Improving the performance of an integrated urban wastewater system under future climate change and urbanisation scenarios
Maryam Astaraie-Imani, Zoran Kapelan and David Butler

ABSTRACT
This paper presents the results of a study to investigate the performance of an integrated urban wastewater system (IUWS) consisting of a sewer system, biological wastewater treatment plant and receiving water under future climate change and urbanisation. In particular, the roles of both system operational control and (re)design were studied as means to ensure future compliance of the system with recipient water quality standards. This is carried out by applying a scenario-based approach to describe the possible features of future climate change and/or urbanisation. Dissolved oxygen and ammonium concentrations are considered as the objectives of the optimisation models. Critical flow rates in the IUWS are considered as decision variables of the operational control model and sewer storage tank volume is considered as the design model decision variable in this study.

It is demonstrated that operational control optimisation alone, under all the scenarios developed, does not have enough potential to cope with these future conditions and only when combined with design optimisation can adequate performance be guaranteed.

Key words | climate change, design, integrated modelling, operational control, optimisation, urbanisation, urban wastewater system

INTRODUCTION
It is becoming increasingly clear that future changes, particularly those associated with a changing climate and growth in urban areas will have a negative impact on urban wastewater system performance (Butler & Davies 2011). To maintain performance, such systems will need to be adapted to cope with these changes and several options are possible. Broadly, these fall within the categories of either improving the operational control of the existing system or redesigning/rehabilitating the system, or indeed both. However, the extent to which either or both options are able to contribute to maintaining or even enhancing performance is not known – hence the motivation for this study.

A review of relevant literature is presented initially. The method used for integrated modelling of urban wastewater systems is then presented after that. Then the methods and parameters used for modelling future climate change and urbanisation in this study are discussed. Afterwards, the urban wastewater systems optimisation models developed are introduced. The case study is discussed along with the results of the IUWS optimisation models developed. Finally, a summary of the work done and conclusions drawn is provided.

BACKGROUND
The feasibility of improving the performance of IUWS using operational control has been studied over several years. The application of multi-objective optimisation methods was introduced by Schütze et al. (2002) and has been successfully used by Fu et al. (2008) to demonstrate how conflicting recipient water-quality criteria can still be met. Muschalla (2008)
developed a new multi-objective evolution strategy in combination with an integrated pollution-load and water-quality model to optimise the performance of an urban wastewater system. Tudor & Lavric (2011) developed a dual-objective optimisation of an integrated water/wastewater network (IWWN) aiming to minimise fresh water consumption and reduce operating costs. However, in all these studies, system performance was improved by optimising operational control under existing (arguably design) conditions, without the complication of considering the implications of future climate and urbanisation changes.

Chen et al. (2004), Butler & Davies (2011) and Even et al. (2007) noted that combined sewer overflows (CSOs) reduce the quality of water in rivers. Storage tanks are common structural measures used to limit CSO discharges in combined sewer systems, although several other management options are available to reduce the input wastewater, including reducing surface runoff and dry weather flow (DWF) to the system. A well established method is to consider additional storage for upgrading the system to cope with the excess wastewater generated from, for example, future urban development (Lau et al. 2002). These storage enhancements are important in reducing the negative impacts of future changes such as those expected relating to climate, as Butler et al. (2007) have shown for a case study in London.

Storage tanks provide a volume for the temporary storage of combined wastewater and storm water during a rainfall event, and the stored water can be released back into the urban wastewater system gradually. Butler & Davies (2011) introduced the primary functions of the storage system in attenuating flow, limiting localised flooding and reducing the volume of polluted storm water discharges into the river. Lau et al. (2002) showed that the minimisation of CSO discharge volume/frequency does not necessarily lead to improved water quality in the receiving waters. In the above studies, system performance is improved solely by optimising the design of the system, i.e. without considering operational control optimisation at the same time. This issue was addressed by Fu et al. (2010) by investigating the optimal distribution and operational control of storage tanks but only by considering the specific aspect of urbanisation (new residential development). The work presented here builds on that approach and develops it further by considering climate change effects in addition to the urbanisation changes. The aim then is to investigate the performance of an IUWS under both future climate change and urbanisation and establish the role of both system operational control and (re)design as in ensuring future compliance of the system with recipient water-quality standards.

**CASE STUDY**

**Description**

The IUWS used here is a semi-real system (see Figure 1). This case study has been used in the past for various purposes, including real-time control (Schütze et al. 2002; Butler & Schütze 2005), system real-time control (RTC) potential analysis (Zacharof et al. 2004) and system impact analysis (Lau et al. 2002). The IUWS is divided into the sewer subsystem, the wastewater treatment plant (WWTP) and the river.

The sewer system analysed here is an example sewer system used by ATV (1992). It has seven sub-catchments with a total area of 725.8 ha. The average DWF is approximately 27,500 m$^3$/d. A storage tank is located at the WWTP inlet to control the CSOs by storing the wastewater in excess of the WWTP pumping capacity. The storage tank release capacity is controlled by means of thresholds on maximum outflow rates. Lessard (1989) provided a dataset of the concentration of different water quality indicators for dry and storm water flows which are applied for the sewer system in this study.

A nitrifying activated sludge plant in Norwich (UK) is used here as the basis of the treatment plant simulations. The system has a treatment capacity of 27,500 m$^3$/d, which is equal to DWF. It consists of a storm tank, primary clarifiers, activated sludge reactor and secondary clarifiers. The storm tank is controlled by the maximum inflow rate to the primary clarifiers. The tank is emptied at a certain pump rate, as soon as the inflow rate to the plant drops below a pre-specified threshold value. The WWTP model has been previously calibrated and validated (Lessard & Beck 1995).

The river system is a hypothetical, 45-km long stretch divided into 45 equal reaches. The runoff generated by rainfall on the upstream catchments enters the system as an additional inflow into the river at reach 1. The CSOs are
assumed to discharge at reach 7 with the storm tank overflows and treatment plant effluent at reach 10.

The base case (BC) in this study is a system assumed to be functioning under existing conditions, i.e. without any climate- or urbanisation-related changes. In the BC, the IUWS described above was subjected to a 6-day rainfall event from 7 to 13 February 1977 with a total depth of 27 mm. The values (nominal) of the operational control and design parameters defined in this study under the BC are presented in Table 2.

INTEGRATED URBAN WASTEWATER SYSTEM MODELLING

The IUWS considered in this study has three subsystems comprising a sewer system, a WWTP and a river (see Figure 1). The IUWS simulator here is the Matlab/Simulink-based SIMBA5 simulation tool, developed by IFAK (2005).

Sewer system model

The hydrological modelling approach included in SIMBA is applied for the sewer system modelling due to its simplicity. The KOSIM (Itwh 1995) approach has been applied to modelling impervious and pervious areas. In this study, the rainfall–runoff simulation wetting losses, depression storage losses, washoff, infiltration and evaporation losses are assumed to be constant. A diurnal pattern of flow and pollution for DWF simulation has been considered in the sewer system including several pollutants (e.g. suspended solids (SS), volatile SS, ammonium, nitrate, total chemical oxygen demand (COD) and soluble COD). Surface runoff and flow within the sewer network are modelled conceptually by the Nash cascades method and by translation (cascades of linear reservoirs). Pollutants in the catchment originate from two sources: DWF and rainfall–runoff. The concentrations of pollutants in these two were considered constant. The storage tanks’ performance is modelled based on a simple sedimentation approach and without any biological processes allowing reduction of pollution in overflows to be modelled.

Wastewater treatment plant model

The Activated Sludge Model 1 (ASM1) established by the IWA work group (Henze et al. 1986) is the model used in this study for operational control and process studies of biological WWTPs. To model the storm tank the same approach used for the storage tanks in the sewer system is applied. In this study, the reactor is considered as a completely mixed nitrifying reactor based on the ASM1 with oxygen input by pressure aeration. The secondary clarifier is an integral part of the activated sludge system. Two pumping stations have been considered in the WWTP to return the activated sludge to the reactor and also to control the wastewater influent to the WWTP.
River model

In this study, SWMM software (from the SIMBA sewer block) is used for modelling flows in the river. For the purpose of water quality modelling in the river the water quality model developed by Lijklema et al. (1996) was applied. This river water quality model was described by Schütze et al. (2002) and it considers self-purification.

CLIMATE CHANGE AND URBANISATION

Rainfall was selected as the indicator of climate change in this work due to its important impact on IUWS operation and design. Hulme et al. (2002) and IPCC (2000) estimated the future pattern changes in rainfall for the UK indicating a future with wetter winters and drier summers for some regions under certain climate change scenarios.

The climate change parameters used in this study are the increase in rainfall depth and/or intensity:

- **RD**: This parameter represents the percentage increase in rainfall depth. This increase under climate change is achieved by applying a fixed percentage value to BC rainfall intensities across an entire event/time-series. As a consequence, total rainfall depth is increased.

- **RI**: This parameter represents the percentage increase in rainfall intensity. This increase under climate change is achieved by applying a fixed percentage value to rainfall intensities across an entire event/time-series whilst maintaining the same cumulative rainfall depth.

The urbanisation parameters used in this study are as follows:

- **POP**: In this study, POP represents the percentage increase in population count over a given period of time. In the UK, population growth by 2030 is predicted at 4.5% as a minimum and 15% as a maximum shown in Table 1 (Astaraie-Imani et al. 2012).

- **PCW**: PCW is defined as per capita water consumption in litres/day/person. This factor has a key role in influencing domestic wastewater quantities. As Butler & Davies (2011) indicate, approximately 95% of the water consumed is returned to the sewer system as wastewater. In this study, a range of PCW values has been considered as shown in Table 1 (Astaraie-Imani et al. 2012).

- **IMP**: IMP represents the percentage of impervious surfaces increase (i.e. urban creep in this study) which has a direct influence on the rate of stormwater runoff in urban areas (Butler & Davies 2011). The range used for this parameter was defined by Astaraie-Imani et al. (2012) and is shown in Table 1.

IUWS OPTIMISATION

Operational control problem

Optimal operational control of the IUWS aims to derive a strategy to gain the best system performance with respect to various criteria (operation control objectives). The ultimate aim of operational control is to maximise IUWS performance so the objectives of the optimisation algorithm are summarised as: maximise the minimum dissolved oxygen (DO) concentration in the river; minimise the maximum ammonium (AMM) concentration in the river (assuming slowly varying conditions). In this case, the optimisation is accomplished by optimising the operational control of the critical flow rates as decision variables in this study. The critical flow rates are: maximum outflow rate from the sewer system ($Q_{\text{maxout}}$); maximum inflow rate to the WWTP ($Q_{\text{maxin}}$); threshold for triggering emptying the storm tank ($Q_{\text{trigst}}$). Nominal values (their values in the BC) and value ranges are presented in Table 2.

In this study, in line with helping decision makers to select appropriate strategies, the region limited to the

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### Table 1 | Climate change, urbanisation parameters’ nominal values and their value ranges (Astaraie-Imani et al. 2012)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Nominal value</th>
<th>Value/value range</th>
</tr>
</thead>
<tbody>
<tr>
<td>RD</td>
<td>%</td>
<td>0</td>
<td>30</td>
</tr>
<tr>
<td>RI</td>
<td>%</td>
<td>0</td>
<td>30</td>
</tr>
<tr>
<td>POP</td>
<td>%</td>
<td>0</td>
<td>[4.5,15]</td>
</tr>
<tr>
<td>IMP</td>
<td>%</td>
<td>0</td>
<td>[5,15]</td>
</tr>
<tr>
<td>PCW</td>
<td>litre/person/day</td>
<td>180</td>
<td>[80,260]</td>
</tr>
</tbody>
</table>
4 mg/l (FWR 1998) concentration indicates the compliant region in which solutions meet both DO and AMM concentration standards (FWR 1998).

(Re)design problem

The IUWS (re)design aims to improve the IUWS performance by increasing the catchment wastewater storage capacity. The objectives of the IUWS design optimisation problem are the same as the objectives described for the operational control optimisation model. There is dependency between the operational control (i.e. decision variables of the operational control model) of the IUWS and its design (i.e. storage capacity of the system in this study). Therefore, in the design case, the volume of the storage tank is added to the operational control model decision variables as an additional decision variable. In effect, design optimisation is carried out at the same time as the operational optimisation. With regard to the aim of the (re)design problem, Figure 2 shows the iterative procedure of the design optimisation process to find the aforementioned wastewater storage capacity increase required. This iterative procedure is comprised of the following steps:

1. Assume the system storage capacity increment (c).

The factor c is the percentage storage capacity increment-coefficient applied to the existing storage capacity of the catchment to increase it. Therefore the upgraded storage capacity of the catchment is estimated by Equation (1):

\[ V_{\text{new}} = V(1 + c/100) \]  

where \( V_{\text{new}} \): increased storage capacity of the catchment (m³); \( V \): current storage capacity of the whole catchment.

The \( V_{\text{new}} \) obtained (see step 1), has to be distributed among the existing storage tanks in the sewer system. The relevant storage tanks' contribution coefficients are introduced here and used as decision variables. The storage tank contribution coefficient is defined as follows:

Storage tank contribution-coefficient (\( a_i \))

The storage capacity increase needs to be distributed among the existing storage tanks in the catchment by a storage tank contribution coefficient and this is calculated according to Equation (2):

\[ S_{Ti} = V_i + V_{\text{new}} \times a_i/100 \]  

where \( S_{Ti} \): increased storage capacity of storage tank \( i \) (m³); \( V_i \): existing volume of storage tank \( i \) (m³); \( i \): storage tank

Table 2 | Decision variables of the IUWS operational control and design optimisation models

<table>
<thead>
<tr>
<th>DV(^d)(unit)</th>
<th>DV(^d) description</th>
<th>Nominal values (BC)</th>
<th>Value/value range in operational control model</th>
<th>Value range used in (re)design model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design parameter</td>
<td>( a_2 ) (%)</td>
<td>CC(^c) of ST(_2)</td>
<td>21.2</td>
<td>21.2</td>
</tr>
<tr>
<td></td>
<td>( a_4 ) (%)</td>
<td>CC(^c) of ST(_4)</td>
<td>10.61</td>
<td>10.61</td>
</tr>
<tr>
<td></td>
<td>( a_6 ) (%)</td>
<td>CC(^c) of ST(_6)</td>
<td>15.15</td>
<td>15.15</td>
</tr>
<tr>
<td></td>
<td>( a_7 ) (%)</td>
<td>CC(^c) of ST(_7)</td>
<td>53.03</td>
<td>53.03</td>
</tr>
<tr>
<td>Operational control parameter</td>
<td>( Q_{ST2} ) (m(^{3}/d))</td>
<td>Maximum outflow rate of ST(_2)</td>
<td>5 \times DWF(^a)</td>
<td>5 \times DWP(^a)</td>
</tr>
<tr>
<td></td>
<td>( Q_{ST4} ) (m(^{3}/d))</td>
<td>Maximum outflow rate of ST(_4)</td>
<td>5 \times DWF(^a)</td>
<td>5 \times DWP(^a)</td>
</tr>
<tr>
<td></td>
<td>( Q_{ST6} ) (m(^{3}/d))</td>
<td>Maximum outflow rate of ST(_6)</td>
<td>5 \times DWF(^a)</td>
<td>5 \times DWP(^a)</td>
</tr>
<tr>
<td></td>
<td>( Q_{\text{maxout}} ) (( Q_{ST7} ) (m(^{3}/d))</td>
<td>Maximum outflow rate of ST(_7)</td>
<td>7 \times DWF(^a)</td>
<td>[3 \times DWF:8 \times DWF(^b)]</td>
</tr>
<tr>
<td></td>
<td>( Q_{\text{minin}} ) (m(^{3}/d))</td>
<td>Minimum inflow to the WWTP</td>
<td>3 \times DWF(^a)</td>
<td>[2 \times DWF:5 \times DWF(^b)]</td>
</tr>
<tr>
<td></td>
<td>( Q_{\text{trigst}} ) (m(^{3}/d))</td>
<td>Threshold triggering emptying the storm tank</td>
<td>24192</td>
<td>[16416,31104]</td>
</tr>
</tbody>
</table>

\(^{a}\)These values are multiplied in the DWF value of their relevant sub-catchment area (see Figure 1).

\(^{b}\)These values are multiples of the treatment capacity of the WWTP i.e. 27,500 m³ for this study.

\(^{c}\)CC: Contribution-coefficient.

\(^{d}\)DV: Decision variable.

\(^{esws}\): sewer system.
There is dependency between the performance of the storage tanks' capacities and their outflow rates (Fu et al. 2010). Therefore, if the storage tank's capacity changes (e.g. increases), its maximum outflow rate (throttle flow) needs to be adjusted accordingly. This provides the potential for more efficient usage of the storage tank capacity to reduce the CSOs. Therefore the values/value ranges of the maximum outflow rates from the storage tanks are changed to the new value ranges (see Table 2).

2. Use $c$ in the design optimisation model.

3. Control whether the minimum $c$ was achieved or not.

To ensure this, $c$ is increased/decreased (see the next steps) until the 50% target is reached (see the next step for definition of the 50% target). If the minimum $c$ was achieved, the algorithm is finished and if not the increase(s)/decrease(s) are continued until reaching this target.

4. Check whether most of the optimal Pareto front solutions (most meaning 50% assumed in this study) fit into the compliant region. The reason for this is that, apart from the water quality impact, cost is an important factor that affects the decisions to upgrade the system.

In this study, a surrogate approach to considering costs is applied to limit the redesign cost by finding the minimum $c$ required to improve system performance. The redesign costs estimated this way have the potential to demonstrate the minimum redesign budget needed to improve the system performance while coping with future changes.

5. If less than 50% of the optimal Pareto front solutions are within the compliant region, increase the value of $c$ and return to step 2. This range increase is variable and depends on how far the optimal Pareto front is from the compliant region. If it is completely out of the compliant region, the increase step is between 50 and 100% but if it is within the compliant region, the decrease step is between 5 and 20%.

6. If, however, more than 50% of the optimal Pareto front solutions were within the compliant region, decrease the value of $c$ until reaching the 50% target.
Climate change and urbanisation scenarios

The IUWS optimisation is performed using a scenario-based approach. These scenarios describe the possible features of the future climate change and urbanisation with the considered input parameters and have the potential to indicate the impacts of climate change and urbanisation on the performance of the IUWS in the boundaries of their decision space (input parameters defined in Table 1). Two groups of scenarios have been developed in this study: climate change scenarios (two scenarios) and combined climate change with urbanisation scenarios (four scenarios were developed but only the two most critical scenarios are presented in this paper). Additionally, the BC was considered as a scenario as well. These scenarios aim to investigate the possibility of mitigating the negative impacts of climate change and/or urbanisation on the quality of water by improving the operational control/design of IUWS. Table 3 shows the climate change and urbanisation parameter values for each scenario. Different combinations of these input parameters form the scenarios’ structure. For all scenarios, the optimisation objectives and decision variables were defined as in the operational control and design problems.

SCA: For this scenario, the operational control optimisation is implemented assuming the BC. The aim is to explore the possibility of improving the quality of water in the river in the current operational control settings of the IUWS (i.e. operational control parameters’ nominal values). All the urbanisation parameters are set to their nominal values (see Table 1) and the applied rainfall is the base rainfall.

The two climate change scenarios defined in this study are scenario B (SCB) and scenario C (SCC) and the two combined climate change with urbanisation scenarios considered are SCD and SCE as shown in Table 3.

SCB: In this scenario, the urbanisation parameters are set to their nominal values (see Table 1) and the climate change parameter is RD (see Table 3).

SCC: In this scenario, the considered urbanisation parameters are similar to SCB. The only difference is the applied climate change parameter RI.

SCD: In this scenario, the climate change parameter is RD. Also the urbanisation parameters are set to their upper values (see Table 3).

SCE: This scenario remains much the same as SCD with the only difference being the climate change parameter considered is RI.

RESULTS

Operational control optimisation results

Operational control optimisation was applied to SCB, SCC, SCD and SCE. This included utilisation of the MOGA-ANNβ meta-model (Astaraie-Imani et al. 2014), applied to the optimisation model, to speed up the computational process.

Figures 3 and 4 show the Pareto fronts representing the optimal trade-off between minimum DO and maximum

Table 3 | Climate change and urbanisation scenarios, parameters and their value ranges

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Climate change parameters</th>
<th>Urbanisation parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RD, RI</td>
<td>POP (%)</td>
</tr>
<tr>
<td>Scenario A (SCA)</td>
<td>Base rainfall</td>
<td>0</td>
</tr>
<tr>
<td>Scenario B (SCB)</td>
<td>RD</td>
<td>0</td>
</tr>
<tr>
<td>Scenario C (SCC)</td>
<td>RI</td>
<td>0</td>
</tr>
<tr>
<td>Scenario D (SCD)</td>
<td>RD</td>
<td>1.15</td>
</tr>
<tr>
<td>Scenario E (SCE)</td>
<td>RI</td>
<td>1.15</td>
</tr>
</tbody>
</table>

*a*This means that the urbanisation parameter value is kept at its nominal value (see Table 1).

*b*This is RD/RI with 30% increase (see Table 1).

Figure 3 | Min-DO and max-AMM concentrations in the BC under SCA (i.e. BCSCA), SCB (BCSCB) and SCC (BCSCC), optimal Pareto fronts of the operational control model in SCA, SCB and SCC.
AMM concentrations for all the scenarios. In Figures 3 and 4, the points BCSCA/BC, BCSCB, BCSCC, BCSCD and BCSCE show the performance of the IUWS under BC, SCB, SCC, SCD and SCE, respectively.

Figure 3 shows that point BCSCA/BC is nearly dominated by the solutions obtained in all the above scenarios. This demonstrates that optimising the operational control has the potential to substantially improve the quality of water in the river under base rainfall (SCA).

The points BCSCB and BCSCC (in Figure 3) are dominated by the solutions obtained under increased rainfall depth (SCB) and also increased rainfall intensity (SCC). The improvements for the SCB are more significant than SCA and SCC. The reason for this is that increasing the rainfall depth in SCB has a greater potential to worsen the quality of water in the river than increasing the rainfall intensity in SCC.

SCD and SCE show significant deterioration in the water quality in comparison with SCA (i.e. BCSCD and BCSCC points in Figure 4). In other words, despite the operational control optimisation, the IUWS cannot cope with the future changes described in these two scenarios. The reason for this considerable worsening in river water quality is linked to the urbanisation parameters, which were set to their upper value range (Table 1). The increase in the urbanisation parameters increases the pollutants’ load in DWF and surface runoff and consequently leads to deterioration of water quality in the river. Figure 4 shows that in SCD water quality deteriorated considerably more than in SCE. This is due to the increased negative impacts of urbanisation parameters under RD in SCD.

Selecting an operational control strategy among the many available is subjective and depends on the decision-maker’s preference. In line with helping decision makers select an appropriate operational control strategy the region limited to the 4 mg/l (FWR 1998) concentration indicates the compliant region; solutions in this region meet both DO and AMM concentration standards (FWR 1998).

With regard to the optimal Pareto fronts in SCB and SCC (Figure 3), the numbers of the solutions within the compliant region decrease significantly in SCB in comparison with SCA and SCC. This indicates that the decision makers have fewer options for an appropriate operational control strategy in the future in SCB. In SCD and SCE (Figure 4), there is no desirable solution for decision makers. Therefore, in these cases, changing the operational control of the IUWS only (even by optimisation), cannot improve water quality in the river to the target level.

As a result the operational control optimisation of the IUWS has potential to improve the water-quality indicators in the river under climate change scenarios in this study to the standard required (i.e. SCB and SCC). However, the same cannot be said for the combined climate change and urbanisation scenarios (i.e. SCD and SCE). Therefore, alternative strategies for these scenarios are required to improve the quality of water such as rehabilitation, redesign of the IUWS or similar.

Comparing the operational control strategies

In order to further illustrate the impact of each scenario’s optimal operational control on the water quality parameters (optimisation objectives) in the river, several operational control strategies could be selected from the optimal Pareto fronts achieved for each scenario (e.g. A1, B1 and C1 in Figure 3 for the climate change scenarios and/or E1 and D1 in Figure 4 for the combined climate change with urbanisation scenarios). For this purpose, in this paper, only the min-DO and max-AMM concentrations in reach 40 and 10 of the river for the selected solution D1 in Figure 4 (as SCD is the most critical scenario to meet the compliant region in this study) are presented in Figures 5 and 6. The
increased min-DO concentrations and reduced max-AMM concentrations in these two figures illustrate the general observation previously made in terms of improving the quality of water by optimising the operational control of the IUWS under future changes but the obtained min-DO and max-AMM concentrations are far from the 4 mg/l standard (DoE 1994) (i.e. compliant region) which shows the significant worsening of water quality under combined climate change with urbanisation (SCD).

(Re)Design optimisation results

Design optimisation was applied to SCD and SCB, which were the most critical scenarios (resulting from the IUWS operational control optimisation), under climate change only and also under combined climate change with urbanisation. Figures 7 and 8 show the Pareto fronts representing optimal trade-off minimum DO and maximum AMM concentrations for the aforementioned scenarios. In addition to the Pareto fronts of the design model, the Pareto fronts of the operational control model are demonstrated for comparisons in these figures.

Using the iterative procedure, it was found that about 100% and 675% storage capacity increases are required in SCB and SCD, respectively, in order to meet the bounds of the compliant region. The system upgrades add 13,200 and 89,100 m³ additional storage capacities to the current storage capacity of the system (13,200 m³).

As can be observed in the above figures, the Pareto fronts achieved from the operational control optimisation (see SCB

![Figure 5](https://iwaponline.com/jwcc/article-pdf/4/3/232/374887/232.pdf) | Min-DO concentration in reach 40 of the river in SCD before and after the operational control optimization.

![Figure 6](https://iwaponline.com/jwcc/article-pdf/4/3/232/374887/232.pdf) | Max-AMM concentration in reach 40 of the river in SCD before and after the operational control optimization.

![Figure 7](https://iwaponline.com/jwcc/article-pdf/4/3/232/374887/232.pdf) | Min-DO and max-AMM concentrations in the BC under SCB (i.e. BCSCB); optimal Pareto fronts of the design optimisation model in SCB.

![Figure 8](https://iwaponline.com/jwcc/article-pdf/4/3/232/374887/232.pdf) | Min-DO and max-AMM concentrations in the BC under SCD (i.e. BCSCD); optimal Pareto fronts of the design optimisation model in SCD.
and SCD optimal Pareto fronts in Figures 3 and 4, respectively) are dominated by the solutions obtained from the IUWS design optimisation. This indicates that the changes exerted in the operational control parameters (see Table 2) and increments of the catchment storage capacity, have enough potential to improve the system performance so that more than 50% of the optimal Pareto fronts’ solutions can meet the compliant region in SCB and SCD. These significant improvements in the performance of the system can highlight the importance of upgrading the existing IUWS to deal with the proposed future changes especially under the combined scenarios in the future (i.e. SCD). For instance, SCD, as the most critical scenario (see Figure 4), needs the maximum storage capacity increase and consequently the maximum redesign budget to cope with the future changes.

The observations in Figures 7 and 8 indicate that under future climate change and urbanisation, improving the design of the system may completely change the behaviour of the system through the existing interactions between the parameters (i.e. operational control and design) aiming to meet the compliant region but likely at a high cost.

Comparison of solutions obtained in operational control and design optimisations

In order to further compare the impact of the optimal design with the operational control, it is useful to look at the design parameters in each scenario. In this paper, only the values of the design parameter $a$ (see Table 2) for all the optimal solutions within the compliant region in scenario SCD are presented (see Figure 9).

Note that the horizontal axis shows the design parameters and the vertical axis shows their corresponding values. The dashed lines show the values of $a$ in the BC (these values were achieved by dividing the volume of each storage tank in the BC into the total storage capacity of the catchment i.e. 13,200 m$^3$). The lines with the descending and ascending slopes show the decreasing or increasing role of the storage tanks in meeting the compliant region arising from the existing interactions among the storage tanks. The following conclusions can be obtained from Figure 9 for design parameter $a$.

The $a_7$ in SCD (Figure 9) needs to be nearly doubled (i.e. 50%, compared with the BC) until the optimal Pareto front can meet the compliant region. This shows that the current volume of ST$_7$ is not large enough in SCD to cope with the future changes. Additionally, in Figure 9 it can be observed that $a_2$, $a_4$ and $a_6$ decrease compared with the BC. This indicates that even with 675% storage capacity increases in SCD, $a_2$, $a_4$ and $a_6$ decrease to about 50, 90 and 85% (compared with the BC), respectively. In other words, the current storage capacities of these tanks are too large, i.e. these tanks are over dimensioned. Therefore, the design optimisation reduces unnecessary storage capacity and consequently the investment budget under the future changes to meet the compliant region.

Figure 10 shows the IUWS operational control parameter for all the optimal solutions within the compliant region in SCD (see Figure 10), respectively. The dashed line on these figures shows the nominal values of the maximum outflow rates presented in Table 2. The slopes between the operational control parameters show the system performance in reducing the CSOs considering the treatment capacity of the WWTP.

Generally, it can be observed from Figure 10 that $Q_{\text{max out}}$ obtained from the design optimisation model has decreased approximately 50% in all scenarios compared with the BC (dashed line). These decreases are prompted by the ST$_7$ using the increased storage capacities efficiently and storing more wastewater generated to partially treat them. A similar decrease of about 38% can be observed for $Q_{\text{ST2}}$ in the ST$_2$.

$Q_{\text{ST4}}$ and $Q_{\text{ST6}}$ increased by about 29 and 9% compared with the BC (dashed line), respectively, these are not
considerable compared with $Q_{\text{maxout}}$. These increases arise from increased wastewater discharge from the storage tanks to reduce the CSOs and hence meet the compliant region. $Q_{\text{maxin}}$ and $Q_{\text{trigst}}$ do not show significant variation from the dashed line as the capacity of the WWTP has not been increased.

The above observations in Figure 10 indicate that under the future climate change and urbanisation, improving the design of the system may completely change the behaviour of the system through the existing interactions between the parameters (i.e. operational control and design) to meet the compliant region.

CONCLUSIONS

In this paper, an attempt to reduce the negative impact of climate change and urbanisation on water quality in the river has been made by optimising (i.e. improving) the operational control and upgrading the storage capacity of the integrated urban wastewater system. In the optimisation models developed here water-quality indicators in the recipient were used as objectives and relevant operational control and/or design parameters were decision variables.

The results obtained in the case study analysed illustrate that operational control optimisation has limited potential in terms of improving the quality of water in the recipient under the considered climate change scenarios. This improvement seems to be more effective in the case of rainfall intensity increase than in the case of increased rainfall depth, as the increased volume of rainfall has a more negative impact on the water quality in the river.

The target water quality in the river is not achievable when climate change is combined with maximum urbanisation growth (scenarios SCD and SCE). In this case, for the case study analysed here, optimal operational control on its own cannot improve the quality of water in the river and an alternative approach involving IUWS redesign/rehabilitation is necessary to improve the system performance.

Finally, the analysis carried out in this study and the results obtained also indicate the importance of considering sewer and WWTP systems as integrated systems as this enables dimensioning IUWS storage capacity adequately so that the system as a whole can deliver the best/target performance.

All of the above findings are based on a semi-real case study in the UK and hence should not be generalised. Future work will involve testing and verification of the optimisation approaches shown here on additional, more realistic case studies in the UK and abroad. Future work will also involve improved representation of climate change by using UKCIP09 (Hulme et al. 2002) scenarios to more accurately quantify the impact of climate change on IUWS performance and, in turn, on the recipient water quality, i.e. aquatic life. Additionally, future work will consider other climate change, urbanisation and system parameters to analyse the IUWS performance under the future changes. Moreover, other rehabilitation techniques can be investigated in the future to improve the system performance.

REFERENCES


ATV (Abwassertechnische Vereinigung V.) 1992 Richtlinien für die Bemessung und Gestaltung von Regenentlastungsanlagen in
Mischwasserkanälen. ATV-Arbeitsblatt A128, Gesellschaft zur Förderung der Abwassertechnik, St. Augustin.


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