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EVALUATION OF DESIGN AND OPERATION OF THE SEWAGE TRANSPORT AND TREATMENT SYSTEM BY AN EQO/EQS BASED ANALYSIS OF THE RECEIVING WATER IMMISSION CHARACTERISTICS

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INTRODUCTION

It is evident to all those involved in urban water management that three important units need to be considered to assess the performance of an urban drainage system: the sewer, the treatment plant and the receiving water. The sewage transport and treatment system have to be designed and operated with respect to the objectives of the investments made: prevent flooding, ensure hygienic quality in the populated area and maximize the quality of the receiving water. It is, however, also evident that the complexity and size of the whole system lowers the tractability of holistic performance evaluations.

Partly because of the complexity and partly because of the heritage of the stepwise development in urban drainage, urban water management is currently typically subdivided into an optimization of the subsystems individually. Necessarily, such subdivision creates interfaces between components. Consequently, a need arises to translate the objectives for the overall system in local requirements (and operational standards) at these interfaces. Examples of such local subgoals are a limitation to the yearly number of combined sewer overflows (CSOs), storage capacity per impervious area in the catchment, wastewater treatment plant effluent quality standards, a required dilution ratio, etc.

In this subdivision approach, possible interactions between the components become neglected, inducing a too important simplification of the system. For instance, it is important to be able to judge whether providing less storm water retention, inducing more frequent overflows, is more harmful to the recipient than increasing storage volumes and in this way impose longer high flow periods to the treatment with negative effects on its performance.
It is also worthwhile to reflect on the fact that the abovementioned interface definitions have been generated from the perspective of engineers specialized in the design and operation of the subsystem they are responsible for. This induces that objectives become technology driven, resulting in BATNEEC approaches (Best Available Technologies Not Entailing Excessive Costs). In addition, this perspective is emission-geared, i.e. the aim is to minimize what is expelled from the system one is responsible for, not to minimize the impact on the receiving system (note that this would ask for in-depth knowledge of that system too). As a side-effect the resulting emission standards are uniform, i.e. they do not consider local circumstances of the recipient and, for instance, its interactions with neighboring subsystems. This approach has been termed UES (uniform emission standards) and has been accepted in most European countries.

In the alternative approach, termed EQO/EQS (Environmental Quality Objective/Environmental Quality Standard) (Tyson et al., 1993), the objectives with respect to the use of the recipient (e.g. fishing, bathing, drinking water production) are set out first and, subsequently, local, temporal and operational standards are defined to ensure compliance with the defined objectives. Evaluation according to this strategy is inherently holistic since it has to assess the impact of all components concerned and, consequently, the interactions between them. Moreover, since the impact on the recipient is focused upon, this approach is clearly immission-based.
A particular feature of practical importance for this approach concerns the temporal detail and time horizon that needs to be covered during the impact assessment. On the one hand a long time horizon is necessary because the history of the individual components has an impact on their behavior that cannot be neglected, e.g. the accumulation of sediments in sewers and rivers. On the other hand, certain events such as CSOs are

characterized by small time constants and need detailed short-term evaluations. As a result, large time series of simulation results are compiled, inducing a need for data reduction techniques to make interpretation feasible. Extreme statistics, in the form of concentration-duration-frequency (CDF) plots of the immission characteristics have been proposed to allow for an analysis of water quality problems in agreement with the EQO/EQS approach.

In this contribution the EQO/EQS approach is adopted and an effort is presented to develop a methodology that makes this approach feasible. All is illustrated on the basis of a realistic case study.

METHODOLOGY

To obtain the functional, spatial and temporal integration required for a comprehensive drainage study as the one strived for here, the use of hydrologic, hydraulic and quality simulation models becomes imperative. For the reasons given above, long and detailed time series of the immission characteristics of the receiving water must be generated and analyzed by means of concentration-duration-frequency (CDF) curves.

To illustrate the potential of the methodology, a study was performed in which different pollution abatement scenarios were compared for a particular catchment (Figure 1). At the level of the sewage transport system, designs without and with CSO control infrastructure were considered: options CSO and BAS. In the CSO option, an overflow structure at the end of each collector discharges excess flow to the river, while in the BAS option on-line storage basins are placed at the same outlets to limit the overflow frequency to 7 per year. For the wastewater treatment plant, four possible operating scenarios were compared. The simulation study evaluates the effect of potential control strategies such as by-pass (S1), retention of first flush in a storm tank (S2), step feed to the aeration tanks (S3) and flow ratio control of the return active sludge (S4).

Fig. 1. Schematic representation of the sewer system, treatment plant and river stretch.

The impact of the effluents on the river system in either scenario is studied. Time series of flows and concentrations from both sewer system and WWTP are used as input to a continuous river model in order to consider the problems from an immission point of view. Only the results of the simulations for 1986 will be discussed here.

The sewer component

For prediction of the flows and concentrations of sewage discharged at the CSO structures and into the treatment plant, the continuous simulation model KOSIM (Harms and Kenter, 1987) was selected. In this model, the sewer system is represented by a number of reservoirs connected in series or parallel. The sewer network, taken on the basis of a part of the Brussels urban drainage system, could be schematized into 50 subbasins using previous detailed hydraulic simulations and differences in land use. In this way internal storage and transport time effects could be taken into account.
A conceptual rainfall-runoff model transforms the 10 minute rainfall data obtained from the Royal Meteorological Institute at Ukkel (near Brussels) into a flow series for the subbasin. The total drainage area is about 5400 ha, one third of which is impervious. Storm flow calculations accounted for wetting losses (0.5 mm), depression storage loss (1.8 mm) and a time dependent runoff coefficient (between 0.25 and 1.00). A population of approximately 300.000 is connected. Pollutant inputs during dry weather conditions were obtained experimentally from in situ measurements of quantity and quality variables (VUB, 1992; ULB, 1992). Daily average flows and loads of BOD, COD and suspended solids are 45000 m^3 and 13, 28 and 10 tons, respectively. Variations indicated by max/min ratios are 1.85 for flows and 6 for the pollutant loads. For storm conditions constant wash-off concentrations were taken from Jolankai (1992) : 0.06, 0.13 and 0.5 g/l for BOD, COD and suspended solids respectively. From a comparison of the resulting emission characteristics at the overflows with data obtained in the Netherlands (Bakker et al., 1989), it could be concluded that the simulation data are realistic (Smeets et al., 1995).
Although the model is capable to take erosion and deposition processes into account in the subbasins, no use of it was made in the study due to lack of data. Within the system, flows and pollutants are conveyed downstream based on a constant travel time. Weirs and different types of storage basins can be included.

In the first option of the sewer system, termed CSO, no storage basins were included. In the BAS option on-line storage was installed at the outlet of the five collectors into the trunk sewer that leads to the treatment plant. Smeets et al. (1995) calculated that for the given system nearly 200.000 m^3 additional storage volume was needed to ensure a maximum of 7 overflows per year. The hydraulic calculations for basins and weirs are based on the continuity equation, on maximal flow capacities and on stage-discharge relations. In storage basins, settling is described by traditional sedimentation theory. 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). No interactions between pollutants and/or the sediment are considered.

The wastewater treatment component

Biodegradation of the pollutants in the sewage conveyed to the treatment plant (expressed as chemical oxygen demand, COD) was calculated using a dynamic model describing the behavior of a typical activated sludge wastewater treatment plant. The sewage treatment works consists of primary clarification, 3 completely mixed aeration tanks in series where biotransformation occurs and a final clarifier where the biosolids are separated (Figure 1). The maximum influent flow rate of 5 times dry weather flow (DWF) was imposed by the sewer system design. Because the effluent quality, i.e. the combination of clarifier effluent and occasional treatment plant bypass, is especially endangered under transient flow conditions, special attention was drawn in the study to possibilities for plant operation that minimize storm event effects. Figure 1 schematizes the four scenarios that were compared:

Scenario S1.	Base Case with bypass and constant recycle flow rate
Scenario S2.	Storm Tank for retention of first flush
Scenario S3.	Step Feed of influent to the aeration tanks
Scenario S4.	Ratio Control of the Recycle Flow Rate

The 300.000 PE treatment facility incorporated in the simulated system was based on traditional design rules. 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 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 which induces oscillations in DO level.

To describe the dynamic behavior of the aeration tanks, a biotransformation model derived from the state-of-the-art IAWQ model nr 1 was used (Henze et al., 1987). Nitrification and denitrification processes were omitted from the model because nitrogen was not considered in this stage of the study. Together with the information on wastewater temperature (obtained from an experimentally verified correlation with air 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 temperature dependency of oxygen mass transfer and biological activity was modeled in the traditional way.

The basic 5-layer model of Lessard and Beck (1988) was used as a starting point to describe the dynamic behavior of the primary clarifier. At DWF a hydraulic retention time of 2 hours is obtained, resulting in a 1.8 m/h surface loading of the primary clarifier. A maximum of 5 DWF is passed through the primary clarifiers. Of this flow 2.5 DWF is sent to the aeration tanks for biological treatment. As indicated by Lessard and Beck (1988) the settling velocity of the transported particulate material increases as the flow in the sewer is increased during storm conditions (as a result of resuspension). The Lessard/Beck model was modified to include scouring as given in Alarie et al. (1980). In the primary clarifier hydrolysis of the slowly biodegradable particulate fraction was also considered and described as a first order process.

For the secondary clarifier, a hydraulic retention time of 6 hours and a design surface loading of 0.6 m/h under DWF was chosen. Motivated by the evaluation of settler models by Grijspeerdt et al. (1994), the 10-layer one-dimensional final clarifier model of Takacs et al. (1991) was adopted. The inlet was positioned at one fourth of the total clarifier height. Normal operation included a constant sludge recycle flow rate defined by a recycle ratio of 25 percent of the mean DWF. The constant waste flow rate was set to 0.75 percent of the mean DWF. With a simulated mean 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 percent of the total clarifier height.

Some more details on the operating scenarios is given next (see also Figure 1). In scenario 2, a storm tank with a volume of 6 hours DWF is installed after the primary clarifiers. Whenever the outflow of the primary clarifiers exceeds 2.5 DWF, the excess is diverted into the storm tank. If the tank is full, the excess is bypassed to the river. As soon as the primary clarifier effluent flow rate drops below 2.5 DWF, the storm tanks are being emptied at a rate such that the max. inflow capacity to the aeration tanks is used (2.5 DWF).
For temporary decrease of the secondary clarifier sludge loading during storm events it is considered valuable to initiate step feeding (scenario 3). Essentially it allows to temporarily redistribute sludge between the final settler and the aeration tanks (Olsson & Jeppsson, 1994). To this end influent is distributed (in this study equally) over the different aeration tanks instead of all being entered into the first one.
Traditional ratio control of the recycle flow rate is evaluated in scenario 4. In this operation mode, the recycle flow rate is no longer constant but is varied proportionally to the influent flow rate (Andrews, 1974). In the simulations, the proportionality constant was set to 0.25.

The river component

In this illustrative case study dissolved oxygen (DO) was considered to be the most relevant immission characteristic for the assessment of the impacts on the receiving water. The SALMON-Q water quality model (HR Wallingford, 1994) was used to predict DO time series in a part of the river Zenne. The river was divided into 18 elements of approx. 850 m in each of which the necessary equations were solved (Figure 1). The hydrodynamic part of the model is based on the mass and momentum conservation laws of de Saint-Venant. Pollutant transport is calculated with one-dimensional advection-dispersion equations.
Inputs to the model consisted of the output of the models given above, i.e. 10 minute interval time series of discharge, temperature, suspended solids, particulate and dissolved BOD (biochemical oxygen demand) and DO from the CSO and WWTP effluents. Second, upstream discharge series and downstream level series are necessary as boundary conditions of the model. Initial conditions, parameters and supplementary boundary conditions have been obtained from the literature and by previous modeling studies on the river Zenne. Temperature influences have been included. No sediment transport has been modeled because of insufficient data.

RESULTS

Relative contributions of the emissions from the sewage transport and treatment systems

In Table 1 the relative contributions of combined sewer overflows and WWTP to the annual emissions into the river are compiled. The total flow volume generated by the system amounts to 29.1 Mm^3, of which 12.4 Mm^3 is storm flow. Depending on the overflow structure considered the number of overflows ranges from 100 to 150 for the CSO sewer option, leading to 27 percent of the sewage not being treated. The average overflow duration is 6.7 hours. With additional storage (BAS option) the average overflow duration increases to about 18 hours for the 7 overflow events, but only 6 percent of the wastewater remains untreated. It should be noted, however, that the reduction of the emission for the 2 major overflow events amounts to less than 20%.

Due to sedimentation in the storage basins - 10% of the COD and 30% of the suspended solids is retained in the basins -, the total pollutant load to the WWTP is nearly the same for both sewer designs. It means that, as more wastewater volume is being treated under the BAS option, the mean pollutant concentration in the influent to the WWTP is lower for this option. This flow increase towards the WWTP also causes a global decrease in the treatment efficiency as indicated in the effluent quality results (Table 1). Surprisingly little benefits could be found for the different operating scenarios of the WWTP: only 5% difference can be observed between the effluent loads of extreme scenarios. The conservative settling properties chosen in the sedimentation model of the activated sludge seem to have masked the beneficial effects of the proposed control actions.
A main result of the study is that the global efficiency of the BAS option with respect to the combined overflow - WWTP emissions remains the best : the COD/BOD emissions are reduced from 18 to 12% and the suspended solids emissions from 43 to 15% of the total generated in the catchment. The sensitivity of this result to, for instance, the settling properties of the sludge is focus of current research.

Table 1. The yearly distribution of the masses (in %) for the different sewer options and WWTP operating scenarios

The behavior of the wastewater treatment plant under transient conditions

Detailed analysis of the concentration and mass flux time series (e.g. Figure 2a and b) obtained from the simulation model is hampered by the size of the datasets ("data drowning"). Frequency distributions as given in Figure 3 can help to understand the behavior by pointing to some trends, that can subsequently be studied in more detail at the level of the original time series.

Fig.2. Time evolution of a) effluent flow and b) DO concentration during a sequence of rain events.

Inclusion of reservoirs in the sewer system (CSO versus BAS) and installation of a storm tank (S1 versus S2) clearly modifies the effluent flow frequency distributions (Figure 3a). Because the BAS reservoirs are emptied at a flow rate of 5 DWF this specific entry in the distribution increases. Evidenly, this will affect the WWTP performance for sustained periods after a rain event, even leading to a constant bypass of 2.5 DWF of sewage. A sewer reservoir emptying rate of 2.5 DWF would possibly be more appropriate but would increase the risk of insufficient on-line storage capacity for a next rain event. Overall, a shift of the frequency distribution to higher influent flow rates is observed for the BAS option because sewer reservoir emptying mainly occurs when dry weather conditions prevail,so that the corresponding DWF entries in the distribution decrease. 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.
Treatment plant operation according to scenario 2 allows for temporary storage of storm waters at the plant that can receive biological treatment after the event. Under this scenario, the occurrence of 5 DWF loads is decreased with a concomitant increase in the distribution entries at 2.5 DWF loads. This is due to the emptying procedure of the storm tank that maintains the max. inflow rate of 2.5 DWF to the aeration tanks.

Fig.3. Frequency distributions of a) effluent flows and b) effluent COD concentrations for CSO (top) and BAS (bottom) sewer options.

For the effluent COD distributions (Figure 3b) the comparison between the two sewer designs reveals better effluent quality for the CSO option, i.e. the entries with higher concentrations are present at lower frequencies. This is not surprising, as the hydraulic loading and bypass frequency for the BAS option is higher. Treatment plant operation according to scenario 2 for the BAS sewer design alleviates this, because the need for plant bypass during reservoir emptying can be reduced (see above). In contrast, scenario 2 has a less pronounced beneficial effect for the CSO option.
Ratio control of the return sludge flow rate (scenario 4) was found to be beneficial at the lower range of effluent concentrations. An explanation can be found in the stabilizing effect this type of control has on the sludge concentration in the aeration tanks, maintaining a more constant biodegradation capacity.

The simulated effluent DO time series (Figure 2b) confirm the increase of the mean loading of the WWTP under the BAS option, i.e. a lower mean dissolved oxygen concentration is predicted. On the other hand, for the CSO option, the more frequent peak storm loadings result in an increased frequency of DO concentrations lower than 1 mg/l. The smoothing effect of the on-line storm reservoirs is hereby illustrated.
In 1986, the simulated period, ten events occur for which the effluent DO concentration under the BAS option drops below 4 mg/l for longer than one day. Again, the emptying procedure of the storm reservoirs is at the basis of this because it results in sustained bypassing of zero DO wastewater. Interpretation of these data has to be made with care since the DO control system also affects the DO readings. In fact, a drop below 2 mg/l for a longer period of time indicates a severe problem, i.e. insufficient aeration capacity is available in the treatment plant.

The river

At the level of the river flows, it is found that peak discharges remain lower and are less frequent under the BAS option. This again points to the smoothing effect of the on-line storage facilities. On the other hand, it is important to stress the point that these basins have a limited effect on the most important storm events that still give rise to overflow. Indeed, the two most important peak discharges over the year 1986 are more or less identical for both options.

Upstream of the WWTP, the DO levels clearly benefitted from the presence of on-line storage (Figure 4, middle). This is obvious as more water is conveyed to the sewage works and the associated pollution is therefore bypassing the corresponding river elements. However, even at these locations the situations most harmful to river life do not differ much between both sewer options: as for the flows the most extreme DO depletions are caused by storms that are not being controlled under the BAS option.
An interesting phenomenon was found during a period at the end of August 1986 (Figure 4). The period is characterized by a succession of overflow events (under the CSO option) that are typically not classified as being extreme. The self-purification capacity of the river was insufficient to restore the dissolved oxygen balance due to a combination of low base flow and high temperature. Hence, this succession lead to a critical situation in the river for nearly 2 weeks. For the river system this period was classified as the most extreme event of the year 1986.
It is important to note that for similar successions of overflows during other seasons, the resulting low DO level is found to very rapidly climb to higher values and therefore does not lead to a critical situation. Similarly, the most extreme storm events (as the one near September 15th) can lead to fast recoveries of the river quality. The events of August and September illustrate very clearly that a combination of factors that influence the water quality of the river must be accounted for when the effect of overflows is analyzed. The conclusion can be drawn that it is dangerous to focus on extreme overflow events only to assess the performance of an urban drainage system.

Downstream of the treatment plant, it is evident that the difference between both sewer design options decreases (Figure 4, bottom) due to the worse treatment plant performance under the BAS option. However, (at least for this period) the BAS option keeps an edge on the CSO option as was expected from the emission results discussed above. CDF curves (plotting the frequency of periods that a variable remains below a critical value for a certain duration) were calculated from the dissolved oxygen time series at this location (Figure 5). Comparison of the graphs show that critical situations (DO < 4mg/l) occur more often and last longer under the CSO option. It can be concluded that the BAS option remains beneficial even downstream from the WWTP. Few differences could be noticed when considering the different operating scenarios of the treatment plant. As could be expected from the interpretation of the plant's behavior, slightly better results are achieved under scenario S2.

Fig.4: Time series for river flow (top) and DO upstream (middle) and downstream (bottom) of the WWTP for sewer CSO and BAS options

Fig.5: CDF curves for DO concentrations downstream of the treatment plant under CSO (left) and BAS (right) options

DISCUSSION

When interpreting the results presented in this paper, it is important to keep the set of hypotheses, assumptions and parameters in mind that originated from the objective of the study. The investigations were conducted to assess the feasibility of an integrated methodology that allows to evaluate effects of (urban) waste disposal on 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.

For the simulated system, a major reduction of CSO emissions (especially with regard to the suspended solids) can be obtained through the installation of storage basins in the sewer network. While the sewage work's performance suffers as a result of the increased wastewater volume being subject to treatment, the net effect of on-line storage reservoirs is still beneficial. An important result of the study was that these investments had only marginal effects on the most important CSOs.

The different WWTP operating scenarios that were evaluated had only minor effects on the effluent quality. As for the sewer system, installing a storm tank (scenario S2) allowed to alleviate the effect of peak loads and decrease the emissions during storm events. Trends in the concentration frequency distributions indicate that the other operating schemes (step feed and ratio control of return sludge flow rate) 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 than in the model used here.

To emphasize the interaction between sewer system and treatment plant, it is good to mention the problem caused by the emptying rate of the sewer basins that does not consider the internal working of the treatment plant, i.e. half of the volume of the stored wastewater conveyed to the plant cannot be treated biologically due to hydraulic limitations. An alternative could consist of limiting the depletion rate of the basins to the WWTP capacity (and consequently adapting the volumes of the basins).

As mentioned in the introduction, this paper is aimed to investigate the problem from an immission point of view. It is the use of the river quality model that allows to answer questions like: what to say about the BAS option which yields an effective decrease of the yearly emission load and somewhat less extreme emission concentrations, but which leaves the major CSO events quasi unaffected and leads to longer periods with 'mediocre' WWTP emissions ? Interactions between subsystems were illustrated and it could be concluded that the extreme events at treatment plant or sewer overflows are not necessarily the ones to consider when evaluating the receiving water quality. 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.

Simulation models are essential for the analysis of the global system. The necessity to evaluate the system under various conditions makes detailed simulations for long periods of time imperative. For the considered system, the application of such models is technically possible. On a PC-486/66 microcomputer, the simulation for one year with the models described in this paper requires less than 30 minutes computation time for the sewer system, less than 2 hours for the WWTP and approx. 30 hours for simulation of the pollutant propagation in the river. While being cumbersome - due to the extremely large amounts of data - the statistical analysis of the simulation results does not represent a fundamental problem either. A profound analysis of the results for each subcomponent of the system is found important for the interpretation of the results and the concomitant understanding of the behavior of this complex system.

The authors do not pretend that the use of the models is straightforward. Several problems remain to be solved with regard to the modeling, including the conceptual representation of phenomena (e.g. surface wash-off and sediment transport in sewers and rivers) and the parameters of the models. In addition, the immission based methodology proposed here is still highly dependent on an adequate translation of the environmental quality objectives towards operational immission characteristics that can be calculated with the models.

CONCLUSIONS

Continuous simulations for the integrated sewer-WWTP-river system have been carried out and the efficiency of different sewer and WWTP scenarios has been evaluated from an immission point of view. The following conclusions can be formulated.

The installation of storage basins does not prevent important storm events to give rise to overflow. A major reduction of the CSO emissions is obtained though. Even if the BAS option leads to an increase in the emissions from the WWTP, the net effect of the basins remains beneficial. Although few differences have been noticed between the different WWTP control strategies, the scenario with the storage tank (S2) gets around the limitation of the capacity of the WWTP.
It was shown that it is very important that the combination of factors, influencing the water quality of the river, is accounted for when analyzing the effect of overflows.
On a 486 PC (66MHz), the simulation of one year with the models described in this paper is feasible. While the large amounts of simulation data make their statistical analysis cumbersome no fundamental problem remains.
A number of problems still remain however at the level of the modeling of the mechanisms of the components

and with regard to the overall immission based methodology, where a major problem is seen to be the translation of the environmental objectives into operational immission characteristics that can be calculated within the proposed methodology.

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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). Review paper: 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, Ministerie Volkshuisvesting, Ruimtelijke Ordening en Milieubeheer, s'Gravenhage, the Netherlands, Report 8.5. (In Dutch).

Bauwens W., Vanrolleghem P. And Smeets M. (1996). An evaluation of the efficiency of the combined sewer-waste water treatment system under transient conditions. Wat. Sci. Tech. (In press).

Degremont (1990). Water treatment handbook.

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, Institüt für Technische-wissenschaftliche Hydrologie, Hannover, Germany. (In German).

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.

HR Wallingford (1994). Salmon Q, version 1.01, User Documentation. Hydraulic Research, Wallingford, 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.

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

Smeets M., Raemdonck N. and Bauwens W. (1995). A methodology to reduce CSO with additional storage capacity. In: Integrated Managements of Urban Environments, Proceedings 2nd International Symposium 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.

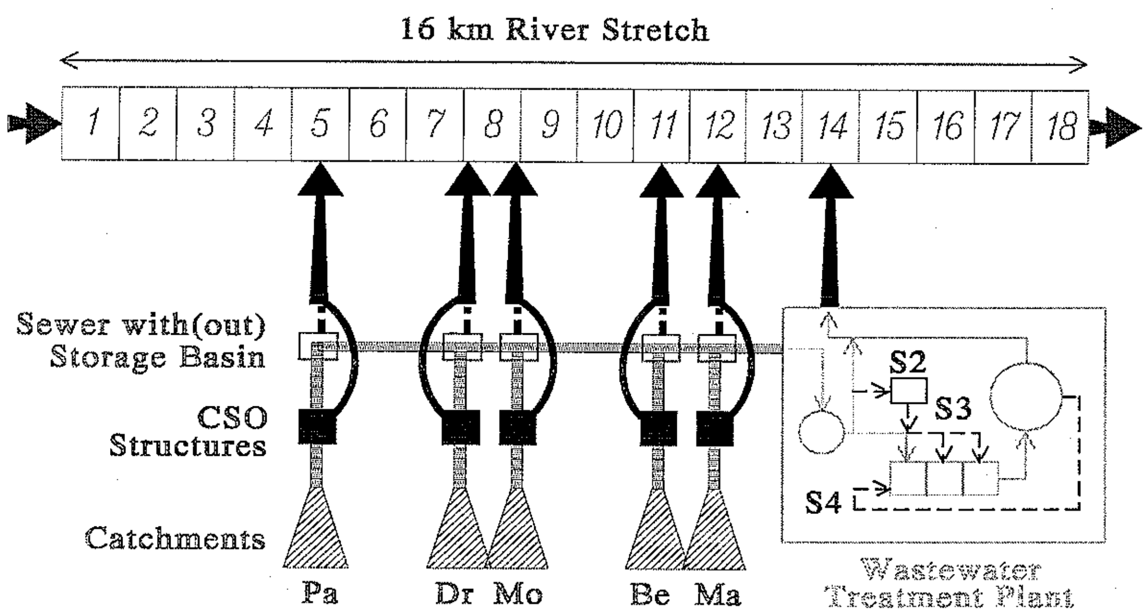
Tyson J.M., Guarino C.F., Best H.J. and Tanaka K. (1993). Management and institutional aspects. *Wat. Sci. Techn.*, **27(12)**, 159-172.

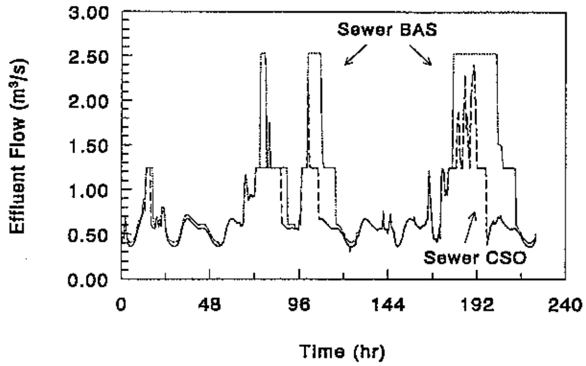
ULB (1992). Réseau de surveillance des écoulements et des charges polluantes dans les collecteurs d'amenée à la future station d'épuration Bruxelles-Nord. Lab. de traitement des eaux et pollution, Université Libre de Bruxelles, Belgium. (In French).

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

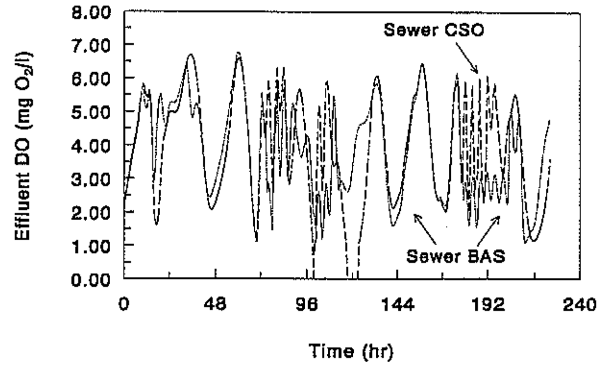
van der Graaf J.H.J.M. (1976). Laten biologische zuiveringsprocessen zich naar temperatuur optimaliseren, *H2O*, **9**, 87-93.(in Dutch).

		Option CSO				Option BAS			
		S1	S2	S3	S4	S1	S2	S3	S4
Flow	CSO	27.0				6.0			
	Effluent	73.0				94.0			
	Total	100.0				100.0			
COD	CSO	12.0				2.5			
	Effluent	6.4	5.9	6.3	6.3	10.2	9.7	10.0	10.0
	Total	18.4	17.9	18.3	18.3	12.7	12.2	12.5	12.5
SS	CSO	39.0				9.0			
	Effluent	4.1	3.9	4.1	4.1	6.4	6.1	6.3	6.2
	Total	43.1	42.9	43.1	43.1	15.4	15.1	15.3	15.2

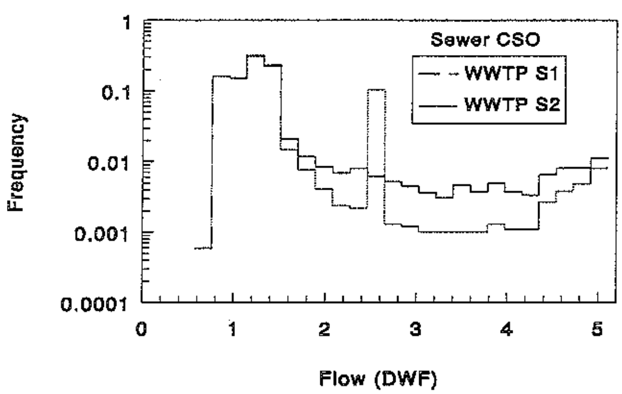




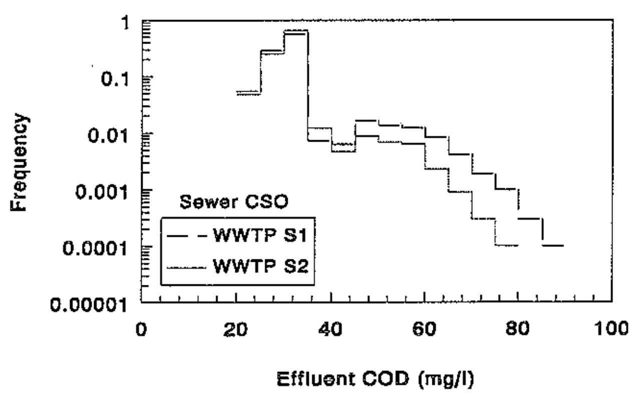
Sewer BAS
Sewer CSO
Effluent Flow (m^3/s)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0
48
96
144
192
240
Time (hr)



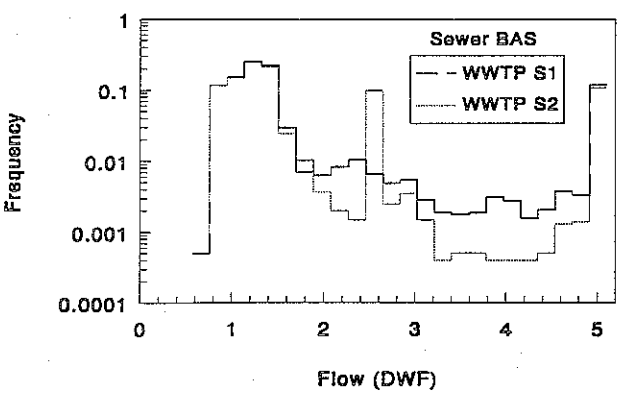
Sewer CSO
Sewer BAS
Effluent DO (mg O_2/l)
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
48
96
144
192
240
Time (hr)



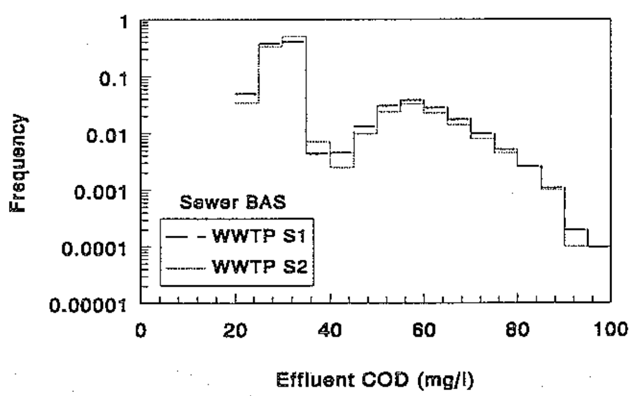
Sewer CSO
WWTP S1
WWTP S2
Frequency
1
0.1
0.01
0.001
0.0001
0
1
2
3
4
5
Flow (DWF)



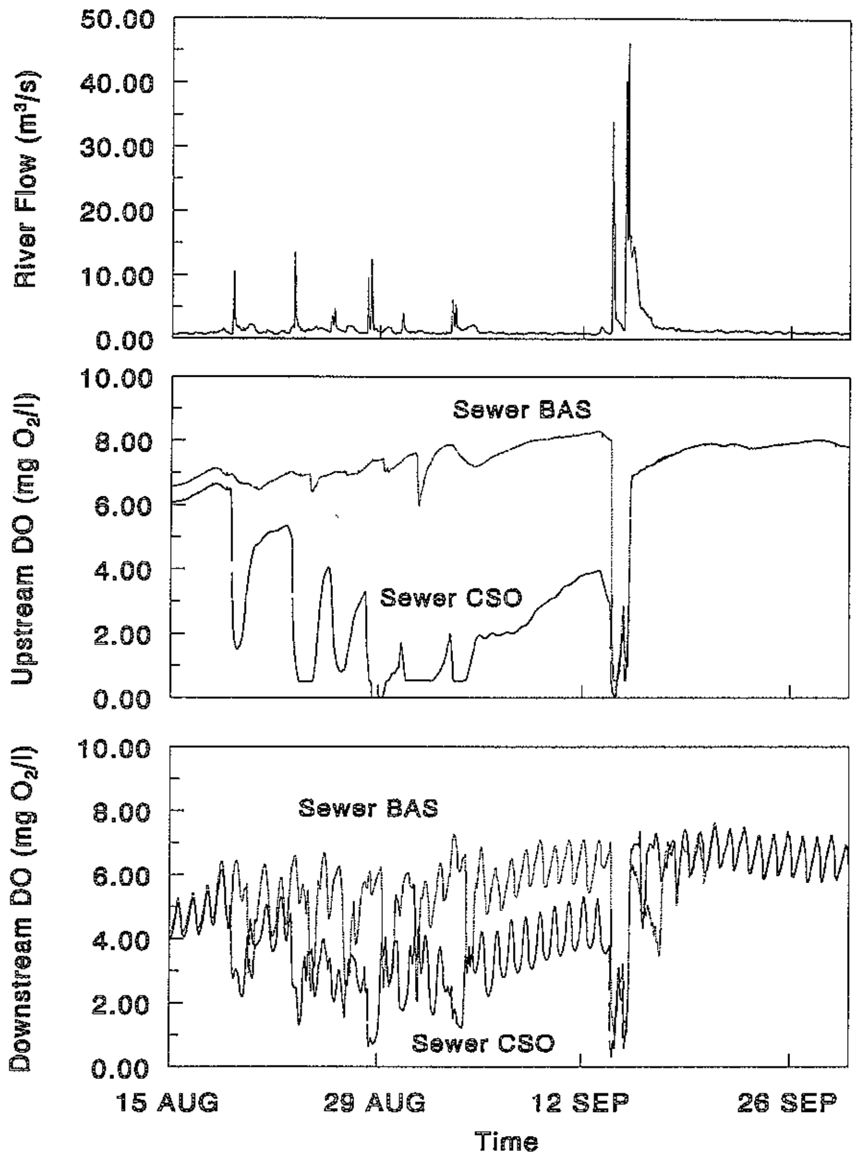
Sewer CSO
WWTP S1
WWTP S2
Frequency
1
0.1
0.01
0.001
0.0001
0.00001
0
20
40
60
80
100
Effluent COD (mg/l)



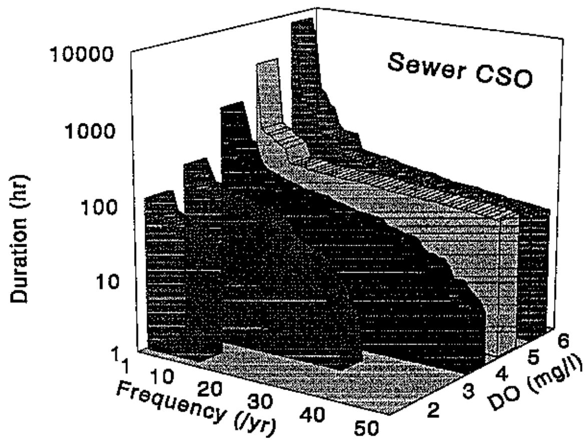
Sewer BAS
WWTP S1
WWTP S2
Frequency
1
0.1
0.01
0.001
0.0001
0
1
2
3
4
5
Flow (DWF)

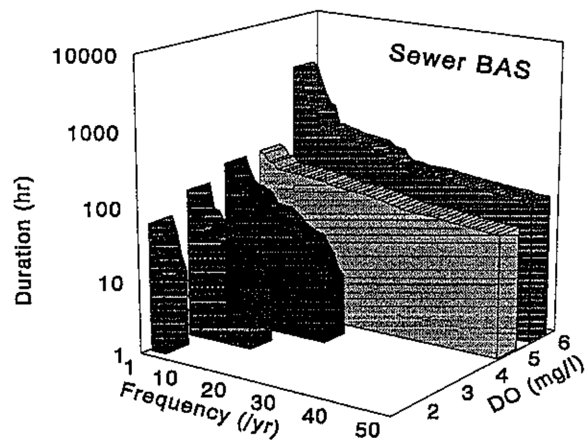


Sewer BAS
WWTP S1
WWTP S2
Frequency
1
0.1
0.01
0.001
0.0001
0.00001
0
20
40
60
80
100
Effluent COD (mg/l)



River Flow (m³/s)
50.00
40.00
30.00
20.00
10.00
0.00
Upstream DO (mg O₂/l)
10.00
8.00
6.00
4.00
2.00
0.00
Sewer BAS
Sewer CSO
Downstream DO (mg O₂/l)
10.00
8.00
6.00
4.00
2.00
0.00
Sewer BAS
Sewer CSO
15 AUG
29 AUG
12 SEP
26 SEP
Time


Sewer CSO
Duration (hr)
10000
1000
100
10
1
Frequency (/yr)
1
10
20
30
40
50
DO (mg/l)
2
3
4
5
6


Sewer BAS
Duration (hr)
10000
1000
100
10
1
Frequency (/yr)
1
10
20
30
40
50
DO (mg/l)
2
3
4
5
6