Optimal distribution of storage tank volume to mitigate the impact of new developments on receiving water quality

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Abstract

Storage tanks are commonly installed in a combined sewer system to control the discharge of combined sewer overflows, which has been identified as a leading source for receiving water pollution. The traditional approach to determine the distribution of storage tank volume in the sewer system is confined to the use of objectives within the system itself due to the limits of separate modelling of urban wastewater systems, consisting of the sewer system, wastewater treatment plant and receiving water. The aim of this paper is to investigate the optimal distribution of storage tank volume with an objective to mitigate the impact of new developments on receiving water quality. An urban wastewater model has been used to test three optimization scenarios: optimal control, storage distribution, and a combination of these two. In addition to the cost of storage tank construction, two receiving water quality indicators, dissolved oxygen and ammonium concentrations, are used as optimization objectives. Results show the benefits of direct evaluation of receiving water quality impact in the context of storage distribution optimization. It is suggested that storage allocation should be considered in

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conjunction with optimal control in order to achieve the maximum effectiveness in water pollution mitigation.

Key words: combined sewer overflow; integrated modelling; multi-objective optimization; new development; storage tank; urban wastewater system; water quality

Introduction

Combined sewer systems are widely used in urban areas to collect and convey both dry weather flow (DWF) and stormwater runoff through a single pipe system to wastewater treatment plants. One particular phenomenon in such systems is the occurrence of direct discharges of combined sewer overflow (CSO) to receiving waters when the flow exceeds the available system capacity. CSO discharges can contain pollutants such as organics, sediments, microbial pathogens, nutrients and toxics, and have been identified as a leading cause of or contributor to receiving water pollution and human health impacts (USEPA, 2004; Even et al., 2007).

Storage tanks are common measures used to limit CSO discharges in combined sewer systems, although many Best Management Practices (BMPs) are also available to reduce surface runoff entering the system. A storage tank provides a physical volume for temporary storage of combined wastewater and stormwater during a storm event, and the stored water can be released back into the urban wastewater system over an extended period. In this way, introducing storage achieves the following primary functions: attenuating flow, limiting localized flooding and reducing the value of polluted stormwater discharged into receiving waters (Butler and Davies, 2004). As far as pollution reduction is concerned, the benefits become more significant in an urban catchment where there exists a first flush phenomenon, in which a particularly high load of pollutants occurs in the initial phase of combined sewer flow when compared with the loadings at a later stage during wet weather events (Gupta and Saul, 1996; Deletic, 1998). The provision of additional storage is a well-established method for system upgrade in order to manage the increased flows generated by new urban developments (Lau, 2002), and may have a role to play to mitigate the potential impacts of future uncertainties, for example climate change, as demonstrated by Butler et al. (2007) for a case study in London.

The distribution of storage tanks in an urban catchment is dependent on the specific purpose for which the tanks are designed. As far as pollution control is concerned, the traditional approach for determination of the position and size of storage tanks is to use objectives based on the sewer system, for example, minimisation of CSO discharge volume/frequency or total pollution loadings (Chen and Adams, 2005). However, research has shown that reduction of CSO discharges does not necessarily result in improvement in receiving water quality (Rauch and Harremoës, 1998; Rauch and Harremoës, 1999; Lau et al., 2002). One possible explanation is that reduction of CSO discharges in the sewer system leads to a higher hydraulic load to the treatment plant and a related prolonged flow period, which produces a negative effect on the effluent quality and ultimately a negative impact on receiving water quality (Lau, 2002). The above problem needs to be addressed in an integrated modelling framework by considering the interactions between the sewer system, treatment plant and receiving water. This has been made possible due to the developments in modelling approaches and computer power in recent years (Rauch et al., 2002; Butler and Schütze, 2005).

Determining the size and location of storage tanks can be formulated as an optimization problem, and various optimization methods have been used in the literature to derive the optimal solutions, for example, dynamic programming (Mays and Bedient, 1982) and controlled random search (Lau, 2002). In many situations, however, storage provision has to consider multiple objectives, for example, cost, pollution control and flood attenuation, due to the different needs of stakeholders. For multi-objective problems, in general, there is no single optimal solution, but rather a set of non-dominated solutions, i.e., the so-called Pareto-optimal solutions, which form the Pareto-optimal front in the objective space. Each of these solutions is optimal in the sense that improvement in one objective leads to deterioration in at least one of the other objectives. Genetic algorithms have proven to be a very powerful technique for many optimization problems in the urban wastewater system (Rauch and Harremoës, 1999), and many potential applications have been demonstrated for multiobjective optimization problems (Muschalla et al., 2006). NSGA II, proposed by Deb et al. (2002), is a state-of-the-art multiobjective genetic algorithm due to its rapid convergence characteristics and generation of well-spread Pareto optimal solutions, thus it has been chosen for this study (Fu et al., 2008).

The objective of the work reported is to investigate the optimal distribution of storage tank volume in an urban catchment to mitigate the impact of new developments on receiving water quality. An existing integrated urban wastewater model was used to simulate system dynamics in the sewer system, treatment plant, and receiving river and the interactions between them. Instead of conventional volume or pollution based design criteria in the sewer system, this study is based on achieving optimal receiving water quality criteria, i.e., dissolved oxygen and ammonium concentrations. Three optimization scenarios are defined to compare the effectiveness of storage distribution with (and without) optimal control to mitigate water quality pollution, and they are demonstrated in the context of an integrated urban catchment.

Materials and Methods

The Existing System

The catchment consists of a combined sewer system, a treatment plant and a river system, as shown in Fig. 1. It was first defined by Schütze (1998) and has been studied in detail for real time control problems (Lau et al., 2002; Butler & Schütze, 2005; Fu et al., 2008).

The sewer system is divided into seven sub-catchments, in total with an area of 725.8 ha and a population of 152 000. Four on-line pass through storage tanks are located downstream of sub-catchments SC2, SC4, SC6 and SC7, respectively. Such arrangements allow a proportion of the particulate pollutants to settle while the flow passes through the tank, and thus result in a reduction in the pollutant concentrations of overflows during rain events. The storage is assumed to be uniformly distributed across the sub-catchments, with an average volume of 18 m^3/ha.

The wastewater treatment plant has the capacity to treat an average dry weather flow of 27 500 m^3/d. It consists of a storm tank, primary clarifier, and activated sludge reactor and secondary clarifier. The tank, with a volume of 6 750 m^3, provides additional storage to the sewer system and increases the average storage of the urban catchment to 27 m^3/ha.

The river system is 45 km in length and is equally divided into 45 reaches for simulation. The CSO discharges are assumed to be at reach 7, and storm tank overflows and treatment plant effluent at reach 10. In order to simulate the impact of treatment plant effluent and CSO discharges on the river water quality, the river base flow was set to be low at 129 600 m^3/d. This results in a 1:5 dilution ratio of dry weather treatment plant discharges to river base flow. The boundary conditions for concentrations are defined as the ‘dry’ scenario used by Schütze (1998), as follows: ammonium: 0.09 mg/l; dissolved oxygen (DO): 9.0 mg/l; slowly biodegradable BOD: 1.80 mg/l; and readily biodegradable BOD is set to zero, assuming that the organic material has degraded upstream of the treatment plant effluent and CSO discharges.

The integrated urban wastewater system can be controlled to improve system performance. Two scenarios, as shown in Table 1, have been previously considered for control of the existing system:

Scenario EI (Pre-development Base Case Control): no optimization is applied and default values are used for control variables.

Scenario EII (Pre-development Optimal Control): the control variables are chosen on the basis of the study by Schütze (1998), and their ranges are given in Table 1.

The New Development

A new sub-catchment SC8 is located upstream of SC5 with a planned development for 20 000 residents (about 13% of the existing population) and an area of 72.41 ha. This will produce an estimated DWF of 3 000 m^3/d.

Separate sewer systems are commonly designed for new developments in the UK, however it is not uncommon for these to be connected ultimately to an existing combined sewer system (POST, 2007). With the aim to demonstrate the proposed method, it is assumed that both stormwater and DWF from SC8 are diverted to the sewer system at SC5. The new development will thus have an impact on the existing system in two respects: increased DWF and stormwater flow (total volume and peak flow). This will lead to an increased load on the treatment plant and frequent CSO discharges during rain events, and will ultimately have an impact on receiving water quality.

Simulation Model

This urban catchment is simulated using an integrated model developed through the commercial tool SIMBA 5.0 (IFAK, 2005). The three sub-systems are constructed in a single simulation environment, thus consideration of dynamic interactions between these sub-systems has been made easier. In the integrated model, the sewer system is simulated by the KOSIM approach (ITWH, 1995), the treatment plant by Activated Sludge Model No.1 (Henze et. al. 1986), and the river system by EPA storm water management model (Huber and Dickinson, 1988). This model was validated using a previously developed model SYNOPSIS (Butler & Schütze, 2005), and a good agreement between them was obtained in terms of goodness-of-fit measures for CSO discharges, treatment plant effluent, river flow and quality. For a detailed model description the reader is referred to Fu et al. (2008, 2009).

A single 7-day rainfall series from historical data is used for simulation, and it has a total depth of 27.1 mm and an average intensity of 9.3 mm/h. This rainfall event leads to some CSO and storm tank usage while the treatment plant capacity is reached in the existing catchment. This rainfall series has been used in earlier studies on the catchment and it is used in this study by way of comparison. Constant event mean concentrations are assumed for various pollutants simulated during this rainfall series, and the first flush phenomenon is not considered in this case study.

Optimization Problem Formulation

Indicators

It has been demonstrated that minimisation of CSO discharge volume/frequency or pollutant load does not necessarily lead to improved water quality in receiving waters (Lau et al., 2002). So, in this study, receiving water quality itself is used as the criteria to allocate the storage tank volume in the sewer system and wastewater treatment plant.

Two indicators are chosen to represent the receiving water quality: minimum DO concentration and maximum ammonium concentration over the entire simulation period and all river reaches, denoted by DO-M and AMM-M, respectively. These two indicators are of importance in order to maintain overall river 'health', and are widely used parameters in the control of wastewater discharges to protect aquatic life. For example, and for a specific return period, a 4mg/l threshold is required for both DO and ammonium concentrations in order to protect aquatic life according to the intermittent standards given in the Urban Pollution Management Manual (FWR, 1998). In the case study, the above threshold for DO and ammonium concentrations are well maintained in the river base flow; however, they are normally exceeded when CSO discharges occur during storm events in the base case control scenario.

Apart from water quality impact, cost is an important factor affecting the decision of system upgrade. The provision of storage involves construction and operating costs. Construction cost can be estimated by use of cost functions in the form of power laws or polynomials, considering the size of volume and area. The detailed formation of a cost function may vary significantly in different times, regions or countries. To demonstrate the multi-objective method, a cost function adapted from Gillot et al. (1999) is used here

$$C = \sum_i 5559 V_i^{0.473}$$

where C is the total cost of system upgrade (million US dollars), and V_i is the provided volume of ith storage tank (m^3). Operating costs are neglected as being similar in all the options considered.

Impact of new development

The impact of new development on receiving water quality was investigated using the two previously developed control scenarios EI and EII given in Table 1. These two scenarios were first applied to the existing system, and the solutions were derived in terms of DO-M and AMM-M. All these solutions were then implemented in the new system with the development of sub-catchment SC8, and their new objective values were calculated and shown in Fig. 2. For Scenario EI, the minimum DO concentration has deteriorated from 3.5 mg/l to 2.4 mg/l, and the maximum ammonium concentration from 3.7 mg/l to 4.1 mg/l. The optimal solutions originally derived from Scenario EII also result in a poorer water quality state, and are mainly converged to two regions: one can achieve a better DO concentration but with a poorer ammonium concentration in comparison to the base case control of the new system, while the other region has a much poorer water quality situation. Deterioration of water quality in both scenarios demonstrates a real need to investigate the ways to maintain or improve water quality.

Optimization Scenarios

Three optimization scenarios are defined in order to mitigate the impact of the new development on receiving water quality. Specifically, the scenarios are described below:

Scenario NI (Optimal Control). The objective of this scenario is to maximize DO-M and minimize AMM-M. This is accomplished by optimizing the control of the critical flow rates in the system, i.e., the variables 7-12 defined in Table 1. No additional storage is provided although the existing storage tanks (Tanks 2, 4, 6 and 7) in the sewer system and the storm tank in the treatment plant are still used. The control variables in this scenario are exactly the same as those of Scenario EII, however, the sewer system is expanded to include the new development SC8. Optimal control has been regarded as an effective way to improve the receiving water quality in an integrated urban wastewater system (Butler & Schütze, 2005; Vanrolleghem et al., 2005; Fu et al., 2008). Therefore this scenario is intended to illustrate the control potential in the existing urban wastewater system and reveal the extent of its effectiveness in mitigation of the impact of the new development.

Scenario NII (Storage Distribution). The objective of this scenario is to maximize DO-M, minimize AMM-M and minimize cost. This is accomplished by optimizing storage distribution (including volume and location) over the system but without optimal control of the existing system including storage tanks. In addition to increase of the volume of four storage tanks and one storm tank in the existing system, a new storage tank (Tank 8) is assumed to be located at the downstream of the new development. The maximum increase in the total storage volume is set to 27 000 m^3, and this will raise the average storage volume in the entire catchment to about 58 m^3/ha, a doubling of the current level. The control of the new tank is set to 5 times DWF, the same as the other storage tanks in the sewer system.

Scenario NIII (Storage and Control). Similarly to Scenario NII, the objective of this scenario is also to maximize DO-M, minimize AMM-M and minimize cost. However, this scenario is defined to concurrently optimize the storage distribution and control strategy. In addition to the general system characteristics, such as rainfall, catchment size, and population size, the storage position and size is also inevitably affected by how the tanks and other parts of the integrated system are controlled. It has been revealed that storage size is one of the most sensitive system properties that affect the control potential of an urban wastewater system in terms of receiving water quality (Zacharof et al., 2004). Conversely, the required storage volume is expected to be significantly affected by system control in order to achieve the best overall system performance. This scenario is used to show the combined effects of scenarios NI and NII.

Results and Discussion

In the optimization process, the following parameter values were used for NSGA II: generation number = 100, population size = 200, crossover probability = 0.9, and mutation probability = 0.1. When generating each solution in the initial population and subsequent new solutions in NSGA II, an increment of 50 m^3 was assumed for tank volumes (variables 1-6), and a minimum step of 0.01 for the other variables. These assumptions make the derived solutions sensible from a cost and logistics perspective in practice. The optimization results for the three scenarios are explained below.

Receiving Water Quality

The Pareto-optimal solutions from Scenarios NII and NIII are shown in Figs. 3(a) and (b), respectively, in terms of DO-M and AMM-M. Many solutions shown are not Pareto-optimal in terms of these two water quality indicators, but these are kept in the optimization results because of the cost objective.

A trade-off relationship between the two water quality objectives can be observed for both scenarios, and it implies that it is impossible for a solution to improve both DO and ammonium concentrations concurrently. This is probably because the nitrification process deteriorates under higher flows to the treatment plant, so the AMM-M indicator worsens while DO-M improves because of fewer CSO discharges to the river, and vice versa.

There are no solutions from Scenario NII to satisfy the 4 mg/l threshold of both DO and ammonium concentrations (Fig. 3a). However, a few solutions have been found under Scenario NIII (Fig. 3b), and these solutions are further analysed below.

Cost

The costs of the solutions from Scenario NIII are shown in a pair-wise comparison with either of the two water quality indicators in Fig. 4. Similarly to Fig. 3, the Pareto optimal fronts for the two pairs of objectives were derived from all the solutions. The Pareto optimal solutions in the Cost and DO-M space as indicated by crosses have relatively high AMM-M values in Fig. 4(b), and the Pareto optimal solutions in the Cost and AMM-M space as indicated by squares have relatively low DO-M values in Fig. 4(a).

Optimal solutions in the DO-M and AMM-M space as indicated by circles have a relatively high cost with a minimum cost of 1.85 million US dollars as shown in Figs. 4(a) and (b). Conversely, the solutions with a low cost (for example, less than 1.0 million US dollars), have low DO-M concentrations and moderately high AMM-M concentrations. None of these solutions can satisfy the 4 mg/l threshold. No solutions were found that achieve simultaneously good water quality (in terms of DO-M and AMM-M) and low cost. Thus, there exists a trade-off between cost and receiving water quality.

Solutions within the water quality threshold of 4 mg/l have a higher cost compared with others (around 2.8 million US dollars). The cost can be regarded as the minimum in order to achieve the water quality thresholds. With the same cost, the water quality achieved in Scenario NII exceeds the thresholds, with a DO-M of 3.6 mg/l and AMM-M of 4.35 mg/l.

Noticeable from Figs. 3 and 4 is that the number of Pareto optimal solutions can be increased to a great extent when one more objective is considered. In the practical decision making process, the number of objectives should be carefully considered and kept to the minimum in order to restrict the feasible solutions.

Comparison of the Three Scenarios

Best attainment surfaces can be used to visualise the biggest objective space that is achieved over multiple runs. The best surfaces for each of the three scenarios, as shown in Fig 5, were derived from 5 runs in terms of the water quality indicators.

Comparing the Pareto fronts of Scenarios NI and NII, the Pareto front of Scenario NII is better than that of scenario NI. This means that the storage increase in this case study has a greater potential to improve receiving water quality compared with optimal control of the existing system. The control potential of the existing wastewater system is limited in this specific case, although its performance can be improved to some extent compared with the base case control. Scenario NII can only explore a limited objective space, with a much shorter trade off curve when compared with that of Scenario NI. The extension of the objective space is related to the range in which storage volume is allowed to increase. In this case, the maximum storage expansion for the total 6 tanks is limited to 27 000 m^3, and if the maximum volume is increased, the objective space explored would expect to increase accordingly.

The Pareto front of Scenario NIII is much improved over that of scenario NII. This scenario results in higher water quality in terms of the two objectives and in fact can achieve higher water quality than the base case control of the existing catchment prior to development, and almost the same level of optimal control of the existing catchment. The objective space achieved in this scenario is also greater than those from the other scenarios, in particular, it further explores the two ends of the Pareto optimal curve where one objective achieves the best and the other reaches the worst. This scenario reveals the combined effect of scenarios NI and NII, i.e., simultaneous optimization of control strategy and storage distribution, which has significantly improved its potential in mitigating the impact of new developments on receiving water quality.

Tank Volume Distribution

The tank volume distribution is illustrated for all the Pareto solutions from Scenario NIII in Fig. 6, in which normalized values are used in order to compare the various valuables. The optimal volumes of the tanks connected to SC2 and SC8 are mainly centred on either the upper or lower bounds of the considered ranges, and the optimal solutions for other tanks are distributed across the whole ranges. This means that the allocated volume for a specific tank is not important, and the optimal solutions are more dependent on the overall combined effect with other tank volumes. In addition to the non-linear characteristics of the urban wastewater system, another reason is probably due to the large objective space covered by the Pareto solutions.

The solutions with a high volume of Tank 8 in the upper bound have moderate DO-M and AMM-M concentrations, and lie in the eclipse area as shown in Fig. 3b. Further, the relatively few solutions within the threshold limits follow a very similar trajectory for the tank volume increases. In these solutions, the new tank has a high volume, thus it can effectively control the outflow from the new development and improve the receiving water quality. This shows the flexibility in tank volume distribution is reduced to a great extent when a threshold is imposed to the water quality indicators.

The trajectories of the other variables (variable number 7 to 12) of the optimal solutions are also shown in Fig. 6. For those solutions satisfying the 4 mg/l water quality threshold as shown in Fig. 3(b), a very similar trajectory of variable values is observed as indicated by the bold solid lines. Table 2 shows the variable values for these optimal solutions and it indicates a very small variation for each variable in those optimal solutions, for example, the volume of Tank 2 varies only from 3850 to 3900 m^3 and the volume of Tank 4 from 1700 to 2050 m^3. The narrow trajectory bounds formed by these solutions represent the optimal parameter space in which the water quality indicators can be restricted in the thresholds. Thus these solutions might be preferred by the decision maker who is more concerned about the receiving water quality than the cost of system upgrade.

Uncertainty Analysis

In a decision making process, apart from understanding the trade-offs between objectives, it is also vital to understand the performance of control strategies under uncertainties. Uncertainties may arise from inputs, model structures and model parameters, including a large set of water quality parameters. Practically, significant uncertainties also surround in the future development of the urban catchment. In this paper, a simple uncertainty analysis was conducted to understand the performance of optimal solutions with different population sizes and longer rainfall series, while model structure and parameter uncertainties are assumed to be similar in each solution allowing us to compare one with another. For real-world applications, however, it might be necessary to conduct a holistic uncertainty analysis considering all uncertainty sources to provide the full implication of each solution and thus assist informed decision making for urban water managers.

Population size is the most important planning parameter for new developments, and it has the greatest impact on receiving water quality compared with other planning parameters such as housing density and occupancy (Fu et al., 2009). The population variation was considered within the range of ±20% of the planned size, i.e., [16 000 24 000]. The cost objective is only related to the tank volumes, so it will not vary according to the change in population size and will not be considered. As the water quality indicators are monotonic to the population size in this case study (Fu et al., 2009), it is only necessary to use the two extreme points of the population range to calculate the output bounds. The lower and upper bounds of water quality indicators are shown in Fig. 7 for the Pareto optimal solutions of Fig. 3b in the DO-M and AMM-M space. Obviously, the solutions in the two ends of the Pareto front have a greater variation than those in the middle. However, the solutions within the water quality threshold of 4 mg/l are not sensitive to population change. This provides more confidence for a decision maker who is willing to choose those solutions.

Instead of the typical 7-day rainfall series used for optimization, a different 5-year rainfall series was used to re-evaluate the original Pareto optimal solutions of Fig. 3b, and the results are shown in Fig. 7. It can be seen that the DO-M values become better for all solutions and the AMM-M values are roughly the same. Further, the shape of the original Pareto front is almost kept unchanged. This confirms that the 7-day rainfall series provides reasonably representative conditions of the catchment as those generated by much longer time series (Schütze, 1998). However, the solutions might get worse when severe storms are observed in the rainfall series. Due to the computational requirement of the integrated model, it is not practical to use long term, continuous simulation in the optimisation process, alternatively, event-based design storm might be used in practice, instead of a short rainfall series used in this paper.

Conclusions

This paper investigates the optimal distribution of storage in an urban wastewater system with an objective to mitigate the impact of new developments on receiving water quality. The conclusions are summarised as follows:

- The optimal control strategies developed from the existing urban catchment are no longer optimal when a new housing area is developed, and some even result in a much poorer water quality situation where both minimum DO and maximum ammonium concentrations in the receiving river deteriorate to a great extent.
- There exists a trade-off between tank construction cost and receiving water quality. The solutions with a low cost have either a very low DO concentration or a very high ammonium concentration. Similarly, the solutions satisfying the 4 mg/l water quality standard have the highest cost amongst the Pareto optimal solutions.
- Optimization results confirm a clear trade-off relationship exists between the two water quality indicators: minimum DO and maximum ammonium (Fu et al., 2008). This makes the decision making situation complicated with regard to balancing different objectives. When suitable water quality thresholds are imposed, however, the number of optimal solutions is reduced considerably.
- Storage distribution (volume and location) should be optimized together with control parameters of the urban wastewater system in order to maximize system performance in terms of receiving water quality. Considering them in a single optimization scenario, the derived optimal solutions not only achieve a much better water quality state, but also explore a wider objective space, offering a greater flexibility in balancing different objectives.
- Uncertainty analysis shows that the optimal solutions with the 4 mg/l water quality threshold are insensitive to a ±20% population variation in the new development, giving greater confidence in their use in practice. Further, the

optimal solutions show similar performance when the system is simulated using a much longer rainfall series than the original one used for optimization.

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Fig.1. Schematic representation of the integrated urban wastewater system. SCx represents the x*th* sub-catchment, and SC8 is the new development. The dashed lines show CSO discharges from the existing storage tanks.

Fig. 2. The water quality impacts of the new catchment with SC8.

Fig. 3. Performance characteristics of the optimal solutions: (a) Scenario NII and (b) Scenario NIII. The circles indicate the Pareto optimal front in the objective space of DO-M and AMM-M. The solutions in the eclipse area have a larger volume of Tank 8 in the upper bound as shown in Fig. 6.

Fig. 4. Performance characteristics of the optimal solutions from Scenario NIII: (a) DO-M and Cost and (b) AMM-M and Cost.

Fig. 5. Comparative results for the optimization scenarios in terms of the two water quality indicators DO-M and AMM-M.

Fig. 6. The variable trajectories for the Pareto optimal solutions from Scenario NIII. The bold solid lines show the solutions within the 4 mg/l water quality threshold.

Fig. 7. The performance of the Pareto optimal solutions in Fig. 3b with population variation and longer rainfall series.

Table 1. Default values of control variables and their ranges (in brackets) for different scenarios. A variable in a scenario is optimized if a range is given; otherwise the default value is used.

Variable No	*Variable description*	*Scenario EI*	*Scenario EII*	*Scenario NI*	*Scenario NII*	*Scenario NIII*
1	The volume of Tank2 at the end of SC2 (m^3)	2800	2800	2800	[2800, 4800]	[2800, 4800]
2	The volume of Tank4 at the end of SC4 (m^3)	1400	1400	1400	[1400, 3400]	[1400, 3400]
3	The volume of Tank6 at the end of SC6 (m^3)	2000	2000	2000	[2000, 4000]	[2000, 4000]
4	The volume of Tank7 at the end of SC7 (m^3)	7000	7000	7000	[7000, 14000]	[7000, 14000]
5	The volume of storm tank in the treatment plant (m^3)	6750	6750	6750	[6750, 13750]	[6750, 13750]
6	The volume of Tank8 at the end of the new development (m^3)	-	-	-	[0, 7000]	[0, 7000]
7	The maximum outflow rate of the storage tank linked to sub-catchment 7 (xDWF)	5	[2, 6]	[2, 6]	5	[2, 6]
8	The maximum inflow rate to treatment plant (xDWF)	3	[4, 6]	[4, 6]	3	[4, 6]
9	The maximum outflow rate of the new tank (xDWF)	-	-	-	5	[2, 6]
10	The threshold triggering emptying the storm tank (m^3/s)	0.28	[0.19, 0.36]	[0.19, 0.36]	0.28	[0.19, 0.36]
11	The emptying flow rate of storm tank (m^3/s)	0.14	[0.08, 0.28]	[0.08, 0.28]	0.14	[0.08, 0.28]
12	The return activated sludge rate (m^3/s)	0.17	[0.08, 0.28]	[0.08, 0.28]	0.17	[0.08, 0.28]

Table 2. Optimal solutions within the 4 mg/l water quality threshold as shown in Fig. 3(b).

Variable No	*units*	*Solution 1*	*Solution 2*	*Solution 3*	*Solution 4*	*Solution 5*	*Solution 6*	*Solution 7*
1	(m^3)	3850	3900	3900	3900	3850	3850	3850
2	(m^3)	2050	1700	1850	1900	2000	1950	2000
3	(m^3)	3900	3500	3600	3600	3850	3850	3850
4	(m^3)	13950	13550	13500	13700	13800	13950	13950
5	(m^3)	11550	11100	10900	11000	11550	11400	11550
6	(m^3)	7000	6200	6900	5950	7000	7000	7000
7	(xDWF)	4.48	4.55	4.56	4.56	4.46	4.47	4.47
8	(xDWF)	3.47	3.53	3.56	3.56	3.53	3.51	3.49
9	(xDWF)	3.31	3.32	3.41	3.36	3.30	3.22	3.18
10	(m^3/s)	0.33	0.33	0.34	0.34	0.34	0.34	0.34
11	(m^3/s)	0.20	0.22	0.21	0.21	0.20	0.20	0.20
12	(m^3/s)	0.24	0.23	0.24	0.23	0.24	0.24	0.24

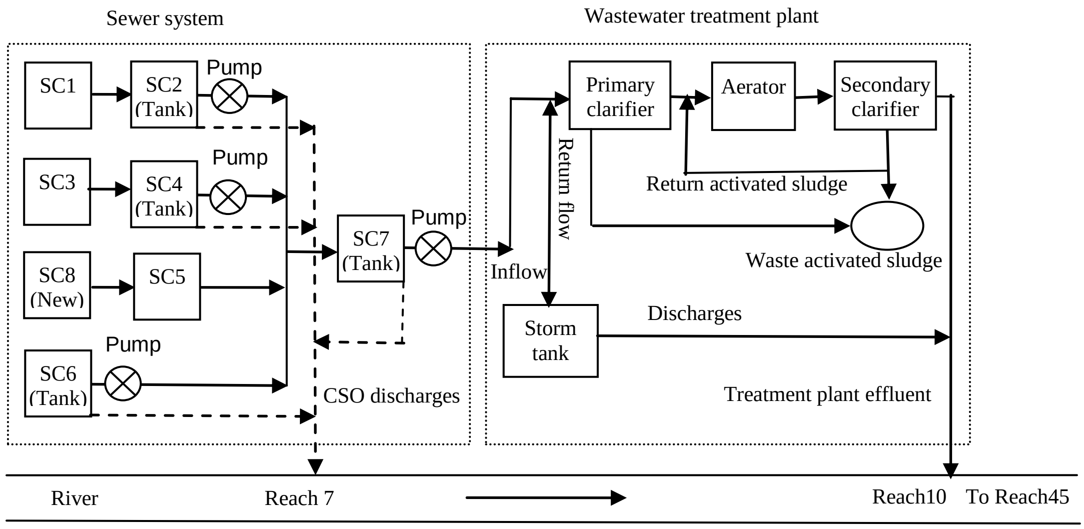


Fig.1. Schematic representation of the integrated urban wastewater system. SCx represents the x*th* sub-catchment, and SC8 is the new development. The dashed lines show CSO discharges from the existing storage tanks.

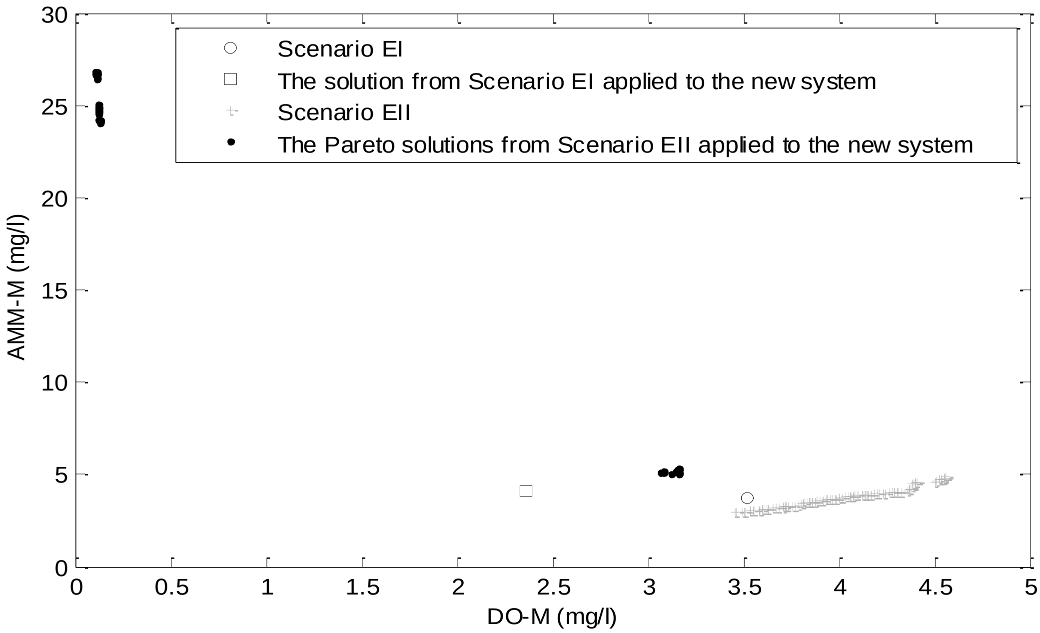
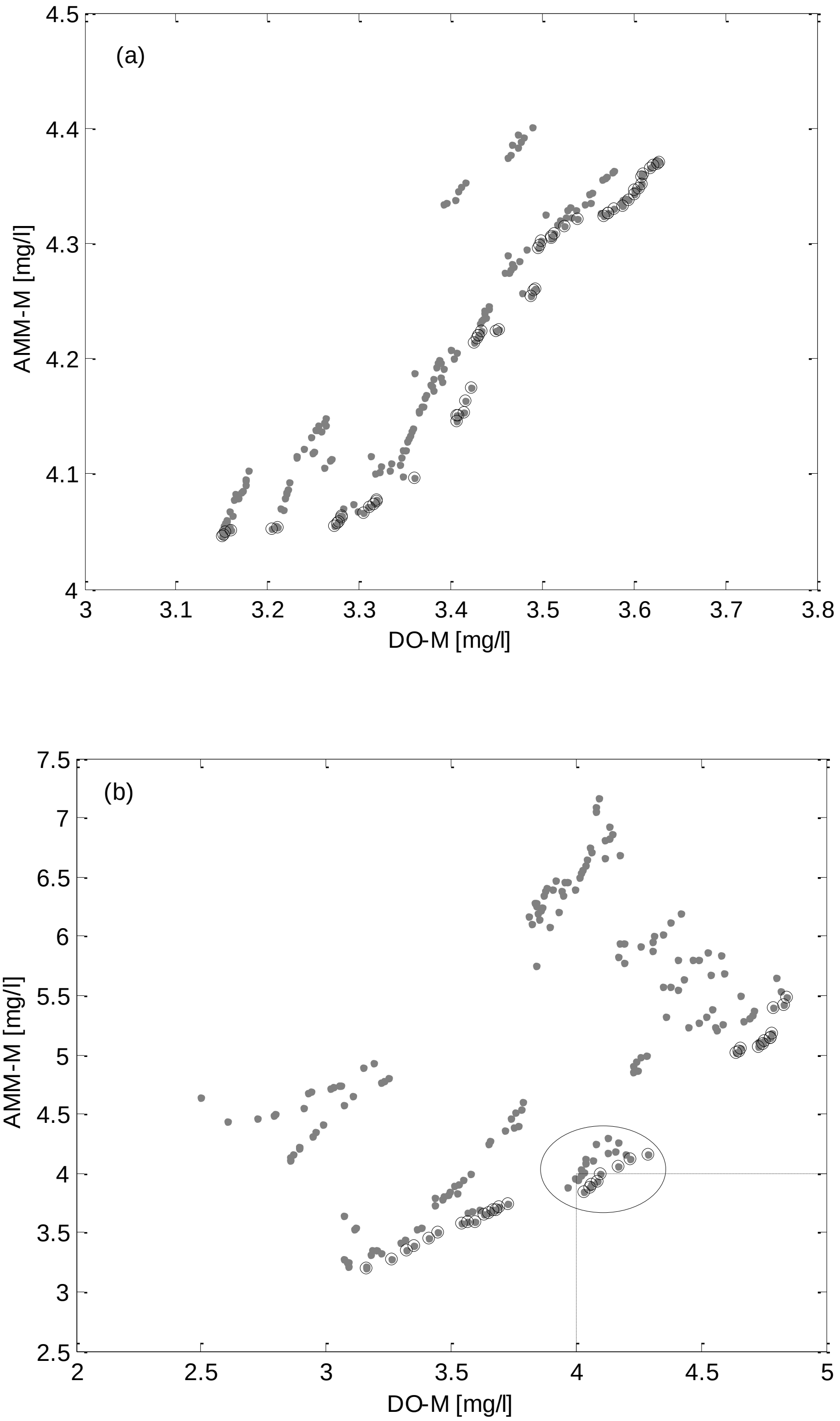


Fig. 2. The water quality impacts of the new catchment with SC8.



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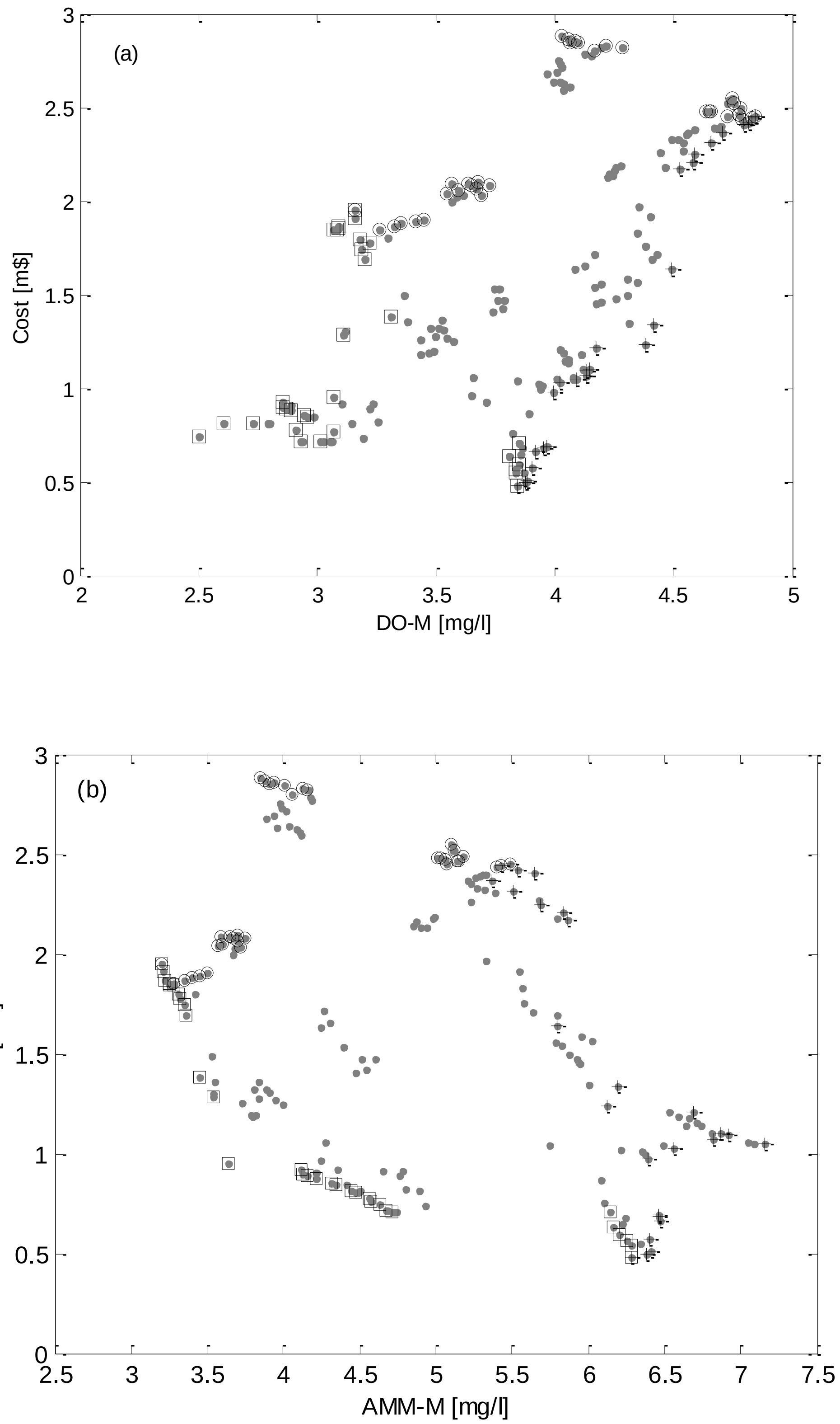

Fig. 4. Performance characteristics of the optimal solutions from Scenario NIII: (a) DO-M and Cost and (b) AMM-M and Cost.

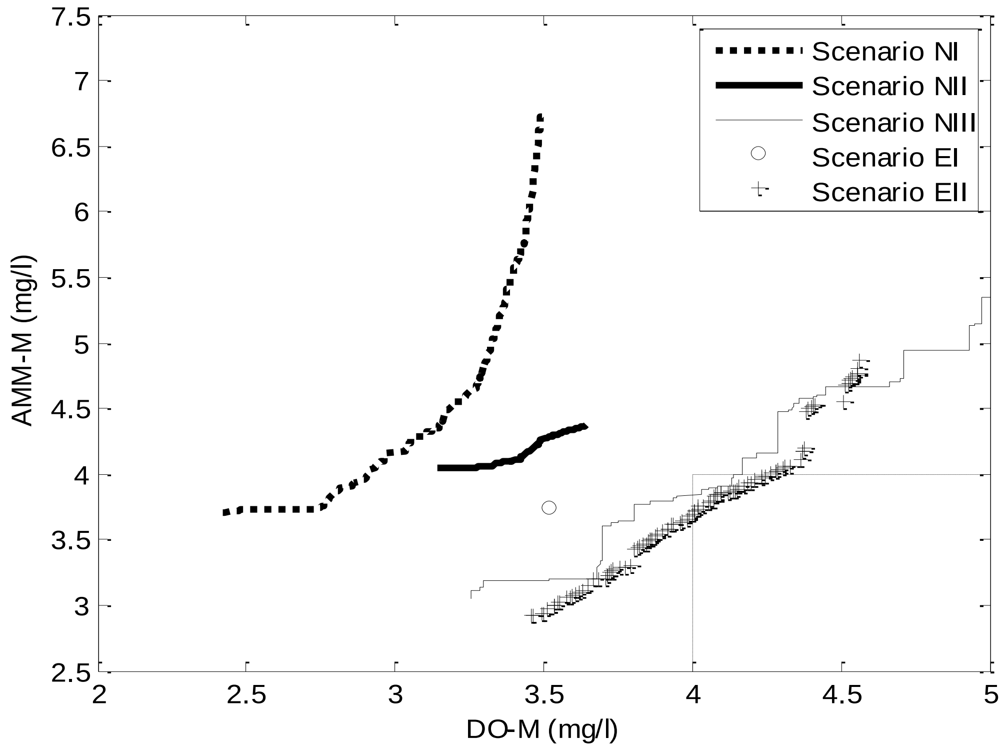

Fig. 5. Comparative results for the optimization scenarios in terms of the two water quality indicators DO-M and AMM-M.

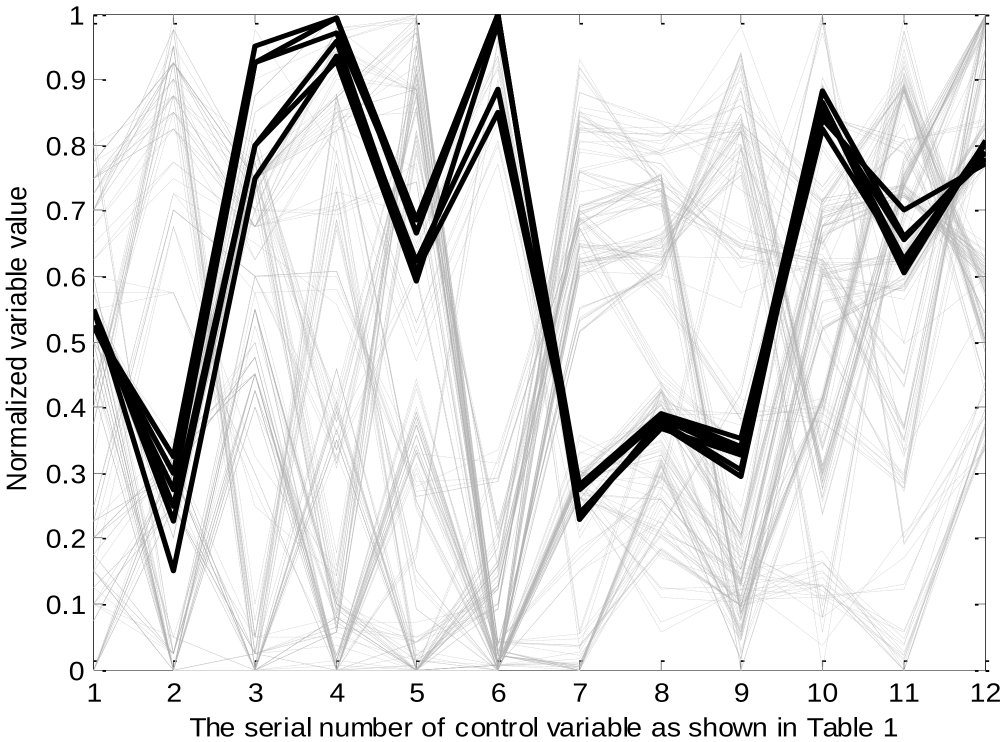

Fig. 6. The variable trajectories for the Pareto optimal solutions from Scenario NIII. The bold solid lines show the solutions within the 4 mg/l water quality threshold.

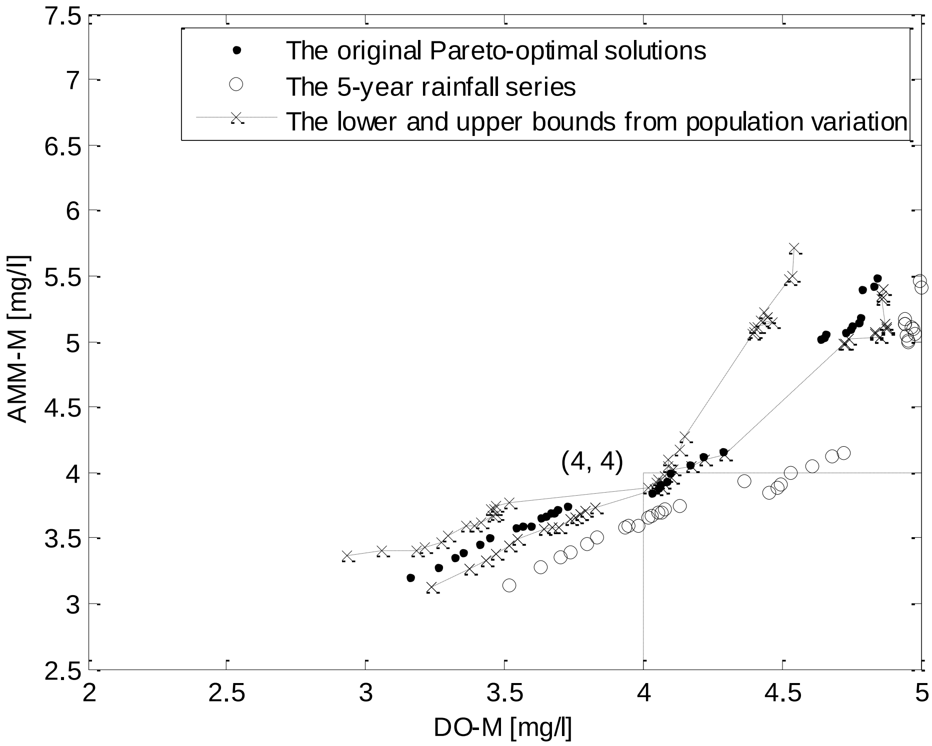

Fig. 7. The performance of the Pareto optimal solutions in Fig. 3b with population variation and longer rainfall series.