q_{i,t} = 0 \tag{2}$$

$$A_{T,i}\frac{\Delta h_{i,t}}{\Delta t} = I_{i,t} - Q_{i,t} \;\; ; h_{i,t} < H_{S,i} \tag{3a}$$

$$A_{T,i}\frac{\Delta h_{i,t}}{\Delta t} = I_{i,t} - Q_{i,t} - O_{i,t} \;\; ; h_{i,t} > H_{S,i} \tag{3b}$$

At the interceptor sewer node i, $q_{max,i}$ is the capacity of the interceptor sewer; $A_{T,i}$ is the water surface area in the overflow chamber and is assumed to be constant; $\Delta h_{i,t} / \Delta t$ approximates the time rate of change of the water level in the overflow chamber. Eqs. 3b represent conditions in which discharge to the receiving water occurs, i.e. when the water level in the overflow chamber is above the spill level $H_{S,i}$ (Figure 1). Eqs. 3a apply when the water level is below the spill level and therefore discharge to the receiving water does not occur. The term on the left hand side of Eqs. 3 represents the rate at which the volume of water in the overflow chamber is changing. Eq. 1 defines the feasibility of the volumetric flow rates in the interceptor sewer system. Eq. 2 defines the continuity of the flows at the nodes of the interceptor sewer system. Eqs. 3 define the continuity of the flows of the combined sewer overflow chambers.

The decision variables of the optimization problem are the combined sewer overflow settings. More specifically, the decision variables are the respective orifice and other settings that control the flows from the sub-catchments to the interceptor sewer. For simplicity we assumed that the flow from the overflow chamber to the interceptor sewer takes place by means of an orifice in the overflow chamber. Thus the decision variables here are the $Q_{i,t}$ values, the time-varying volumetric flow rates from the overflow chambers to the interceptor sewer. Inherently, the hydraulic simulation model SWMM 5.0 accounts for Eqs. 2 and 3 that are thus satisfied automatically. The concept of constraint dominance in Deb *et al.* (2002) was used to address Eqs. 1. The procedure is not described here as it is well established.

2.1 Pollution load evaluation

The effluent quality index is a parameter that considers the effects of several important wastewater contaminants in aggregate and includes the total suspended solids, chemical oxygen demand, five-day biochemical oxygen demand, total Kjeldahl nitrogen and nitrates/nitrites. The effluent quality index has been used previously as a performance and sensitivity index for wastewater treatment (Copp 2002, Kim *et al.* 2006, Lee *et al.* 2006). Additional information on the effluent quality index is available in Mussati et al. (2002) and Rathnayake and Tanyimboh (2012a). This parameter is particularly useful as it aims to quantify the total amount of pollution as shown in Eq. 4.

$$E = \frac{1}{t_f - t_i} \int_{t_i}^{t_f} \left[2C_{TSS}(t) + C_{COD}(t) + 2C_{BOD}(t) + 20C_{NOX}(t) + 20C_{TKN}(t) \right] Q_w(t) dt \qquad (4)$$

E is the effluent quality index that represents the *effective pollution rate* (for example, in kg/day with the units of the terms in Eq. 4 selected accordingly); t_i and t_f respectively represent the start and end of the period during which the wastewater discharge takes place; Q_w is the volumetric flow rate of the wastewater; C_{TSS}, C_{COD}, C_{NOX}, C_{BOD} and C_{TKN} are the concentrations of total suspended solids, chemical oxygen demand, nitrates/nitrites, five-day biochemical oxygen demand and total Kjeldahl nitrogen, respectively.

Various catchment characteristics e.g. land-use i.e. residential, commercial, agricultural, etc. influence the concentrations of the various contaminants in the run-off from rainfall. For example, Duncan (1999) described the composition of stormwater run-off for different land-uses. Also, the pollutographs for various contaminants have been derived empirically in previous studies (Li et al. 2007, Yusop et al. 2005). We carried out a detailed and extensive investigation of the properties of the relevant pollutographs and thus developed pollutographs for the interceptor sewer system discussed in the next section (Rathnayake and Tanyimboh 2012a, Rathnayake 2013). The objective function for the pollution load was taken as

$$F_1 = \sum_{t=1}^{N_t} \sum_{i=1}^{N_i} E_{i,t} \qquad (5)$$

The subscripts t and i represent the time steps and overflow chambers, respectively; N_t is the number of time steps; N_i is the number of overflow chambers; $E_{i,t}$ is the pollution load contributed by overflow chamber i during time step t; F_1 is the total pollution load to the receiving water for the entire duration of the storm. Eq. 5 accounts for the changes in the concentrations of the contaminants in the wastewater throughout the duration of a storm.

2.2 Wastewater treatment cost

A wastewater treatment plant typically has an overall capacity that may be approximately about twice the secondary treatment capacity. Any excess flow above the secondary treatment capacity is held in stormwater detention tanks which have a role similar to primary sedimentation tanks. If the total volume of wastewater exceeds the overall capacity of the plant, then the stormwater detention tanks fill completely and overflow to the receiving water (Metcalf and Eddy 2004). We developed a generic indicative cost model for wastewater treatment (Rathnayake and Tanyimboh 2012a,b; Rathnayake 2013) based on various models and data in the literature (Friedler and Pisanty 2006, Hernandez-Sancho and Sala-Garrido 2008, United Nations 2003). For example, Friedler and Pisanty (2006) observed that the operation and maintenance cost, as a proportion of the total treatment cost, rises as the design flow rate increases and has the general form of an "S shaped" curve. The generalised approximation to the wastewater treatment cost was taken as

$$C = \begin{cases} aQ_w^f; \quad Q_w \le 3Q_d & \text{(6a)} \\ bQ_d^f + cQ_w - dQ_d + e; \quad 3Q_d \le Q_w \le 6Q_d & \text{(6b)} \\ bQ_d^f + dQ_d + e; \quad Q_w > 6Q_d & \text{(6c)} \end{cases}$$

where C (€/year) is the cost; Q_w (m^3/s) is the volumetric flow rate of the wastewater; and Q_d (m^3/s) is the dry-weather flow, i.e. the average daily flow rate to the wastewater treatment plant under dry-weather conditions (IWEM 1993); a to f are empirical coefficients, where $a = 1.642\times10^6$, $b = 1.8912\times10^3$, $c = 1.13$, $d = 3.38$, $e = 7.584\times10^3$ and $f = 0.659$. The cost model in Eq. 6 is for the total operating cost that includes wastewater treatment, personnel, energy, maintenance, etc. Eq. 6 can be replaced easily with any suitable alternative cost function. The objective function for the cost of wastewater treatment was taken as

$$F_2 = \sum_{t=1}^{N_t} C_t \tag{7}$$

where C_t is the wastewater treatment cost for time step t and F_2 is the total cost for the entire duration of the storm.

2.3 Optimization problem and solution

The optimization problem can be summarized as follows. Minimize $\boldsymbol{f} = (F_1, F_2)^T$ subject to the system constraints. The conservation of mass and energy and other system equations were satisfied by the hydraulic simulation model SWMM 5.0. However, the constraints for the capacity of the interceptor sewer i.e. Eqs. 1 were addressed using the binary constraint-tournament method in the Non-dominated Sorting Genetic Algorithm (NSGA) II (Deb *et al.* 2002). We wrote a routine in the C computer programming language to link the optimization and simulation models.

NSGA II is a computationally fast and elitist multi-objective evolutionary optimization algorithm that has been applied successfully in many practical problems in various disciplines (Tabari and Soltani 2013, Takbiri and Afshar 2012) including urban wastewater systems (Fu *et al.* 2010). Additional applications include a memetic algorithm with local search and cultural learning (Barlow and Tanyimboh 2014) and recently introduced penalty-free methods for water distribution systems (Siew and Tanyimboh 2012, Siew *et al.* 2014, Saleh and Tanyimboh 2013, 2014). The above-mentioned enhanced implementations have been shown to be particularly effective in terms of the computational efficiency and quality of the solutions. However, in view of the complexity of the problem addressed here, and as a first attempt, we used the binary constraint-tournament method (Deb *et al.* 2002).

It was assumed without loss of generality that wastewater from the combined sewer overflow chamber to the interceptor sewer is controlled by a rectangular orifice at the bottom of the chamber. For the purposes of the optimization a number of relatively small time steps were defined to cover the entire duration of the storm. For the first time step the heights of the orifices were generated randomly to create the initial population of solutions. Then, a hydraulic simulation was carried out. The results of the hydraulic simulation were used to calculate the pollution load and the wastewater treatment cost. For subsequent time steps the optimized orifice heights from the preceding time step were taken as the initial conditions.

3 PRACTICAL APPLICATION

A realistic generic interceptor sewer system derived from modifications to the interceptor sewer system described in Thomas *et al.* (1999, 2000) and Thomas (2000) that is based on the interceptor sewer system in Liverpool, UK, was used for illustration purposes, for a storm duration of 2.5 hours. A schematic diagram of the interceptor sewer system is shown in Figure 1c. We extended the system's capacity and operational complexity by adding two storage tanks, i.e. tanks 8 and 9, at overflow chambers 2 and 5, respectively. These new tanks were assumed to operate on-line; i.e. they start to fill as the volumetric flow rate in the interceptor sewer approaches the maximum through-flow rate. The wastewater in the on-line tanks returns later to the interceptor sewer when the flow from the upstream section plus the inflow from the subcatchment is less than the capacity of the sewer.

Referring to Figure 1c, the maximum flow rate for sewers C1 to C3 is 3.26 m^3/s and for C4 to C7, 7.72 m^3/s. The diameter for sewers C1 to C3 is 1.66 m and for C4 to C7, 2.44 m. Depths of overflow chambers T1 to T7 and storage tanks T8 and T9 are 5.42, 6.91, 7.95, 8.04, 8.18, 8.47, 9.26, 6.91 and 8.18 m, respectively. The volumes of overflow chambers T1 to T7 and storage tanks T8 and T9 are 1533, 940, 400, 1365, 2685, 1415, 1370, 940 and 2685 m^3, respectively. Orifices O1 to O7 are rectangular. The widths are 1.25, 1.7, 1.5, 2.08, 2.65, 1.8 and 1.65 m, respectively. Orifices O1 and O5 are 1.45 m high and the rest are 0.625 m high. Additional data (e.g. sewer invert levels, slopes, etc.) are available in Thomas (2000), Rathnayake and Tanyimboh (2012a) and Rathnayake (2013).

The various sub-catchments have stormwater run-off hydrographs the details of which are available in Thomas (2000) and references therein (Thomas et al. 1999, 2000; etc.). The present demonstration is for the average dry-weather flow for *sanitary wastewater* in conjunction with a single storm. Due to the relatively short duration of the storm, and as in Thomas (2000), we did not consider the diurnal variations in the flow rate of *sanitary wastewater*. The surface water run-off hydrographs and temporal and spatial variations in the concentrations of contaminants (Eqs. 4 and 5) are available in Rathnayake (2013). Figure 2 shows run-off hydrographs and suspended solids concentrations for sub-catchments 1 to 3.

For the computational solution we allowed 10,000 function evaluations (i.e. hydraulic simulations) for each time step. The population size was 100; crossover probability was 1.0; distribution index (see Deb *et al.* 2002) for both crossover and mutation was 20; and mutation rate was 0.6. The sensitivity analysis carried out suggests the results are stable with respect to the mutation rate (Rathnayake 2013). The time interval used in the finite difference solution of the hydraulic equations in SWMM 5.0 was 30 seconds while the time interval specified for each time step in the optimization was 15 minutes. After the first time step in the optimization, the optimal solutions for the minimum pollution load and the minimum wastewater treatment cost were available. The respective orifice heights for the above-mentioned solutions were used as two alternative starting points for the second time step in the optimization. The optimal orifice heights for the second time step (i.e. t = 15 to 30 minutes) were obtained from their respective Pareto-optimal fronts. This sequence was repeated for all successive time steps until the end of the storm. This gave two sets of time-varying orifice heights, for the minimum pollution load and minimum treatment cost, respectively. It was not possible to start the hydraulic simulation model SWMM 5.0 at any time other than $t = 0$, due to restrictions in the software. Consequently, the simulation for each successive time step started at time t = 0 and continued until the end of the time step. Therefore the execution time of the optimization algorithm was increased significantly. We used a Pentium 4 desktop personal computer (2.7 GHz Core 2 Duo processor, 4 GB RAM).

4 RESULTS AND DISCUSSION

An optimization run with 10,000 function evaluations in each time step for the minimum pollution load and minimum treatment cost options for a storm of 2.5 hours with ten time steps took about 76 minutes and 18 minutes, respectively. Table 1 shows that the orifices for the minimum treatment cost option are essentially closed in practical terms. The exception is orifice 1 for time steps 1 and 2. The orifice closures reduce effectively the size of the sewer system that is simulated for the fitness assessments and, consequently, the hydraulic simulations require less time. The Pareto-optimal fronts obtained for ten different randomly generated initial populations were essentially the identical; Figure 3a shows the Pareto-optimal front at time t = 15 minutes. Solution $A_{T1}(S)$ gives the minimum pollution load whereas solution $B_{T1}(S)$ gives the minimum treatment cost. Figures 3b and 3c illustrate the progress of the optimization. It can be seen the algorithm convergences quickly at about 2000 function evaluations.

At time t = 30 minutes, the solution for the minimum pollution load was obtained by executing the optimization from time t = 0 to 30 minutes, with the control settings from Solution $A_{T1}(S)$ (Figure 3a) as the initial condition. Similarly, for the minimum treatment cost option, the optimization was run from time t = 0 to 30 minutes, with the control settings from Solution $B_{T1}(S)$ (Figure 3a) as the initial condition. The optimized solutions achieved have significant differences in terms of the treatment cost and pollution load, i.e. €11,000/year and 7.1 million kg/day for the minimum treatment cost option compared to €6.8 million/year and 4.6 million kg/day for the minimum pollution option. As mentioned previously the orifices for the minimum treatment cost option are essentially closed (Table 1). This minimizes the flow to the interceptor sewer and the treatment plant which keeps the treatment costs low. This low cost is achieved at the expense of a high pollution load to the receiving water. For the minimum pollution load option more wastewater reaches the interceptor sewer and the treatment plant which reduces pollution.

On completion of the optimization, the respective sets of orifice settings obtained for orifices 1 to 7 were used to conduct full hydraulic simulations to check the solutions achieved. Figure 5 shows the volumetric flow rates in the conduits at the end of each time step for the minimum pollution option; the dashed lines represent the respective capacities. The maximum flow rates did not exceed the capacities, i.e. the solutions obtained may be considered feasible. Furthermore, conduits 4 to 7 have higher flow rates than conduits 1 to 3. This is consistent with the normal expectation that downstream sewers would carry more wastewater than those upstream. The results show a consistent spatial progression of increasing flow rates for conduits 1 to 7.

Figure 5 shows the wastewater volumes in the overflow chambers and storage tanks for the minimum pollution option; the dashed lines represent the respective capacities. Wastewater volumes above the dashed lines represent the combined sewer overflows. It may be noted that storage tanks 8 and 9 remain full throughout, except at the start of the storm. This suggests the model is effective as it seems to utilise the available storage in full. Figure 6 shows the overflow volumes. Much higher volumes can be seen between 0:30:00 and 1:30:00 hours. During this time interval high inflows from stormwater run-off enter the combined sewer system (Rathnayake 2013), with a corresponding increase in the spillage volumes. However, the volumes for the overflow chamber 7 reveal an interesting pattern. All other chambers have low volumes at 02:30:00 hours. This is expected also, due to the reduced stormwater flow rates near the end of the storm. However, overflow chamber 7 at the most downstream end of the sewer system has relatively constant rates of discharge during the entire storm as it carries combined wastewater from the entire catchment.

Figure 7 shows the cumulative pollution loads, wastewater treatment costs and combined sewer overflows, for both the minimum pollution load and minimum treatment cost options. It is noted that the minimum pollution load option at the end of any time step has a lower pollution load than the minimum treatment cost option (Figure 7a). Conversely, the minimum treatment cost option has lower treatment costs throughout the storm than the minimum pollution load option (Figure 7b). Figure 7c illustrates an interesting result. Until 01:30:00 hours, the two control options have roughly comparable spillage volumes. However, Figure 7a shows the corresponding pollution loads are significantly different during this time period. This demonstrates clearly that a volumetric optimization approach that does not address the water quality aspects explicitly may not be sensitive enough to control the sewer system satisfactorily. Besides the wastewater treatment cost, the proposed optimization model has an objective function that addresses water quality explicitly. The effluent quality index considers *at once* five major water pollution parameters. This represents an advance on previous approaches (Fu *et al*. 2008 and 2010; Vanrolleghen *et al*. 2005). Using one objective function for the overall pollution, difficulties associated with optimization problems with many objectives are avoided (Saxena *et al*. 2013; Sinha *et al*. 2013).

At the end of the storm, the combined sewer system discharged 17.72 million kg/day of pollution to the receiving water and the treatment cost was €41.526M/year for the minimum pollution load option. By contrast 26.53 million kg/day of pollution was discharged to the receiving water and the treatment cost was €1.74M/year for the minimum treatment cost option. These figures illustrate the potential for active system control, for the system considered here. However, further investigation and improvements to the optimization model may be beneficial, including further verification based on field data. This inevitably poses many challenges related to issues such as land-use, rainfall run-off and pollution wash-off characteristics and the cost objective function. Also, the response of the receiving water and details of the wastewater treatment plant were not addressed.

5 CONCLUSIONS

A novel multi-objective evolutionary optimization approach for control of combined sewer overflows that considers unsteady sewer flows, pollution load to the receiving water and wastewater treatment costs was developed. It can handle temporal and spatial variations in water quality in the rainfall run-off from different sub-catchments and considers various contaminants including suspended solids, nitrates/nitrites and ammonia at once. The decision variables of the optimization problem are the time-dependent combined sewer overflow settings. The optimization model was applied to a realistic interceptor sewer system. The results demonstrate the benefits of the multi-objective optimization. The non-dominated solutions provide a range of alternative control options that offer choice and flexibility to the sewer system controllers to enable the best control settings based on the environmental regulations, costs and other relevant factors to be chosen. Verification through simulations in SWMM 5.0 of the control options identified further indicates that the solutions found are both feasible and effective. With further developments in the technology to measure flows and water quality parameters and send feedback to a control location, this dynamic optimization model is proposed as a contribution in the active control of integrated urban wastewater systems.

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Table 1 Combined sewer overflow settings for the minimum treatment cost and pollution load options

Time steps	Time slots (minutes)	Optimized orifice heights for the orifice numbers indicated (cm)						
		1	2	3	4	5	6	7
1	0-15	41.98[a] *(5.02)*[b]	0 *(0)*	21.84 *(0)*	24.08 *(0)*	0 *(0)*	0.02 *(0)*	0 *(0)*
2	15-30	21.06 *(1.40)*	13.45 *(0.02)*	0 *(0)*	12.34 *(0)*	1.64 *(0)*	18.11 *(0)*	0 *(0)*
3	30-45	39.63 *(0)*	0 *(0)*	1.18 *(0)*	0.03 *(0)*	0 *(0)*	6.58 *(0)*	26.63 *(0)*
4	45-60	40.58 *(0.01)*	0 *(0)*	0.03 *(0)*	0.34 *(0)*	0.01 *(0)*	16.45 *(0)*	14.85 *(0)*
5	60-75	14.34 *(0)*	0.05 *(0)*	13.85 *(0)*	0.04 *(0)*	0.25 *(0)*	15.00 *(0)*	11.56 *(0)*
6	75-90	25.05 *(0)*	0.56 *(0)*	11.37 *(0)*	1.81 *(0)*	14.70 *(0)*	55.49 *(0)*	0 *(0)*
7	90-105	29.32 *(0)*	0.01 *(0)*	9.17 *(0)*	0.10 *(0)*	20.87 *(0)*	0.00 *(0)*	0 *(0)*
8	105-120	38.41 *(0.03)*	0.02 *(0)*	3.53 *(0)*	0.03 *(0)*	19.67 *(0)*	5.01 *(0)*	0.39 *(0)*
9	120-135	41.71 *(0)*	0.93 *(0)*	0.77 *(0)*	0.06 *(0)*	21.89 *(0)*	0.67 *(0)*	0 *(0)*
10	135-150	19.07 *(0.07)*	0.01 *(0)*	62.39 *(0)*	4.95 *(0)*	15.17 *(0)*	0.33 *(0)*	0 *(0)*

[a] Values for the minimum pollution load option are in normal type. [b] *Values for the minimum wastewater treatment cost option are italicised in parentheses. For the minimum wastewater treatment cost option, except for orifice 1 during the first two time steps the orifices are closed for practical purposes.*

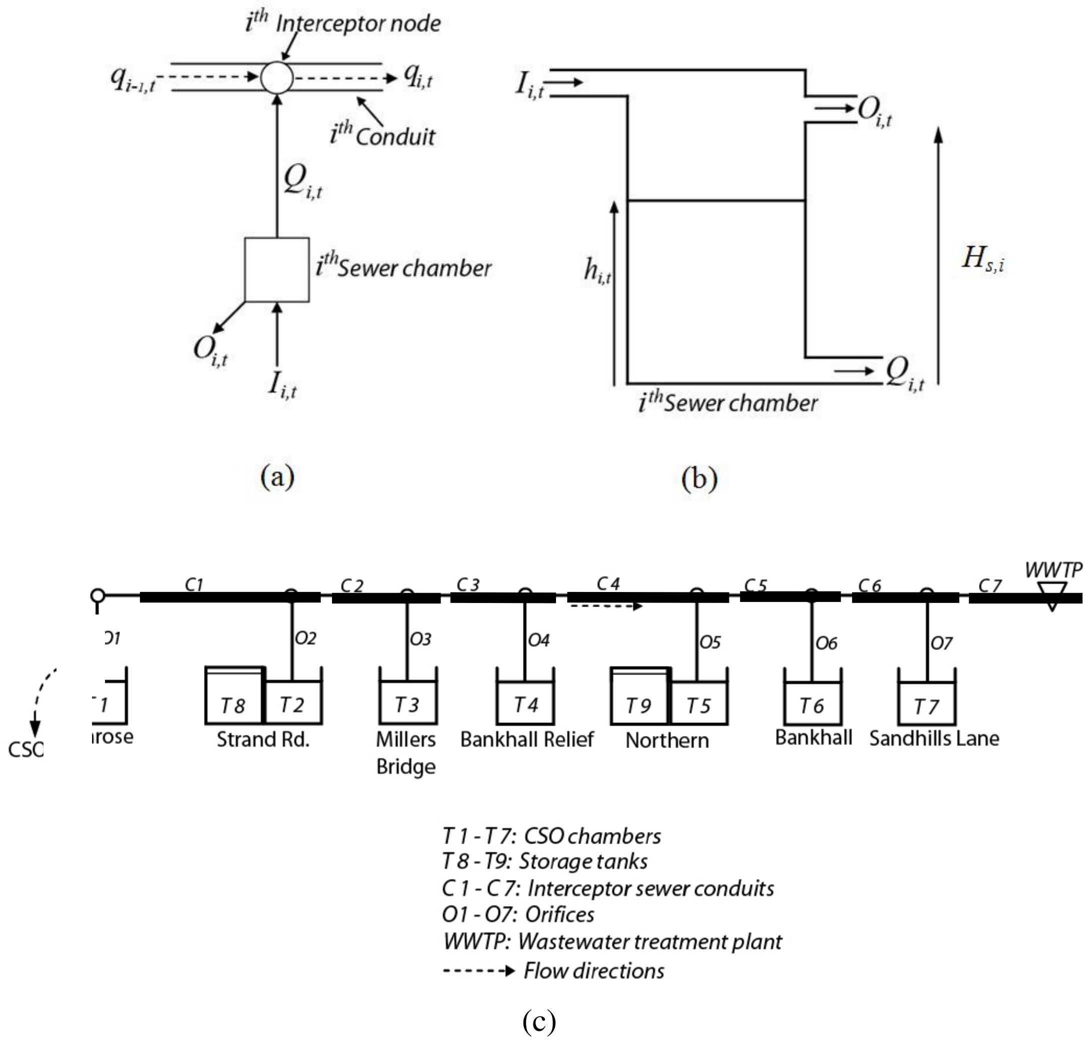
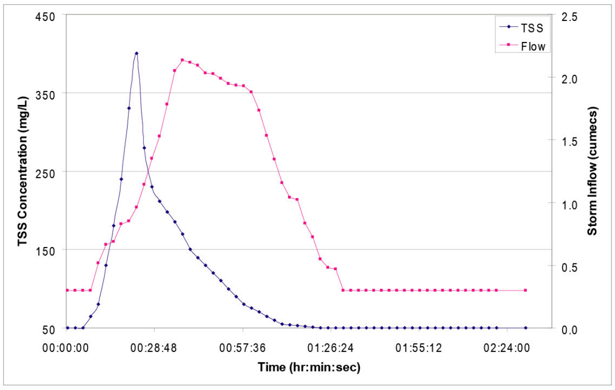
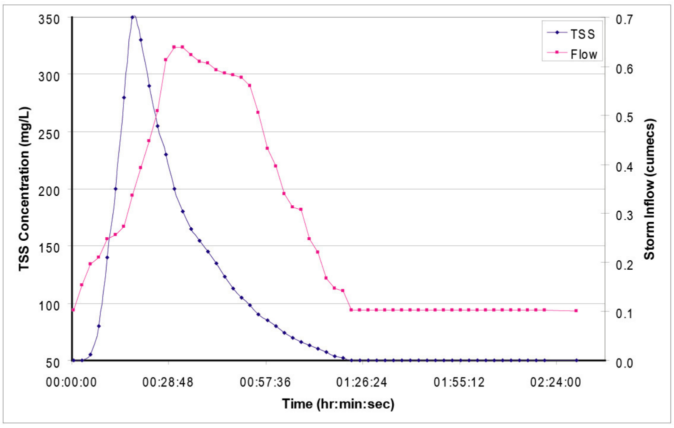


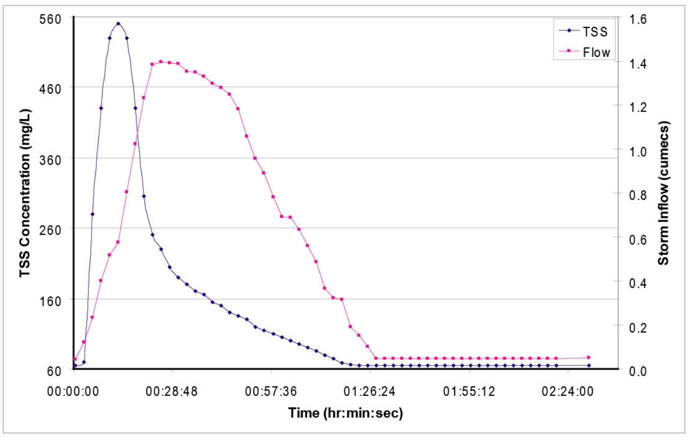
Fig. 1 Interceptor sewer system. (a) Flows at interceptor sewer node (b) Overflow chamber. $H_{s,i}$ is the spill level; $h_{i,t}$ is the depth of water in the chamber at time t. (c) Layout of the interceptor sewer system with overflow chambers and storage tanks.



(a) Sub-catchment 1 (Rimrose)

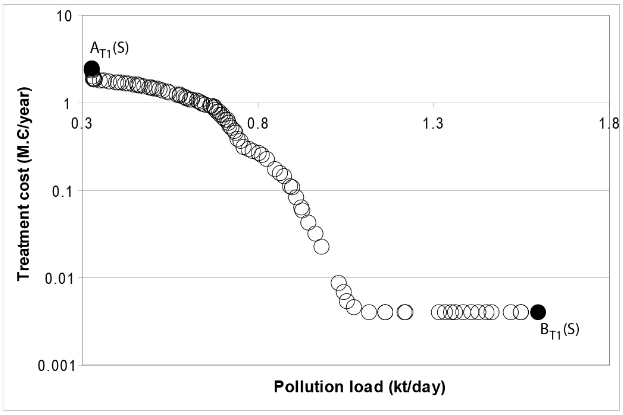


(b) Sub-catchment 2 (Strand Road)

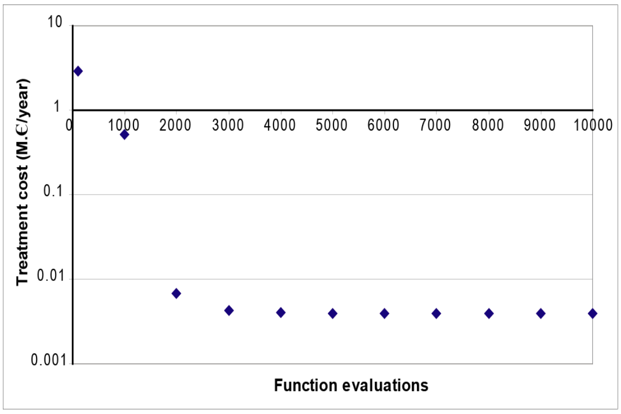


(c) Sub-catchment 3 (Millers Bridge)

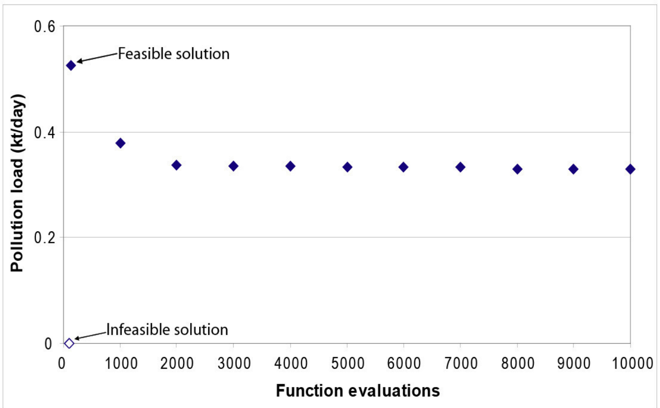
Fig. 2 Rainfall run-off hydrographs with suspended solids concentrations for three sub-catchments



(a) Pareto-optimal front. The vertical scale is logarithmic.

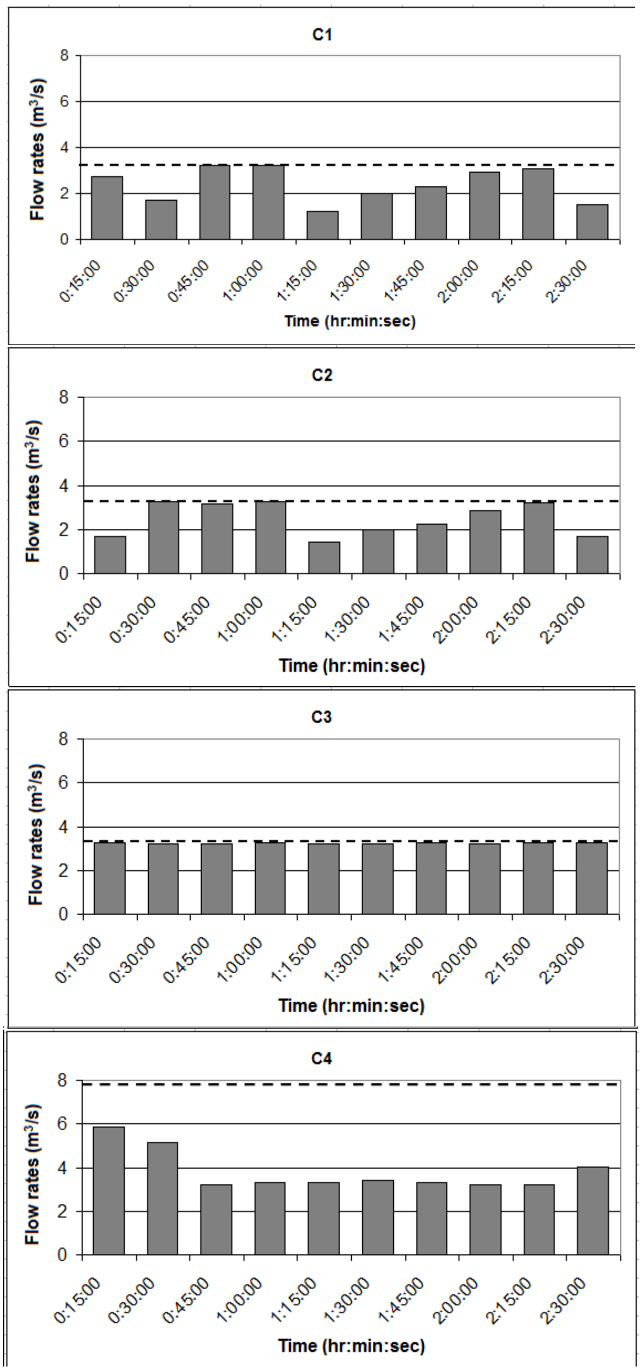


(b) Progress of the treatment cost. The vertical scale is logarithmic.



(c) Progress of the pollution load

Fig. 3 Convergence rate and Pareto-optimal solutions for the period 0-15 minutes


C1
C2
C3
C4
Flow rates (m³/s)
Time (hr:min:sec)
0
2
4
6
8
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00

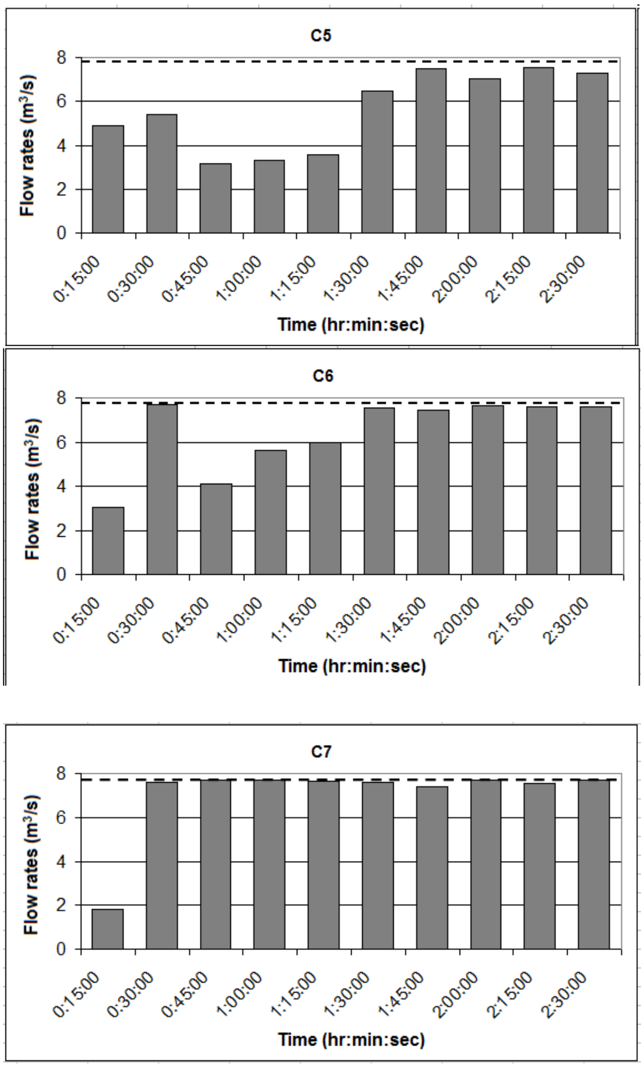
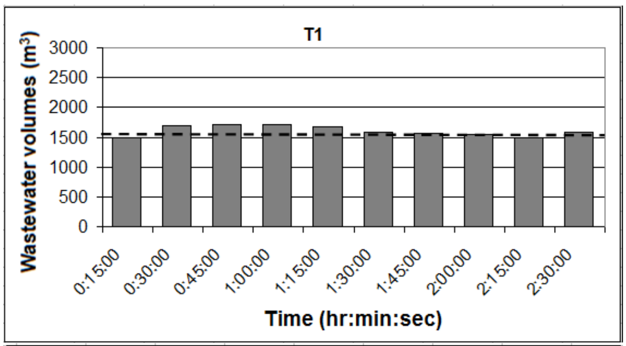
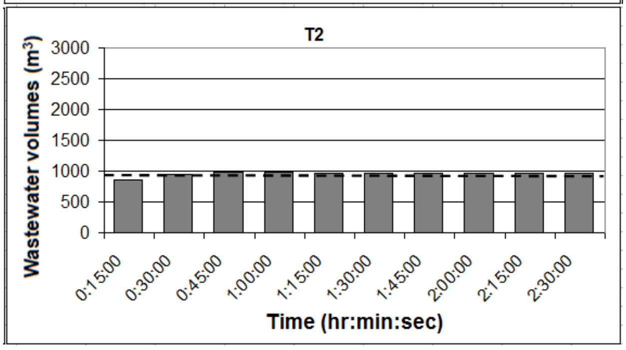
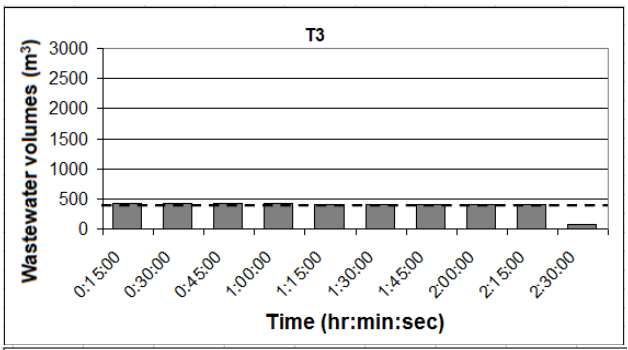
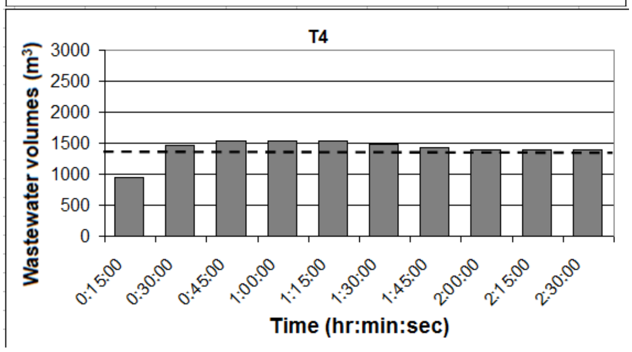


Fig. 4 Volumetric flow rates in interceptor conduits for minimum pollution load control option. The dashed horizontal lines represent the respective capacities.


T1
Wastewater volumes (m³)
3000
2500
2000
1500
1000
500
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)

T2
Wastewater volumes (m³)
3000
2500
2000
1500
1000
500
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)

T3
Wastewater volumes (m³)
3000
2500
2000
1500
1000
500
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)

T4
Wastewater volumes (m³)
3000
2500
2000
1500
1000
500
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)

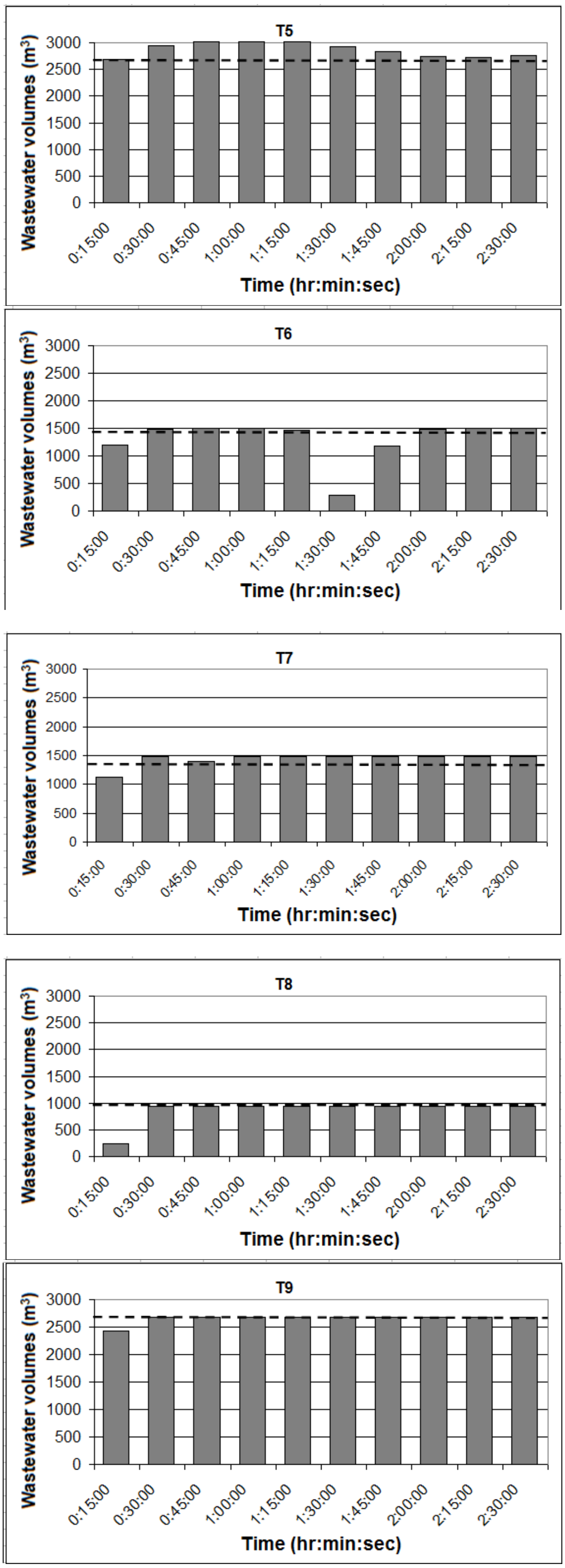
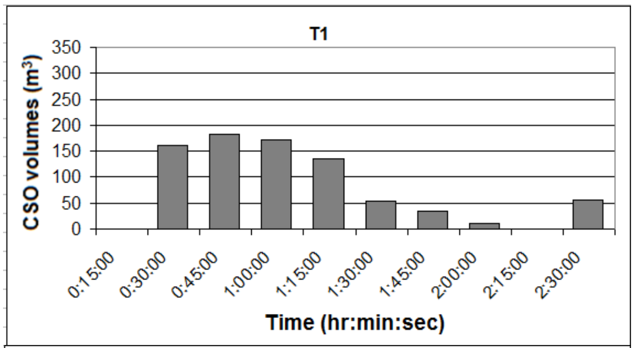
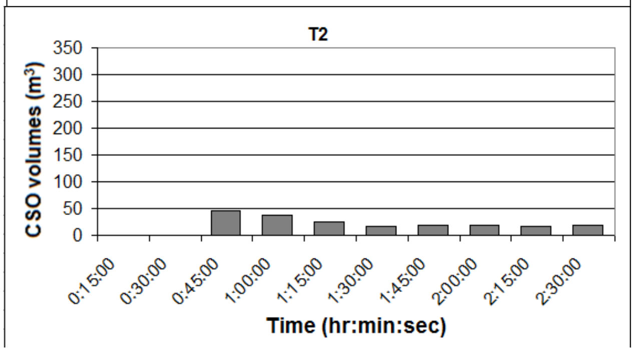
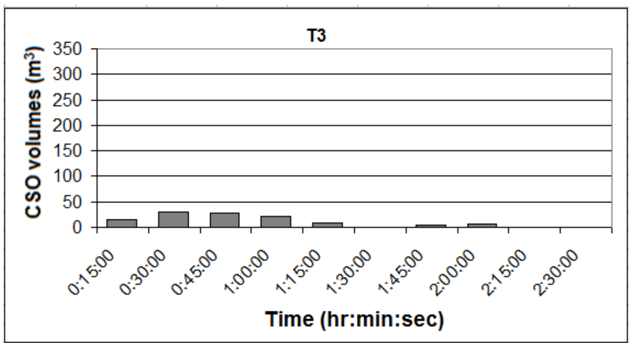
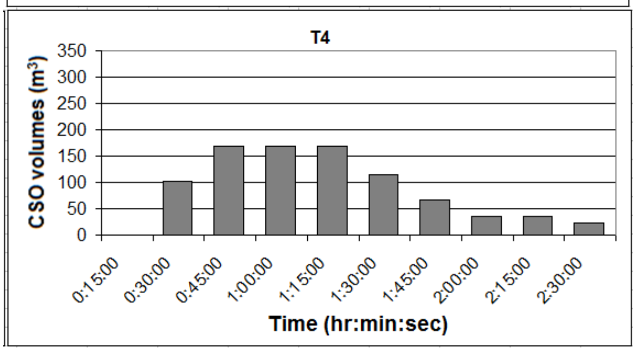


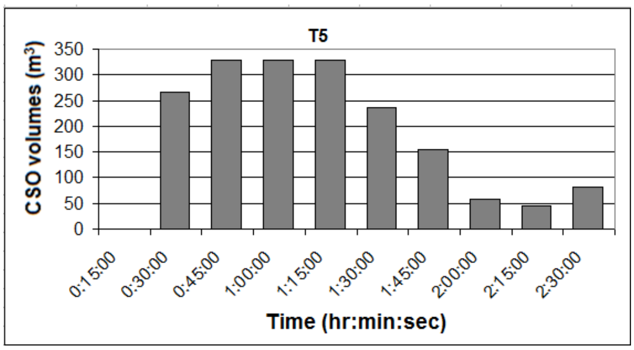
Fig. 5 Wastewater volumes in storage tanks and overflow chambers for minimum pollution control option. The excess volumes superimposed above the dashed horizontal lines that represent the respective capacities of the chambers are the overflows shown in Figure 6.


T1
CSO volumes (m³)
350
300
250
200
150
100
50
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)


T2
CSO volumes (m³)
350
300
250
200
150
100
50
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)


T3
CSO volumes (m³)
350
300
250
200
150
100
50
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)


T4
CSO volumes (m³)
350
300
250
200
150
100
50
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)


T5
CSO volumes (m³)
350
300
250
200
150
100
50
0
0:15:00
0:30:00
0:45:00
1:00:00
1:15:00
1:30:00
1:45:00
2:00:00
2:15:00
2:30:00
Time (hr:min:sec)

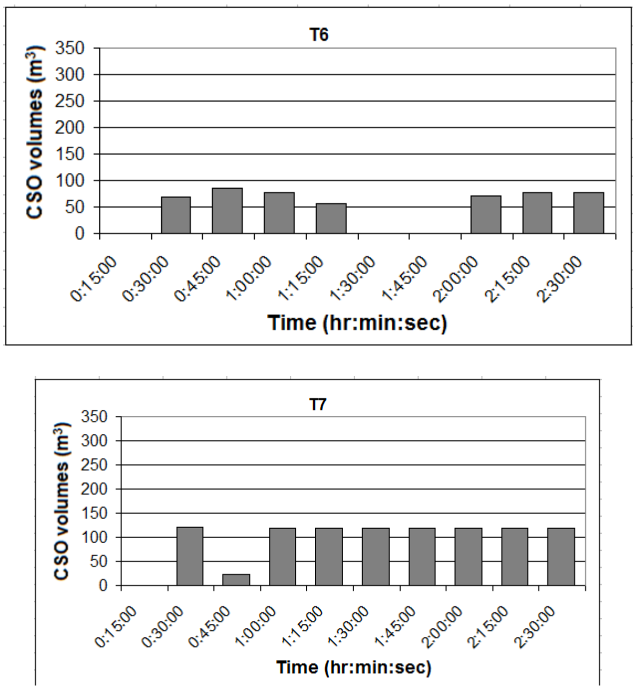
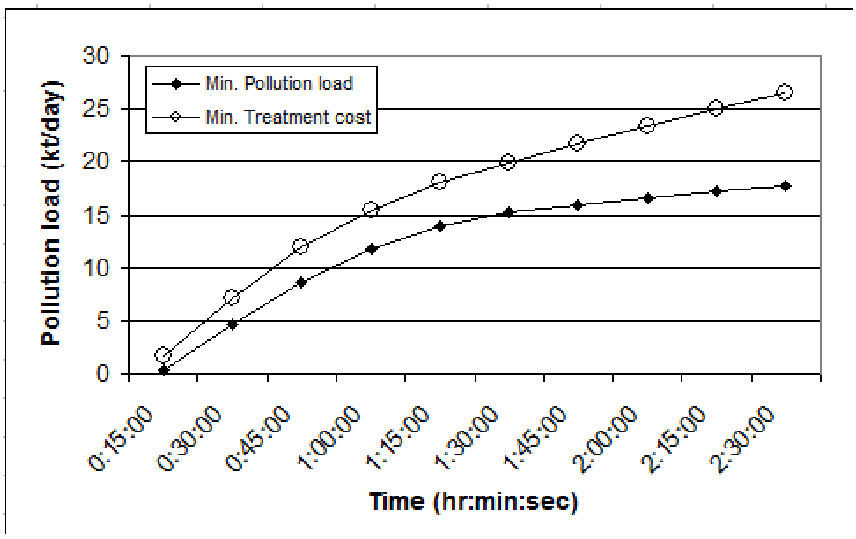
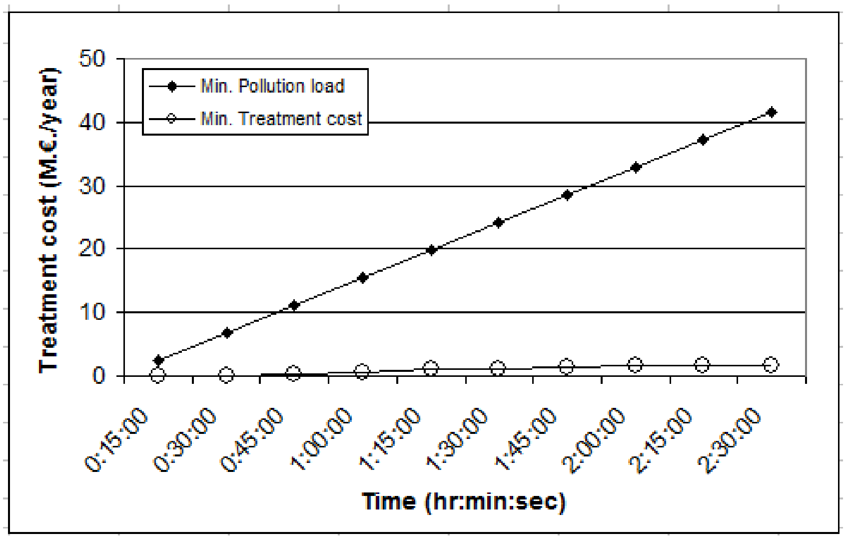


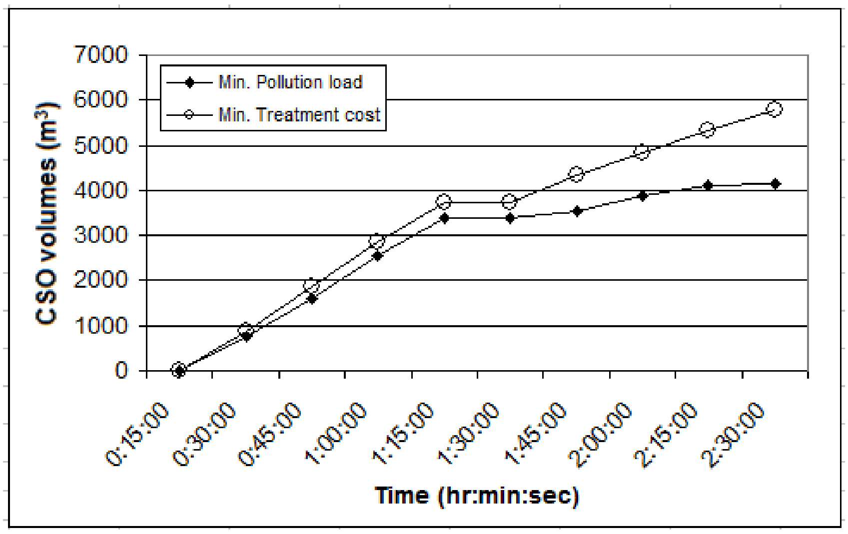
Fig. 6 Combined sewer overflows for minimum pollution load control option



(a) Cumulative pollution loads



(b) Cumulative treatment costs



(c) Cumulative combined sewer overflow volumes

Fig. 7 Cumulative pollution loads, wastewater treatment costs and overflow volumes