Pergamon

Wat. Sci. Tech. Vol. 33, No. 2, pp. 199–208, 1996.

0273–1223/96 $15·00 + 0·00

PII: S0273-1223(96)00201-6

AN EVALUATION OF THE EFFICIENCY OF THE COMBINED SEWER – WASTEWATER TREATMENT SYSTEM UNDER TRANSIENT CONDITIONS

W. Bauwens*, P. Vanrolleghem** and M. Smeets*

* *University of Brussels (VUB), Laboratory of Hydrology, Pleinlaan 2, 1050 Brussel, Belgium*
** *University of Gent, Department Applied Mathematics, Biometrics and Process Control, Coupure Links 653, 9000 Gent, Belgium*

ABSTRACT

The paper considers the efficiency of alternative sewer and wastewater treatment plant management schemes with respect to the effluents to the receiving waters. The input time series for the flows and concentrations at the CSO structures and at the treatment plant intake are obtained through a continuous sewer simulation model. The wastewater treatment plant model is based on a structured dynamic model describing COD removal and final settling. Special emphasis is put on the sludge inventory of the plant since this is considered to be the main problem area under storm conditions. The methodology is illustrated on the combined sewer network of Brussels. Scenarios without and with CSO control measures in the sewer are considered. At the treatment plant, the simulation study evaluates the effect of potential control strategies such as ratio control of the RAS, step feed and retention of first flush in a storm tank. Copyright © 1996 IAWQ Published by Elsevier Science Ltd

KEYWORDS

Activated sludge; combined sewer overflow; modeling; sewer systems; wastewater treatment plant.

INTRODUCTION

On a yearly basis, the emission of pollutants from combined sewer systems into the receiving water represents only a fraction of the total pollutant amount that will be processed by the wastewater treatment plant. Nevertheless, the impact of combined sewer overflow (CSO) on the receiving waters cannot be neglected, mainly with respect to peak concentrations and to the accumulation of toxic substances. Moreover, this impact will increase with the improvement of the treatment efficiency. In order to reduce the impact of the CSO, additional storage and real time control may be considered in the sewer system. In combination to – or as a consequence of – the latter actions, the loading of the water treatment plant (WWTP) may be increased. This, on its term, results in a larger variability of the effluent characteristics of the WWTP.

With regard to water quality regulations, two alternative approaches can be distinguished: the Uniform Emission Standard (UES) and the Environmental Quality Objective / Environmental Quality Standard

(EQO/EQS; Tyson *et al.*, 1993). Under the EQO/EQS approach, the objectives with regard to the use(s) of the water are defined first and case specific standards are derived to achieve compliance with the objectives. The UES approach focusses on emission standards, irrespective of local circumstances but considering technology based criteria. Most actual regulations concerning CSO can be classified under the UES approach. They prescribe a limitation of the overflow frequency (3–10/year) and/or on a certain dilution ratio (e.g. 1:5) that must be obtained before CSO is allowed. The actual, case specific, environmental impacts of the overflows on the receiving waters are hereby not properly considered.

The aim of good drainage design – in pollution terms – is to balance the effects of continuous and intermittent discharges against the assimilation capacity of the receiving water, in order to optimize the quality of the receiving water at minimal cost. If this definition is accepted, the analysis of the problem in compliance to the EQO/EQS approach imposes itself. Here, the effects of pollution abatement scenarios are to be considered through a statistical analysis of immission characteristics of the receiving water. Also, a holistic view becomes imperative: all flow and pollutant sources should be considered on a basin scale and all flow situations and the dynamics of the system have to be considered. Only through such an approach, can a real risks analysis of the system can be performed.

The results presented in this paper are the partial results of such an integrated analysis. However, the emphasis will be placed on the interactions between the sewer system and the WWTP, and especially on the behavior of the latter under transient conditions, from an emission point of view.

METHODOLOGY AND MODELS USED

General

In order to perform a statistical analysis of the immission characteristics of the receiving water – e.g. by means of concentration-duration-frequency (CDF) curves – long time series of those characteristics must be generated for each of the scenarios considered. The use of hydrologic, hydraulic and quality simulation models becomes hereby imperative.

Conceptual models that incorporate the fundamental quantity and quality determining processes are available for the river system, the treatment plant and for the sewers. However, due to their complexity – and consequently large computation times – their application is limited to the simulation of specific events. If continuous simulations are aimed at for the generation of long time series, models with a simpler conceptual background and whereby also the system is represented in a simplified way have to be used.

The sewer model

For this study, the KOSIM model (Harms and Kentor, 1987) has been selected. In this model, the sewer system is represented by a number of reservoirs connected in series or parallel. A conceptual rainfall-runoff model transforms the rainfall series into a flow series for the subbasin. Pollutant inputs are generated from the dry weather flow cycle and concentrations and – during rainfall – through the principle of constant concentration associated to the storm flow. Sedimentation and resuspension are modeled for each subbasin, based on critical discharges for settlement and sediment removal. Within the system, flows and pollutants are conveyed downstream by accounting for a constant travel time. Weirs and different types of storage basins may be considered. The hydraulic calculations for these structures are based on the continuity equation, on maximal flow capacities and on stage-discharge relations. In the storage basins, the settlement of pollutants and sediments is described by classical sedimentation theory. No interactions between pollutants and/or the sediment are considered.

The wastewater treatment model

A traditional carbon removal wastewater treatment system has been used. It consists of primary clarification, 3 completely mixed aeration tanks in series and a final clarifier (Fig. 1). As the biotransformation model was the IAWQ model No. 1 with elimination of nitrification and denitrification processes (Henze *et al.*, 1987), 5 state variables were considered for all compartments in the plant (in IAWQ nomenclature): X_H (heterotrophic biomass), X_I (inert particulate material), X_S (slowly hydrolyzable particulate substrate), S_S (soluble substrate) and S_O (oxygen).

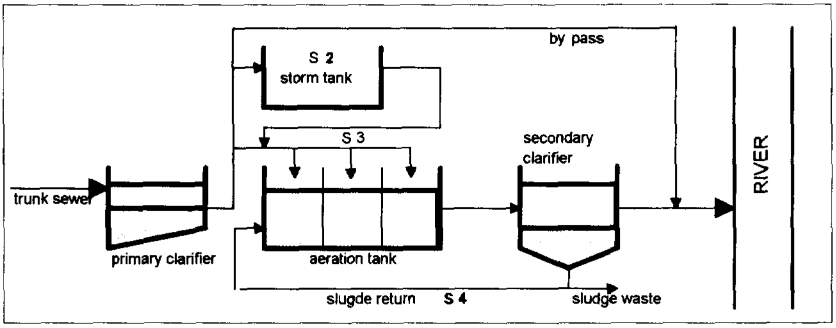

Figure 1. Schematic representation of the treatment plant.

Together with the information on wastewater temperature, an overall heat balance involving different heat loss and generation terms (van der Graaf, 1976) was used to model the dynamics of the mixed liquor temperature. The dependence of mass transfer and biodegradation on temperature was modeled in the traditional way. For the primary clarification a model was developed on the basis of the 5-layer model of Lessard and Beck (1988). A first order hydrolysis reaction of the slowly biodegradable particulate fraction (X_S) is included and the settling velocity depends on sewage features, i.e. storm sewage has better settling properties than normal sewage. Scouring is taken from Alarie *et al.* (1980). Secondary clarification was modelled according to Takacs *et al.* (1991) using a 10-layer one-dimensional model. This choice was motivated by the results of Grijspeerdt *et al.* (1994). More details can be found in Bauwens *et al.* (1995).

THE SYSTEM

The sewer system

Part of the Brussels urban drainage system was chosen as a model to illustrate the problem. The system consists of five collectors on the left bank of the river Zenne, which receives the used waters of the whole of Brussels. The total drainage area is about 5400 ha, of which ca. 1800 ha is impervious. The average inhabitant density is around 53 capita/ha.

The five collectors are connected to a WWTP by a trunk sewer with a capacity of 5 times the dry weather flow (DWF). At the outlet of each collector, an overflow structure discharges the excess flow to the river (option CSO, Fig. 2). In a second scenario, on-line storage basins are placed at the outlet of each collector, to limit the overflow frequency to 7 per year (option BAS, Fig. 2). The total additional storage volume amounts to ca. 195,000 m^3, as calculated by Smeets *et al.* (1995).

To compute the flows in this system with KOSIM, the network was schematized by means of 50 subbasins. This schematization was based on the differences in land use and on detailed hydraulic simulations, to account for internal storage and transport time effects.

The storm flows were calculated, accounting for wetting losses (0.5 mm), depression storage loss (1.8 mm) and a time dependent runoff coefficient (between 0.25 and 1). Literature data were used for the washoff concentrations during rainfall events (Jolankai, 1992): 0.06, 0.13 and 0.5 g/l for, respectively, BOD, COD and suspended solids. With these concentrations, the average daily loads amount to ca. 2, 4.5 and 17 t for

BOD, COD and suspended solids. From a comparison of the resulting emission characteristics at the overflows (see Smeets *et al.*, 1995) with data obtained in the Netherlands (Bakker *et al.*, 1989), it can be concluded that the data are realistic, except maybe for the BOD concentration which is probably somewhat high.

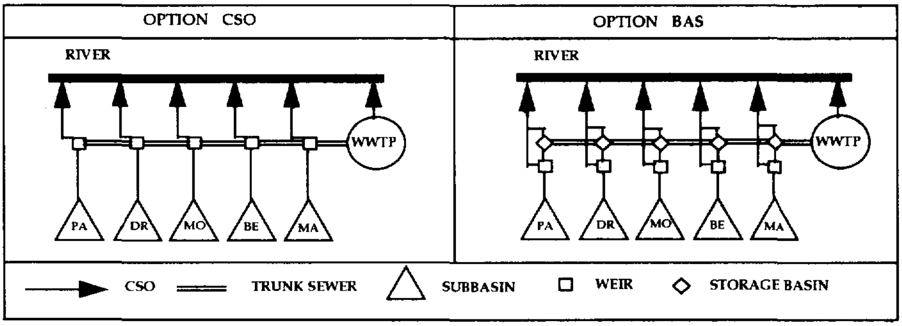

Figure 2. The simplified sewer system for the CSO and BAS options.

The daily cycle of the dry weather flow characteristics are based on *in situ* measurements of quantity and quality variables (VUB,1992; ULB,1992). The average daily flow amounts to 45x103 m^3, while ca. 13 t of BOD, 28 t of COD and 10 t of suspended solids are released on a daily basis. The latter data correspond to concentrations of 0.35, 0.6 and 0.24 g/l, respectively. The daily maximum/minimum ratio is 1.85 for the discharge and 6 for the pollutant flows.

With regard to the sedimentation problem, it was assumed that no sedimentation takes place in the pipe network. In the storage reservoirs, the sedimentation efficiency depends on the degree of filling of the basin. The maximal efficiencies were set to 0.12 for BOD and COD and to 0.27 for the suspended solids (Degremont, 1991).

The wastewater treatment system

Four operating scenarios were evaluated with respect to the impact on the effluent quality, i.e. the combination of clarifier effluent and treatment plant bypass, especially under transient flow conditions (Fig. 1):

S1. Base Case
S2. Storm Tank
S3. Step Feed
S4. Ratio Control of the Recycle Flow Rate

For the primary clarification a hydraulic retention time of 2 hours is obtained at DWF, resulting in a surface loading of 1.8 m/h. In the base case, a maximum of 5 DWF is passed through the primary clarifiers during storm conditions of which 2.5 DWF is sent to the aeration tanks, the remainder being bypassed to the river.

The hydraulic retention time in the aeration tanks is 9 hours during DWF and the biomass loading rate is around 0.25 g COD/g COD/d. Dissolved oxygen (DO) control is based on a 3 intensity level system allowing one to increase the aeration capacity two or fourfold above base aeration if DO decreases below 2 mg/l. Similarly, reduction of the aeration intensity occurs when DO increases above 4 mg/l. Minimum aeration is always maintained to fulfill mixing needs. The status of the aeration system is only allowed to change once per 2 hours. Evidently, this operating scheme induces oscillations in DO level.

For the secondary clarification, a hydraulic retention time of 6 hours and a design surface loading of 0.6 m/h under DWF was chosen. The inlet was positioned at one fourth of total clarifier height. Normal operation included a constant sludge recycle flow rate defined by a recycle ratio of 25 per cent of mean DWF. The constant waste flow rate was set to 0.75 percent of mean DWF. With a typical waste sludge concentration of 14 kg COD/m^3, a sludge age of approximately 9 days is found. The sludge blanket was typically found between 20 and 50 per cent of clarifier height.

In scenario 2, a storm tank with a volume of 6 hours DWF is installed after the primary clarifiers. It is operated in fill-and-bypass mode (Lessard and Beck, 1990): whenever the outflow of the primary clarifiers exceeds 2.5 DWF, the excess is diverted into the storm tank as long as it is not filled. If full, the excess is bypassed to the river. This scenario ensures that the often highly polluted primary clarifier contents at the beginning of a storm event (one third of the storm tank capacity) and the first flush are not bypassed, but are stored for later treatment. Emptying of the storm tank is initiated as soon as the primary clarifier effluent flow rate drops below 2.5 DWF. The emptying rate is adjusted so that the inflow capacity to the aeration tanks is used (2.5 DWF).

The step feed option (S3) is evaluated as it is considered excellent for temporary decrease of the secondary clarifier sludge loading during storm events (Olsson and Jeppsson, 1994). Essentially it allows one to redistribute sludge between the final settler and the aeration tanks. To this end influent is distributed over the different aeration tanks instead of all being entered into the first one. The distribution is performed equally over all three tanks. It is good to keep in mind that during such operation treatment efficiency drops due to decreased biocatalyst concentration and hydraulic retention time. One relies on absorption to remove most pollutants.

In the last operating scenario (S4), the effect of the classical ratio controlled recycle flow rate is studied (Andrews, 1974). In this approach, the recycle flow rate is no longer constant but is varied proportionally to the influent flow rate. The proportionality constant was set to 0.25. A direct result of such an approach is that the biomass concentration in the aeration tanks is more stable, while the underflow concentration in the final settler is prone to larger variations. This indicates that settler performance will deteriorate under this scenario, while beneficial effects can be expected at the level of the plant's biodegradation capacity. It is important to note that the time delays in the propagation of hydraulic disturbances should be taken into account in order to have a more realistic description of plant behavior, especially under such control actions (Olsson and Stephenson, 1985; Olsson and Jeppsson, 1994).

RESULTS

Overall behavior of the system

The calculations were performed, using 10 minute rainfall data of the Royal Meteorological Institute at Ukkel. The results of the simulations for 1986 will be presented. Figure 3 gives an overview of the distribution of the annual loads. The total flow volume generated by the system amounts to 29.1 Mm^3, of which 12.4 Mm^3 is stormflow. For the COD, 16% of the 9900 t produced and for the suspended solids 64% of the 9700 t originate from stormflow.

For the CSO option, the number of overflows ranges from 100 to 150, depending on the overflow. The average overflow duration is 6.7 hours or a total of 660 hours. This results in 27% of the flow volume which is sent to the river without treatment. For the COD (BOD) and for the suspended solids this fraction is 12 and 39%, respectively. If additional storage is implemented, these fractions reduce to 6, 2.5 and 9%. The average overflow duration increases to about 18 hours for the 7 overflow events. It should be noted, however, that the reduction of the emission for the 2 major overflow events amounts to less than 20%.

As a consequence of the sedimentation in the storage basins – 10% of the COD and 30% of the suspended solids is retained in the basins – the total mass influent to the WWTP is almost the same for both scenarios.

With the increase of the flow that passes through the WWTP under the BAS scenario, a global decrease in the treatment efficiency can be observed: while ca. 6% of the COD/BOD and 4% of the suspended solids were found in the effluent of the WWTP under the CSO option, respectively, 10 and 6% are found for the BAS option. The global efficiency of the latter option with respect to the combined overflow – WWTP emissions remains however the best: the COD/BOD emissions are reduced from 18 to 12% and the suspended solids emissions from 43 to 15%.

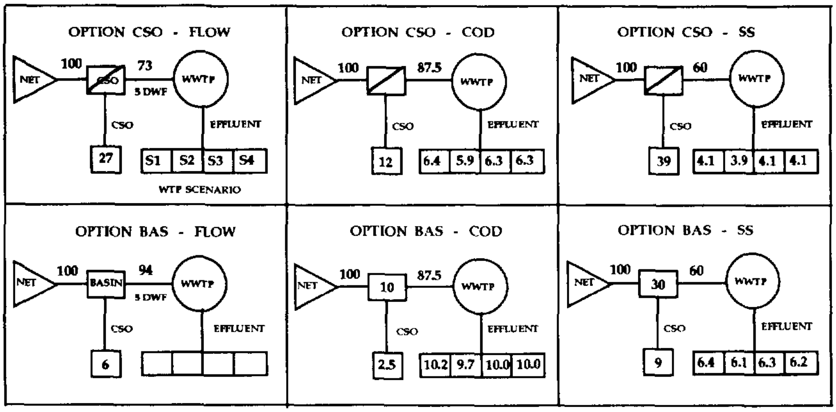

Figure 3. The yearly distribution of the masses.

The different operating scenarios at the level of the WWTP have (surprisingly) little effect on total or peak flows of the effluent : only 5% difference is observed between the effluent loads of extreme scenarios. It must be noted, however, that the alternative scenarios will have their most beneficial effects in situations where the final clarifier is nearly overloaded. This is not the case with the treatment plant considered here.

A more detailed analysis of the WWTP behavior under transient conditions

Overall figures such as mean or extreme values may provide a biased view on the performance of the system. Indeed, one can imagine that the maxima are identical because the plant behavior for all scenarios is similar for one particular event that surpasses the capacity of the WWTP. Mean values may be governed mainly by performance during DWF, so that little differentiation can be made between operating scenarios. In contrast, detailed analysis of the concentration and mass flux frequency distributions (e.g. Fig. 4) show some trends. It should be noted that those trends may be amplified considerably for a less efficient treatment plant than the one used in this study.

At the level of the sewage flow, the distributions show a clear effect of the inclusion of reservoirs in the sewer system (Fig. 4). For the BAS option, the frequency of 5 DWF inflow is increased significantly compared to the CSO option, due to the depletion of the reservoirs at this rate. This will affect the WWTP performance after rain events, as this operation will induce sustained bypass of 2.5 DWF of wastewater receiving only physical treatment. Operation at the treatment plant according to scenario 2 will allow for temporary storage of such storm waters at the plant. This ensures that the occurrence of 5 DWF loads is decreased with a concomitant increase in 2.5 DWF loads, partly due to depletion of the storm tank installed at the treatment plant (which also leads to an inflow of 2.5 DWF to the aeration tanks). Hence, scenario 2 significantly decreases the amount of wastewater that receives no biological treatment.

However, in the case of longer rain periods (e.g. Fig. 5), this operating scenario is less effective. Because emptying of the sewer reservoirs occurs mainly during DWF conditions, it is evident that the corresponding entries in the distribution have decreased. Overall, a shift of the frequency distribution to higher influent flow rates is observed for the BAS option, leading to a higher hydraulic load of the treatment plant.

Combining this effect with the result that a similar waste load is sent for biological treatment (see above), one can conclude that more dilute wastewaters have to be treated at a shorter residence time.

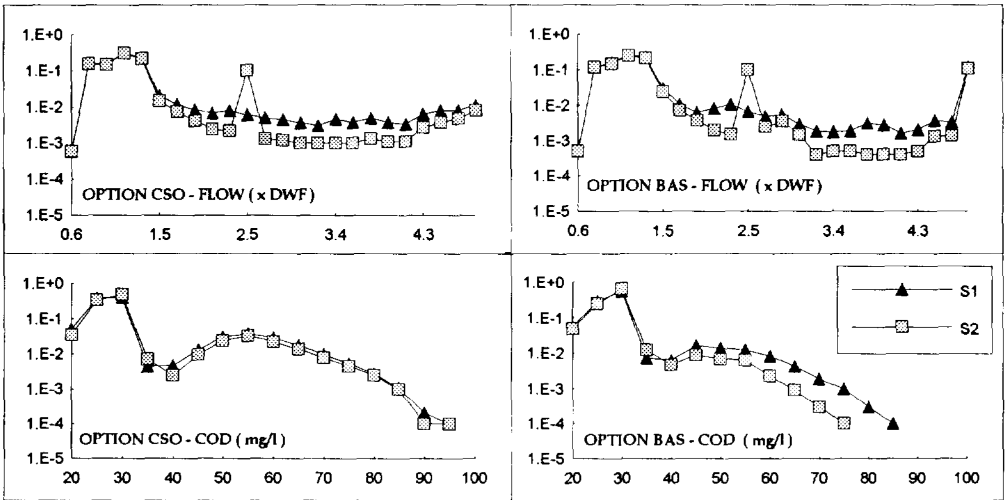

Figure 4. Frequency distributions of the flows and the COD concentrations of the effluent.

The comparison between the sewer designs shows a trend of a better effluent quality for the CSO option. This is not surprising, as the hydraulic loading and bypass frequency for the BAS option is higher. This induces a higher frequency of low quality effluents. Operation of the treatment plant according to scenario 2 alleviates this, as the bypass frequency is reduced substantially (see above). Again, longer rain periods will reduce the beneficial effect of this scenario.

It was found that the ratio control of RAS (scenario 4) has a beneficial effect on the lower range of effluent concentrations. This may be explained by the stabilizing effect ratio control has on the MLSS concentration in the aeration tanks, which should decrease the variability in biodegradation capacity (up to 25 per cent) observed for the other operating scenarios at higher flow rates and therefore shorter hydraulic retention times. This effect is most apparent in the low COD range.

The results of the effluent DO concentrations confirm the increased loading of the plant for the BAS option. However, for the CSO option, peak loadings result in an increased frequency of DO concentrations lower than 1 mg/l. The smoothing effect of the storm tanks is hereby illustrated. The CDF curves given on Fig. 6 show the occurrence of DO-drops below certain limits. The main observation made is that approx. 10 events occur in the simulated period for which the DO concentration in the effluent drops below 4 mg/l for longer than one day. This is again a result of the fact that the emptying of the storm reservoirs gives rise to sustained bypassing of (zero DO) wastewater to the river. It should be noted that interpretation of these data has to be made with care, as the DO control system also affects the DO readings. In fact, only a drop below 2 mg/l for a longer period of time is significant as this indicates that insufficient aeration capacity is available to treat all waste.

When one takes a closer look at the mass fluxes discharged, very similar conclusions and interpretations can be extracted from the frequency charts. A particular result appears in the fluxes of discharged oxygen in case of scenario 2 where a particular mass flux occurs at a considerably high frequency. This peak is caused by the depletion of the storm tank into the treatment plant at a reasonably low waste and maximum hydraulic loading resulting in a high DO mass flux.

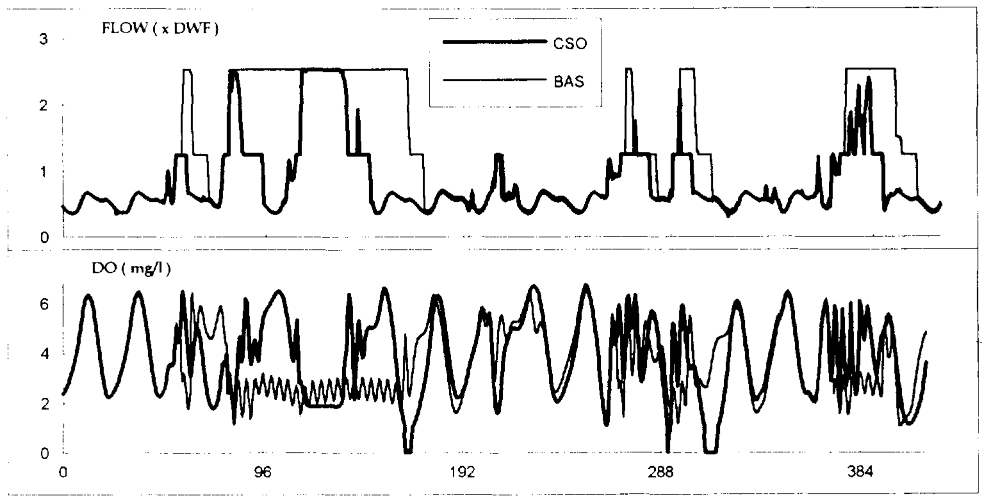

Figure 5. Evolution of effluent flow and DO concentration during a long rain event (time in hours).

Finally, it should be noted that the COD remaining in the effluent from the WWTP is mainly due to suspended solids. The analysis can therefore be reduced to one of both. This again points out that the function of the clarifier is one of the bottlenecks in wastewater treatment and it is good to remind the reader that models for this are still not well established.

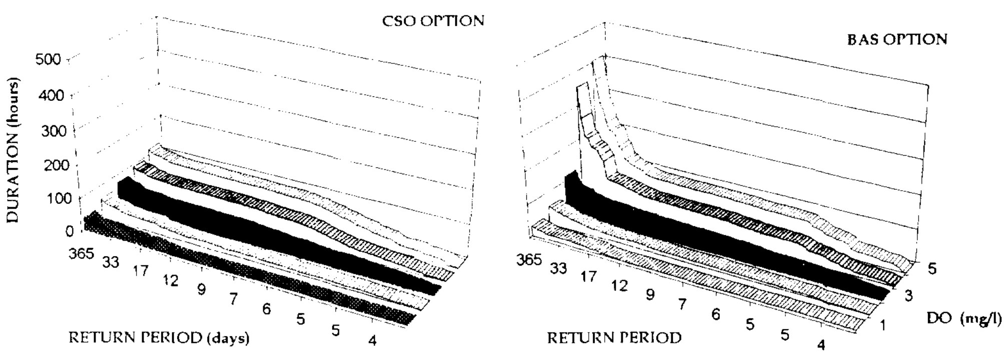

Figure 6. DO concentration-duration-frequency curves for scenario 1.

DISCUSSION AND CONCLUDING REMARKS

The results presented form only a part of a feasibility analysis of an integrated methodology for the evaluation of effects of (urban) waste disposal on the receiving water quality. The scenarios, hypotheses and parameters used were therefore not selected to find an optimal control strategy for the given realistic – but virtual – urban catchment, but in the light of the above mentioned goal.

For the interpretation of the results, the combination of hypotheses and parameters should however be kept in mind. With regard to the sewer system, measured data were available for DWF, but literature data were used for the quality during rainfall runoff. Also, the storm washoff concentrations were taken as constant; no real first flush – in the sense of an increase of concentrations caused by washoff and/or the resuspension of sediments in the sewer – was considered. For the BAS option, on-line storage basins were chosen and the pollutants that settled in the basins were not accounted for. The storage volume was chosen in order to limit the overflow frequency to seven per year, for a depletion rate in compliance with the maximal trunk sewer capacity of five times the dry weather flow. For the WWTP, it should be remembered that it was designed for an influent capacity of 2.5 times the dry weather flow and that a relatively high efficiency was obtained, mainly as a result of the assumed settling characteristics of the sludge.

Under these assumptions, a major reduction of the CSO emissions is obtained through the installation of the storage basins in the sewer network, especially with regard to the suspended solids. Although this leads to an increase in the emissions from the treatment plant by about 50%, the net effect of the basins is still beneficial. It is also important to keep in mind that the basins have only a marginal effect on the – from the point of view of volumes – most important overflow events.

For the system considered, the global emissions for the different scenarios show only minor differences. The best overall scenario is the one with the storage tank (S2) which gets round the limitation of the capacity of the WWTP. With respect to this scenario, it should be mentioned that an alternative could consist of limiting the depletion rate of the basins to the WWTP capacity (and consequently adapting the volumes of the basins). The more complex management schemes for the WWTP do not prove to be globally superior to the S2 scenario. However, some trends in the concentration and mass flux distributions indicate that these schemes might be useful in the framework of a dynamic – influent driven – management of the WWTP under transient conditions. This could especially be the case if the settling characteristics of the sludge are less good. Also, these schemes could prove to be favorable with respect to the fate of NH_4, as this soluble compound is highly sensitive to the decreased reaction time that will occur due to hydraulic overloading (Durchschlag *et al.*, 1992). Further research has to be conducted along these lines of thought.

As mentioned in the introduction, this paper is limited to the investigation of the problem from an emission point of view. A difficulty hereby is that the comparison between different scenarios is not straightforward. What to say e.g. about the BAS option which yields an effective decrease of the yearly emission load and somewhat less extreme WWTP emission concentrations, but which leaves the major CSO events almost unaffected and leads to longer periods with 'mediocre' WWTP emissions? The integration of the problems – and consequently the actual risk assessment for the system – is only possible by looking at the effects of the proposed measures on the receiving waters.

The methodology hereto requires the use of simulation models for the analysis of the global system under various conditions, i.e. through continuous modeling. For the sewer system and the WWTP, the application of such models is technically possible. On a 486 microcomputer, the simulation for one year with the models described in this paper requires less than 30 minutes computation time for the sewer system and less than 2 hours for the WWTP. Investigations on the simulation of the pollutant propagation in the river show that the computation time may be estimated to be in the order of 24 hours. Although cumbersome – due to the extremely large amounts of data – the statistical analysis of the results does not represent a fundamental problem either. The importance of a profound analysis of the results for each subcomponent of the system should hereby be stressed.

By this, however, the authors do not pretend that the use of the models is straightforward. Several problems remain to be solved with regard to the modelling, including the conceptual representation of phenomena (e.g. surface washoff and sediment transport in sewers) and the parameters of the models (especially with regard to the biochemical reactions and the settling properties). With regard to the overall immission based methodology, a major problem is seen to be the translation of the imission characteristics (which are the relevant variables and what are the relevant stochastic characteristics of the series?) towards the environmental impacts.

ACKNOWLEDGEMENT

The authors wish to thank the Administration for Natural Resources and Environment of the Brussels Region, under which authority part of this study was conducted.

REFERENCES

Alarie, R.L., McBean, E.A. and Farquhar, G.J. (1980). Simulation modeling of primary clarifiers, *J. Environ. Eng.*, **106**, 293–309.

Andrews, J.F. (1974). Dynamic models and control strategies for wastewater treatment processes. *Wat. Res.*, **8**, 261–289.

Bakker, T., Timmer, J.L. and Wensveen, L.D.M. (1989). De vuiluitworp van gemengde rioolstelsels. Nationale Werkgroep Riolering en Waterkwaliteit, Min. Volkshuisvesting, Ruimtelijke Ordening en Milieubeheer, s'Gravenhage, the Netherlands, Report 8.5.

Bauwens, W., Vanrolleghem, P., Smeets, M. and Fronteau, C. (1995). An emmission based evaluation of the efficiency of the combined sewer – wastewater treatment system under transient conditions. Technical Report, Lab. Hydrology, VUB, Brussels, Belgium.

Degremont (1990). *Water treatment handbook.*

Durchschlag, A., Hartel, L., Hartwig, P., Kaselow, M., Kollatsch, D., Otterpohl, R. and Schwentner, G. (1992). Joint consideration of combined sewerage and wastewater treatment plants. *Wat. Sci. Tech.*, **26**(5–6), 1125–1134.

Grijspeerdt, K., Vanrolleghem, P. and Verstraete, W. (1994). Selection of one dimensional sedimentation models for on-line use. *Wat. Sci. Tech.*, **31**(2), 196–204.

Harms, R.W. and Kenter, G. (1987). Mischwasserentlastungen, KOSIM V.3.0, Mikrocomputer in der Stadtenwasserung, Institut fur technische-wissenschaftliche Hydrologie, Hannover, Germany.

Henze, M., Grady, C.P.L., Jr, Gujer, W., Marais, G.V.R. and Matsuo, T. (1987). Activated sludge model n° 1, IAWPRC Scientific and Technical Reports n°1, London, UK.

Jolánkai, G. (1992). Hydrological, chemical and biological processes of contaminant transformation and transport in river and lake systems: a state of the art report. Technical documents in hydrology, IHP, UNESCO, pp. 99.

Lessard, P. and Beck, M.B. (1988). Dynamic modeling of primary sedimentation. *J. Environ. Eng.*, **114**, 753–769.

Lessard, P. and Beck, M.B. (1990). Operational water quality management: control of storm sewage at a wastewater treatment plant. *J. Wat. Pollut. Control Fed.*, **62**, 810–819.

Olsson, G. and Jeppsson, U. (1994). Establishing cause-effect relationships in activated sludge plants – what can be controlled? *In: Proc. Workshop Modelling, Monitoring and Control of Wastewater Treatment Plants.* Med. Fac. Landbouww. Univ. Gent, 59, 2057–2070.

Olsson, G. and Stephenson, J.P. (1985). The propagation of hydraulic disturbances and flow rate reconstruction in activated sludge plants. *Environ. Techn. Lett.*, **6**, 536–545.

Smeets, M., Raemdonck, N. and Bauwens, W. (1995) A methodology to reduce CSO with additional storage capacity. *In: Integrated Managements of Urban Environments, Proc. 2nd Int. Symp. on Urban Environments, Melbourne, Australia,* Vol. 2, 329–334.

Takács, I., Patry, G.G. and Nolasco, D. (1991). A dynamic model of the clarification-thickening process. *Wat. Res.*, **25**, 1263–1271.

Tyson, J.M., Guarino, C.F., Best, H.J. and Tanaka, K. (1993). Management and institutional aspects. *Wat. Sci. Tech.*, **27**(12), 159–172.

ULB (1992). Réseau de surveillance des écoulements et des charges polluantes dans les collecteurs d'amenée à la future station d'épuration Bruxelles-Nord, Lab. de traitement des eaux et pollution, Université Libre de Bruxelles, Belgium.

VUB (1992). Studie voor het beheer en de modellering van het toekomstig waterbehandelingssysteem van het rioolstelsel Brussel-Noord, Dienst Hydrologie, Vrije Universiteit Brussel, Belgium.

Van der Graaf, J.H.J.M. (1976). Laten biologische zuiveringsprocessen zich naar temperatuur optimaliseren, H2O, 9, 87-93.