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URBAN STORM WATER QUALITY AND EFFECTS UPON RECEIVING WATERS

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URBAN STORM WATER QUALTIY AND EFFECTS UPON RECEIVING WATERS

PREFACE

The progress in the construction of waste water treatment plants, in legislation and in the prevention of water pollution has shifted the proportions of sources of pollutants discharged in surface waters. Increasingly attention is being paid to the role of combined sewer overflows and storm water discharges in water pollution.

Historically design and control of sewers and pumping stations has been based on quantitative considerations.

Substantial experience on rainfall, on transformations during run-off and transport in sewers and on overflows is available and quantitative descriptions are being developped.

Comparatively little is known of the quality aspects and managers and administrators often need answers urgently.

Hence in several countries research programmes are being carried out in which the water quality of rainfall, run-off from paved areas in separate and combined sewer systems and in overflows is measured and analysed. Moreover we see an increasing attention for the effects of these discharges upon the receiving waters, not only with respect to the bacteriology and chemistry of water and sediments but including the influence upon the structure and function of ecosystems. The contributions to this Conference clearly illustrate the complexity of the problem to assess the systems behaviour and the variety of techniques used by scientists and technologists to generate appropriate answers.

We wish to thank the authors for their contributions and CHO-TNO and the EWPCA for the support given to the organisers of this Conference.

On behalf of the Programme Committee,

Lambertus Lijklema, Chairman
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ROLE OF HIGHWAY GULLIES IN DETERMINING WATER QUALITY IN SEPARATE STORM SEWERS

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Abstract

Data are presented from field and laboratory studies of water quality and pollutant attachment to sediments in road-side gully pots in a residential catchment. Results of correlation analysis of water quality parameters are presented and the effect of discharge to a separate storm sewer via gully pots are illustrated for two events, one displaying enhanced suspended solids concentration due to resuspension of pot basal deposits, the other showing improved quality of outflow as a result of sedimentation within the gully. The importance of pH variation in determining the exchange of heavy metals from liquor to sediment phases of pollutant transport, and vice versa, is demonstrated.

1 Introduction

The paper presents results of a partly completed three year research programme undertaken to investigate the influence of gully pots on the quality of urban storm water discharge from highway surfaces to a separate storm sewer system.
Typically in the UK, gullies in residential areas receive

storm runoff from some $200m^2$ highway surface. Below the inlet grating at the highway surface is a gully pot of some 100l capacity, which provides a water seal to the downstream sewer connection and is intended to trap sediments, preventing their forward transmission through the sewer system.

The investigation reported set out to determine in what ways the discharge quality of highway runoff, with its associated sediments and pollutant loads, was affected by passage through the gully pot prior to entering the sewer system. Three aspects were studied in some detail: gully pot liquor and deposited basal sediment quality; storm runoff quality at inflow and outflow from the gully pot; and factors affecting pollutant transfer to and from sediment-associated and disolved phases.
The study has been based on three cul-de-sacs in a well-maintained, residential housing estate at Clifton, Nottingham. The three sub-catchments selected were all similar in land use but had highway slopes which ranged from 0.5 to 5.5%. The sites had been locations of overland flow research (Pratt & Henderson (1981)),and the catchment as a whole had been previously studied to determine general sediment washoff (Pratt & Adams (1981)) and stormwater discharge quality (Fletcher et al. (1978); Fletcher & Pratt (1981)).

2 Research Programme

2.1 Systematic sampling: liquor

The research has built up a data set of results of 26 water quality variables, including 12 heavy metals (Table 1) from weekly sampling, undertaken in three periods between May '84 and April 1985. Liquor samples were collected from twelve gully pots, four from each of the three sub-catchments. The gullies were all located at the closed end of each cul-de-sac with two on either side of the highway. The sampling programme was conducted over a twelve week period during the

Table 1 Parameters analysed in the gully liqour programme

Water Quality Parameters		Heavy Metals (mg/l)	
Temperature	(^{o}C)	Sodium	(Na)
Dissolved oxygen	(mg/l)	Potassium	(K)
pH	(pH unit)	Iron	(Fe)
BOD_5	(mg/l as BOD)	Lead	(Pb)
COD	(mg/l as COD)	Calcium	(Ca)
Conductivity	(microsiemen/cm)	Magnesium	(Mg)
Suspended solids	(mg/l)	Zinc	(Zn)
Alkalinity	(mg/l as $CaCO_3$)	Manganese	(Mn)
Total hardness	(mg/l as $CaCO_3$)	Copper	(Cu)
Phosphate	(mg/l as P)	Cadmium	(Cd)
Sulphate	(mg/l as SO_4)	Nickel	(Ni)
Chloride	(mg/l as Cl)	Chromium	(Cr)
Nitrate	(mg/l as N)		
Ammonia	(mg/l as N)		

summer 1984 and over two further five week periods during winter 1984/85.

A one litre sample was taken at a depth of half the depth of standing water in the pot chamber. The purpose of this part of the study was to assess and quantify the range of the selected water quality parameters on a temporal and spatial basis. Data are presented for both the total data set to show the overall variability of the twelve gullies within three sub-catchments (Table 2) and for each sub-catchment separately (Table 3).

2.2 Systematic sampling: sediments

Basal sediments from the twelve gully pots were collected on a monthly basis for six months from October 1984 and analysed to determine the heavy metals attachment. The metals were digested in a mixture of 1:1 by volume of

Table 2 Descriptive statistics for the total water quality data set for the gully liquor sampling programme

Statistic	n	mean	max.	min.	σ
Temperature	264	12.7	25.0	2.0	6.2
Dissolved oxygen	264	3.5	8.5	0.2	2.1
pH	264	7.1	11.4	5.5	0.9
BOD_5	264	19	62	0.0	16.7
COD	230	105	976	0.0	154
Conductivity	264	5.0	114.1	0.0	15.8
Suspended solids	216	114.0	2449	0.0	227
Alkalinity	240	139.5	500.0	20.0	91.7
Total hardness	240	299.0	1240	20.0	260
Phosphate	240	1.8	13.0	0.0	1.9
Sulphate	228	377.0	2268	0.0	382
Chloride	240	1808	54593	5.0	6956
Nitrate	180	18.6	266.0	0.1	35.2
Ammonia	240	2.5	26.6	0.0	4.6
Sodium	240	1596	45000	0.00	4852
Potassium	240	5.42	45.00	0.30	7.40
Iron	240	1.24	50.00	0.00	3.90
Lead	240	0.09	0.78	0.00	0.17
Calcium	240	73.00	460.00	0.00	90.90
Magnesium	240	7.40	130.00	0.00	18.40
Zinc	240	0.15	1.18	0.00	0.13
Manganese	240	0.21	2.50	0.00	0.29
Copper	240	0.02	0.23	0.00	0.04
Cadmium	240	0.06	2.00	0.00	0.24
Nickel	240	0.09	2.08	0.00	0.31
Chromium	240	0.02	0.90	0.00	0.09

Table 3 Descriptive statistics for the water quality data set on each of the three highway sub-catchments

	Tame Close			:Churnet Close				:Twyford Gardens				:
	n	min.	max.	mean	n	min.	max.	mean	n	min.	max.	mean
Temp.	88	2.0	25.0	12.8	88	2.5	25.0	12.7	88	2.5	24.5	12.6
DO	88	0.6	8.5	3.3	88	0.3	8.3	3.6	88	0.2	7.9	3.5
pH	88	5.7	10.7	7.0	88	5.8	11.4	7.1	88	5.5	9.9	7.1
BOD_5	88	0.0	62.0	22.3	88	0.0	62.0	19.5	88	0.0	60.0	16.3
COD	77	0.0	976.0	173.0	75	0.0	537.0	85.9	78	0.0	356.0	57.3
Cond.	88	0.0	93.3	5.7	88	0.0	114.1	5.7	88	0.0	47.6	3.7
SS	72	0.0	610.0	110.0	72	0.0	2449	111.0	72	0.0	1256	121.0
Alk.	80	20.	500.0	158.0	80	20.	320.0	131.3	80	20.	390.0	129.4
TH	80	40.	1220	313.0	80	20.	1240	306.0	80	40.	1060	279.0
Phos.	80	0.0	13.0	2.5	80	0.0	8.2	1.6	80	0.0	5.3	1.3
SO_4	76	0.0	2268	399.0	76	0.0	1710	381.0	76	0.0	1345	350.0
Cl	80	7.0	42894	1922	80	7.0	54593	2268	80	5.0	20916	1235
Nit.	60	0.1	266.0	20.3	60	0.5	109.2	16.1	60	0.3	217.0	19.3
Amm.	80	0.0	23.8	3.0	80	0.0	26.6	2.2	80	0.1	26.6	2.5
Na	80	1.0	22500	2236	80	1.0	45000	1875	80	0.0	12000	677.0
K	80	0.3	45.0	5.5	80	0.4	42.0	5.5	80	0.4	34.0	5.3
Fe	80	0.0	50.0	1.6	80	0.0	17.0	1.3	80	0.0	16.0	0.9
Pb	80	0.0	0.7	0.1	80	0.0	0.8	0.1	80	0.0	0.5	0.1
Ca	80	6.8	460.0	71.9	80	5.0	430.0	78.0	80	0.0	420.0	69.3
Mg	80	0.0	130.0	6.8	80	0.0	115.0	7.2	80	0.0	120.0	8.1
Zn	80	0.0	0.8	0.2	80	0.0	1.2	0.1	80	0.0	0.4	0.1
Mn	80	0.0	1.0	0.2	80	0.0	0.9	0.2	80	0.0	2.5	0.2
Cu	80	0.0	0.2	0.0	80	0.0	0.2	0.0	80	0.0	0.1	0.0
Cd	80	0.0	2.0	0.1	80	0.0	2.0	0.1	80	0.0	1.0	0.0
Ni	80	0.0	2.1	0.1	80	0.0	2.1	0.1	80	0.0	0.8	0.1
Cr	80	0.0	0.8	0.0	80	0.0	0.9	0.0	80	0.0	0.2	0.0

concentrated hydrochloric and nitric acids, and atomic absorption spectrophotometry was used to determine the metal concentrations in the resulting supernatent. The metal concentrations determined are given in Table 4.
A correlation analysis has been carried out to help identify important associations between parameters under study within the gully liquor system to assist in the identification of pollutant sources within the sub-catchments. A summary of the correlation analysis is given in Table 6, where parameters which have correlation coefficients in excess of 0.5 are indicated.
Figure 1 shows plots of some of the quality parameters plotted against time for gully 305.54 on the Churnet Close sub-catchment. The results clearly example the temporal variability of the parameters during periods within the sampling year.

3 Storm runoff sampling

The second part of the field study has been concerned with monitoring the changes in liquor quality during rainstorm events. A number of events have been recorded over the period June 1984 to December 1985 to determine the changes between inflow runoff quality and outflow quality from the gully pot chamber, and to relate such changes to the hydrological conditions and to interactions between inflow, gully liquor and basal sediments.
One gully in each sub-catchment was used for the storm runoff sampling, being one of those at the head of the cul-de-sac where by-passing was not possible and all flow entered the gully (gullies 301.39, 304.104 & 305.53; see Pratt & Henderson (1981)).
The sampling equipment was housed in a steel cabinet bolted to the highway above the gully frame. A small inflow reservoir chamber of some 4 litre capacity was fitted on the

Table 4 Descriptive statistics for the gully pot basal sediment data set on each of the three highway sub-catchments

	Tame Close				Churnet Close				Twyford Gardens			
	n	min.	max.	mean	n	min.	max.	mean	n	min.	max.	mean
Na	24	0.040	24.800	3.400	24	0.100	113.500	6.900	23	0.040	27.030	2.780
K	24	0.000	45.900	4.500	24	0.000	122.100	8.000	23	0.020	5.560	0.700
Fe	24	0.590	10.770	4.270	24	0.390	13.920	13.920	23	0.900	177.800	12.100
Pb	24	0.006	0.820	0.143	24	0.004	0.322	0.094	23	0.002	0.682	0.078
Ca	24	1.800	255.800	38.700	24	1.400	145.300	29.900	23	2.500	201.400	25.800
Mg	24	0.100	88.900	7.100	24	0.000	39.300	8.200	23	0.130	21.940	3.750
Zn	24	0.011	0.737	0.126	24	0.006	0.748	0.103	23	0.014	1.087	0.196
Mn	24	0.049	0.820	0.278	24	0.043	0.405	0.172	23	0.000	4.028	0.481
Cu	24	0.001	0.089	0.013	24	0.002	0.051	0.016	23	0.001	0.247	0.032
Cd	24	0.000	0.010	0.003	24	0.000	0.013	0.002	23	0.000	0.028	0.006
Ni	24	0.003	0.039	0.011	24	0.001	0.047	0.011	23	0.002	0.395	0.041
Cr	24	0.003	0.246	0.053	24	0.000	0.263	0.051	23	0.005	0.528	0.071

Minimum, maximum and mean values in mg/g.

Table 5 Parameters analysed in the storm runoff water quality programme

Suspended solids	Pb
COD	Zn
Total organic carbon	Cu

Table 6 Correlation analysis for the total water quality data set for the gully liquor sampling programme

	pH	TH	Cond.	Cl	SO_4	Na	K	Pb	Ca	Cu	Ni
Temp:											
DO :											
Alk :											
TH :	*										
Cond:	*	*									
SS :											
BOD_5:											
COD :											
Phos:											
Cl :		*	*								
SO_4 :	*	*	*								
Amm.:											
Nit.:											
Na :		*	*	*	*						
K :											
Fe :											
Pb :		*	*	*	*						
Ca :	*	*	*	*	*	*	*	*			
Mg :											
Zn :											
Mn :											
Cu :	*	*	*	*	*	*		*	*		
Cd :											
Ni :	*	*	*	*	*	*		*	*	*	
Cr :		*	*	*		*			*	*	*

* indicates correlation coefficent greater than + 0.5

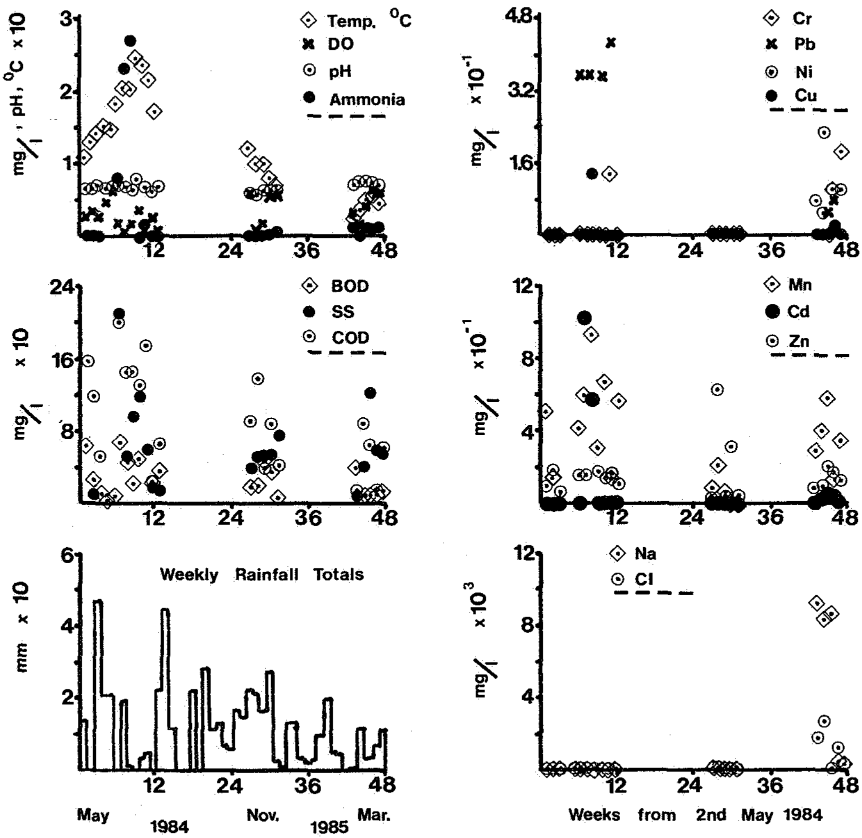

Figure 1 Temporal variation of water quality parameters for gully 305.54

upstream face of the gully pot just below the gully grate. The reservoir chamber contained a float switch and a sampler head which was connected to a 24 bottle vacuum sampler. A hole at the base of the reservoir allowed it to drain during dry weather, but to fill to overflowing during storm runoff from the highway given sufficient inflow rate. Laboratory trials showed that the float switch was activated at flow rates into the reservoir in excess of 0.03 l/s. This value set the minimum threshold for sampling a storm event: once started the sampler was activated for 24 samples regardless

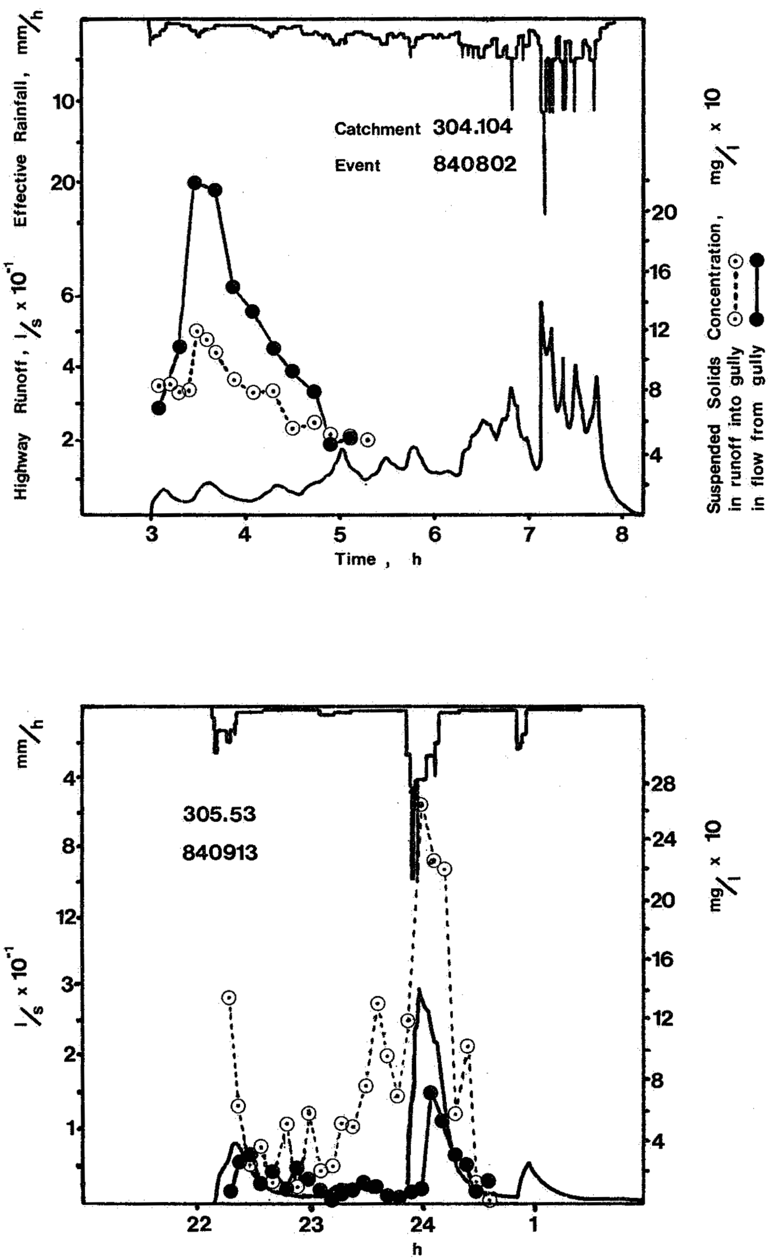

Figure 2 Gully inflow hydrographs and suspended solids concentrations within flows into and from a gully

of subsequent inflow rates. A second sampler collected gully liquor from half depth in the gully pot itself. Once the float switch was activated by storm inflow, simultaneous samples were collected at pre-set time intervals, of either 1 or 6 minutes, from both the inflow waters and from the pot liquor. The pot liquor samples were assumed to be similar in quality to discharge quality from the gully to the downstream sewer. A clock mechanism on samplers recorded the time of first sample, which could be related to observed rainfall. Rainfall was measured using a 0.1mm tipping bucket raingauge, logged at 30s time intervals, and situated within 200m of all sub-catchments.

A previously developed linear reservoir rainfall-runoff model was used to predict inflow hydrographs to each gully. Flow monitoring equipment installed within a gully was thought likely to significantly modify the flow and quality performance of the gully pot. Figure 2 shows the water quality-quantity relationships for two runoff events on two different gully catchments. The figure illustrates the variation in performance which may occur with different storm characteristics. Event 840802 displays a worsening of outflow quality as runoff causes the disturbance of the basal sediments and their resuspension. Effective sedimentation occurred in event 840913 and outflow suspended solids concentrations are markedly reduced. Flow rates during the sampling periods in these events are low: flow rates of the order of design inflow to a gully, about 10 l/s, seem likely to cause considerable worsening of outflow quality.

4 Laboratory studies

Table 7 shows a sample of the results of an experiment conducted to show the influence of sediment size on Zn, Cu and Pb adsorption and release mechanisms for various pH values from 5 to 8. Similar results were found for all three metals.

Table 7 Results of laboratory study of interchange of zinc between liquor and sediments at different pH

Sediment size range (mm)	Concentration of Zn in liquor after :- Day 1	Day 2	Day 3	Day 4	Day 5
pH 5.0					
> 10	2.20	2.20	2.08	2.45	2.70
4 - 10	2.20	2.85	4.20	2.45	2.40
1 - 4	3.10	3.80	4.20	4.20	4.50
0.15 - 1	2.50	3.80	4.00	4.00	4.20
< 0.15	2.80	2.20	2.20	2.45	4.60
pH 8.0					
> 10	0.77	0.33	0.27	0.23	0.08
4 - 10	0.88	0.42	0.38	0.21	0.15
1 - 4	0.68	0.42	0.38	0.19	0.12
0.15 - 1	0.63	0.27	0.27	0.17	0.15
< 0.15	0.43	0.33	0.33	0.25	0.15

Initial liquor Zn concentration 2mg/l in all cases.

Acknowledgements

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THE VARIABILITY OF SOME CONTROLLING PARAMETERS FOR TOXIC POLLUTANTS DURING STORM RUNOFF EVENTS

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Abstract

The temporal variations in water quality parameters within a gullypot system and at stormwater outfalls in the UK and Sweden are discussed. Typical behaviour patterns are identified and related to storm characteristics. Soluble components, such as dissolved organic carbon are predominantly washed through the gullypot system early in the storm due to contributions from road runoff, gullypot liquor and sediment interstitial waters. In contrast the delivery of solids and their associated pollutants to the sewer system is highly dependent on the re-suspension of basal gullypot sediments. The pollutant loadings at the outfalls are interpreted in terms of both the gullypot processes and those occurring in the below-ground system. In the case of dissolved organic carbon two distinct peaks are observed which are labelled 'readily available' and 'residual matured' organics respectively. The probable sources of these peaks are identified by comparison with the temporal behaviour of dissolved chloride and the later 'residual matured' organic peak is related to the mobilisation and transport of gullypot sediment maturation products as well as to the large scale removal of organics from the road surface.

1 Introduction

The evaluation of the controls on, and impacts of, toxic pollutants discharged to urban receiving waters by stormwater runoff has received relatively little attention despite the increasing concern for the potential magnitude and severity of the problem. The distribution and fate of such priority pollutants is partly controlled by those physical and biogeochemical processes which stimulate the transformation and mobilisation of the toxic compounds during the stormflow event. The partitioning of heavy metals for example, between the dissolved and particulate phases of stormwater runoff is dependent on parameters such as flow, suspended solid concentrations, dissolved and particulate organic carbon contents, pH, dissolved chloride levels and the prevailing oxic/anoxic conditions. In addition biochemically mediated reactions can be responsible for the mobilisation of metals.

Morrison et al. (1984a) have shown that metal distributions throughout individual storms show increases in the proportions of Cd, Cu, Pb and Zn in the soluble phase as the storm progresses. Although the increase in the water-suspended sediment ratio is a relevant factor, the levels of dissolved chloride (for Cd and Pb) and dissolved organic carbon (for Cu and Zn) were shown to be the controlling parameters. During snowmelt events, the high ionic strength of the runoff due to the high dissolved chloride levels which are present can result in peak bioavailable dissolved metal concentrations of 146 $\mu g\ l^{-1}$, 330 $\mu g\ l^{-1}$ and 558 $\mu g\ l^{-1}$ for Cu, Pb and Zn, respectively (Morrison et al., 1984b).

This paper reports the concentration and loading values for a number of known water quality controlling parameters and discusses their variation through individual storm events monitored at a discrete gullypot catchment and at the outfalls to two separately sewered, residential catchments in NW London, UK and Gothenburg, Sweden.

The variability of the 'first flush' phenomena is well recognised by stormwater engineers with both separate and combined systems showing typical pollutant peaks early in the storm event which have been related to scouring of below ground deposits and later secondary peaks associated with above ground sources. The temporal variation in the SS pollutograph will be of particular importance for controlling the behaviour of those toxic pollutants, such as Pb, which have a high affinity for the particulate phase. This response is very different from that which can be expected as a result of increases in dissolved organic carbon (DOC) and dissolved chloride concentrations which will assist greater solubilisation of metals in the stormwater. Therefore a knowledge of the nature and occurrence of such controlling variables is important in evaluating toxic emissions from urban sewer systems.

2 Catchment details

The discrete gullypot catchment used in this investigation was located in a car park attached to Chalmers University of Technology, Gothenburg, Sweden. The data obtained from this small study was scaled-up to interpret the results obtained for the stormwater reaching the outfall pipes of two urban catchments in Sweden and in the UK.

Oxhey, UK: a 247 ha, 18% impervious, residential catchment situated in the north west fringes of Greater London.

Bergsjon, Sweden: a 15.6 ha, 42% impervious, multi-storey residential catchment in the north eastern suburbs of Gothenburg.

Chalmers, Sweden: a 390 m^2 catchment constituting one part of the asphaltic surface of a car park on the site of Chalmers University of Technology.

Full details of the instrumentation and sampling equipment, which were used to obtain the results for all three catchments have been described by Harrop (1984), Morrison (1985) and Morrison et al. (1985).

3 Relationships between Storm Characteristics and Water Quality Parameters in a Gullypot System

The gullypot outflow loadings of three water quality parameters together with selected hydrological characteristics are shown in Table 1 for five storm events which were monitored between July and September 1984. Storm D, which like storm A occurred after a relatively long dry period was the most intense storm with a maximum of 2.7 mm rain falling in a 5 minute period. As a result, the runoff for this thunderstorm was close to the design capacity of the sewer network (1 l s^{-1} over 100 m^2 for 15 minutes) with a maximum flow rate of 4.6 l s^{-1} recorded at the gullypot. Storm A was a long heavy storm which attained a maximum road runoff flow rate of 2.03 l s^{-1} and an overall flow volume of 6923 l.

Table 1 Hydrological Characteristics and Water Quality Parameter Outflow Loadings for Selected Storm Events for the Gullypot System

Storm Code	Storm Duration (hours)	Rainfall Volume (mm)	Road Runoff Volume (Litres)	Antecedent Dry Period (days)	Parameter Outflow Loadings (g)		
					Dissolved Organic Carbon	Particulate Organic Carbon	Suspended Solids
A	5.8	18.5	6932	10*	28.3	45.3	467
B	8.7	2.9	1667	3	15.1	2.4	23
C	4.3	4.6	2155	1	10.3	1.5	28
D	1.0	9.7	3951	10	21.4	26.2	314
E	2.5	6.5	3992	1	6.5	7.6	48

*0.6 mm rainfall on preceding day.

The data presented in Table 1 indicates that the outflow gullypot loadings of SS and particulate organic carbon (POC) are dependent primarily on the intensity of the storm event and the overall runoff volume although these relationships will obviously be subject to the availability of material within the gullypot itself. Thus the gullypot chamber contributed 66.5% and 11.3% of the SS in the outflow for storms A and D, respectively. During low intensity events, such as storms B, C and E, there is however a net deposition of sediment within the gullypot, with less than 50 mg l^{-1} of SS leaving the gullypot. In these instances the sediment associated metal outflow is therefore the result of road runoff after the larger, heavier particulates have settled out in the gullypot sediment.

DOC loadings show a positive linear relationship with antecedent dry period. This suggests that a continuous accumulation of organic carbon, which becomes available for washout, occurs on the road surface as well as in the gullypot liquor and sediment during the dry period. The relationship between SS mass flow and antecedent dry period, although similar, is weaker due to the involvement of the previously mentioned gullypot sediment-mobilisation processes.

4 Temporal Variations in Water Quality Parameters within the Gullypot System

An important process which occurs within the gullypot during storm conditions is the mobilisation of interstitial sediment waters and this is indicated by changes of controlling parameters such as conductivity, dissolved oxygen, redox potential and DOC. During large storm events the interstitial waters are disturbed at the beginning of the event leading to conductivity increases due to the release of sediment associated salts. In addition, dissolved oxygen and redox potential decrease due to oxygen utilisation by bacteria and mixing of anoxic sediments. Associated increases in DOC are observed of up to 41 mg l^{-1} in storm A, and this is largely due to the pre-storm formation of sedimentary maturation products. The rate of interstitial water mobilisation is predominantly related to the incoming overland flow rates which for the monitored storms are greater during the early part of the storm flow event. However, rapid increases in conductivity can occur for low volume storms if they are accompanied by initial high rainfall intensities. The changes in conductivity provide a useful and sensitive index of the time of mobilisation for the dissolved, metal-enriched interstitial water during storm events, although other factors, notably an initial large decrease in pH, will also act as controlling parameters for soluble metal release.

Storm A, which is characterised by a long early flush of 90 minutes duration, demonstrates the initial changes in the parameters which have just been described. In addition, continuous pH measurements show a progressive decrease from 6.3 to typical rainfall levels (pH 4.1 for this event) indicating that the dissolved buffering agents have been depleted after 90 minutes. During this initial period it is therefore envisaged that incoming road runoff is mixed with mobilised interstitial waters and removed through the gullypot outflow into the sewer pipe.

A subsequent increase in rainfall intensity during the storm event resulted in higher runoff flows and a large discharge of solids due to the disturbance and re-suspension of the basal gullypot sediments (Figure 1). An associated decrease in redox potential coincides with the further release of interstitial waters. The loading pattern for POC clearly follows the same trend as SS (Figure 1). The small differences in the relative intensities of the storm profiles for these pollutant parameters can be explained by their different source characteristics. During the two periods of increased flow activity which occurred between 130 and 190 minutes the mobilisation of basal sediments results in a lowering of POC profile relative to the suspended solids. This is a consequence of the lower POC composition of these solids compared to road runoff and gullypot liquor solids which are removed earlier in the storm.

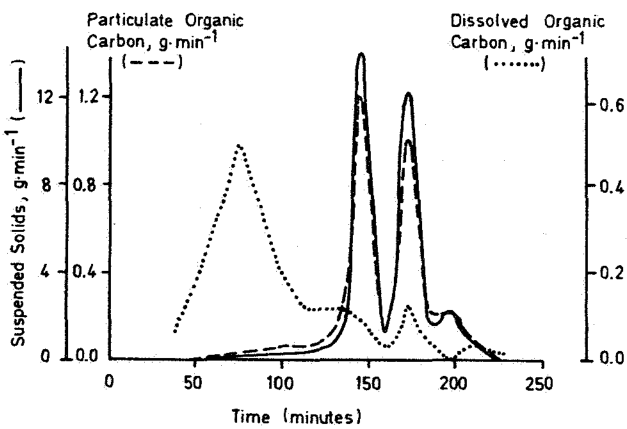

Figure 1 Water Quality Parameter Loadings in Gullypot Outflow for Storm A

DOC is predominantly washed out in the first low flow stage of the storm which can be related to the presence of this material in road runoff, gullypot liquor and sediment interstitial water. The further but reduced removal of DOC at 173 minutes can be ascribed solely to a road runoff contribution.

Storm D represents a heavy thunderstorm and the variations in conductivity, pH, dissolved oxygen and redox potential indicate that the gullypot liquor and interstitial water were washed out within 5-10 minutes of the commencement of high runoff flows. In the case of dissolved oxygen this process is characterised by a sharp inverse deficit peak. As a consequence of the intensity of this storm event both dissolved and suspended solid components are rapidly washed out of the gullypot and exhibit marked 'first flushes' . The loading profile for DOC shows an additional peak following maximum flow at which time the gullypot basal sediment appears to have been completely mixed (to give 2145 mg l^{-1} SS) and the observed secondary peak may be due to the mechanical stripping of organic compounds from the re-suspended basal sediment.

5 Relationships between Storm Characteristics and Water Quality Parameters for the Oxhey and Bergsjon Catchments

The hydrological characteristics for 9 storm events as well as the average concentrations of water quality parameters at the outfalls of the two catchments are given in Table 2. The parameters selected are those which are considered to be particularly important in controlling the metal species which are subsequently discharged to the sewer and receiving waters. Dissolved chloride and DOC concentrations are influential in controlling the levels of soluble metals and in particular those which may be bioavailable to the aquatic flora and fauna whereas SS and POC levels will act as scavengers for the removal of dissolved pollutants. The soluble/sorbed metal distribution is also highly dependent on the pH which at the outfall can be seen to be close to neutral for all the monitored storms (Table 2) and therefore

Table 2 Hydrological Characteristics and Water Quality Parameter Outfall Concentrations for Selected Storm Events on the Oxhey and Bergsjon Catchments

Storm Code	Catchment	Storm Duration (hours)	Urban Runoff Volume (m^3)	Average pH	Average Outfall Parameter Concentration (mg l^{-1})			
					Dissolved Chloride	Dissolved Organic Carbon	Suspended Solids	Particulate Organic Carbon
1	Oxhey	2.2	1058.3	7.3	64.2	8.4	160.6	-
2	Oxhey	2.7	1362.3	7.4	9.2	6.4	51.5	-
3	Oxhey	0.8	320.3	6.2	-	-	131.8	27.9
4	Bergsjon	0.7	33.4	6.6	15.9	9.0	122.5	24.4
5	Bergsjon	1.7	63.1	7.1	21.4	5.5	69.6	14.6
6	Bergsjon	0.4	79.9	6.2	-	4.1	65.0	13.4
7	Bergsjon	4.4	281.3	6.3	13.5	4.2	34.1	-
8*	Bergsjon	6.6	111.7	7.1	5631.8	5.5	242.5	79.1
9*	Bergsjon	9.0	77.1	6.8	1140.4	5.0	38.7	3.2

*snowmelt events

of lesser importance as a controlling parameter. The near neutral pH values observed at the outfall are consistent with the buffering processes which have been described in the gullypot and which continue within the pipe system.

Storms 1 and 2 are representative of typical rainfall events at the Oxhey site, which occurred during the winter of 1983, and possess an interval of 2.7 days between the two events. The weather for the preceding two months contained isolated snowfalls and the road salting activities which took place during this time are reflected by the elevated dissolved chloride concentrations observed during Storm 1 (Table 2) with a maximum level of 179 mg Cl l^{-1}.

The maximum flow intensity, for all the monitored catchment storms, was highest for Storm 1 with a value of up to 231 l s^{-1} which is equivalent to 4.7 l s^{-1} (impervious hectare)$^{-1}$. The highest runoff intensity sampled at the Bergsjon site was 82.3 l s^{-1} in Storm 6 which represents 13.5 l s^{-1} (impervious hectare)$^{-1}$.

Storms 4, 5, 6 and 7 are typical of the frequent low to medium intensity rainfall events which occur throughout the year in Gothenburg. As such, they do not present any extreme concentration values for the measured parameters although the SS level is slightly elevated for Storm 4.

Snowmelt events 8 and 9 were collected at the Swedish outfall during the winters of 1984 and 1985 respectively, but represent events which are quite different in character. Snowmelt 8 typifies a fairly rapid melt following several days deposition of snow. Melting was due to a combination of increased temperatures and rainfall which resulted in runoff flushes containing concentrations of SS and dissolved chloride of up to 1624 mg l^{-1} and 17 g l^{-1}, respectively. In contrast snowmelt 9 occurred at lower ambient temperatures and was therefore a much slower process which consequently produced lower concentrations of dissolved chloride and SS (Table 2).

Comparison of snowmelt and stormwater runoff events clearly demonstrates the tendency for SS concentrations, and therefore loadings, to be higher in the former (snowmelt runoff range: 4.5 - 1624 mg l^{-1}; stormwater runoff range: 24.9 - 306.0 mg l^{-1}). This has been observed by other studies (Malmqvist 1983) and one explanation is the addition of solid material to the road surface during salting operations. However, a more satisfactory explanation may be that deposited particulates are mixed with the snow and the resulting snow and particulate emulsification provides a more effective and active mechanism for releasing SS to the subsequent runoff. In the direct release of road dusts to stormwater runoff the surface and the particulates require initial wetting to reduce the frictional shear stress before such efficient removal can occur.

The average DOC concentrations (Table 2) do not show a great deal of variation although there is a tendency for them to be higher in stormwater compared to snowmelt runoff events. This may be a consequence of the reduction of biological gullypot and in-pipe sediment maturation processes, which are known to release DOC, during colder weather. The range of DOC concentrations measured for all 9 storm events was 2.9 to 14.2 mg l^{-1} with the highest concentrations predominantly occurring at the beginning of the events which is consistent with the early release of this controlling parameter from the gullypot system.

Stormwater solids appear to be highly organic enriched, with up to 33% POC composition, which indicates a clear enhancement between the gullypot outflow and the catchment outfall. This enrichment is undoubtedly due to the hydraulic release and transport of highly organic material from within the in-pipe system as well as to the ability of the re-entrained particles to adsorb organics released by microbial action within the below-ground system. The consequences of these high POC levels in terms of metal release to the receiving waters are that those metals such as copper which have a high affinity for organics will be strongly attracted to the solid phase.

6 Temporal Variations in Water Quality Parameters at the Stormwater Outfall

The temporal variations in water quality parameters observed during individual storm events show distinct trends which can be related to gullypot behaviour and the effects of varying source contributions. The pollutographs for Storm 5 (Figure 2) are fairly typical of those observed at the stormwater outfall. SS parallel the flow hydrograph with the increase observed during the initial subdued flow peak at 15 to 25 minutes, primarily representing in-pipe washout although it is accompanied by some contributions derived from the easily mobilised, fine solids contained in road surface runoff. The main SS peak coincides with the peak flow although the maximum solids loading rates precede the flow peak by some 7-10 minutes; this feature is consistent with the results obtained from the gullypot study. The 'first flush' of solids from road runoff is delayed through gullypot settling but the chamber contents are subsequently remobilised by the increasing turbulence and unstable flow conditions associated with the rising limb of the hydrograph. In the case of snowmelt events there is a much greater tendency for SS loadings to peak before maximum flow which is indicative of the greater activity and more rapid delivery of snowbound surface particulates.

There is a substantial early peak of DOC commonly observed at the storm outfall which coincides with the initial flow stage (Figure 2). Whilst there may be some contribution to this early loading from below-ground sources, the pattern is very similar to that observed for Storm A (Figure 1) at the gullypot outflow and strongly suggests that the major source of this input is from surface runoff. This interpretation is confirmed by the occurrence of a coincident peak in

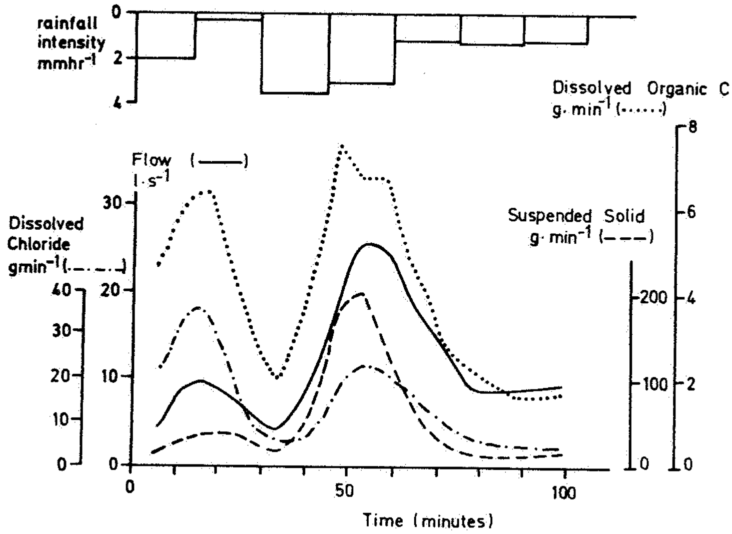

Figure 2 Water Quality Parameter Loadings at the Stormwater Outfall for Storm 5

dissolved chloride which is certainly derived from road runoff. These early pollutant loadings pass quickly through the system as 'plug flows' and show a rapid decline which may be partly related to dilution by 'clean' roof waters discharging through the sewer during this early flow phase. The leading peaks in dissolved material can therefore be related to both the scouring of in-pipe deposits as well as to the rapid removal of roof and road products and may be described as 'readily available' organics.

The substantial secondary organic peaks accompanying the main flow peak can most probably be related to the mobilisation and transport of gullypot sediment maturation products as well as to the large scale removal of organics from the road surface. The increased hydraulic flow effect is required to entrain and transport road and chamber sediments and to achieve the associated release of organic compounds. The DOC released in this way can be defined as the 'residual matured' organics. The existence of peak loadings which can be assigned to the two phases of 'readily available' and 'residual matured' organics are clearly observed for all the runoff events studied. This pattern is not consistently demonstrated however in the case of dissolved chloride which is normally depleted after the initial flow peak and reflects a surface exhaustion effect. During snowmelt events and winter stormflows, the dissolved chloride levels tend to produce higher loadings throughout the event with peak levels corresponding to peak flows confirming the surface source of delayed, secondary pollutant peaks. The difference in behaviour between dissolved chloride and DOC clearly indicates the more complex biochemical transfer processes which operate for the latter parameter within the below-ground system in comparison to the more direct surface washoff and dilution mechanisms controlling the outfall discharge of chloride and other soluble inorganics.

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URBAN WATER QUALITY IN LELYSTAD; RAINFALL AND RUNOFF FROM SELECTED SURFACES

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Abstract

In the new town Lelystad, the overground runoff is collected by storm sewers, which discharge into the urban surface waters. Following the precipitation on its way to the canals, significant water quality changes can be observed. The precipitation itself is contaminated by air pollution. During surface runoff the water can pick up or drop pollutants. The nature and amount of picked up or dropped pollutants depend on the characteristics of both the rainfall and runoff and the surface involved.

In this study the water quality of rainfall and runoff from selected surfaces such as a roof, a motorway and galvanized railings was investigated. Concerning the runoff from a sloping roof, it is a striking fact that pollutants seem to accumulate on the roof while only high rainfall intensities wash them off; especially lead seems to accumulate.
Runoff from a town motorway was heavily polluted by traffic induced substances like lead, zinc, oil and polycyclic aromatic hydrocarbons. Concentrations in the runoff lie for some constituents far beyond Dutch standards for open waters. Corrosion of galvanized railings explains the bulk of the zinc concentration in the stormwater runoff from a housing area.

1 Introduction

The excess rainfall on an urban area flows off by way of roofs, pavements, streets, gullypots, etc. into the sewer. With a separate sewer system like the one in Lelystad, the excess precipitation is discharged by way of the stormwater sewer directly into the canals, while the waste water is discharged to a waste water treatment plant. Starting-point for the separation from these two discharge streams was the idea that stormwater was "clean" and direct discharge into open water was allowed.
Nowadays it is quite clear that stormwater is polluted in many ways. A large number of substances discharged with the stormwater, can affect the quality of the aquatic environment. The main point of research now is, besides assessing the annual load, the analysis of quality data from discharge-events and the reaction of the receiving open water upon such a discharge. The quality of the runoff from urban surfaces depends on the type of the surface - roof, street, sidewalk - and on the incoming load of pollution, the accumulation and the removal. Part of the Urban Water Research Project Lelystad is the analysis of the quality of the runoff from these selected surfaces. In this paper some results of this study will be presented.

2 Runoff pathways

On it's way to the receiving water, the runoff picks up pollution from the surfaces at several places. Part of this pollution is transported only over a short distance, due to e.g. settling or (re)attaching. The occasions where the stormwater can become polluted after reaching the urban ground surface are indicated in figure 1.

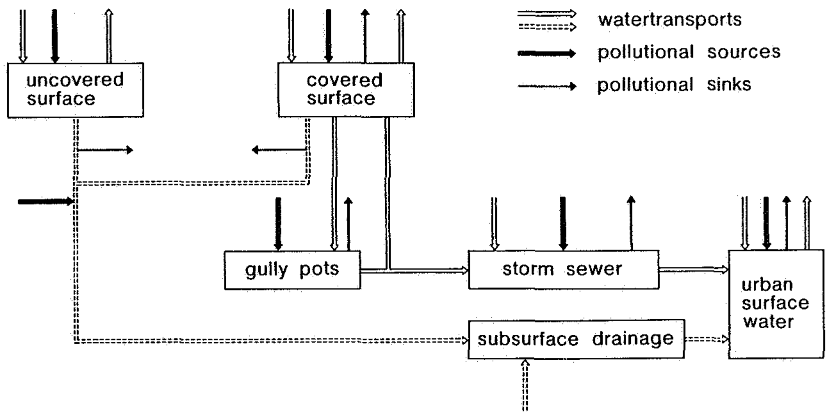

Figure 1 Stormwater runoff: pathways and pollutional sources and sinks (Uunk, 1985)

The most important sources of pollution for both the covered and uncovered surfaces are the wet and dry deposition, the traffic, the animal excrements, corrosion and organic and inorganic waste from litter, leaves, etc. The quality of the stormwater discharge can also be affected by illegal discharges into gullypots, wrong connections between waste and stormwater sewers and biodegradation processes. On the other hand some pollution will be removed by adsorption to the soil, by streetsweeping and by cleaning the gullypots and manholes.

3 Water quality measurements in Lelystad

In the housing area Bastion, a section of Lelystad, research is going on since 1982 considering the pollution of the stormwater discharges and the effects on the receiving water. The stormwater discharge and the precipitation are measured continuously. The stormwater discharge is sampled about every 12 m^3. At the same time the wet deposition is

measured with a closable raincollector, which is opened during rainfall only. Moreover, an extensive measuring programme including continuous water quality monitoring is executed in the receiving water. The measurements and the control over the measuring-system is executed by a data acquisition system based on a desk calculator (HP 9825).
In the Bastion area 270 single family houses are realized in 2 to 4 storey buildings. The roofs of these buildings have an angle of about 25° with the horizontal. The total area is 4.5 ha; 66% is covered; the slope of the surface amounts between 0 and 0.5%.

In addition to the water quality analysis of the runoff from the whole basin, the runoff from several small basins is sampled in order to measure the contribution of these selected surfaces to the pollution that is found at the stormsewer outfall. For that purpose not only the quality of the precipitation, is assessed, but also of the discharge from a city mainroad and from a roof. Moreover the sources of the zinc load of the area is investigated.
Recently research has started to examine the content of gullypots, the amount of pollution transported over the paved surface during rainfall and the effect of streetsweeping.

The precipitation used for assessing the quality of the wet deposition is collected in the Bastion area with a collector, which is closed automatically during dry periods. The collector is situated about 2.5 m above surface level and is made of inert material. The results of 16 samples were available.

For sampling the roof discharge, a roof of 8 m^2 in the Bastion area has been selected. Of these 8 m^2, 6.50 m^2 have an angle of 35° with the horizontal and the remaining 1.50 m^2 is situated vertically. In the horizontal projection, the roof surface is 5.25 m^2. The roofing material is asbestos-cement. Its surface is slightly rough. On about 1 m^2 algae have developed; the rest of the surface looks more or less clean. In 1985 6 rainstorms have been sampled.

A part of a city mainroad that drained to a gullypot was used to collect samples of the discharge of a larger road. The surface area amounts 176 m^2 and the road is made of asphalt. The roadcover was smooth and showed no cracks. The outflow of the gullypot was connected to a large underground tank, to collect the total discharge from a storm. Samples of 11 storms have been analysed. The traffic intensity was about 3200 cars per day.

For 3 rainstorms a simultaneous quality analysis was available for the precipitation, the roof discharge, the road discharge and the Bastion storm sewer discharge.

The water trickling down from 2 galvanized railings during rainfall was collected in 10 storms. These railings are used extensively on the balconies and outside corridors of the appartments. The (galvanized) metal surface of 1 railing is 1.3 m^2. The total surface in the Bastion area is estimated as 2,700 m^2.

4 Results

4.1 Rainfall and roof discharge

Precipitation itself is already polluted with particles and dissolved substances out of the air. Table 1 gives the quality measurements of the rain water during the research period and those measured in Lelystad and in The Netherlands during a longer period.

Table 1 Quality of rain water in Lelystad and The Netherlands

parameter	units	(1)	(2)	(3)
NH_4	mg/l	1.35	1.64	3.08
$NO_3 + NO_2$	mg/l	.80	.75	5.53
P-tot	mg/l	.04	.08	
P-ortho	mg/l	.01		.47
Cl	mg/l	5	9	5
pH		4.8		4.6
Fe	mg/l	.24		.06
Pb	µg/l	23.5	10.0	18.0
Cd	µg/l	.5		.5
Cu	µg/l	13.3		12.7
Cr	µg/l	5.0		.5
Zn	µg/l	74.0		85.0

(1) Bastion '83 - '85 median

(2) Lelystad '72 - '80 mean

(3) The Netherlands '78 - '82 median of 27 local mean values (Van de Ven, 1986)

The mean quality of the rain water measured during the research period differs only slightly from that of earlier measurements in Lelystad and The Netherlands, albeit those earlier measurements comprised the composition of dry and wet depositions.
The mean values for ammonia and nitrate in The Netherlands however, are remarkably higher than those in Lelystad, due to the difference between wet and dry deposition, although the fact that Lelystad is located at larger distance from ammonia and nitrate sources than other locations may be another reason. A striking but inexplicable difference is furthermore found for Chromium. Nevertheless, the values measured in different periods in Lelystad are in the same order of magnitude.

Table 2 gives the mean values of the quality of runoff from the selected surfaces. The water quality analyses of the precipitation, the roof runoff, the motorway runoff and the stormwater runoff as well as the rainfall and runoff intensities for the three rainstorms are entered in table 3. The difference between the quality of the rain water and that of the roof runoff gives an indication in which way the roof influences the quality of the discharged rain water.

Table 2 Water quality data from the experimental basin "Bastion" (precipitation, roof- and stormwater runoff) and from a "town motorway". Units: NH_4-Oil in mg/l; Pb-ΣPAC in µg/l

	Precipitation (1) **		Roof (2)		Motorway (3)		Stormwater runoff (4)
	wm.*	med.	wm.	med.	wm.	med.	med.
NH4	1.08	1.35	.94	.66	1.22	1.30	.95
$NO_3 + NO_2$	.72	.80	.66	.65	.91	.90	1.97
N-kjeld.	2.41	1.50	1.48	1.35	3.33	2.70	1.90
N-Kjeld. f.	1.09	1.20	1.22	1.10	1.52	1.60	1.39
Total-P.	.09	.04	.08	.07	.34	.26	.45
Total-P.f.	.05	.05	.05	.02	.02	.03	.12
Ortho-P	.07	.01	.04	.01	.01	.01	.16
BOD					8.9	5.0	3.7
COD					119	78	30
SS					149	90	45
SS-org.					47	33	14
Oil					6.5	4.1	1.1
pH	4.7	4.8	6.7	6.8	6.7	7.1	
Pb	32.6	23.5	10.1	11.1	443.5	342.5	72.7
Cd	2.5	.5	.5	.6	2.4	1.6	.7
Cu	10.9	13.3	5.7	5.0	77.4	49.2	14.0
Cr	7.8	5.0	5.0	5.0	16.0	10.3	9.8
Zn	94.1	74.0	33.4	39.7	323.9	247.5	505.5
ΣPAC (Borneff 6)					4.4	2.2	.5

* wm. = discharge weighted mean, med. = median

** (1) = 16 events; (2) = 6 events; (3) = 11 events; (4) = July '82 - Dec. '85

Table 3 Water quantity and water quality of three storm events

Quantity		storm 1	storm 2	storm 3
dry weather period	d	.8	1.5	.5
max. runoff intens.	l/s/ha	4.6	3.5	36.2
max. rainfall int.*	l/s/ha	17.5	18.9	307.0
rainfall depth	mm	14.1	5.7	10.4

Quality

	storm1	storm2	storm3	storm1	storm2	storm3	storm1	storm2	storm3
	NH4		mg/l	NO3 + NO2		mg/L	N-Kjeld		mg/l
P**	.8	1.9	-	.5	2.6	-	.7	2.6	-
R	.38	1.1	2.44	.4	2.0	.9	.4	1.9	3.5
M	.29	1.5	2.41	.36	2.3	.6	1.4	2.7	10.0
Q	.58	.64	1.8	.7	2.4	1.0	.8	1.2	5.2
	N-Kjeld.filt.		mg/l	P.tot		mg/l	P.tot.filt		mg/l
P	.6	2.6	-	.01	.08	.04	.01	.05	-
R	.2	1.7	3.1	.16	.02	.09	.14	.01	.02
M	.2	2.1	2.8	.24	.18	.76	.02	.04	.03
Q	.5	.9	2.4	.20	.80	1.10	-	.17	-
	P-ortho		mg/l	pH			Pb		µg/l
P	.01	.05	.01	4.6	4.3	-	7.6	41.8	8.8
R	.14	.01	.01	6.5	7.0	7.0	5.0	20.4	22.0
M	.01	.03	.01	7.1	7.4	7.5	333.1	209.0	1289.0
Q	.14	.48	.17	-	-	-	7.0	28.3	16.1
	Cd		µg/l	Cu		µg/l	Zn		µg/l
P	.3	1.1	.3	13.4	23.3	15.9	61.0	538.0	58.0
R	.4	.7	.4	5.0	7.1	8.0	30.0	68.0	40.0
M	1.6	1.6	6.5	45.0	42.0	175.0	189.7	242.0	719.0
Q	.4	-	-	11.8	14.8	40.9	410.0	955.0	1205.0

* rainfall intensity measured during small intervals; ca. 20 secs.

** quality of: P = rainfall; R = roof; M = motorway; Q = stormwater discharge

In table 2 can be seen that the concentration of almost all constituents in the roof runoff are lower than in the precipitation. Only total-P gives a higher value for the roof runoff, most likely caused by faecal droppings from birds. From the concentrations measured in both the rain water and the roof runoff during the three rainstorms (table 3) one can draw the same conclusions.
The difference between the nutrient concentrations are not substantial; the heavy metal concentrations however decrease sharply during roof runoff. In particular this is the case with lead and zinc. Obviously these metals are bound to particles and/or algae present on the roof, which are not washed off by the stormwater in first instance, but only during high rainstorm intensities.

Rainstorms 3 (table 3) gave a very high rain storm intensity of 307 l/s/ha (precipitation measurements are performed within small time intervals of about 20 seconds). Nevertheless table 3 shows that only lead is washed off substantially during this event, while zinc still accumulates on the roof. Another possible removal mechanism is, that the accumulated material dries out first and erodes by the wind later on. Further investigation on these processes are needed.

After contact with the surface the pH of the rain water increases, in the first case due to a change in the carbon dioxide equilibrium.

4.2 Runoff from a motorway and a housing area

4.2.1 Motorway

The ammonia and nitrate concentrations in the motorway runoff differ only slightly from the concentrations in the precipitation. The Kjeldahl nitrogen content however is much higher than in the precipitation, which is already rather high on itself. The entrainment of traffic induced organic particles must be the reason.
The total phosphate content is substantially larger than in the precipitation. The concentration in the filtrate however, hardly differs from that in the prcipitation; the phosphate is bound to the washed off particles.

In fact the concentration of the organic and inorganic suspended solids in the motorway runoff is 2-3 times that in the stormwater runoff from the housing area "Bastion". At first glance one would expect the contrary, but a lot of particulated matter erodes from the verges by the runoff and the traffic induces whirlwinds. In the runoff from the motorway as well as that from the housing area the organic material is one third of the total suspended solids load.

In comparison with the concentrations in the rain water, the content of the heavy metals lead, zinc and copper in the motorway runoff is rather high. The lead originates from the combustion of "leaded" petrol; zinc principally comes from the wear of car-tires while copper concentrations are much higher than those in the stormwater runoff from the Bastion, which is not surprising in view of the low traffic load in the housing site.

4.2.2 Housing area

The zinc concentration in the runoff from the housing area is fairly high. The main source of this zinc must be the galvanized railings in the area, considered the results of the analyses of the zinc concentration in the rain water dripping off from two railings (table 4).

Zinc railings are used throughout the housing area on balconies and upper-footpathes. The corrosion of these galvanized railings by rain water contributes to a large extent to the zinc content of the stormwater runoff (table 4).

Table 4 Mean zinc concentrations in runoff from railings used on balconies and upper-foothpathes (10 collect samples)

	Zn mg/l	Zn-filt. mg/l	pH	Total rainfall (mm)	Total load (mg)	Total surface of railings (m^2)
Mean	75.6	71.9	5.97	200	2286	2.6
Highest	191.0	190.0	6.59	<u>Specific load</u>:		
Lowest	19.5	18.6	5.20	4.4 mg $Zn/m^2/mm$ rainfall		

The total galvanized surface in the area is about 2700 m^2. Using the values from table 4 this surface emits about 2.4 kg zinc anually. With a covered surface of 3 ha, the concentration in the stormwater runoff - caused by corrosion - amounts to 400 µg/l. The observed mean zinc concentration in the stormwater runoff is 505 µg/l; so the influence of the railings must be considerable. The rain water itself contents about 75 µg Zn/l, added to the 400 µg/l from the railings, gives an almost perfect balance of the amount of zinc washing off from the Bastion area.

The BOD en COD of the motorway runoff are considerably higher than those of the stormwater runoff from the Bastion. Discharging this runoff on the receiving water could result in problems with the oxygen balance in those waters. However, the amounts of motorway runoff are generally small. The COD/BOD ration of 13 indicates a very low biodegradability of the organic matter in the runoff.

Finally some comments on the contents of oil and polycyclic aromatic hydrocarbons. A median oil content of 4.1 mg/l in the motorway runoff was measured, which is about 4 times the concentration in the runoff from the Bastion: a very clear pollutional impact by the motorized traffic. Concentrations of the polycyclic aromatic hydrocarbons - the "Borneff six" - in the motorway runoff are also about 4 times those in the Bastion runoff (2.2 - .5 µg/l). Considered the Dutch standard for surface waters of .1 µg/l(IMP, 1985), especially the first value of 2.2 is very high, the impact of exhaust emissions by the motorized traffic is evident.

5 Conclusions

- Differences in the water quality of the runoff from selected surfaces within one basin are considerable.
- Surprisingly most concentration of the analyzed constituents decrease, comparing rainfall quality with roof runoff quality.
  Only total-P gives a higher value, most likely caused by bird droppings.
  Obviously lead is bound to particles and/or to algae on the roof and is washed off only during high rainstorm intensities.

The behaviour of zinc is not quite clear, because only about half the rain water concentration is found back in the roof runoff.
Further investigation of the processes on the roof are needed.

- The runoff from a "town-motorway" is polluted mainly by lead, zinc, copper, organic (micro)pollutants like oil and polycyclic aromatic hydrocarbons (both about 4 times the concentration in the housing area runoff) and a high content of suspended solids, from which 1/3 is of organic origin. Motorway runoff is the most polluted runoff from the surfaces considered here. Treatment is expedient.
- The influence of corrosion of galvanized railings in the housing area on the zinc concentration of the runoff is evident. Analysis of the rain water dripping off from those railings showed a specific load of 4.4 mg Zn/m^2/mm rainfall.
  The total zinc discharge from the housing area is almost completely explained by the corrosion of the galvanized railings and the zinc content in the stormwater.
- To obtain a complete insight in the change of the stormwater quality caused by contact with various runoff elements - like streets gully pots, sewer pipes - more research is needed.

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THE IMPACT OF WET WEATHER FLOW ON THE HYDROGEN SULPHIDE FORMATION IN COMBINED SEWERS

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Abstract

The primary objective of the study was to investigate hydrogen sulphide formation rates and concentration levels in pressure mains serving combined sewered areas. The investigations were carried out in a 3.9 km pressure main which transports the sewage from a combined sewered town of 12,000 equivalent persons of COD and a catchment area of 106 ha. During dry as well as wet weather periods flow conditions, temperature and concentrations of hydrogen sulphide, sulphate and soluble COD were recorded. Based on these data and sewer design parameters an empirical formula which estimated the hydrogen sulphide concentration was developed and evaluated. This model was the fundamental basis on which to establish an optimal procedure for regulation of chemicals to the wastewater in sewers in order to reduce the hydrogen sulphide problem. The investigations showed that the model depended on the daily variation and dry/wet weather conditions of the flow, the temperature and the COD concentration in a given sewer. Reduced COD concentration and residence time in the sewer during rain events would considerably depress the hydrogen sulphide concentration and reduce the need for chemicals.

1 Introduction

Generation of sulphides in sewerage systems is a well known phenomenon in countries with a warm climate. A wide complex of problems are associated with the hydrogen sulphide formation among which the following are common:

- Corrosive effect on concrete pipes and pumping stations due to biological oxidation to sulphuric acid on moist surfaces.
- Odor and human health problems.

Thistlethwayte (1972) and USEPA (1974).

Under anoxic conditions in the sewers bacterial metabolism in the wall slimes, i.e. the biofilm, causes hydrogen sulphide production. The generation of the sulphides takes place through decomposition of domestic and industrial wastes while inorganic sulphate is reduced to sulphide. The microorganisms involved in this process are usually referred to as sulphate reducing bacteria, Postgate (1984). Sanitary sewage generally contains the substrates - sulphate and organic matter - which are the basic elements required for the sulphide generation. Under the temperature conditions prevailing in the Northern part of Europe, e.g. Denmark, the bacterial activity in the partly filled gravity sewers will usually not deplete the dissolved oxygen concentration to zero although examples of hydrogen sulphide build-up in these systems are known, Kuntze (1983). Until recently the gravity sewer system was absolute predominant in Denmark. However, during the past decade large, central sewage treatment plants have become more and more common. This trend has resulted in an increased demand for full flowing pressure mains as the most efficient way of transportation of sewage to the centralized treatment plants. In these closed systems the dissolved oxygen in the sewage is rapidly consumed leaving the sewer in an anoxic state where hydrogen sulphide production may take place.

An important design parameter for the pressure main which is closely related to the potential for hydrogen sulphide formation is the ratio between the highest and the lowest value of the sewage flow. In a sewer serving a separate sewered catchment area this ratio is typical in the order of 4:1 indicating the daily and annual flow pattern. From combined sewered areas the ratio may easily reach a value of 10:1 in order to protect sensitive receiving waters from combined sewer overflows. This fact results in increased residence time for the sewage in the pipe during dry weather conditions and a subsequent higher sulphide concentration in a combined sewer compared with a separate sewer system.

It is the purpose of the paper to highlight the impact of wet and dry weather conditions on the hydrogen sulphide formation in pressure mains serving combined sewered areas. Especially, the development of a model for forecasting sulphide production will be emphasized. This model will be evaluated and compared with models proposed by Boon and Lister (1975) and Pomeroy and Parkhurst (1977).

2 Investigated sewer system

The investigations were performed on a 3.9 km pressure main connecting two pumping stations, p4 (Vejby) and p3 (Boerglum) (Figure 1). This pressure main is a part of a sewerage system serving a wastewater treatment plant in Loekken located in the Northern part of Jutland, Denmark (Figure 1). Sewage equivalent to 12,000 persons from a combined sewered town (Vraa) is pumped through the pipe.

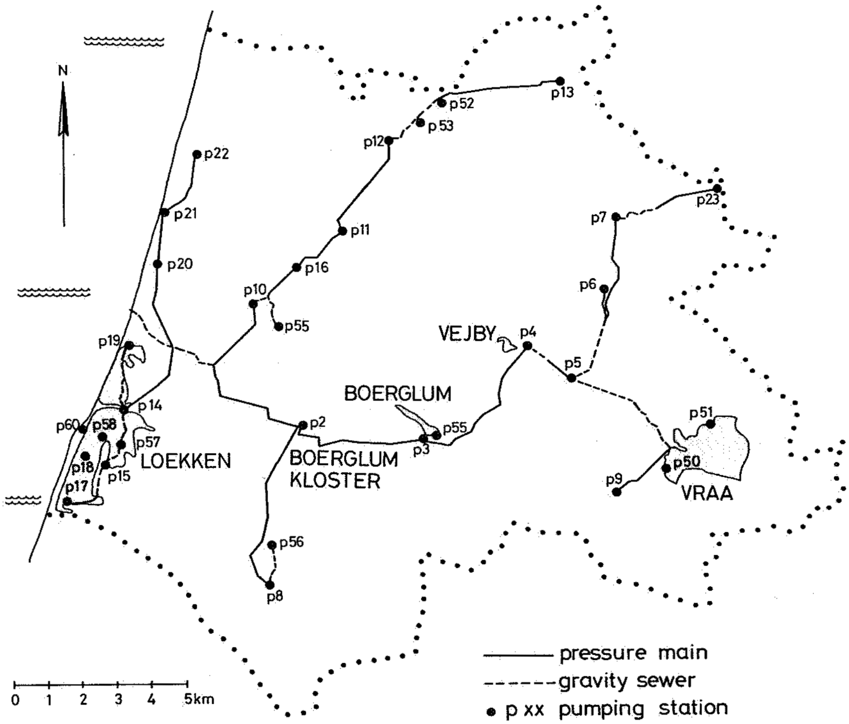

Figure 1 The main sewerage system for the Loekken-Vraa municipality

Table 1 and 2 summarizes the main characteristics for the pressure main investigated.

Table 1 Chief characteristics for the pressure main connecting pumping station p3 and p4

Diameter of pipe:	
inside	376.6 mm
outside	400 mm
Total length of pipe	3890 m
Total volume	433 m^3
Inside area of pipe	4602 m^2

Table 2 Data for the pumps in pumping station p4

Number of pumps in operation	Maximal flow in pipe (m^3/hr)	Velocity in pipe (m/s)
1	260	0.65
2	430	1.07
3	500	1.25

3 Methodology

The investigations were carried out in July-November 1985. During this period of time the daily dry weather flow varied between approximately 25 and 80 m^3/hr. Based on data from table 2 this implied that one pump on average operated between approximately 5 and 20 minutes per hour. In pumping station p4 one pump was in action for at least 3-4 minutes continuously. Therefore, the flow pattern in the pipe was intermittent with a maximal velocity of 0.65 m/s. During a rain event the pattern may change from intermittent to continuous flow and up to three pumps may be in operation at a time (Table 2). The flow pattern was followed intensively throughout the whole period of investigation. The continuous registration of number of pumps in operation gave the information needed to identify and calculate the residence time for each unit of volume arriving at pumping station p3.

Sampling took place in the wet wells in the two pumping stations. Samples were collected automatically every second minute while the pumps were in operation and one hour composite samples were prepared for analysis.

Samples for sulphide and sulphate analysis were preserved with zincacetate and to samples for COD analysis sulphuric acid was added. This procedure required two sample collectors in station p4. Twentyfour samples were collected to cover the variations throughout one day and night.

The samples were carried to the laboratory during the sampling period, and samples for sulphate and COD were filtered (0.45 µm) and refrigerated at 4 ^{o}C until analyzed.

The samples were analyzed for COD (dichromate method), sulphate (turbidimetric method) and sulphide (methylene blue method). All methods were slightly modified according to Standard Methods (1980).

It is important to notify that the analysis for the total sulphide concentration (H_2S, HS^-, S^{--} and FeS) makes it possible to calculate the flux of sulphide from the biofilm whether this was precipitated in the sewer or not.

Based on previous experiments and calculations only a minor relative change in the concentrations of COD and SO_4^{--} in the sewer was expected. Therefore, only the samples from pumping station p4 were analyzed for soluble COD and SO_4^{--}. Duplicate or triplicate analyses of all the individual collected samples were performed.

Also pH, redoxpotential and temperature were measured at intervals.

4 Experimental results and discussion

All experimental results will not be shown in this paper. However, table 3 gives an overview of the results obtained. Every period, day and night, comprises 24 data of the parameters given.

4.1 Model for hydrogen sulphide formation during a dry weather period

Generally, a model for forecasting sulphide production in sewers must

Table 3 Overview of data from the study related to hydrogen sulphide formation

Period no.	Date (1985)	Total sulphide concentration, C_S (gS/m^3)		Sulphate conc., C_{SO_4}	Soluble COD, (g/m^3)	Residence time t_h(hr)	Temperature t(°C)	Weather
		p4	p3	(gS/m^3)				
1	July 1-2	0-0.4	0.1-4.5	∿20 (few up to 75)	130-570	6.0-11.7	11-13	dry
2	Aug. 19-20	0.1-1.0	0.1-0.9	20-35 (single: 138)	50-350	1.0-11.7	12-16	wet
3	Sep. 30 -Oct. 1	0.1-0.6	0.1-1.9	∿20 (single: 104)	50-440	3.7-7.3	12-13	wet
4	Nov. 5-6	0-0.3	0-1.2	8-22	50-370	1.7-6.2	4-7	wet

take into account the substrate for the sulphate reducing bacteria, i.e. the concentrations of sulphate and the organic carbon source (COD). In the study reported here the COD concentration, but not the sulphate concentration, affected the rate of sulphide production (Figure 2).

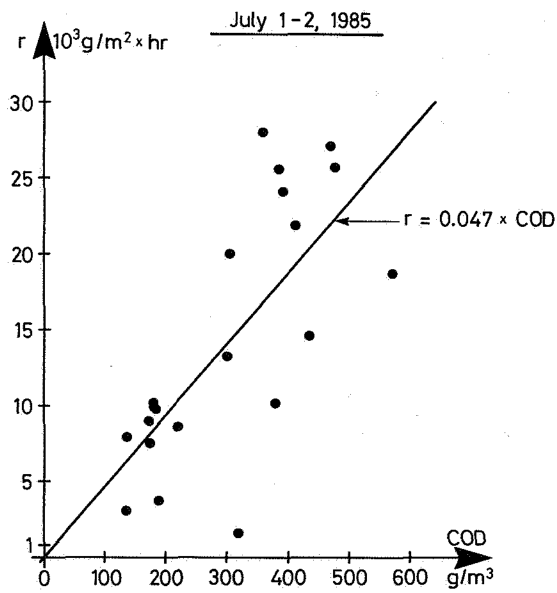

Figure 2 Correlation between sulphide production rate (flux of sulphide from the slime layer), r, and the soluble COD conc. in the sewage

The sulphate concentration level was about 20 gS/m^3 and very few samples were below 15 gS/m^3 (Table 3). Calculations showed that above this level there was no effect of the sulphate concentration on the sulphide production rate. However, under conditions with a higher COD concentration the sulphate concentration may turn out to give an effect on the rate, Thistlethwayte (1972).

The rate shown in Figure 2 was based on hourly mass balances for sulphide production in the sewer involving increase in the total sulphide concentration for hourly pumped volumes from pumping station p4, residence time in the sewer and pipe design parameters.

Based on the findings mentioned the equation of the regression line (Figure 2) was:

$$r = 0.047 \times 10^{-3} \times COD \qquad (1)$$

where

r = rate of sulphide production (flux of sulphide from the biofilm), $g/m^2 \cdot hr$

COD = chemical oxygen demand for filtered samples, g/m^3

Equation 1 was evolved at 12 ^{o}C. It appears that the over-all effect of temperature on the sulphide production in sewer slimes is about 0.07 $^{o}C^{-1}$, Pomeroy and Bowlus (1946). At 20 ^{o}C equation 1 resulted in:

$$r = 0.080 \times 10^{-3} \times COD \qquad (2)$$

The specific sulphide flux coefficient (in units of m/hr) corresponded with a preliminary study carried out in May 1985.

Equation 2 is similar to the following two empirical models forecasting sulphide production in full pipes:

$r = 1.0 \times 10^{-3} \times BOD$ Pomeroy and Parkhurst (1977), USEPA (1974)

$r = 0.228 \times 10^{-3} \times COD$ Boon and Lister (1975)

With a BOD/COD conversion factor of about 0.7 for the case in question it is clearly shown that the specific sulphide flux coefficient cannot be considered as a universal constant. Probably, the type of wastewater characterized by the relative amount and the quality of industrial wastes will play an important role. It is well known that the sulphate reducing bacteria have preference for certain organic species like short chain fatty acids, Postgate (1984). However, little is known about the utilization in the sewer slimes of the organic substrate available in sanitary sewage. At the time being the soluble COD or BOD is probably the best available measure of the degradable carbon source for the sulphate reducing bacteria due to the fact that only soluble and colloidal organic matter can penetrate the biofilm and undergo degradation. The particulate organics may to some extent be hydrolized, but due to a relative short residence time in the sewer this process is not expected to be of great importance.

The activity of the sulphate reducing bacteria can be typified by the following reaction:

$$SO_4^{--} + 2 \text{ lactic acid} \rightarrow H_2S + 2 \text{ acetic acid} + 2\ CO_2$$

This reaction shows that approximately 6 g of COD is consumed per g of SO_4^{--}-S which is reduced to H_2S. Probably, far below 50% of the soluble COD in wastewater is directly degradable by the sulphate reducing bacteria.

Equation 1 led to the following expression for the formation of hydrogen sulphide in a pressure main where anoxic conditions exist:

$$\Delta C_S = 4.7 \times 10^{-5} \times 1.07^{t-12} \times COD \times \frac{t_h \cdot A}{V} \qquad (3)$$

where

ΔC_S = formation of total sulphide in a pressure main, gS/m^3

t = temperature, ^oC

t_h = residence time of sewage in the sewer, hr

A = inside area of the pipe, m^2

V = total volume of the pipe, m^3

As indicated (Figure 2) equation 3 is verified for COD values below about 500 g/m^3. Furthermore, it ought to be mentioned that V/A in equation 3 equals the hydraulic radius.

It is important to notify that equation 3 is an empirical model which require a calibration on the sewerage system in question. Until now we do not have a sufficient theoretical basis for development of a management model taking microbial biofilm kinetics into account although attempts have been made, Holder et al. (1984).

4.2 Formation of hydrogen sulphide during a wet weather period

Based on equation 3 the important parameters for the hydrogen sulphide formation in a pressure main is COD, residence time, temperature and hydraulic radius. When a change from dry to wet weather conditions takes place temperature may be different but usually not considerably. Therefore, the main parameters characterizing the difference between dry and wet weather with respect to hydrogen sulphide formation is COD and residence time.

Three of the four periods in which investigations were carried out include rain events (Table 3). Comparison of the daily variation of COD and residence time, t_h, in a dry weather period (Figure 3) with the corresponding patterns in wet weather periods (Figure 4a, b and c) showed typical differences.

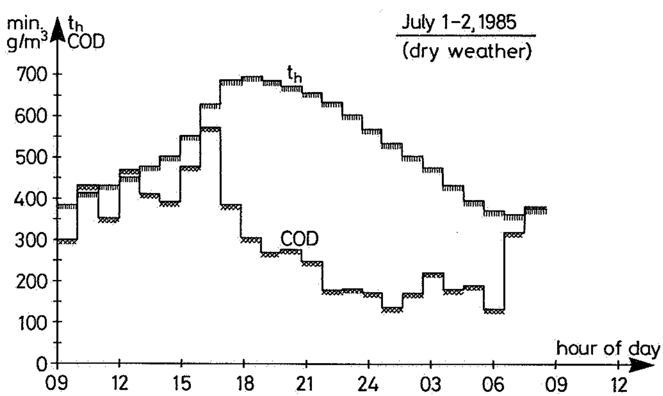

Figure 3 The daily variation of COD and residence time, t_h, during a dry weather period

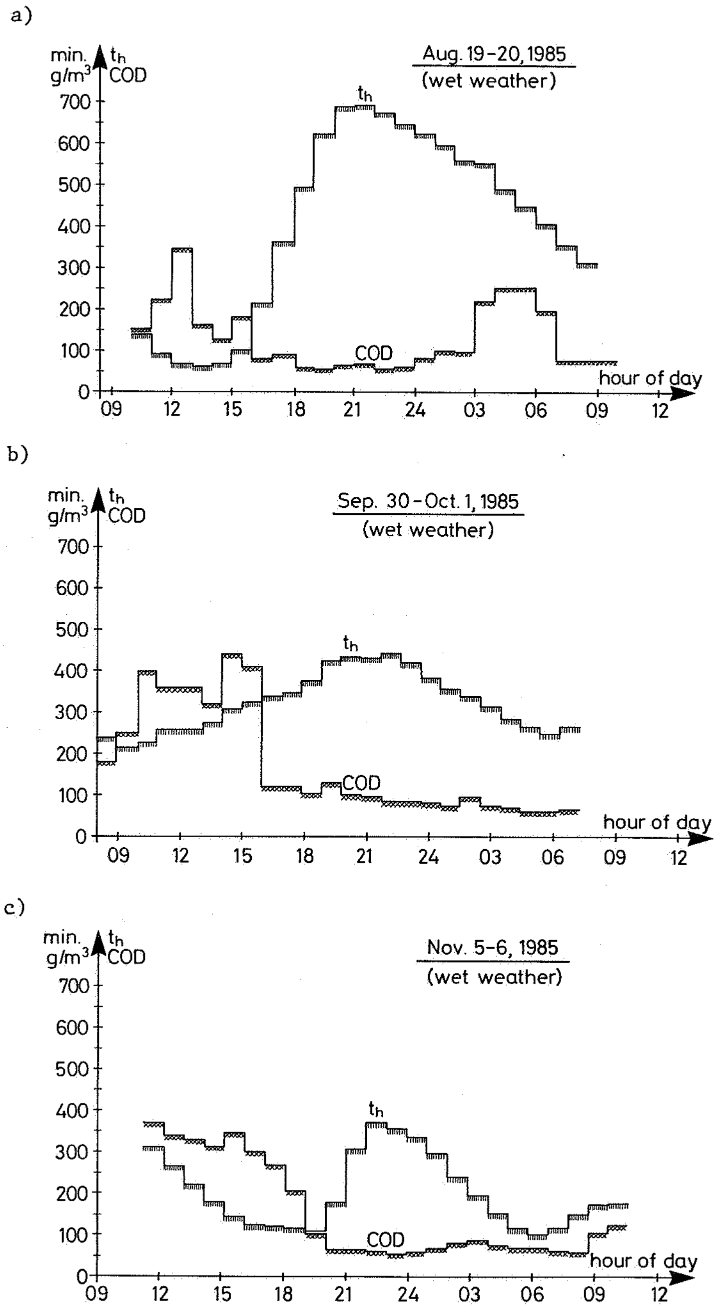

Figure 4 The daily variation of COD and residence time, t_h, during wet weather periods

The lower limit for the COD concentration in combined sewers during a rain event is determined by the concentration in the stormwater runoff overlaid by the contribution of resuspended materials from the sewer itself. Based on a study carried out in the main sewers of two typical combined sewered areas in Copenhagen, Denmark, Johansen et al. (1981) estimate the soluble COD concentration at 30 g/m^3. This number and the results from this study (Figure 3 compared with Figure 4) showed that the COD concentration may be lowered considerably during a rain event. Furthermore, it might be expected that the COD originating from urban runoff is of poor quality as carbon source for sulphate reducing bacteria.

A main problem in combined sewered areas is related to receiving water impacts from overflows. This problem calls for increasing capacity to be available in the sewerage system. However, as seen through this study, the serious adversity related to hydrogen sulphide formation was the increased residence time under dry weather conditions. As an example it was shown that the lowest residence time in the 3.9 km sewer from pumping station p4 to p3 was 52 minutes (Table 1 and 2). This value was almost reached on Aug. 19 (Figure 4). During a typical dry weather period the capacity needed was only about 15% of what was actually available (Figure 3). The importance of this fact for the hydrogen sulphide formation was readily indicated (Equation 3). Consequently, design of pressure mains without considering urban storm runoff would considerably reduce the hydrogen sulphide problem.

It is important to ensure that the model (Equation 3) which was developed to forecast the sulphide production could operate under a wide range of flow values and COD concentrations in order to be usefull for combined sewer systems. Therefore, the measured production of sulphide from pumping station p4 to p3 was compared with calculated values based on equation 3 (Figure 5).

As seen (Figure 5) the model (Equation 3) which was developed based on dry weather data yielded fairly good predictions for sulphide production under wet weather conditions too. The relative error may be high, but the level was generally well estimated.

However, as seen from Figure 5, in parts of the wet weather periods the observed values for sulphide production were higher than the prediction showed (Figure 5c and d). The reason for this inconsistency was not

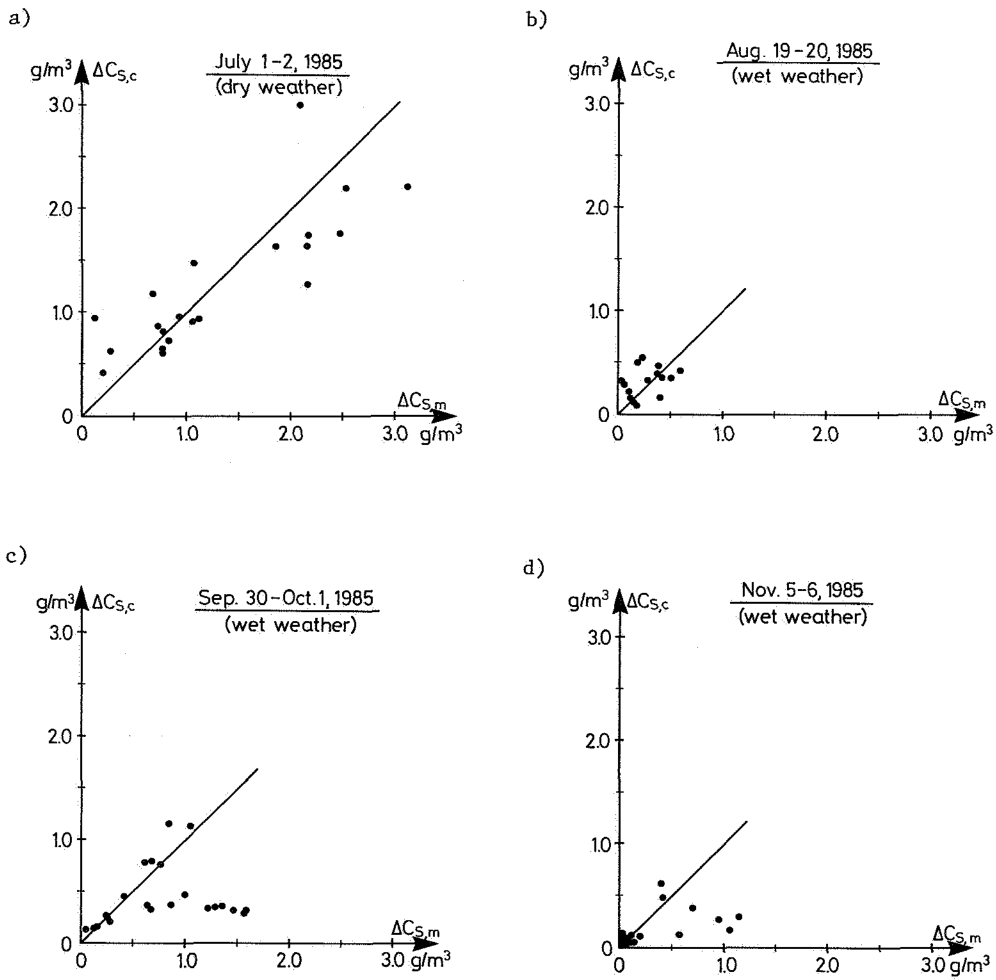

Figure 5 Comparison between measured, $\Delta C_{S,m}$, and calculated, $\Delta C_{S,c}$, values of sulphide production in a 3.9 km pressure main; a: dry weather; b, c and d: wet weather

clear but may be caused by other factors than bacterial activity, e.g. resuspension of materials settled in the sewer pipe and a scouring effect on the wall slimes, both phenomena caused by increased velocity in the pipe. From a previous study, Pomeroy and Bowles (1946) report that the scouring effect is probably not of much consequence if the velocity is under 0.9 m/s. This limit was exceeded during a rain event which resulted in two or three pumps in operation at a time (Table 2).

5 Conclusions

Based on dry weather conditions an empirical model forecasting sulphide production in pressure mains was developed. The factors which form the model were the soluble COD of the sewage, the residence time in the pipe, the temperature and the hydraulic radius of the pipe. The present study showed that the use of an empirical model required that a calibration on the sewerage system in question is performed.
The model developed yielded fairly good predictions on wet weather conditions characterized by low COD and high flow rates. Therefore, the model is a valuable tool for forecasting the sulphide production in combined sewer systems and a basis for optimizing the use of chemicals in order to reduce the hydrogen sulphide problem.

Acknowledgement

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DISCHARGES OF POLLUTANTS FROM DUTCH SEWERAGE SYSTEMS; AN OVERVIEW OF MEASUREMENTS, EXPERIENCES AND RESULTS

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Abstract

Since 1981 an extensive data collection and research programme has been carried out in the Netherlands on quantity and quality aspects of storm water discharges from sewerage systems and their effects on the quality of receiving waters.

In six drainage areas served by typical Dutch sewerage systems the precipitation and storm water discharges are measured continuously. Samples of the discharges are taken and analysed for the most important pollution parameters.

The results of these investigations show that pollution emission increases considerably when both the rainfall intensity and the overflow discharge are high. No relation could be established between pollution emission and duration of the dry weather period before a shower.

In typical Dutch sewerage systems, characterized by small hydraulic gradients and large sewer diameters, the sewage sludge settles easily. As soon as flow velocities in the sewers increase, the sludge can be stirred up and transported to the overflow. High flow velocities generally occur when the overflow discharge is extremely high.

1 Introduction

For many years concern has been expressed about the pollution of surface waters caused by storm water discharges from sewerage systems. In 1977 STORA - a foundation of Dutch governmental bodies for water quality control and treatment of wastewater - commissioned DHV Consulting Engineers to carry out a study on the pollution emission from sewer outfalls.

The goal of the study is to provide insight into the internal processes of sewerage systems, particularly with regard to the discharge of pollutants into receiving waters. Moreover, the results of the study should allow prediction of the quantity and quality of the overflows from various sewerage systems during storm water periods. This knowledge is a prerequisite for a costeffective approach to the design and upgrading of sewerage systems for environmental improvement.

After a comprehensive literature survey and a selection of the most appropriate method of data analysis, a prototype measuring apparatus was developed and installed in 1979. This measuring apparatus allows the collection of data on rainfall and the quality and quantity of storm water overflow. The prototype was tested for reliability and accuracy during the year 1980.
Since 1981 investigations have been carried out on the discharge of pollutants from combined and separate sewerage systems. This paper comprises an overview of the results and a brief description of the experience gained, with specific reference to typical Dutch combined sewerage systems.

2 Characteristics of Dutch combined sewerage systems

The traditional sewerage design in the Netherlands differs considerably from other countries in the world, mainly because of the flatness of our country. The small difference between streetlevel and water table of receiving waters is very small. This flatness, together with high ground water tables and bad soil conditions, means that sewerage systems with very slight slopes are inevitable. Furthermore, this flatness results in a very low hydraulic gradient for the entire sewerage system. In the Netherlands, hydraulic gradients are generally from 1:500 to 1:1000, so that velocities in Dutch sewer systems are rather low. All these aspects result in rather stable flows in the system and settling of sewage sludge should be taken into account.

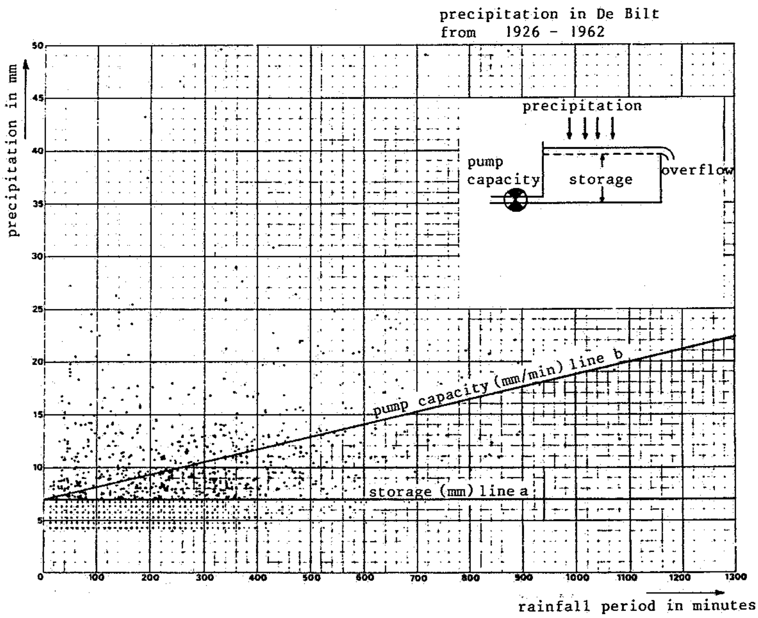
During heavy storms all sewers get filled completely, which justifies a design method based on permanent flow conditions and a permanent design rainfall intensity. Due to the flatness of the drainage areas and the slight slopes of the sewers the storage capacity of Dutch sewerage systems is considerably larger than in other countries. This storage capacity of the sewerage system is a very important aspect of Dutch design methods. The occurrence of storm water overflows mainly depends on this storage capacity and on the pumping capacity to the sewage treatment plant.

Design criteria for limiting discharges of pollutants on receiving waters are also based on the Dutch principle of making good use of the storage capacity of the sewerage system. A theoretical overflow frequency per year is established on the basis of historical series of rainfall (see Figure 1). The permitted theoretical overflow frequency of combined sewerage systems is 5 to 10 times per year.

3 Form of the investigations

To quantify the pollution emission, an extensive monitoring programme was set up; the main elements were the registration of intensity, amount and duration of precipitation, and the flow, duration and pollution of storm water discharges.

To limit the extent of the investigations, for practical and economical reasons, the measuring areas selected were drainage areas served by fairly standard Dutch sewerage systems with only one outfall.



showers above line b give an overflow
showers beneath line b give no overflow

Figure 1 The "Kuyper model" with historical rainfall data

The measuring apparatus is located near this outfall and continuously registers the rainfall and the sewage waterlevel, and measures each storm water discharge. In addition, the waterlevel of the receiving water is registered. During the overflow, volume-proportional samples are taken automatically and stored in a refrigerator. These samples are analysed for the main chemical and physical parameters that influence the quality of the receiving water. Table 1 shows the analysis programme of the samples. The concentrations of heavy metals are determined on the basis of a single composite sample from all the samples collected.

Table 1 Analysis programme of samples from combined sewerage systems

Analysis of original sample	Analysis after 1 hr settlement	Analysis of composite sample per overflow
- COD	- COD	- lead
- BOD	- BOD	- zinc
- N_{Kj}	- N_{Kj}	- iron
- P_{total}	- P_{total}	- chromium
- Cl	- sediment	- copper
- dry residue	- dry residue	- nickel
undissolved solids	suspended solids	
- pH		- mercury
- conductivity		- cadmium
- sediment		

For some areas a supplementary investigation programme was set up, in cooperation with the Ministry of Housing, Planning and Environment, to study the effects of pollution emissions from sewerage systems on the quality of receiving waters.

4 Measuring areas

The monitoring programme was started in 1981 and will continue till the end of 1986, covering six drainage areas. An overview of the monitored areas, terrain slopes, types of sewerage system and periods of monitoring is given in Table 2.

As an illustration the major area characteristics of two combined sewerage systems (Loenen and Oosterhout) are given in Table 3.

Table 2 Measuring areas

Area	type of sewerage system	terrain slopes	period of monitoring*
Loenen	combined	moderate	1981-1985
Oosterhout	combined	flat	1982-1986
Heerhugowaard	separate	flat	1984-1985
Bodegraven	combined	flat	1983-1986
Kerkrade	combined	steep	1983-1984
Amsterdam	separate	flat	1984-1985

* The period of monitoring depends on the type of sewerage system and frequency of storm water discharges.

The theoretical overflow frequency in Table 3 was calculated from the standardized "Kuiper model", using the point plot of rain data of De Bilt (ref. 3).

Table 3 shows that the observed overflow frequencies are higher than the theoretical values. Due to the relatively short period of monitoring, no firm conclusions can be drawn from this difference. Several aspects have to be taken into account, such as:

- a possible relatively "wet" monitoring period
- the rainfall pattern in Loenen and Oosterhout may differ from that of De'Bilt
- the performance of the "Kuiper model" for calculations of the theoretical overflow frequency

To overcome the influence of these aspects on the overflow frequency an additional study was started. The reliability of the "Kuiper model", the value of the run-off coefficient and the differences of precipitation across the Netherlands were studied. The complete results of these studies will be available within a year.

Table 3 Some characteristics of two areas served by combined sewerage systems

Aspect	Loenen	Oosterhout
Population [inh]	2050	2270
Size of drainage area [ha]	56.5	22.0
Impermeable surface [ha]	15.8	11.6
Impermeable surface [m^2/inh]	77	51
Storage capacity sewerage system [mm]	5.7	5.3
Storage capacity sewerage system [m^3]	895	620
Pump capacity*) to treatment plant [mm/hr]	0.88	0.97
Pump capacity [m^3/hr]	140.7	113
Theoretical overflow frequency [1/year]	9	9
Observed overflow frequency [1/year]	15.7	12.4
Number of monitored overflow events	55 (6/'81-1/'85)	31 (6/'82-1/'85)

*) dry weather flow not included

Provisional results of the studies show that the "Kuiper model" is not accurate, due to the assumption that the complete storage capacity of the sewerage system is always available at the beginning of a shower. Another inaccuracy is the supposition that the run-off coefficient of impervious areas equals 1. Especially in Loenen it has been observed that during the winter this coefficient can be considerably higher and in the summer lower. For rather flat areas this range amounts 0.8-1.2.

The study about differences in heavy storm water events across the Netherlands, carried out by the Royal Dutch Meteorological Institute (ref. 2) pointed out that compared to "De Bilt" (in the centre of the Netherlands) differences may occur from -20 to +30 percent.

5 Results of the investigations of combined sewerage systems

5.1 *Overflow discharge data*

The quantity of discharged sewage differs per overflow. Figures 2 and 3 show the distribution of the discharges per overflow expressed in m^3 and also in mm related to impermeable surface.
It can be seen that most overflows have rather small quantities of discharge expressed in mm related to impermeable surface. In the flat area of Oosterhout only a few overflows have been measured with discharges of more than 10 mm.

In the Loenen area the discharges are relatively larger, possibly as a result of run-off from permeable surfaces to the sewerage system.
An overview of the average quantity of discharges per year is given in Table 4.

Table 4 Average quantities of overflow discharges from combined sewerage systems

Drainage area	Average quantity of storm water overflows per year (mm related to impermeable surface)
Oosterhout	74
Loenen	120

5.2 *Emission data*

The discharge of pollutants into receiving water is the main issue the study. Therefore each overflow has been sampled in proportion to volume.

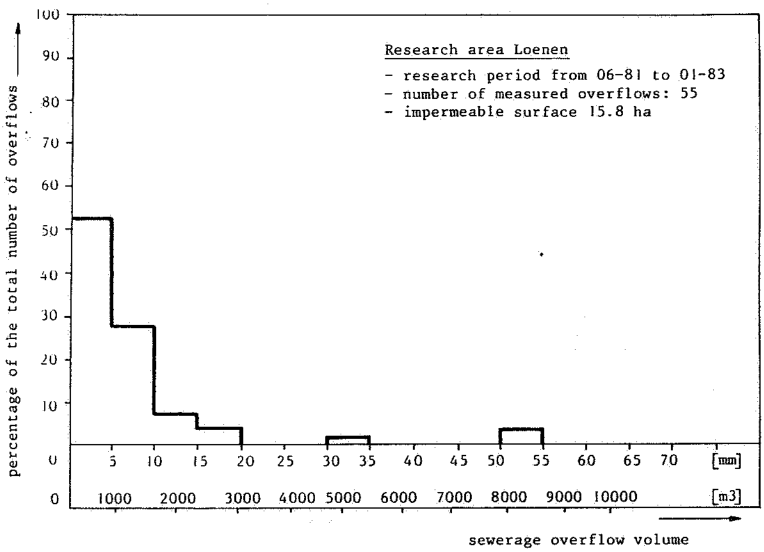


Figure 2 Storm water discharges in Loenen

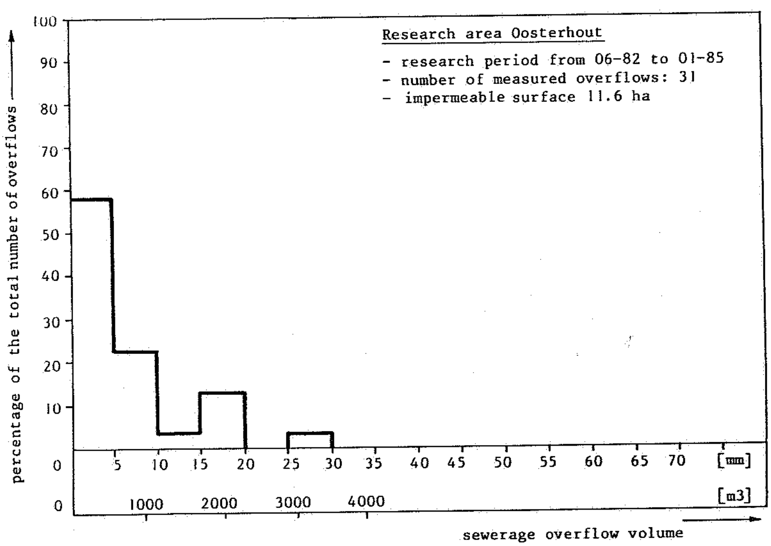
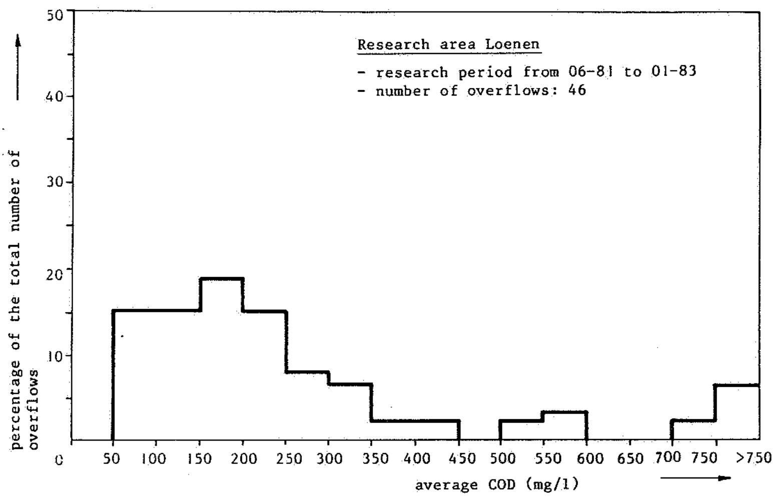


Figure 3 Storm water discharges in Oosterhout



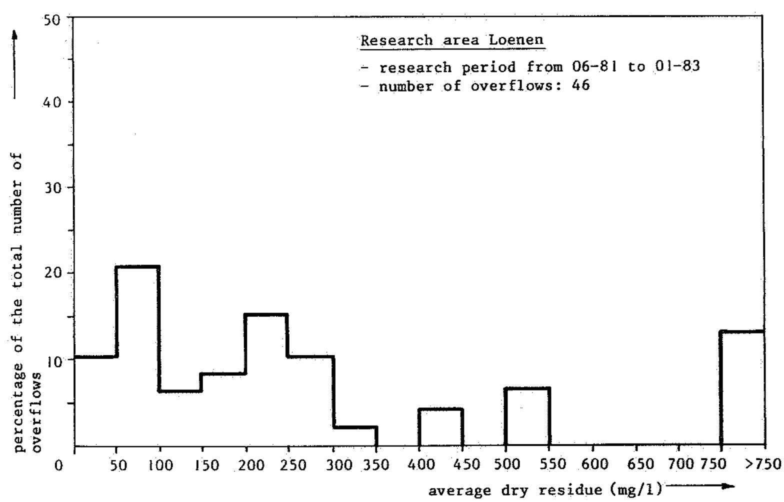
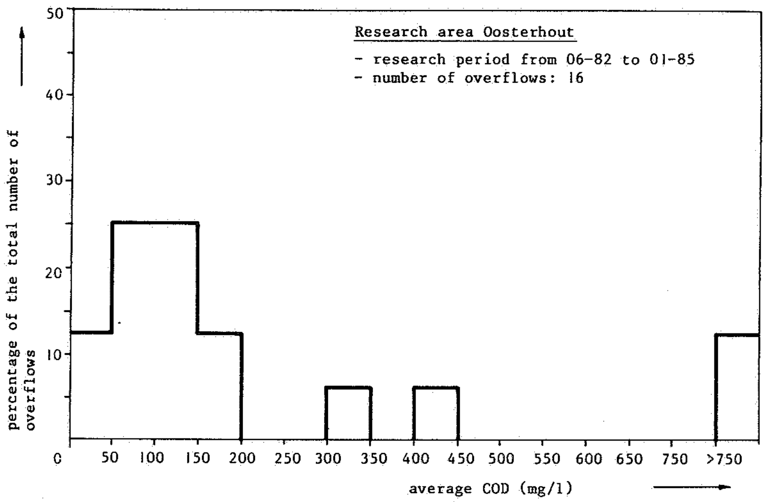


Figure 4 Distribution of the average pollutant concentration per overflow in Loenen



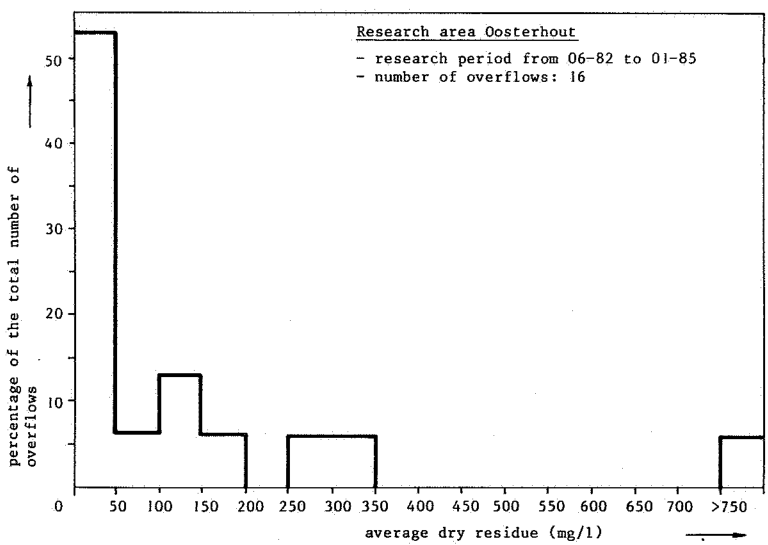


Figure 5 - Distribution of the average pollutant concentration per overflow in Oosterhout

All samples are analysed for physical and chemical parameters (see Table 1). Here only the results of the analysis for COD and dry-residue undissolved constituents are presented, see Figures 4 and 5. These figures show a COD range between 50 and 750 mg/l. The average values for Loenen and Oosterhout amount to 271 mg/l and 345 mg/l res-pectively. For the flat area of Oosterhout the dry residue results generally show a lower value than those for the slightly hilly Loenen area. Obviously these terrain circumstances cause a larger transport of anorganic material and other undissolved constituents.

5.3 *Relationship between parameters of pollution*

To increase the insight into the major processes, the average concentrations of the various parameters are compared for mutual relations. The relation between the average dry residue concentrations and several other concentrations is also examined to confirm anticipated effect of stirring up the sewage sludge.

These relations for the average concentrations of COD, BOD and dry residue are shown in Tables 4 and 5.

6 Evaluation of results

A great number of processes in and outside the sewerage system influence discharges of pollutants into receiving water. A rough distinction can be made between processes in the atmosphere, run-off to the sewerage system and the internal processes in the sewers. These processes can be characterized by process parameters such as rainfall intensity, rainfall depth, dry weather period etc. A preliminary search has been made for relations between process parameters and pollution emission.

During dry weather periods a combined sewerage system transports rather small quantities of sewage from domestic and industrial users. The flow velocities are low, so settlement occurs. The amount of settled sludge

depends on the duration of dry weather periods.
Nevertheless, from the investigations up to now, no significant relation between discharge of pollutants and dry weather period could be indicated. The reason for this discrepancy is that the sewerage system is not flushed clean during each storm event. Dependent on the rainfall intensity, the sludge in the sewers will be stirred up and flushed away, or will settle again due to the low velocities. So even during storm water events, there is no total mixing of sewage, rainwater and sludge in the sewerage system. The rainfall intensity is a better parameter for the process of stirring up deposits than the wash out of dirt and sediment from the street and permeable surfaces.
A study of the maximum average rainfall intensities during periods of 15 or 30 minutes and the average pollution of the overflowing sewage has shown a remarkable correlation.
The correlations for the pollution parameters COD and dry residue are shown in Figure 6.

Table 4 Best possible correlation between pollution parameters in Loenen

pollution parameter		event	correlation coefficient	best possible correlation formula
X*	Y**	n	ρ	
BOD	- N-kj	47	0.82	$Y = -14.4 + 5.16X$
BOD	- dry residue	47	0.65	$Y = 21.7 + 0.059X$
COD	- BOD	47	0.71	$Y = 12.4 + 0.098X$
COD	- N-kj	47	0.87	$Y = 5.2 + 0.019X$
COD	- P-tot	47	0.95	$Y = 1.0 + 0.0072X$
COD	- Pb	26	0.82	$Y = 22.1 + 0.45X$
COD	- Zn	27	0.68	$Y = 162.3 + 0.65X$
COD	- Fe	18	0.66	$Y = 91.0\ X^{0.72}$
dry residue	- P-tot	46	0.95	$Y = 1.53 + 0.0048X$
dry residue	- Pb	26	0.88	$Y = 5.3\ X^{0.595}$
dry residue	- Fe	18	0.71	$Y = 327.4\ X^{0.51}$
dry residue	- COD	46	0.97	$Y = 78.4 + 0.65X$

Table 5 Best possible correlation between pollution parameters in Oosterhout

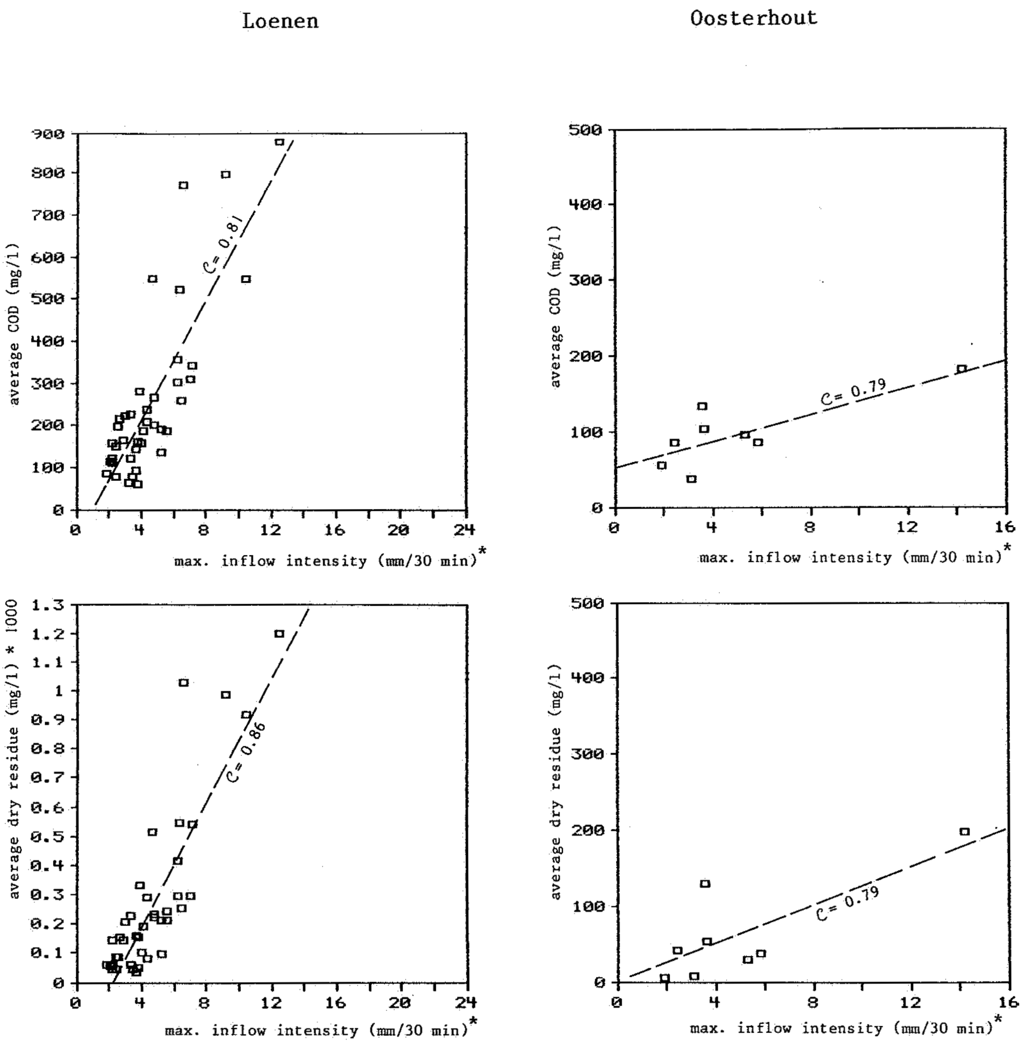
pollution parameter		event correlation coefficient		best possible correlation formula
X*	Y**	n	ρ	
BOD	- N-kj	15	0.91	Y = 5.09 + 0.084X
BOD	- dry residue	14	0.98	Y = -45.9 + 3.50X
COD	- BOD	15	0.99	Y = 2.6 + 0.27X
COD	- N-kj	15	0.89	Y = 5.4 + 0.022X
COD	- P-tot	15	0.90	Y = 1.46 + 0.0083X
COD	- Pb	13	0.72	Y = 38.7 + 0.18X
COD	- Zn	13	0.84	Y = 98.6 + 1.02X
COD	- Fe	13	0.82	Y = 447.9 + 8.26X
dry residue	- P-tot	15	0.86	Y = 1.9 + 0.0084X
dry residue	- Pb	11	0.97	Y = 35.6 + 0.22X
dry residue	- Fe	13	0.87	Y = 557.8 + 9.27X
dry residue	- COD	14	0.99	Y = 43.3 + 1.05X

* in mg/l

** for Pb, Zn and Fe in µg/l, others in mg/l

7 Conclusions and follow-up investigations

Characteristic for Dutch sewerage systems are the large diameters of the sewers especially to the sewer outfall. The large sewers are used as storage in case of heavy storm-waters. Because of these large diameters, the velocities in the sewerage system during dry periods are rather low, causing settlement of sewage sludge during dry weather periods.



* max. inflow intensity = run off * max. average precipitation intensity during 30 minutes of the shower

Figure 6 Relation between average pollutant concentration of the discharge and the rainfall intensity in Loenen en Oosterhout

From the extensive data collection programme (STORA 38b) it can be concluded that such the deposits will only be stirred up, if the precipitation intensity is sufficient to produce a considerable increase of flow velocity in the sewerage system.

In a typical Dutch sewerage system the flow velocities will only increase considerably if the rain intensity, as well as the overflow discharge, is high. Velocities under normal flow conditions to the sewage treatment plant are too low to stir up sewage sludge. As soon as the flow velocities decrease, the deposits can settle down again and the pollution concentration of the overflowing sewage will decrease.

From the measurements made, it cannot be concluded that the sewerage system is free of sludge after heavy storm water. On the contrary, the data give reason to believe that a typical flat Dutch sewerage system contains a certain amount of sludge. Therefore, as soon as high velocities occur in the sewerage system the pollution concentration of the overflowing sewage increases.

The follow up investigations are focused on the relation between the pollution concentration and the velocities in the sewers before and during the overflows. The process of stirring up sludge through flow velocities is considered to be the most important and will form the basis of a pollution emission prediction model. Up till now the following second order processes have been considered to be important:

- Mixing of the rainwater with the fluctuating amount of wastewater.
- Wash out of sewers during very heavy stormwaters.
- Seasonal fluctuations in the run-off process

Up till now the accuracy of the measurements give reason to believe that a reliable pollution emission model can be made on the basis of the data collected. Nevertheless, futher effort is needed before the model can be satisfactorily completed.

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VARIATION OF COMBINED RUNOFF QUALITY AND RESULTING POLLUTANT RETENTION STRATEGIES

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Abstract

Pollutant concentrations and loads in combined runoff are variable in time and space. This is demonstrated on the basis of field data. First flush occurrence at the collecting point of a 540 ha residential area was found to be dependent on dry weather flow levels rather than on the duration of dry spells. Knowledge of the variation of combined runoff quality allows for effective measures to retain the runoff with higher pollution levels in the sewer system, thus reducing receiving water loadings. The performance of common retention basin design and the effectiveness of sewer system control are discussed in more detail. Here, for instance, it was found that with sewer system control first flush, if existent, easily could be retained by utilizing the existing in-line storage capacity.

1 Introduction

For hygenic reasons and living comfort waste water has been quickly removed from urban areas in the past. Cities in central Europe predominantly are drained with combined systems, where waste water flows and storm runoff are collected in common sewers. Combined sewer overflows become necessary, as under storm conditions for technical and economical reasons not all of the combined flows can be conducted to sewage treatment plants. In consequence varying pollutant loads are discharged irregulary to receiving waters affecting the biocenosis and in turn water use.

When the magnitude of storm and combined runoff pollution was recognized in the 60ies and early 70ies biological treatment and later advanced wastewater treatment was added to mechanical treatment. The more treatment facilities went into operation the more it was evident, that further investments in wastewater treatment only are effective if at the same time direct discharge of storm and combined flows into receiving waters is controlled. Control of combined runoff, however, faces opposing demands:

- guarantee of hygenic and floodsafe drainage, which in the past was achieved by immediate discharge into receiving waters;
- receiving water protection, which due to the technical limitation of treatment plant capacities requires detention of flows or retention of matters in the sewer system;
- assurance of treatment plant efficiency, which requires equalization of large inflow variations.

Aside from stormwater infiltration and detention on the surface storage and treatment within the sewer system present common solutions to these demands.

The characteristics of combined runoff pollution and the effectiveness of different measures for combined sewer overflow control are discussed in the following on the basis of field data collected in Munich-Harlaching from 1977 to 1981, a residential area of 540 ha in size. Collection and analysis of the data were done within the research project "Niederschlagsabfluß und -beschaffenheit in städtischen Gebieten" within the special research area SFB 81 "Abfluß in Gerinnen", which was sponsored by the German Research society (SFB 81, 1983). Some of the experience with the continuous measurement scheme was summarized by Geiger (1981).

2 Sources and local variation of combined runoff pollution

Combined runoff is composed of dry weather flow and storm runoff. Dry weather flow includes domestic, commercial and industrial wastewater and infiltration water. Dependent on the composition and the interaction of pollution sources combined runoff quality varies not only from area to area but also within one sewer system as runoff develops.

2.1 Development of combined runoff pollution

Domestic sewage mostly contains organic matters, commercial and industrial wastewaters often are loaded with unorganic material. Contamination of storm runoff may result from dust particles washed out from the atmosphere and from the material picked up along its flow paths. Excreta from animals, paper, cigarettes, fruit and vegitable wastes determine the organic loadings of stormwater. Unorganic and toxic matters often result from traffic and industrial land uses. Oxygen, hydrogen and sunlight sometimes leads to decomposition of the deposits in a way that erodable toxic components originate. In addition unconsolidated surfaces may erode and substantially add to the unorganic loadings of stormwater. Dependent on flow velocities and flow quantities the materials are dissolved,picked up and transported, whereby some material might be completely washed off, some only transported for some distance and deposited again. Same applies to the transport of matters in the sewer system, where dry weather flow and storm runoff are combined. Therefore the transport of matters through an urban drainage system is a continuous shifting process which depends on many arbitrary influences as the schematic of Figure 1 also indicates. Obviously along with runoff development runoff quality constantly may change.

2.2 Determination of combined runoff pollution

If the microorganisms, anorganic and organic matters contained in combined runoff are useful or dangereous depends on their quantity and on water use. The characterization of combined runoff pollution usually refers to matters which form or limit physical, chemical or biochemical degradation in the treatment process and selfpurification in receiving waters. The chemical and biochemical load of combined runoff primarily

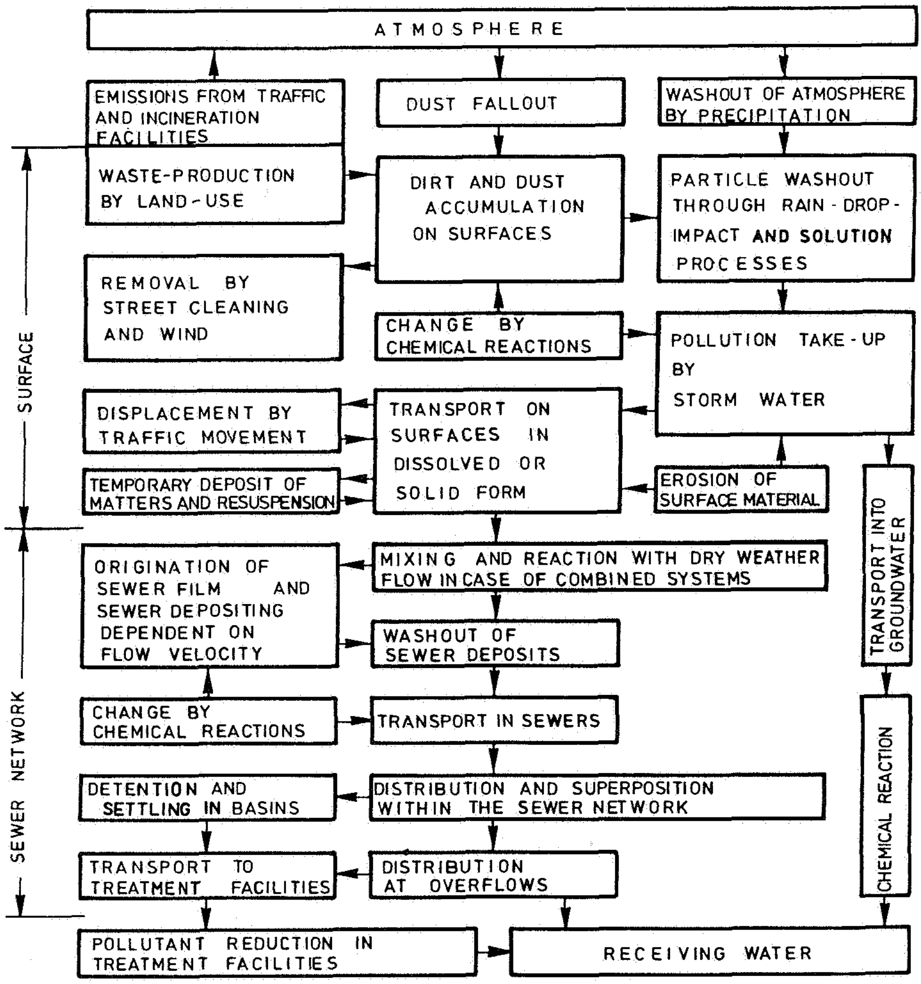

Figure 1 Development of runoff pollution in urban drainage systems (after Geiger, 1984)

is described by composite parameters. Undissolved organic and mineral matters are quantified by suspended and settleable solids. Organic matters are not homogeneous in respect to their degradability. While the biochemical oxygen demand in five days (BOD_5) reflects the dissolved and easily degradable matters, the chemical oxygen demand (COD) presents the hard degradable or nondegradable matters. Total organic carbon (TOC) is a direct measure for the carbon contained in dissolved and undissolved organic compounds. Phosphorus and nitrogen describe the nutrient conditions.

Further pollution parameters such as oil products, different types of bacteria, heavy metals and other toxic compounds would be needed to give a complete description of combined runoff pollution. Engineering practice, however, assumes that there is some correlation between individual contaminants and composite parameters.

2.3 Average concentrations and loads in different areas

Averages of concentrations provide general estimates of runoff quality and reflect the extend of physical, chemical and biological processes. Mean loads are a measure for receiving water impacts in relation to treatment plant charges. The values summarized in Table 1 for suspended solids, BOD_5 and COD illustrate the scatter of mean values for different areas. Suspended and settleable solids are influenced by surface texture and use and especially by construction activities. BOD_5 and COD often reflect the local waste water fraction in combined runoff. In literature the relevant conditions frequently are not reported together with the data. In addition BOD_5 and COD values are influenced by the methods chosen for laboratory analysis, transport and conservation of samples. Extremely important is the number of samples analyzed. Reliable figures only can be derived from longterm data.

From the five year continuous data base established for Munich-Harlaching combined runoff concentrations were derived in two ways, averaging the individual sample values and dividing loading figures by runoff volumes on an event basis. Both results are compared in Table 2

Table 1 Mean concentrations and loads of suspended solids, BOD_5 and COD measured in different combined sewer systems

area of investigation	Suspended solids				COD				BOD_5			
	mg/l	kg/ha·a	$kg/ha_r \cdot a$	kg/E.a	mg/l	kg/ha·a	$kg/ha_r \cdot a$	kg/E·a	mg/l	kg/ha·a	$kg/ha_r \cdot a$	kg/E·a
Atlanta (Ga.,USA)												
-Confed.Ave.									210			
-Boulevard									84			
-McDon Str.									286			
Berkeley(Ca.,USA)	100				200				60			
Bradford (Engl.) -Coopers Lane	237	362	1230	4,0					43	149	505	1,65
Brighouse (Engl.) -Rastrick	647	167	1542	11,4					86	66	611	4,53
Bucyrus (Oh.,USA)												
-Station 8									120			
-Station 17									107			
-Station 23									108			
Cleveland (Oh.,USA)					308				92			
Columbia (DC.,USA)	622				382				71			
Milwaukee (Wisc.,USA)												
- reference 1	321				264				59			
- reference 2	212				161				44			
Des Moines (Iowa,USA)	413								64			
Detroit (Mich.,USA)	274								153			
Hürth (FRG)					222				87			
Lancaster (Engl.) -Stevens Ave.	271				209				56			
Minneapolis-St.Paul (Minn.,USA)	413								141			
New York (N.Y.,USA)												
-Newton Creek	306				481				222			
-Spring Creek	347				358				111			
Northampton (Engl.) - St.Andrews	370	2486	4919	24,2					95	680	1345	6,60
Oslo (Norway)	721	1867	2708	5,5	530	1373	1991	4,0	200+	518+	751+	1,51+
Poissy (France)	751				1005				279			
Racine (Wisc.,USA)												
- reference 1	551								158			
- reference 2	178								90			
Rochester (N.Y.,USA)	273								65			
Sandefjord (Norway)	424	537	4436	21,5	268	340	2809	13,6	103+	131+	1083+	5,3+
San Francisco(Ca.,USA)												
-Baker Str.	91				138				22,9			
-Brotherhood Way	655				100				46			
- Laguna Str.	211	605		3,6	145	538			46,3	152		0,9
-Marinosa Str.	172				188				43			
-Selby Str.	215	708		4,3	148	501			38,1	113		1,9
Seattle (Wa.,USA) -Ctr.Busin.Distr.	162				176				64			
Stuttgart-Büsnau(FRG)	177	1426	3683	11,0	88++	846++	2186	6,6	114	919	2374	7,1
Trondheim (Norway)	510	1757	4731	18,9	352	1209	3256	13,0				
Zürich(Switzerland) -Oerlikon					70							

+ = BOD_7
++ = sedimented sample

Table 2 Mean of concentrations of biological and chemical-physical parameters in combined runoff and dry weather flow of Munich-Harlaching

Parameter	Dimension	Dry weather flow: individual values number*	Dry weather flow: individual values mean	combined runoff: individual values number*	combined runoff: individual values mean	combined runoff: load/volume number*	combined runoff: load/volume mean
TSS	mg/l	4103	177	2729	153	99	163
BOD_5	mg/l	477	199	711	102	29	89
COD	mg/l	4009	443	2595	275	97	274
TOC	mg/l	462	113	635	51	31	48
NKJ	mg/l	3886	45	2537	21	97	22
PHO	mg/l	770	18	712	8,4	29	8,3
temp.	°C	251252	16,3	41687	14,2	-	-
cond.	µS/cm	275988	1002	42275	894	-	-
turb.0°	-	144102	22	28907	22	-	-
turb.25°	-	45061	38	9200	31	-	-
turb.90°	-	60059	28	11056	23	-	-
turb.135°	-	43186	25	7263	24	-	-

*dimensionless

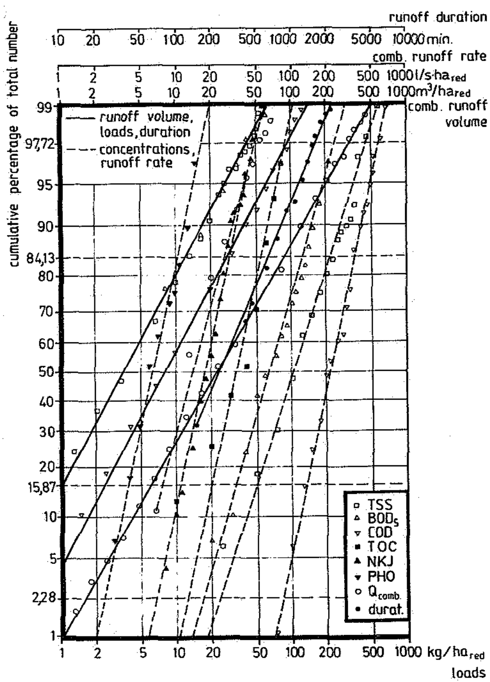

Figure 2

Distribution of flow, concentration and load figures in the combined runoff of Munich-Harlaching (after Geiger, 1984)

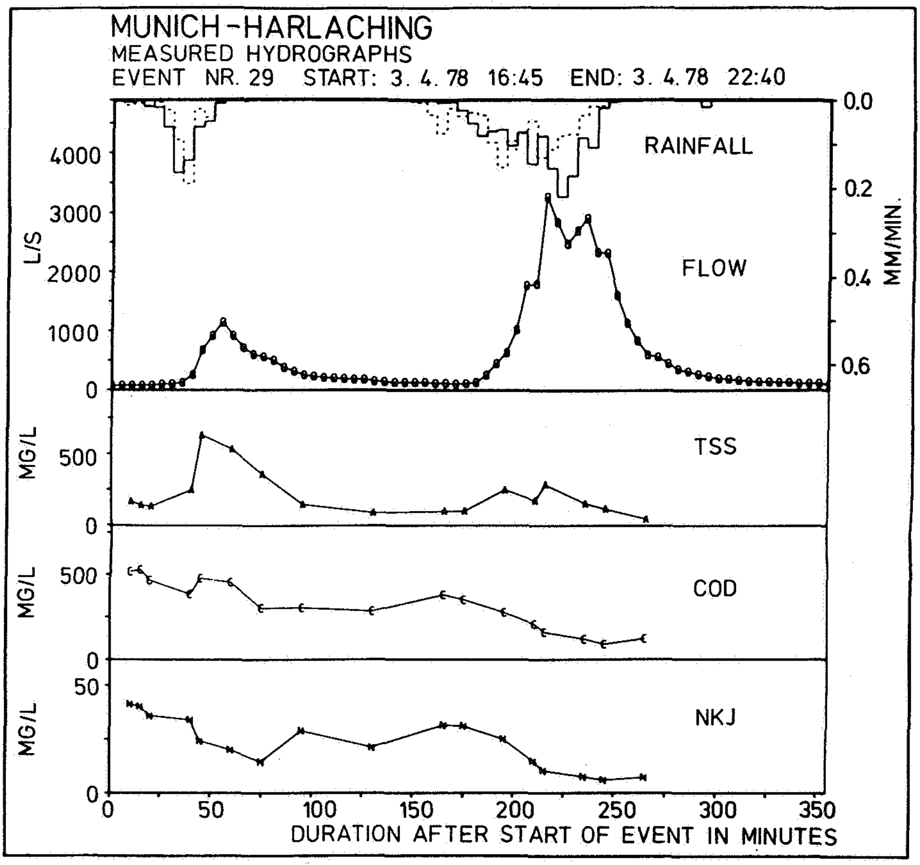

Figure 3 Variation of flow and concentrations of TSS, COD and NKJ of a rainfall-runoff event recorded in Munich-Harlaching (Geiger, 1984)

with the measured dry weather flow concentrations. Both means compare very well, which indicates that the samples are representative. The dry weather flow concentrations for BOD_5, TOC, Kjeldahl-nitrogen (NKJ) and phosphorus (PHO) are double their combined runoff concentrations, while total suspended solids (TSS) show similar magnitudes. This is explained by the different sources of the individual pollutants. Total suspended solids in particular under wet weather conditions derive from the dry weather flow portion and from washoff of oversurfaces and sewer deposits. Same partially applies to COD. The portion of dry weather flow in the combined runoff of Munich-Harlaching was found to be 23 % for runoff volume, 32 % for TSS, 31 % for BOD_5, 44 % for COD, 42 % for TOC, 53 % for NKJ and 43 % for PHO. Figure 2 demonstrates that all combined runoff concentration and load figures were distributed lognormally. In general the variance of runoff volume and loads was observed to be twice the variance of flow rates and concentrations. The large variances indicate different sources and the variability in combined runoff composition.

3 Time dependency of combined runoff pollution

So far only local variation and global differences of combined runoff quality were discussed. For effective combined runoff control, however, knowledge of the time dependency of runoff quality within one rainfall-runoff-event is essential.

3.1 Variation of combined runoff concentrations

Start and end of a rainfall-runoff-event was defined by the exceedance of dry weather flow. Of course a high density of measurements is required to investigate the variation of concentration with time. This for example was the case for all events, for which loads were evaluated in Table 2. Figure 3 illustrates, that shortly after combined runoff starts for some pollutant parameters concentrations increase. This, however, is not necessarely true for all pollution parameters and for all events. Averaging the pollutographs, concentrations were found to ra-

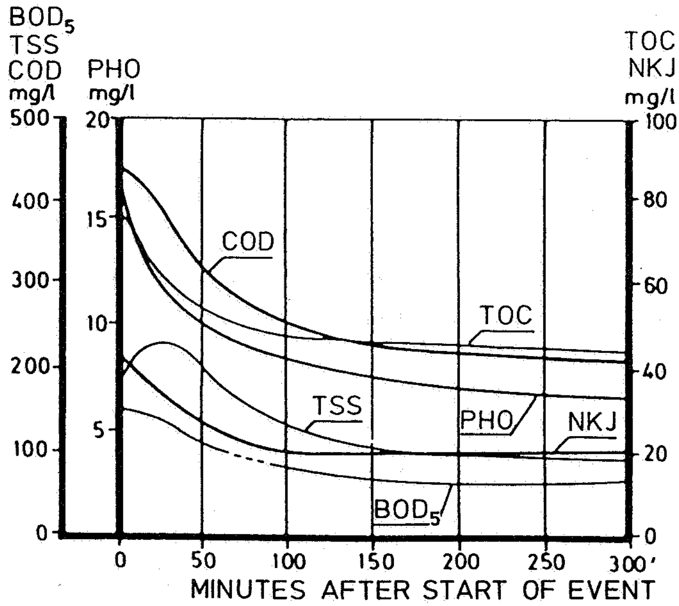

Figure 4 Dependency of concentrations on combined runoff duration (after Geiger, 1984)

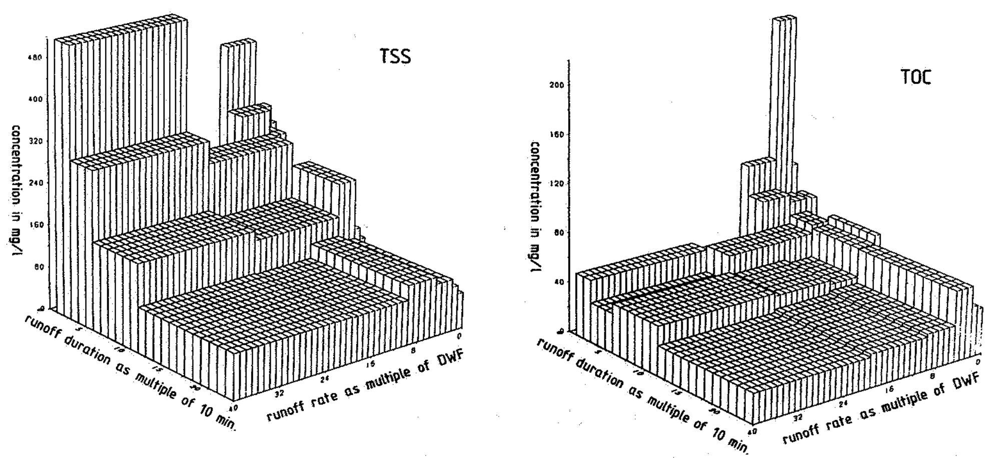

Figure 5 Dependency of concentrations on flow rates and runoff duration (after Geiger, 1984)

pidly decrease within the first hour of runoff levelling thereafter (Figure 4). Total suspended solids concentrations first increased within the initial 15 minutes of runoff indicating first flush effects. Considering the simultaneous dependency of the concentrations from the flow rate Figure 5 was obtained. COD responded similar to the dependency shown for total suspended solids, BOD_5, Kjeldahl nitrogen and phosphorus analogous to total organic carbon.

3.2 Scatter in cumulative loads and first flush behaviour

To differentiate between typical behaviours in combined runoff pollution for the data of Munich-Harlaching the increase in load was plotted against accumulation of runoff. The coordination of all events and pollutants to one of the types in Figure 6 or combinations thereof yielded the shares of Table 3. A deviation of the cumulative load from the diagonal of less than 5 % was considered indifferent, a deflection of 5 % to 20 % moderate and a divergence of more than 20 % strongly positive or negative respectively. Despite of the catchment size flushing effects occured more frequent at the start of runoff than towards its end. First flush was obtained for total suspended solids in 25 % of all events, while it was less than 15 % for all other parameters. However, the cumulative loads were so inhomogeneous, that in total all parameters exhibited an indifferent behaviour on an average with a slight tendency to moderately positive for total suspended solids. Detentions and superpositions of runoff and loads within the rather large catchment explain this overall appearance.The cumulative loads were investigated for their dependency on preceding dry spells, differentiating between periods of up to one hour, 1 to 6, 6 to 24, 24 to 48 and over 48 hours. Also no difference was obtained averaging the cumulative loads separately for events with different runoff peak numbers.

Deriving dry weather flow patterns for total suspended solids it was suspected that during night hours more material is deposited than during the day and that dry weather flow peaks already cause flushing. Therefore the cumulative loads of all storms occuring between 8 a.m. and 8 p.m. and between 10 p.m. and 6 a.m. were averaged seperately. For total suspended solids and COD the average cumulative loads were indifferent for day events and almost strongly positive for night events. For all other parameters the cumulative loads were more or less indifferent for the day and night periods. The potential of deposits of total suspended solids follows to be a function of the time of day.

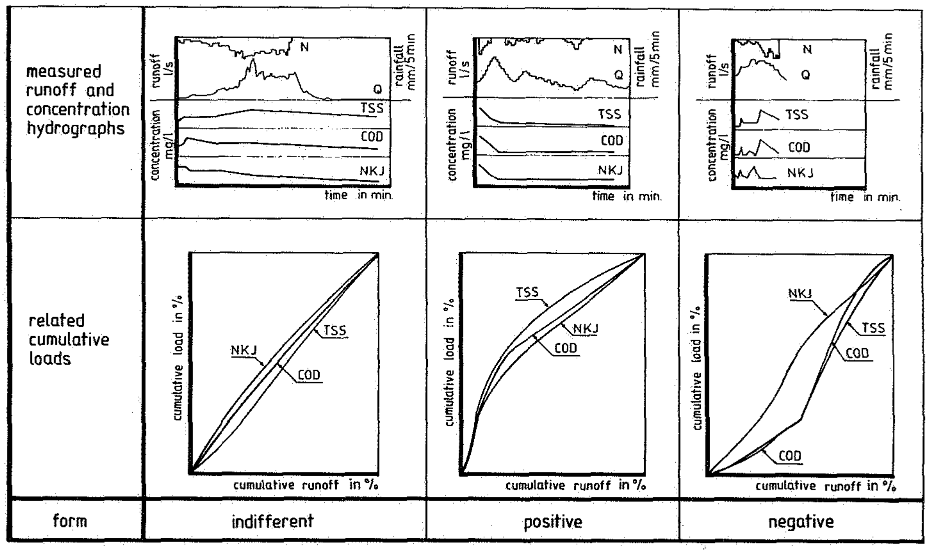

Figure 6 Classification of cumulative loads in combined runoff

Table 3 Characteristics of cumulative loads in percent of all events investigated

Parameter	TSS	BOD_5	COD	TOC	NKJ	PHO
number of events	125	32	123	34	122	33
indifferent	7,2	0,0	7,3	2,9	4,1	6,1
positive-moderate	31,2	21,9	40,7	8,8	28,7	24,2
-strongly	25,6	12,5	13,8	11,8	6,5	3,0
negative-moderate	12,8	12,5	15,4	14,7	27,1	24,2
-strongly	7,2	15,6	9,8	20,6	11,5	9,2
positive-negative	12,0	31,3	13,0	41,2	22,1	33,3
negative-positive	4,0	6,2	0,0	0,0	0,0	0,0

3.3 Comparison of frequencies of different runoff quality parameters

Conventional design procedures assume same frequencies for rainfall and runoff. Using extreme value analysis of partial series assuming exponential distribution showed that the recurrence frequencies of rainfall and runoff properties of individual events differ randomly. Same was found comparing the frequencies of runoff volume and loads (Figure 7). This is a result of the variances of concentrations and load with time. For the parameters investigatedthe number of values taken did not influence the computed recurrence. In addition the assumption of the exponential distribution was replaced by an extreme value type 3 distribution which gave nearly identical results. This indicates that the standard procedures used for sewer design ought to be reconsidered.

4 Pollutant retention strategies

As urban areas in the Federal Republic of Germany as elsewhere were growing around old existing settlements today's pollutant retention strategies are more concerned with rehabilitation measures of existing sewer systems and with expansions of such systems, rather than completely new designs. Most of the designs are done on the basis of rather simple regulations or calculation procedures derived thereof. For larger and more complicated sewer networks sometimes rainfall-runoff models including quality calculations are employed for detention basin placement and sizing.

4.1 Effectiveness of conventional combined sewer overflow control

In the Federal Republic of Germany the regulations widly used in designing combined sewer overflow controls distinguish mainly three cases (ATV, 1977):

- overflow structures not providing additional storage volume except for the storage capacity utilized in the sewers due to the height of the weir crest;

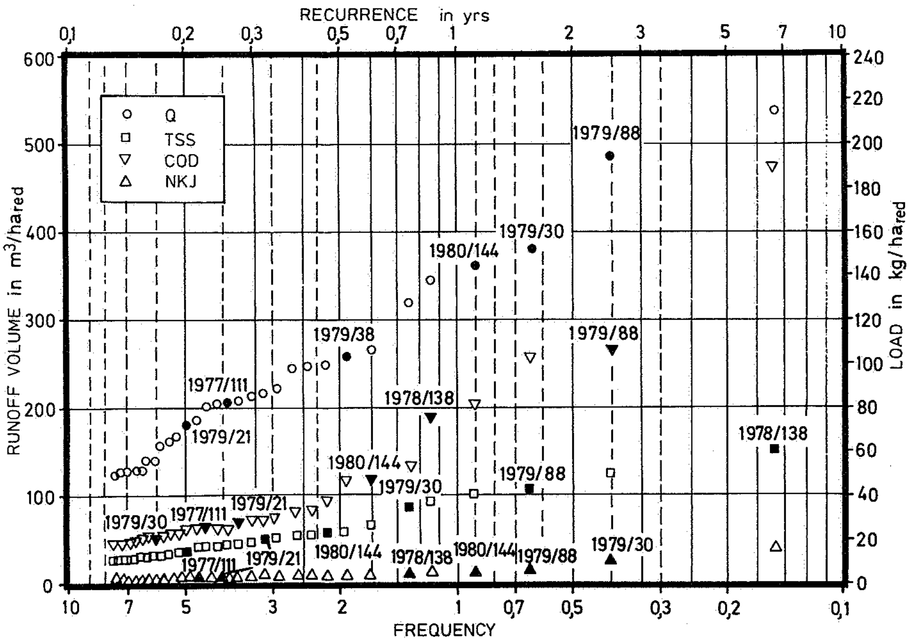

Figure 7 Identification of individual events within the partial series of runoff volume (Q) and different pollutant loads

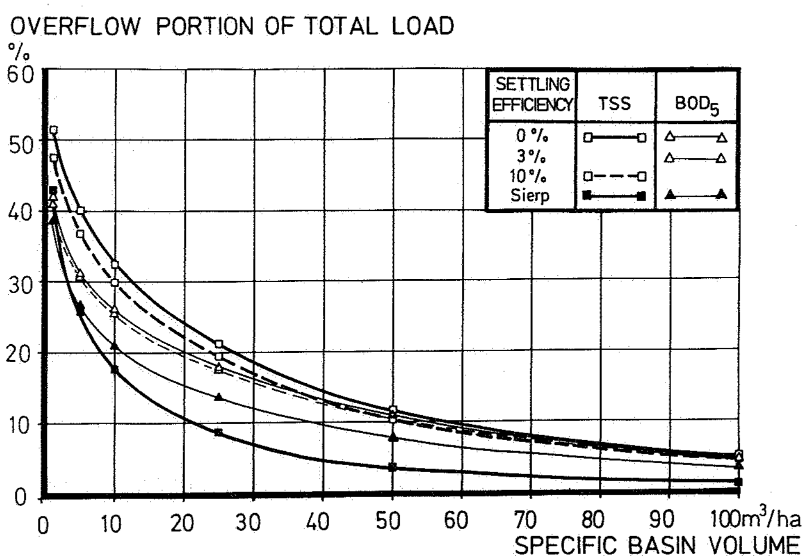

Figure 8 Annual percentage of overflowing TSS- and BOD_5-loads for different settling assumptions for a fictive basin in Munich-Harlaching (after Geiger, 1984)

- overflow structures providing enough volume that for areas with flow times of less than 15 minutes first flush may be retained in the sewer system;
- overflow structures providing enough volume that for areas with flow times of more than 15 minutes matters can settle in the basin and be retained.

The design procedures for all three cases mainly involve the determination of critical flows where overflow starts and for the basins in addition of the maximum flow which can be diverted to the treatment plant. It is assumed, that if the procedures are followed 90 % of the biological degradable matters and of settleable solids are retained in the sewer system.

To check the validity of this assumption the continuous time series of combined runoff measurements of Munich-Harlaching from 1977 to 1981 - missing quality data were added according to the relationships of Figure 5 - was imposed on a spectrum of basin volumes, outflow capacities and settling efficiencies. For the case of a basin structure functioning as a settling tank - the flow time in Munich-Harlaching is 60 minutes - Figure 8 shows the percentages of annual TSS and BOD_5 loads discharged to receiving waters under different settling assumptions. The basin outflow was limited to twice the dry weather flow, the overflow was dimensioned for a rainfall intensity of 10 l/s·ha. For Munich-Harlaching according to the ATV-regulation (1977) a basin volume of 15 m^3/ha_{red} -ha_{red} reflects the impervious areas-would have to be chosen.Such a basin still would discharge between 14 % and 28 % of the annual TSS-load and between 17 % and 23 % of the annual BOD_5-load to receiving waters dependent on its settling efficiency. The ideal settling rate according to Sierp , however, never could be achieved. Figure 8 also shows that exceeding basin volumes of 25 m^3/ha_{red} only a few but intensive events lead to overflows that a further enlargement of the basin volume would increase costs tremendeously without reducing overflowing loads significantly. Although anoverflow basin design according to ATV (1977) does not meet the goal of 90 % BOD_5-load to be retained, the design volume at least is realistic in view of economic acceptability.

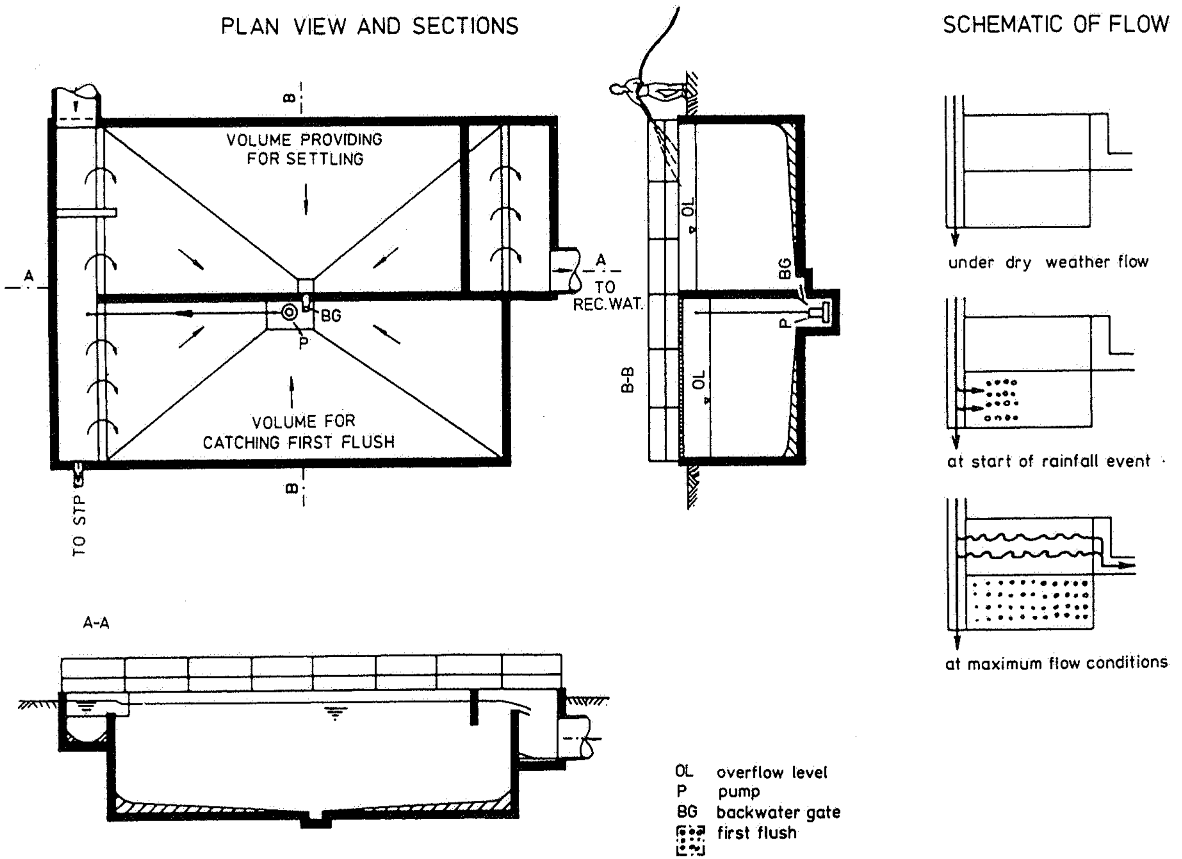

Figure 9 Combination of overflow basin volumes providing for catching first flush and for settling (after Koral, 1984)

4.2 Consideration of first flush behaviour for overflow basin design

The discussion on first flush behaviour indicated that overflow basin designs which only follow one of the principles, namely catching first flush or settling are bound to fail. Koral (1984) promoted a combined design, which mainly is followed in Switzerland (Figure 9). One part of the basin volume is filled at the beginning of an event catching a first flush if existant. Another part only is loaded when the first portion is full and functiones like a settling tank. Aside from its effectiveness the subdivision into two parts eases maintenance as for smaller events only the first portion is filled.

Hailer (1984) suggested to preferably place overflow basins after flat portions of a combined sewer system rather than making the choice of catching or settling basins dependent on catchment size. Low flow velocities will cause deposits, which in case of storm events are flushed and may be caught if a retention basin is placed properly. On the basis of a survey of 285 basins Hailer (1984) also demonstrates the importance of design details for effective operation.

4.3 Operational control and other strategies

The effect of basins for pollutant retention strongly depends on their location in the sewer network. For the combined system of Munich-Harlaching it was found that provision of individual basins only reduced overflow by a small amount. Due to the system configuration utilization of existing sewer volume proved to be more effective. Operational controls were investigated within the research projekt "Entwicklung niederschlags- und ortsspezifischer Kriterien zum wirtschaftlichen Einsatz der Kanalnetzsteuerung " (DFG-Ge 459/1), which again was financed by the German Research Society. For this operational control fictive gates at seven points in the network were introduced. For operation two monitoring points were selected, where closing and opening of the gates are triggered. Simulating runoff under these conditions with a hydrodynamic model SESIM (Broeker and Schulze, 1980) the effectiveness of the opera-

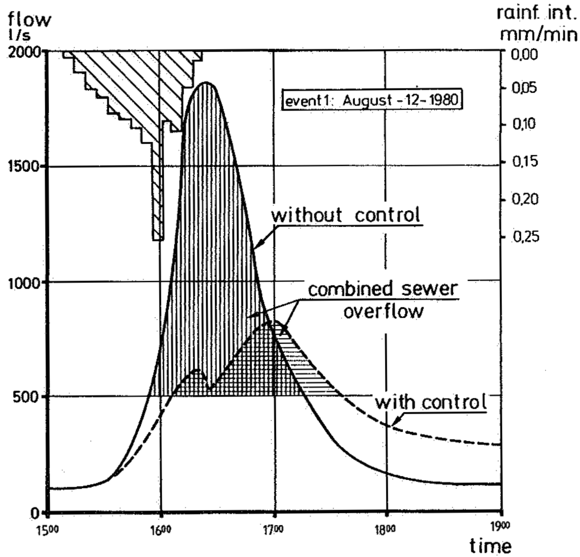

Figure 10 Retardation of flow by controlled utilization of sewer volume for event of August-12-1980 (Geiger and Becker, 1985)

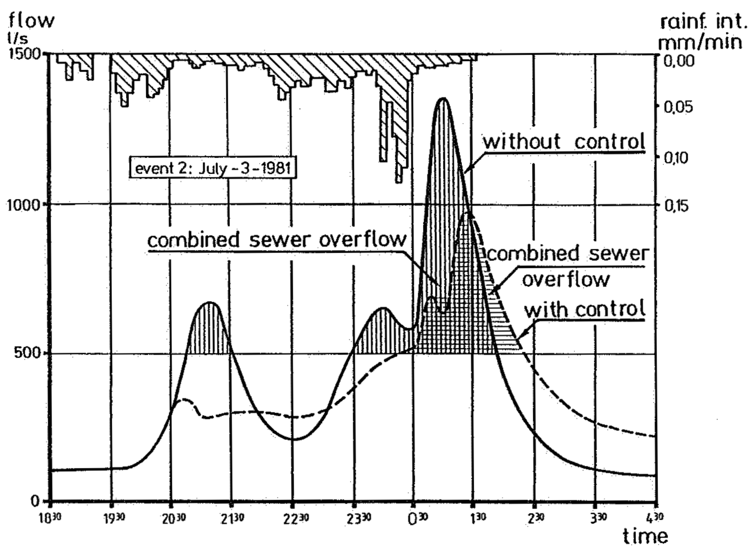

Figure 11 Retardation of flow by controlled utilization of sewer volume for event of July-3-1981 (Geiger and Becker, 1985)

tional controls can be demonstrated. Figures 10 and 11 compare the runoff and overflow hydrographs for conditions without and with operational controls. In both cases the rising limb of the runoff hydrograph is diminished which results in less overflow at the beginning of the storm. This is very important when first flush occurs, which then can be brought to the treatment plant. In both cases operational control reduces runoff volume significantly whereby for event 2 (Figure 11) the first overflow is eliminated. On an average of 13 events investigated for controlled conditions the overflow volume was reduced by 25 % as compared to uncontrolled conditions. In consequence flow above dry weather flow level to the treatment plant is prolonged. This again influences treatment plant efficiency.

An alternative to detention of flow and pollutant retention is the mechanical, chemical and biological treatment of combined sewer overflows. Screens and microstrainers have been applied successfully for mechanical treatment, precipitation and coagulation was used mainly to reduce the PHO-load. As yet these possibilities, however, are seldom used. Chlorination has been tried, whereby the organic matters in combined runoff may lead to chlorinated hydrocarbons.

Stormwater and combined runoff detention, pollutant retention and treatment are "end of the pipe"-measures. An alternative is, to reduce runoff quantity and quality at their sources. Runoff from roofs, backyards and parking lots may be infiltrated into the ground, which significantly reduces runoff peaks and consequently overflows. Inasmuch stormwater infiltration is acceptable in urban areas depends on the quality requirements of the grondwater and on the possibilities to quarantee continuous operation of infiltration facilities.

5 Conclusions

Storm and combined runoff pollution due to the intermittend occurrence of storms and due to the arbitrary accumulation of dust, dirt and other wastes on surfaces and in channels is highly variable and strongly depends on local conditions. The variation of flow and of pollutant concentrations with time is significant. This leads to accumulation of loads in combined runoff, which differ from pollutant to pollutant and from storm event to storm event. Consequently pollutant retention strategies should provide for catching first flush and settling. The differences obtained in recurrence intervals for different properties of the same event suggest reconsidering the standard procedures for sewer design, which try to conclude from rainfall frequencies on all runoff properties. One possibility is continuous simulation with statistical analysis of the computed runoff figures. Any strategy, however, must consider its overall effect on receiving waters including sewage treatment plant efficients, as the treatment plant efficiency may well be influenced by the pollutant retention strategy chosen.

Acknowledgement

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AN INTRODUCTORY ANALYSIS OF FACTORS AFFECTING THE CONCENTRATION OF POLLUTANTS IN THE FIRST FOUL FLUSH OF A COMBINED STORM SEWER SYSTEM

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Abstract

This paper describes the results of a fieldwork program and subsequent statistical analysis in which the quality of storm sewage flows have been monitored, by sample and analysis, at the downstream end of the combined sewer system at Great Harwood, Lancashire. During a two-year period, 113 separate storm events were monitored and the samples extracted were analysed for suspended solids, chemical oxygen demand, ammonia and conductivity. The results of this analysis were then subjected to regression analysis in order to identify the factors controlling the pollutant concentrations in the first flush.

For approximately 90% of the storm events sampled, a distinctive first flush of suspended solids and chemical oxygen demand was observed. Ammonia and conductivity almost always follow a dilution pattern inversely related to sewer flow.

Two types of first flush have been identified and the factors which influence the magnitude and temporal variation of SS and COD concentration are the length of antecedent dry weather period, the pollutant concentration in the dry weather flow and the maximum rainfall intensity.

1 Introduction

The WRC/WAA sewerage rehabilitation manual points to the excessive or too frequent operation of combined sewer overflows and the subsequent degradation of receiving watercourses as being the major failure of sewer systems. However, at this time little consideration is given to the quality aspects of combined sewer design and what is now required is a quality model for use in the design and rehabilitation of sewer systems. In order to produce such a model, it is necessary to develop an understanding of the sources of pollutants and the relationship of their loads to climatic and catchment characteristics.

One major characteristic of the magnitude and temporal variation of pollutants in storm sewage flows is the "first foul flush", which has been identified in a number of previous studies (Ellis 1977; Hedley and King 1971; Thornton and Saul 1986), though this phenomenon has been observed to vary widely in magnitude and duration. Also, large catchments may not experience a distinctive flush (Geiger 1984). For this reason more information is required on the quality of combined sewer flows in relation to the factors that control the concentrations of pollutants as this information is vital to the design of overflows and their impact upon receiving watercourses.

The aim of this paper is to analyse the relationship between the concentration of pollutants in the first foul flush and the factors that are likely to play a role in determining their levels. These include the length of the antecedent dry weather period (ADWP), the intensity of the rainfall, the size of the preceeding storm event and its ADWP and the quality of the dry weather flow.

2 The Field Measurement System

The combined sewer catchment analysed in this study is that which drains the town of Great Harwood, Lancashire, which has an area of 1160 ha, 67ha of which is impermeable, and a population of 12,500. A schematic diagram of the 160 pipe system is shown in Fig.1a. The sampling and monitoring system was sited within the high side-weir storage overflow chamber at the downstream end of the combined sewer system, for which a peak flow rate, corresponding to a design storm of return period one in two years, of 4.5 m^3/s, was computed using the simulation program WASSP-SIM of the Wallingford procedure.

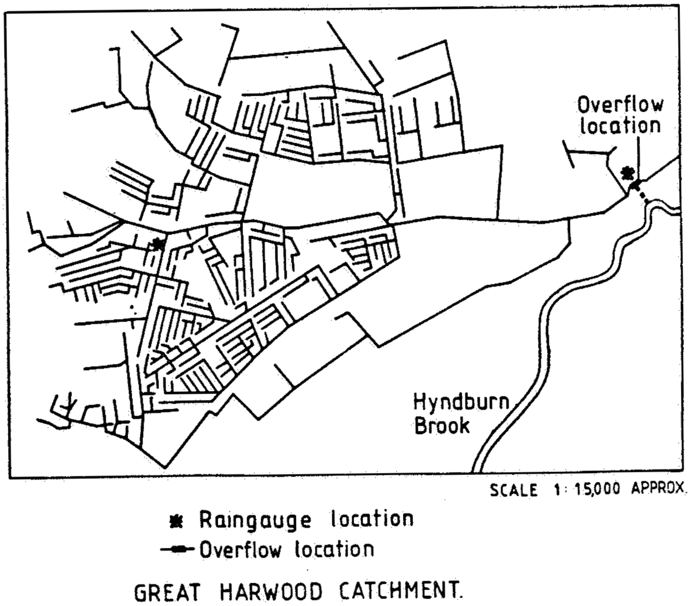

Fig.1a A schematic diagram of the Great Harwood combined sewer catchment

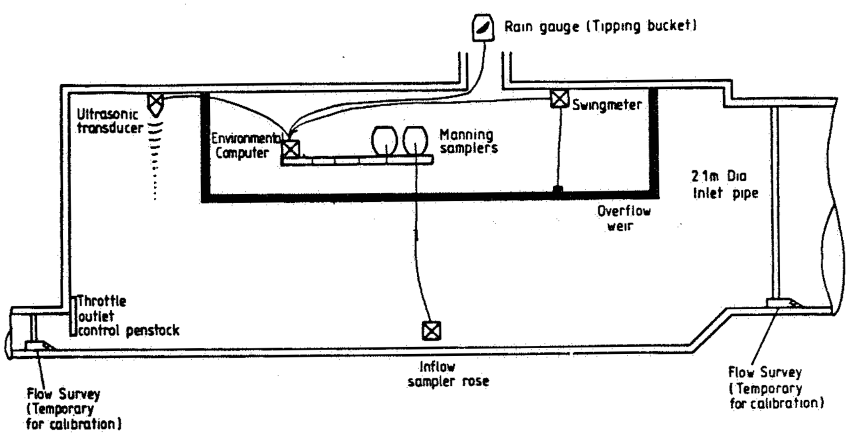

Fig.1b The sampling and monitoring system

The sewer flow was sampled during storm events using a portable discrete sampler externally triggered by a float switch and programmed to extract samples at pre-set time intervals. The samples were analysed for Suspended Solids (SS), Chemical Oxygen Demand (COD), conductivity and ammonia (NH_3) by North West Water. The level of flow in the chamber was measured using an ultrasonic transducer and a swingmeter designed by the Water Research Centre (based on a swinging metal rod with a wire wound potentiometer).

Finally, tipping bucket raingauges were installed in the centre of the catchment and at the sampling station in order that rainfall intensities and totals could be measured. All the data from the level monitoring equipment and the raingauges was recorded either continuously or on a storm event basis using commercially available data loggers.

3 Results

3.1 First Foul Flush

The first foul flush may be defined as the initial period of storm flow during which the concentration of pollutants is significantly higher than those observed during the latter stages of the storm event. However, it should be stressed that the above definition is very subjective and may differ from that of other researchers. It does, however, provide a useful indicator of system performance and a useful basis for comparison.

Pollutant concentration data have been recorded and analysed for 113 separate storms at which 102 showed a recognisable first flush of SS and COD. Conductivity and NH_3 concentrations almost always exhibit a temporal pattern of behaviour inversely related to the combined sewer flow which suggests that the levels of these dissolved pollutants is dependent upon the degree of dry weather flow dilution.

The 102 storms which exhibited a first flush can be divided into two groups, termed 'Type A' and 'Type B', dependent upon the characteristics of the first flush of SS and COD. Type A storms are characterised by SS and COD concentrations that are less than, or equal to the concentrations in the prevailing dry weather flow and Type B exhibits concentrations exceeding those in the dry weather flow. Examples of both types are given in Fig.2.

In the case of the Type A flush the first flush of COD and SS lasts less than 30 minutes and there is a sharp drop of SS and COD concentrations following the initial inflow of storm water to the chamber. The Type B flush is characterised by a longer first flush lasting between 35 and 45 minutes with an initial increase in SS and COD concentrations to a peak which almost coincides with the peak of storm flow.

Observation of the variations between the types of first flush indicates that they are controlled by different factors and it is possible to put forward two hypotheses relating to the influences controlling each type of flush. It may be hypothesised that a Type A flush results from the mixing of dry weather sewage and storm water at the front of the advancing flood wave and that SS and COD within this flush derived almost entirely from the dry weather flow.

In pipe deposition during dry periods has often been put forward as an explanation of the first foul flush (Fletcher et al 1978; Lindholm 1984). Thornton and Saul (1986) noted that Type B flushes tend to occur after an ADWP of at least three days and in storm events with a maximum rainfall intensity of 6-7mm/hr or more. From this evidence it is possible to hypothesis that Type B flushes are controlled by the length of ADWP and rainfall intensity.

These two types of first flush and the two associated hypothesis indicate a need for a more detailed analysis of the data in order to produce a more accurate explanation of the variation observed. For this reason the data were put through a step-wise multiple regression analysis using a Statistical package developed for social scientists. This program introduces each independent variable into the step-wise multiple regression according to pre-calculated partial correlations.

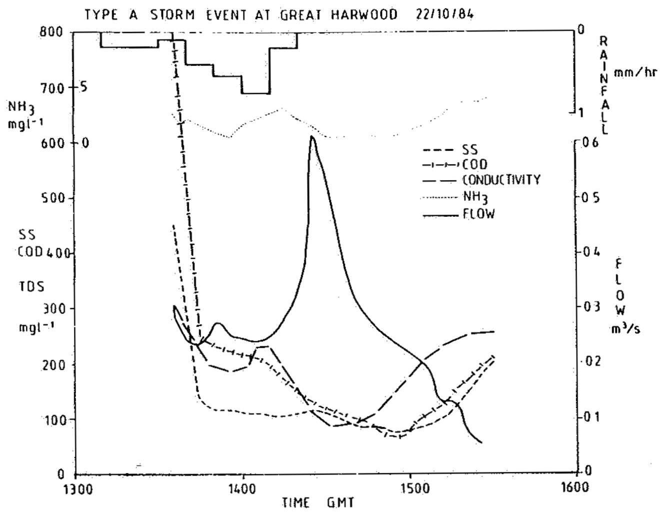

Fig.2a A Type 'A' first foul flush

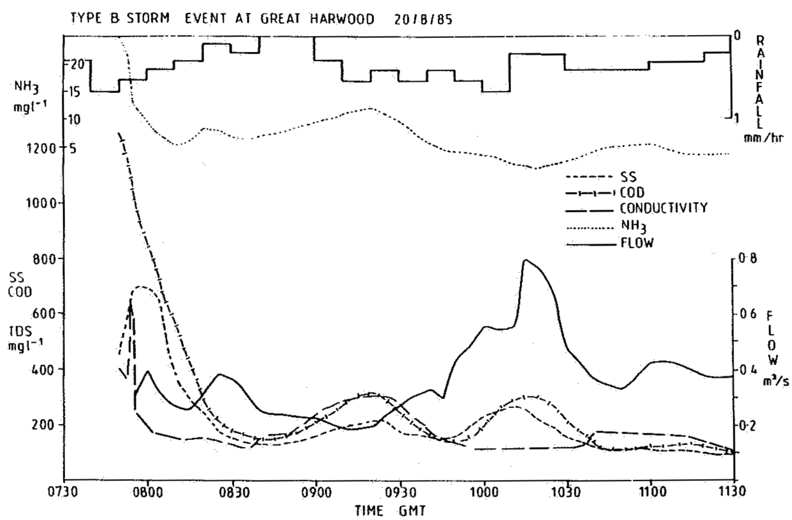

Fig.2b A Type 'B' first foul flush

3.2 Multiple Regression

The data that is used in this study was obtained from 102 storms sampled between February 1984 and February 1986. It is composed of the peak concentrations found within the first foul flush for SS, COD, NH_3 and conductivity which make up the dependent variables. The independent variables are the length of ADWP, maximum rainfall intensity for each storm (MINT), the size of the preceding storm (LSTM) and its ADWP (ADWPB) and, finally, the concentrations of SS, COD, NH_3 and conductivity found in the dry weather flow, (SSB, CODB, NH_3, CONDB). These latter concentrations are mean hourly values derived from a series of six 24-hour periods of dry weather flow sampling. It may be hypothesised that sewer flow would play a role in determining the magnitude and duration of the first flush but it is thought that flow is not a truly independent variable, being a function of rainfall intensity, volume and duration. Furthermore, as flow data was not available for all storms at the time of writing this paper, it was therefore not included in the multiple regression.

3.2.1 All storms

Firstly, the step-wise regression program was applied to the complete data set of 102 storms in order to determine the major overall influences upon the concentrations of SS, COD, NH_3 and conductivity within the first flush. The accumulated correlation coefficients for each step of the multiple regression are given in Table 1.

Table 1

Accumulated Correlation Coefficients For All Storms

	SS		COD		NH_3		COND	
STEP 1	ADWP	0.58	ADWP	0.65	ADWP	0.40	ADWP	0.28
STEP 2	MINT	0.67	MINT	0.68	NH_3B	0.51	ADWPB	0.34
STEP 3	SSB	0.67	CODB	0.70	ADWPB	0.53	MINT	0.38
STEP 4	ADWPB	0.68	ADWPB	0.70	LSTM	0.53	CONDB	0.39
STEP 5	LSTM	0.68	LSTM	0.71	MINT	0.53	LSTM	0.39

The results show that in the case of SS and COD there is a definite relationship between the concentration of the determinands in the first flush and the independent variables of the the multiple regression, which are in order of importance antecedent dry weather period, maximum rainfall intensity, the concentration of SS and COD in the dry weather flow, the ADWP prior to the last storm and the size of the last storm. However, it must also be said that although the length of the ADWP is of most importance, only 33% and 44% respectively of the variation of SS and COD concentrations can be explained by the variation in length of the ADWP. Thus, much of the variation is still left to be explained as the following independent variables only increase the explanation to 46% and 50%.

As would be expected the concentrations of the dissolved determinands NH_3 and conductivity show little, if any, relationship with the independent variables, and although ADWP is of greatest importance, it only provides 16% and 7% of the explanation. Therefore, the hypothesis that the concentration of NH_3 and conductivity in the first flush is largely controlled by the degree of dry weather flow dilution is upheld.

3.2.2 Type A and Type B flushes

Following on from the hypotheses previously discussed relating to the two types of flush, the data were divided into the two groups which were composed of 57 Type A storms and 45 Type B storms respectively. These were then introduced to the regression program in order to improve the level of explanation and to show how Types A and B flushes may be controlled by different factors. The accumulated correlation coefficients for the two sets of data are shown in Tables 2 and 3, which indicate no improvement in the level of explanation.

In the case of SS, COD and NH_3 for the Type A first flush storms, the most important factor in determining their concentration in the first flush is the dry weather flow concentration which tends to uphold the hypothesis that the major controlling factor is the degree of dry weather flow dilution, although the degree of explanation is less than 31%. In the case of conductivity there is no obvious relationship with any of the independent variables as the accumulated correlation coefficient is only 0.38.

Table 2

Accumulated Correlation Coefficients for the Type A Flushes

	SS		COD		NH_3		COND	
STEP 1	SSB	0.56	CODB	0.55	NH_3B	0.47	ADWPB	0.23
STEP 2	MINT	0.60	MINT	0.59	MINT	0.48	ADWP	0.33
STEP 3	ADWP	0.64	ADWP	0.60	ADWPB	0.49	CONDB	0.35
STEP 4	ADWPB	0.68	LSTM	0.62	ADWP	0.49	MINT	0.37
STEP 5	LSTM	0.69	ADWPB	0.63			LSTM	0.38

Table 3

Accumulated Correlation Coefficients for the Type B Flushes

	SS		COD		NH_3		COND	
STEP 1	ADWP	0.42	ADWP	0.54	NH_3B	0.44	ADWP	0.28
STEP 2	MINT	0.45	CODB	0.61	ADWP	0.49	ADWPB	0.34
STEP 3	SSB	0.48	LSTM	0.62	MINT	0.50	MINT	0.38
STEP 4	LSTM	0.49	ADWPB	0.63	LSTM	0.51	LSTM	0.40
STEP 5	ADWPB	0.50	MINT	0.63	ADWPB	0.52	CONDB	0.42

In the case of the Type B first flushes of SS and COD the hypothesis that they are largely controlled by the length of the ADWP is still upheld, but the relationship and the degree of explanation (max 31%) are not as strong as in the case of all storms (Table 1). This is probably brought about by the reduction in the data set (from 102 to 57 and 45) producing a cluster of results rather than a representative spread. In addition, the concentration of NH_3 appears again to be controlled by the degree of dry weather flow dilution and conductivity shows no relationship.

4 Conclusions

The data from the Great Harwood catchment has provided some useful information concerning the quality of combined sewer flows and particularly of the pollutant concentrations within the first foul flush.

A clearly defined first flush of SS and COD has been identified in 90% of the monitored storm events and this first flush has been shown to take two forms. NH_3 and conductivity, however, almost always exhibited a dilution pattern inversely related to sewer flow.

The maximum recorded SS and COD concentrations in the first foul flush are related to the length of the antecedent dry weather period, the quality of dry weather flow and the maximum rainfall intensity.

This study has gone some way to show the effect upon the first flush of these factors, but the levels of explanation within the relationships (max 44%) show that more work is required to produce a fuller understanding.

Acknowledgement

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THE HYDRAULIC PERFORMANCE OF TWO ON-LINE STORAGE CHAMBERS IN COMBINED SEWER SYSTEMS

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Abstract

This paper describes the results of a fieldwork program of research, financed by the Water Research Centre and North West Water (U.K), to assess the hydraulic performance of two on-line high side weir storage storm overflow chambers. Rainfall and sewer flow data have been recorded over a two year study period and analysed to examine the three main aspects of hydraulic performance. Firstly, the paper shows the extent to which the frequency of discharge to receiving streams is reduced throughout a given study period. In this respect the paper goes on to show that storage chamber operation may be predicted with a high degree of certainty from rainfall data. Secondly, there is an examination of the flow retention performance by each storage chamber for a selected number of storm events. This performance is then related to a number of prevailing rainfall, runoff and sewer flow characteristics. Thirdly, this paper examines the extent to which the time of first spill is delayed by the on-line storage facilities and discusses its relevance to the behaviour of the receiving stream. In conclusion, this paper considers the present value and future repercussions of the hydraulic behaviour and performance of on-line storage chambers.

1 Introduction

In the United Kingdom there is a national need for a much improved hydraulic control at stormwater overflows on combined sewer systems. In some cases of sewer rehabilitation the main method for achieving better hydraulic control has been the provision of on-line storage chambers at the point of overflow. Other than maintaining a continuous volume of flow to treatment on-line storage chambers have three objectives ; to reduce the frequency of overflow, to reduce the volume of storm sewage spilled and to delay the time of first spill. In effect, each objective leads to reduced impact upon the receiving watercourse. In the absence of a co-ordinated national policy for the construction of storage chambers there are neither universally accepted design criteria nor many attempts to assess the hydraulic performance of these storage chambers built to date. For example, the storage capacity of an overflow chamber may be determined according to one of many suggested methods; a multiple of dry weather flow, a volume per unit area of catchment, a volume per head of catchment population, a fraction of a one or two year design storm and a volume according to the volume of first foul flush in the sewer system.

In the United Kingdom there have been only a few studies of the performance of on-line or off-line storage chambers. The Ministry Of Housing And Local Government (1970) concluded from a study of four storage chambers that, with a capacity of six to nine hours of dry weather flow, the provision of storage could reduce the frequency of overflow by 22% to 45% and reduce the volume of spill by 30% to 50%. The Scottish Development Department (1977) concluded from a study of two storage chambers that, with a capacity of 68 litres per head of the catchment population, storage could reduce the frequency of overflow by 58% and 77% and reduce the volume spilled by 71% and 89%. Despite these results storage chambers have not been taken up as a nationally accepted method of achieving hydraulic control at redesigned stormwater overflows.

2 The Overflow Storage Chambers

This paper describes results from the study of two on-line storage chambers each at the downstream end of a combined sewer system in the Borough of Hyndburn, Lancashire. The characteristics of each chamber and its catchment are provided in Table 1. Both chambers are designed in accordance with the recommendations outlined by Ackers et al (1968), so that the storage capacity is equal to the volume of dry weather flow within the sewer system overtaken by the toe of the advancing storm wave. In effect the chamber volume is a function of the volume of foul flow, its approach time, the time of rise, the initial foul flow and the flow allowed to treatment. The chamber penstock is set according to Formula A is described in the Technical Committee Report 1970.

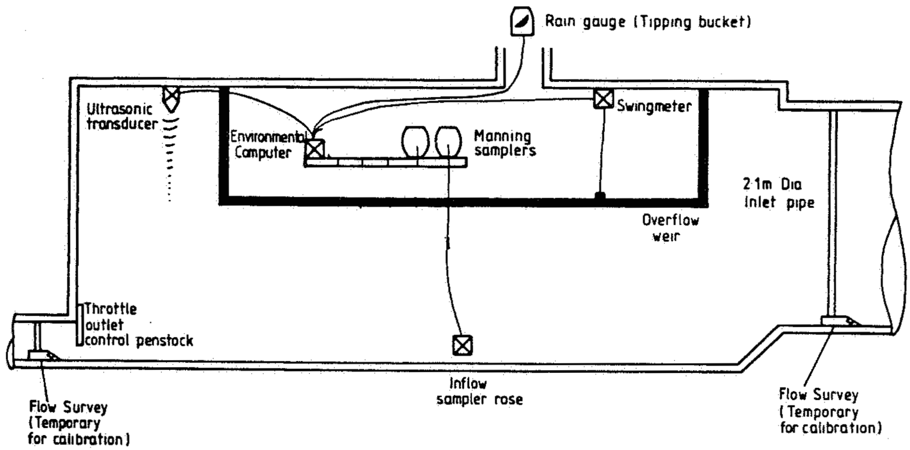
Table 1 The characteristics of the storage chamber and their catchments

	Population	Impervious Area (ha)	Total Area (ha)	Mean Dry Weather Flow (m^3/s)
Great Harwood	12,500	56	121	0.30
Clayton le Moors	6,500	29	41	0.20

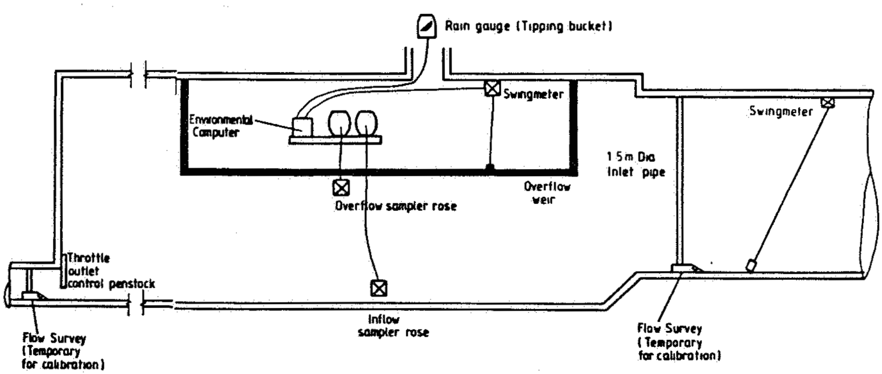
	Maximum Flow to Treatment (m^3/s)	Two Year Design Flow (m^3/s)	Storage Volume (m^3)
Great Harwood	0.27	7.80	138**
Clayton-le-Moors	0.13*	3.78	126**

* A flow survey carried out during the study period has shown that the actual maximum flow discharged to treatment at Clayton-le-Moors is 0.33 m^3/s.

** The storage capacities of the two chambers are very similar despite quite different characteristics due to the Ackers method of calculation. The storage capacities of the Great Harwood and Clayton-le-Moors chambers are small in comparison to those suggested by the earlier studies of the Ministry of Housing and Local Government (1970) and the Scottish Development Department (1977). As a means of comparison the Great Harwood chamber has a capacity of 11 litres per head or 77 minutes of dry weather flow, and the Clayton-le-Moors chamber has a capacity of 19 litres per head or 104 minutes of dry weather flow. The Scottish Development Department predicted that the chambers would reduce overflow frequency by 30% and 45% and reduce the volume of spill by 35% and 50% respectively.

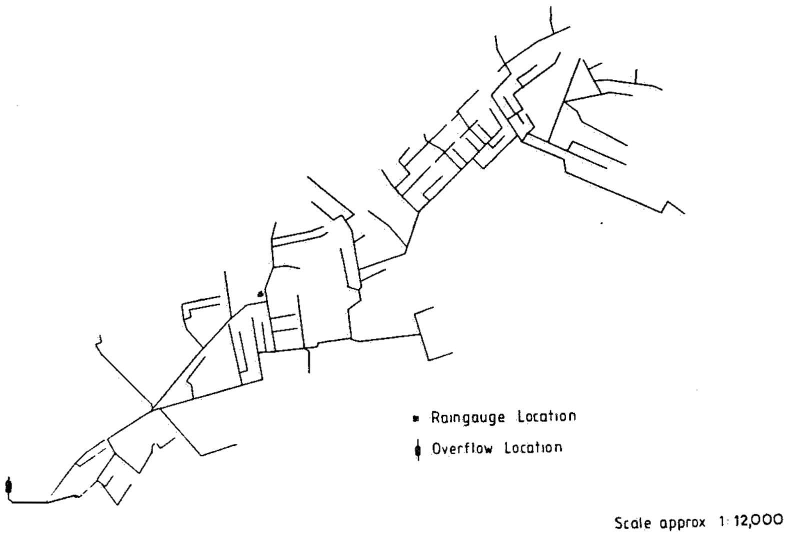


GREAT HARWOOD SAMPLING SYSTEM.

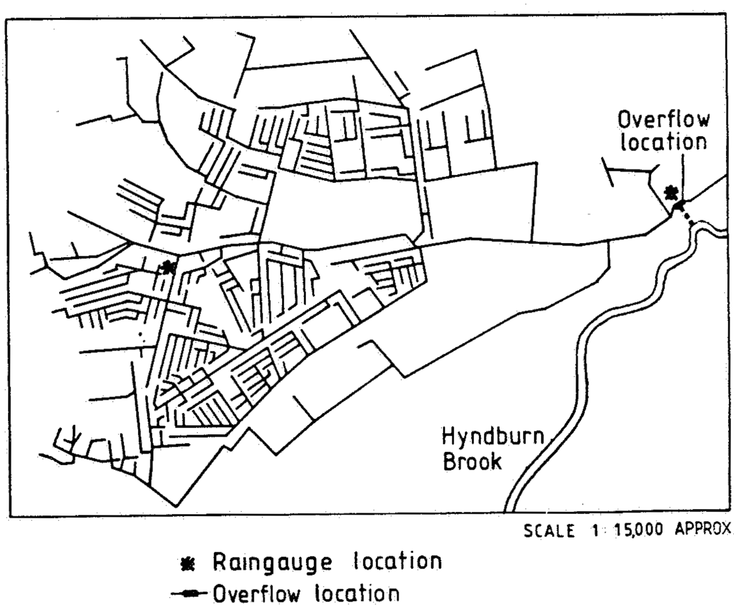


CLAYTON-LE-MOORS SAMPLING SYSTEM.

Figure 1 The Field Measurement Systems at the Great Harwood and Clayton Le-Moors Storage Chambers



CLAYTON-LE-MOORS CATCHMENT



GREAT HARWOOD CATCHMENT.

Figure 2 The Great Harwood and Clayton Le-Moors Combined Sewer Catchments

3 The Field Measurement System

The field measurement systems were established at Great Harwood in February 1984 and at Clayton-le-Moors in April 1985. The details are shown in Figure 1. Inflow, throttle flow and overflow measurements at Great Harwood are derived from a computer program that relates flows to the increase/decrease of stormwater level as monitored by an ultrasonic level transducer in the chamber and a Water Research Centre designed level sensing "swingmeter" (Harman and Forbes, 1983) located over the overflow weir. Flows derived by this method have been checked by a flow survey. Inflow and overflow data at Clayton-le-Moors are derived from the direct calibration of level in the inflow pipe and over the overflow weir as measured by "swingmeter". The calibration is provided by a flow survey. All level data are recorded by Golden River data loggers or environmental computers. Rainfall is continually monitored by a tipping bucket raingauge located in the centre of each catchment, Figure 2.

4 Results

The rainfall and sewer flow data have been analysed to test the hydraulic performance of each storage chamber in terms of their operational frequency, their retention of flow and the delay of first spill.

4.1 The Frequency Of Storage Chamber Operation

Table 2 presents a synopsis of the rainfall and overflow frequency data derived from a 36 week study period at Great Harwood and a 37 week study period at Clayton-le-Moors between June, 1985 and March, 1986. Table 3 presents a synopsis of the data derived from the whole 90 week study period at Great Harwood; April, 1984 to March, 1986.

Table 2 A synopsis of the rainfall and overflow frequency data for Great Harwood and Clayton-le-Moors

	Total Number of Storms *	Number of Storms per week	Total Number of overflows	Number of overflows per week	Percentage Number of storms that caused overflow
Great Harwood	158	4.4	49	1.4	31.0%
Clayton le Moors	161	4.4	32	0.9	19.9%

Table 3 A synopsis of the rainfall and overflow frequency data for the whole study period at Great Harwood

	Total Number Storms*	Number of storms per week	Total Number of overflows	Number of over-flows per week	Percentage Number of storms that caused over-flows
Whole Study Period	340	3.8	113	1.3	33.2%
Spring/Summer 1984	43	2.4	16	0.9	37.2%
Autumn/Winter 1984/85	111	4.3	37	1.4	33.3%
Spring/Summer 1985	85	4.1	29	1.4	34.1%
Autumn/Winter 1985/86	101	4.0	31	1.2	30.7%

* A storm is defined as an event in which flow increased above Formula A i.e. the theoretical penstock setting for each chamber.

These results show that storage chambers effectively reduce the frequency of overflow. At Great Harwood at least 63% of storms that would normally cause overflow (i.e. flow that exceeds Formula A) were retained, a reduction from 4.4 per week to 1.4 per week. At Clayton-le-Moors at least 80% of storms were retained, a reduction from 4.4 per week to 0.9 per week. Incidentally, this is twice the reduction in overflow frequency than that predicted for chambers of this size by the Scottish Development Department (1977). Clayton-le-Moors retains a higher percentage of storm events than Great Harwood, this is almost certainly due to the very high penstock setting allowing an unusually large proportion of flow to be discharged to treatment. Table 3 shows that although the number of storms and overflows per week varies according to season and that during summer and autumn the retention performance is slightly reduced, the storage chamber continues to retain at least 63% of storms. The lower retention performance during spring and summer of the Great Harwood storage chamber is due to the incidence of higher rainfall intensities over the catchment.

Table 4 depicts the rainfall required to cause an inflow or an overflow for each of the storage chambers. These figures are provided from a short test period that is one third of the overall study period (one third was chosen so as to ensure enough storms in both the test period and the model-verification period that followed).

Table 4 The rainfall required to cause an inflow or overflow at each overflow site

	Rainfall Required To Cause Inflow		
	Volume (mm)	Overall Intensity (mm/hr)	Peak Intensity (mm/hr)
Great Harwood	0.5	1.5+	2.0*
Clayton-le-Moors	1.5	4.0+	5.0*

	Rainfall Required To Cause Outflow		
	Volume (mm)	Overall Intensity (mm/hr)	Peak Intensity (mm/hr)
Great Harwood	4.0	3.0+	5.0*
Clayton-le-Moors	3.0	4.0+	7.0*

* This rainfall is required for at least 10 minutes.
\+ This rainfall is required for at least 30 minutes.

In comparison, the Great Harwood storage chamber requires at least 1.5mm/hr of rain for at least a 30 minute period to cause an inflow and a peak rainfall of 5.0 mm/hr for at least 10 minutes to cause an overflow. Due to its high penstock setting the rainfall required to cause an inflow to the Clayton-le-Moor chamber is 4.0 mm/hr for 30 minutes and the rainfall required to cause an overflow is 7.0 mm/hr for 10 minutes. These simple thresholds apply to almost all storms, but there are two exceptions. Overflow may occur at lower rainfall intensities when either baseflow levels are high due to a very short antecedent dry period since the last storm (one or two hours) or when the prevailing rainfall is prolonged for over two or three hours. The rainfall thresholds shown in Table 4 and the exceptions noted above provide the basis for a simple model for predicting storage chamber operation. This model has been tested using the remaining two thirds of the study periods at Great Harwood and Clayton-Le-Moors.

The rainfall data records for both catchments were examined and each individual storm event was classified as either a rain event causing no chamber inflow or a rain event causing chamber inflow or a rain event causing chamber overflow. The results of this prediction model are shown in Table 5.

Table 5 The prediction of storage chamber inflow and overflow

	Rain event but no inflow		Rain event causing inflow		Rain event causing overflow	
	Number of Storms	Percentage Prediction	Number of Storms	Percentage Prediction	Number of Storms	Percentage Prediction
Great Harwood	41	95%	130	91%	65	94%
Clayton-Le-Moors	19	100%	12	83%	17	94%

At Great Harwood, out of a total number of storms of 236, 216 (93%) were predicted correctly by this model. At Clayton-Le-Moors out of a total number of storms of 48, 45 (94%) were predicted correctly. In the prediction of overflows the model achieved a 94% success rate at both Great Harwood and Clayton-Le-Moors. The success of this prediction is a reflection of the very tight hydraulic control storage chambers exhert over storm overflow operation and its close affinity with rainfall characteristics within their catchments. It is the opinion of the authors that simplistic models such as this can form a positive opportunity for the prediction of overflow frequencies for storage chambers in simulation programs such as WASSP-SIM of the Wallingford Procedure (NWC/DOE Standing Committee, 1981). Furthermore, such models when operated in conjunction with continuously monitored rainfall events may form the basis for the future real time control of combined sewer overflows.

4.2 Flow Retention Performance

Values of the percentage of the total flow retained by each storage chamber have been calculated from the flow data for of 21 and 13 individual storm events at Great Harwood and Clayton-Le-Moors respectively. These storms reflect the variety of rainfall intensities and durations encountered during the study period. The mean value of flow retention for all storms was 77.6% and 86.1%, while the lowest percentages of flow retained for an individual storm were 53.8% and 70.5% at Great Harwood and Clayton Le Moors respectively. The better performance of the Clayton-Le-Moors storage chamber reflects the effect of the high penstock setting and the singular shape of the storm hydrograph that includes an unusually long "tail", possibly a result of some infiltration of stormwater into the sewer system after overflow has ceased.

It is clear from these calculations that these storage chambers retain the major proportion of total stormflow. In fact, a much larger proportion of stormflow than was predicted by the Scottish Development Department in 1977. This may be desirable for the prevention of watercourse pollution, but it may have an adverse effect upon the effluent treatment works that receives this greater proportion of retained storm water. During the past few years a study has been carried out by North West Water to ascertain why the Hyndburn Effluent Treatment Works (at which all Great Harwood and Clayton-Le-Moors foul sewage is treated) has a comparitively high cost of effluent treatment (Hudson and Torevel, 1985) when compared to other works in the area. The report noted that this very high cost was a result of treating unusually "weak" foul sewage derived from a sewer system that delivered a very high proportion of its stormwater. The North West Water study concluded that this high proportion of relatively "clean" stormwater was a direct result of routing all the system's combined sewage through a number of on-line storage chamber overflows.

The mean values of flow retention disguise a considerable variability between individual storms. This variability is dependent upon a combination of hydraulic factors that control the rates of chamber fill and overflow to receiving streams. These factors include : the penstock setting, the time of rise to first spill, the rate of flow spilled and the proportion of the total storm time that the overflow operates. These factors are in turn a function of the prevailing rainfall and run off characteristics such as : the volume and duration of rainfall, the initial, overall and peak rainfall intensities during each storm and the length of the preceding dry weather period. Of these the initial rainfall intensity and its effect upon the time of rise to first spill is of particular importance.

The considerable variability of flow retention performance of each storage chamber between individual storm events and its complex causal relationship with so many features brings into question the use of design storm hydrographs for the design of stormwater overflows incorporating storage. It is the opinion of the authors that a more effective method of storage volume estimation would involve the consideration of a time series of rainfall events (Fiddes, 1984).

4.3 The Delay Of First Spills

The delay time has been taken as the time from the first rise of flow into the storage chamber to the point of first spill to the receiving water course assuming that the storage chamber was empty at the start of the storm event. Table 6 presents a synopsis of the delay times associated with all the storm events characterised by overflow in a 38 week period at Great Harwood (35 storms) and Clayton-Le-Moors (28 storms).

Table 6 The mean, minimum and distribution of delay times

	Mean Delay Time (mins)	Minimum Delay Time (mins)	Number of storms with Delay 0-10 mins	Number of storms with Delay 0-20 mins	Number of storms with Delay 0-30 mins	Number of storms with Delay 0-60 mins
Great Harwood	53	4	7 (20%)	10 (29%)	17 (49%)	22 (63%)
Clayton Le-Moor	67	9	1 (4%)	5 (18%)	12 (43%)	18 (64%)

Table 6 shows that over the 38 week study period these storage chambers delay the point of the first spill for a mean time of about one hour. This allows for the storage of any first foul flush present at the start of storm events and gives the receiving watercourse some time to attain a certain degree of dilution of any spilled effluent. The delay times of the Clayton-Le-Moors storage chambers are longer than those of Great Harwood as a result of the difference in penstock settings. The delay times at both storage chambers vary according to the initial rainfall intensity and in some storm events the delay may be less than 5 minutes at Great Harwood and less than 10 minutes at Clayton-Le-Moors. In storms of this kind there is a definite threat of pollution to the receiving watercourse because results have shown that the first foul flush lasts for an average of 30 to 40 minutes (Thornton and Saul, 1986). As Table 6 shows, at Great Harwood and Clayton-Le-Moors greater than 40% of storms that cause overflow have delay times of less than 30 minutes.

The Hyndburn Brook receives the spilled effluent from the two storage chambers studied here. Figure 3 depicts a storm hydrograph from the Hyndburn Brook in which there is a clear visible inpact on stream flow by the four main on-line storage chamber overflows located in the Hyndburn borough (Great Harwood, Clayton-Le-Moors, Accrington and White Ash Brook). In this example a high intensity rainfall event reduced the delay time of first spill and created very high rates of overflow to the Hyndburn Brook. At the Great Harwood overflow chamber, for example, the delay time was 11 minutes and the peak rate of overflow was 0.97 m^3/s. As a result the stream hydrograph has an excessively peaked appearance where the highest flows are associated with overflow spill before the period of normal river flow. An effect such as this raises the question as to whether the type and capacity of storage chamber overflows should be designed to suit the sewage system or the receiving river system.

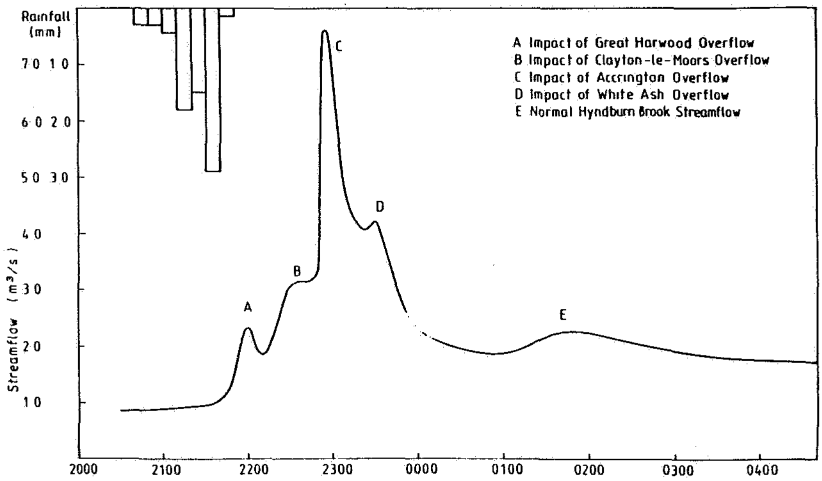


Figure 3 A Hydrograph From The Hyndburn Brook

5 Conclusions

A fieldwork program of research at two on-line storage overflow chambers has shown that good hydraulic control may be achieved if construction follows the design criteria suggested by Ackers et al (1968). This hydraulic control includes, a minimum reduction in the frequency of overflow of 60%, a mean reduction in the total flow spilled of at least 75% and a mean delay time of first spill of one hour.

A simple model for the prediction of storage chamber operation was formulated (based on rainfall characteristics). Tests showed that such a model may have a role in the simulation and real time control of storage chamber behaviour.

Both storage chambers in this study showed good flow retention efficiency and delayed the time of first spill. However, there was considerable variability of performance between the two storage chambers and between individual storm events of the study period. This variability has raised questions concerning :

a) The use of simplistic design criteria such as design storms and per capita based storage estimation methods. An approach based on time series rainfall events is recommended.

b) The design of storage chambers to suit sewer systems rather than the receiving river systems. The discharge from overflows may have a significant effect of the volume of river flow.

c) The day-to-day operation and maintenance of storage chambers such as the setting of penstocks. Such settings should always be monitored in situ.

d) The peripheral effects of storage chamber construction and operation such as those influencing the sewage system effluent treatment works.

Finally it is concluded that the hydraulic efficiency of on-line combined sewer storage overflow chambers of this design are inarguable, but further work is required to refine their construction, application and operation particularly with reference to the quality as well as the quantity of combined sewer effluents.

Acknowledgements

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EFFECTIVENESS OF STORM SEWAGE OVERFLOW STRUCTURES IN HANDLING GROSS POLLUTING SOLIDS

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Abstract

The paper describes the criteria that can be used to assess the performance of storm sewage overflow structures and explains why the ability to separate gross polluting solids is particularly important. Basic design concepts for effective hydraulic control and good solids separation are presented and a comparison made with some design defects from the past. The author describes the results of model tests on four modern types of overflow structure, in which plastic particles were used to simulate the gross polluting solids. A comparison is made between the performance of these structures and recommendations are given for use in design.

1 Introduction

Storm sewage overflow structures are widely used in combined sewerage systems throughout the world to relieve overloading and to reduce the volumes of sewage that have to be dealt with at the treatment works at times of storm. They have been identified as one of the principal causes of urban river pollution so that in upgrading sewerage systems in the U.K. in recent years particular attention has been paid to their rationalisation and improvement.

Many of the existing sewers were built during the 19th century when the sewers themselves were constructed. Since then additional overflows have been added, often on an ad-hoc basis as the needs arose. The simplest types of overflow used were the 'hole in manhole' and leaping weir overflows, neither of which were capable of diverting large volumes of flow. Where a large proportion of the flow had to be diverted the low side weir was used. These early types of overflow structure are illustrated in Figure 1 and are still in widespread use in the U.K. today.

None of these early overflow structures were fitted with throttles on the continuation pipe so there is no precise control over the discharge at which first spill occurs, and the flow to treatment rises considerably as the combined inflow increases (Figure 2). Due to a lack of understanding of their hydraulic performance the amount of water overflowing to the river was often underestimated, and this problem was compounded by increased flows as urban areas expanded. Additional overflows therefore became necessary and this has led to a proliferation of overflow structures in the larger urban areas in the U.K. Since these early structures do not exercise proper hydraulic control, nor provide for any separation of polluting solids, urban watercourses are frequently polluted by storm sewage. In England and Wales alone there are over 2000 unsatisfactory overflow structures of which nearly 25% discharge to Class 1 rivers.

2 Criteria for Effective Storm Overflow Structures

The principal objectives of a sewerage system are:

a) to carry foul sewage to a suitable point for treatment and/or disposal;

b) to drain paved areas;

c) to safeguard natural watercourses from pollution.

When improving existing combined systems the engineer is faced with something of a dichotomy. On the one hand he would wish to remove all storm overflow structures in order to safeguard the watercourses from pollution, yet on the other he may need to retain discharge to the river in order to prevent overloading. The problem can be overcome, however,

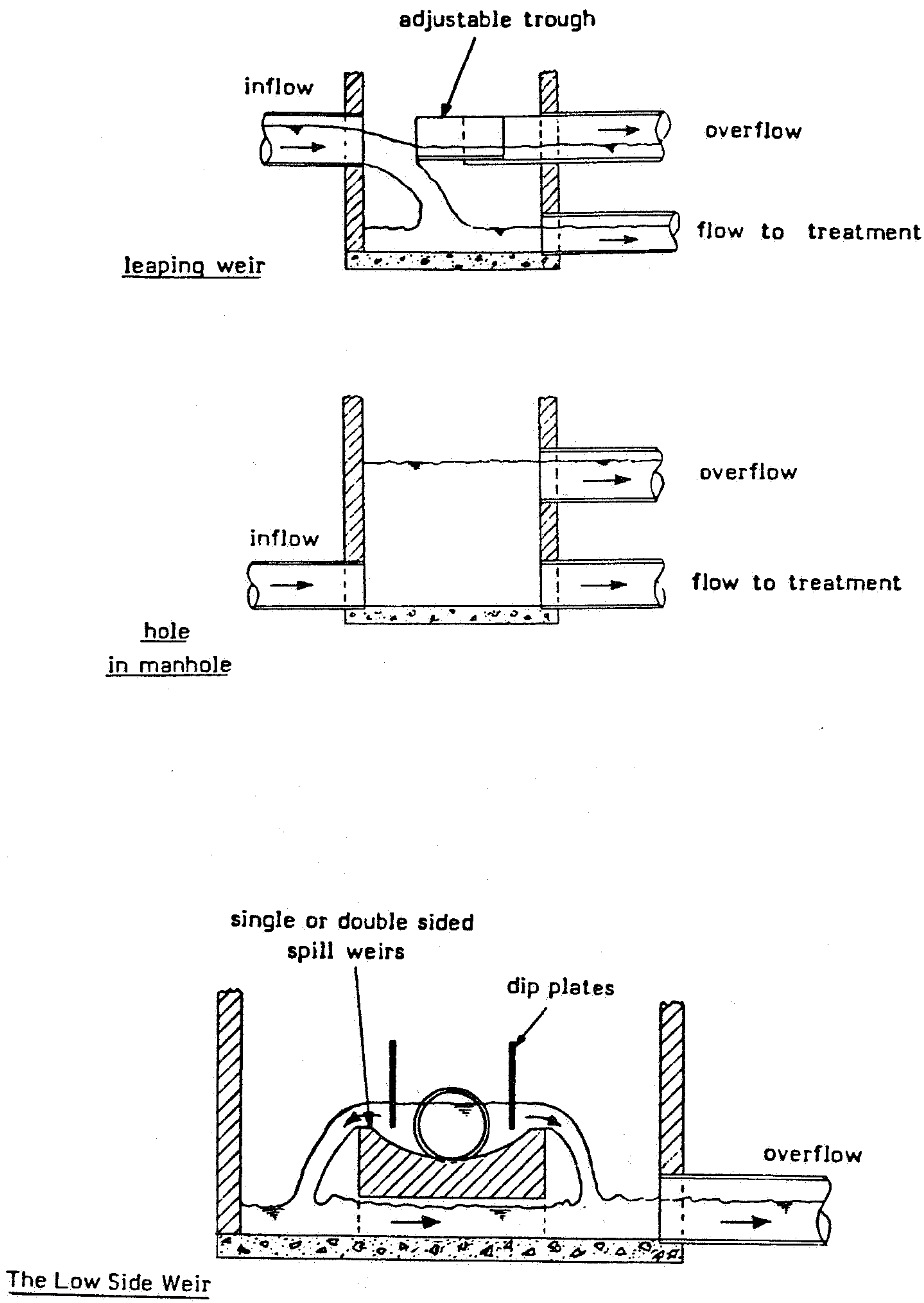
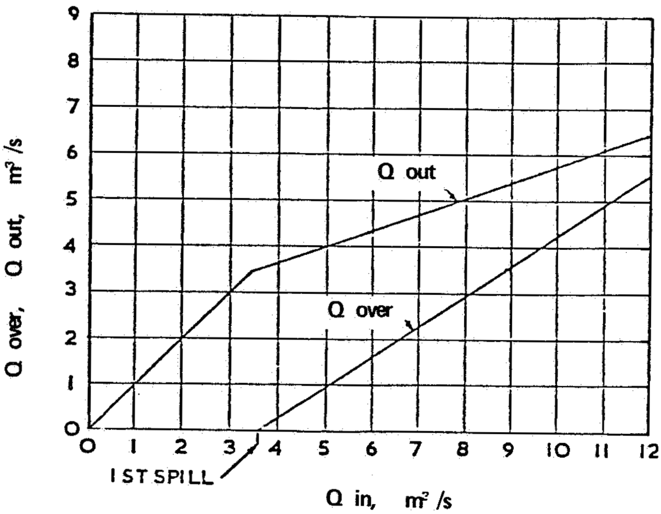
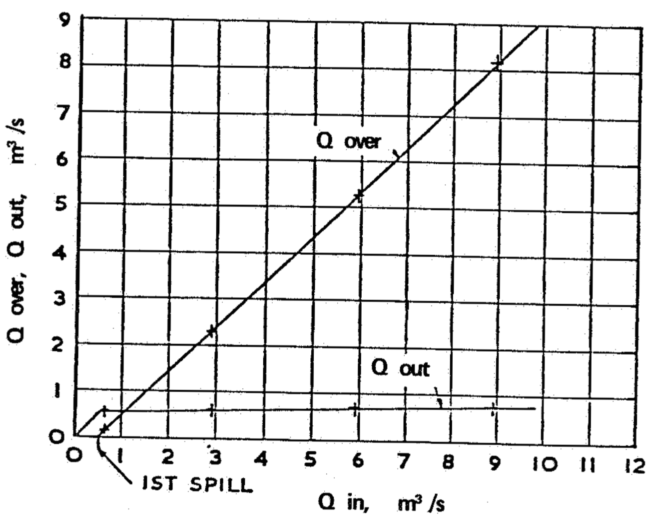


Figure 1 Early Storm Sewage Overflow Structures



Unsatisfactory Low Side Weir



Satisfactory High Side Weir

Figure 2 Discharge Characteristics

by rationalising storm overflow structures, i.e. by removing those structures with unsatisfactory performance and replacing them with a small number of effective overflow structures.

In order to assess which of the existing structures are unsatisfactory and to be able to design effective new structures it is necessary to identify the criteria for effective storm overflow structures, as follows:

a) fully automatic operation;

b) the setting (flow at which first spill occurs) should be appropriate to the location and desired function of the overflow;

c) the flow to treatment should not increase appreciably as the amount overflowed increases;

d) separation of pollutants so that the maximum amount of polluting material passes forward for treatment;

e) minimum maintenance requirements;

f) minimum cost of construction.

Both existing structures and proposed designs can be evaluated against the above criteria. Where field data is unavailable the frequency and operation of overflow structures may be assessed by computer simulation using a suitable hydraulic model such as the Wallingford Procedure (1981).

The setting of an overflow structure is particularly important. The setting should take into account the composition of the dry weather flow (e.g. is it particularly polluted by industrial effluent?), the capacity of the downstream sewer and treatment works, the impact on the receiving watercourse, and the current and proposed use of that watercourse. Because of the continued growth of urban areas many overflow structures discharge at little more than dry-weather flow, whilst others overflow into small streams passing through residential areas. These are prime targets for rationalisation.

3 Guidelines for New Storm Overflow Structures

Before considering specific designs several general guidelines can be given for effective overflow operation. To ensure that the overflow operates at the prescribed setting the outlet should be throttled, using an orifice plate, penstock or throttle pipe, and the overflow weir set above the centreline of the incoming sewer. This will ensure a controlled gentle motion in the chamber not only ensuring a predictable first spill, but also providing the required regulation of the flow passing forward to treatment. Such conditions are rarely achieved in older overflow structures (Figure 2).

The velocity in the incoming sewer should be as low as possible (but not so low as to cause a build up of sediment in the overflow chamber). This is to encourage separation of polluting solids. Generally inlet conditions where Q_{in} is greater than $1.5D^{2.5}$ (Q_{in} in m^3/s, D in metres) will not allow significant separation of pollutants.

To reduce maintenance costs it is important that the chamber floor is self cleansing and the outlet throttle does not block. Experience has shown that a central 300 mm half round channel with a longitudinal slope no less than 1 in 50 and with benching sloping at 1 in 4 will ensure self cleansing in most cases, and a minimum dimension of opening of 200 mm at the throttle will reduce the risk of blockage.

4 Recommended New Designs

Three types of overflow structure are recommended for general application. They are the High Side Weir, the Stilling Pond and the Vortex with Peripheral Spill.

4.1 The High Side Weir

The High Side Weir gives good hydraulic control of the flow and may be constructed with single or double weirs. The downstream throttle ensures that subcritical flow occurs over the length of the weir and a simple graphical method developed by DeMarchi (1934) may be used to determine the required length of weir. The author has shown (Balmforth and Sarginson (1983)) that the weir coefficient is the same for side weirs as for transverse weirs.

Saul and Delo (1982) determined appropriate dimensions for the width of chamber, crest height and scumboard dimensions to give good separation and retention of settleable and floating solids. They also recommended a short "inlet length" before the weir and a small storage area between the downstream end of the weir and the throttle. The recommended chamber dimensions are given in Figure 3.

4.2 The Stilling Pond

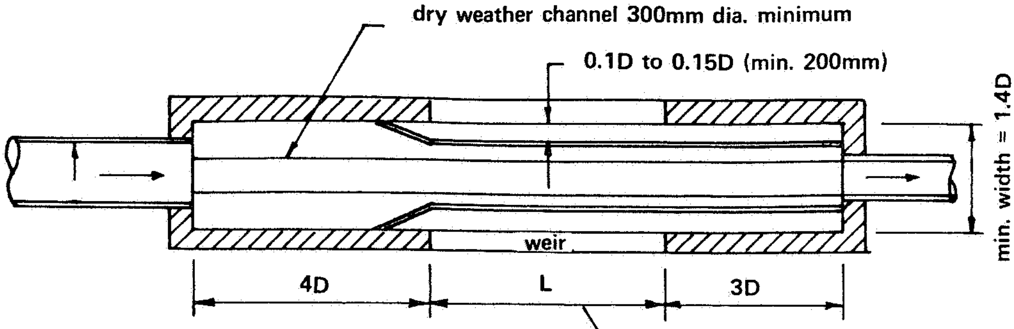
The Stilling Pond overflow structure was devised by Sharpe and Kirkbride (1959) and improved and extended by the author (Balmforth (1982)). It is widely used throughout the U.K., and it also gives good hydraulic control. The recommended dimensions are given in Figure 4 and Table 1. When constructing the chamber on an existing sewer the requirement for D_{min} is sometimes ignored and the chamber proportioned from the diameter of the existing upstream sewer. The length to the scumboard of $7D_{min}$ in the figure refers to the extended stilling pond, and compares with the earlier $4.5D_{min}$ recommended by Sharpe and Kirkbride.

Table 1

Dimensions of the Stilling Pond Overflow (Figure 4)

Height of weir crest H_W	Minimum Diameter of upstream sewer D_{min}	Maximum depth in chamber H_m	Height of Scumboard H_s
0.90D	$0.848Q_{in}^{0.4}$	1.60D	0.50D
1.00D	$0.828Q_{in}^{0.4}$	1.70D	0.60D
1.20D	$0.815Q_{in}^{0.4}$	1.85D	0.80D

The dimensions in Table 1 are D in metres and Q_{in} in m^3/s. The best performance is obtained from the 1.2D weir height. All vertical dimensions are measured from the invert of the upstream sewer at entry to the chamber.



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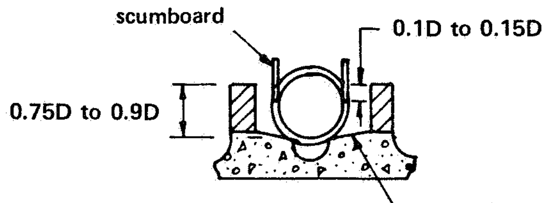

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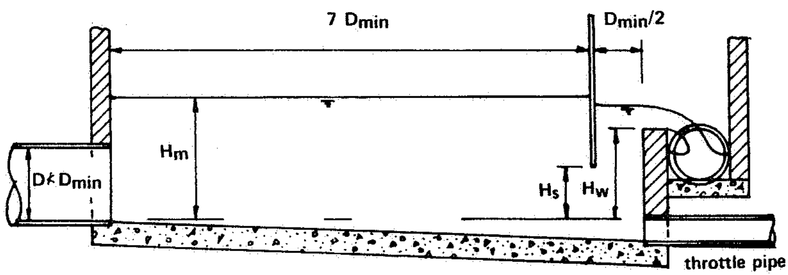
Figure 3 High Side Weir



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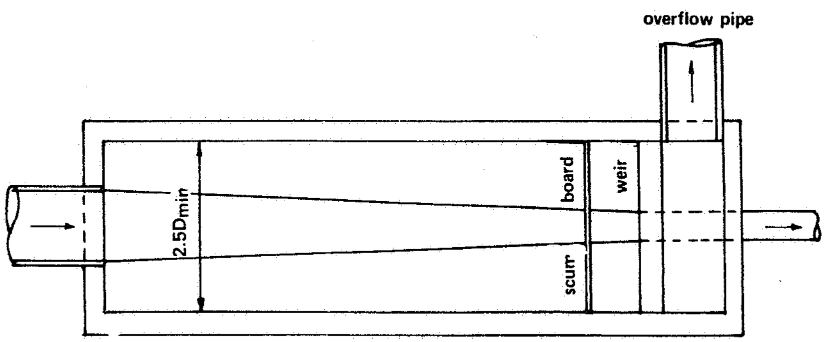

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Figure 4 Extended Stilling Pond

4.3 The Vortex with Peripheral Spill

This is a new type of device (Balmforth et al (1984)) which uses vortex motion to regulate the outflow and to separate the polluting solids. Due to the vortex motion at entry to the continuation pipe the hydraulic control of the flow is marginally better than the High Side Weir and Stilling Pond overflows. The formation of a forced vortex in the chamber greatly assists in the concentration of settleable solids in the centre of the chamber, and these then pass through the outlet and carry forward for treatment. Floatables are retained behind the scum-board and are removed through the central air core. The dimensions of the Vortex Chamber are given in Figure 5 and Table 2.

Table 2

Recommended dimensions for the vortex with peripheral spill

Weir Length L	=	1/4 of circumference
Weir Height H	=	1.5D
Chamber diameter Ø	=	4D
Inlet channel width A	=	1.15D
Inlet channel length B	=	1.2D
Slope of chamber floor	=	1 in 4
Diameter of outlet pipe d	=	0.2 - 0.4D*

* The diameter of the outlet pipe will determine the setting and may be obtained using the theory of the vortex drop devised by Ackers and Crump (1960).

5 Comparison of Performance

The three types work equally well hydraulically. To compare their capabilities at handling polluting solids the results of model tests using plastic particles have been used. The hydraulic models are based on the Froude Law of scaling which has been shown by Saul (1977) to give a reliable prediction of prototype performance. He also demonstrated that the performance of sewage solids could also be modelled using particles with the same ratio of settling/rise velocity to mean

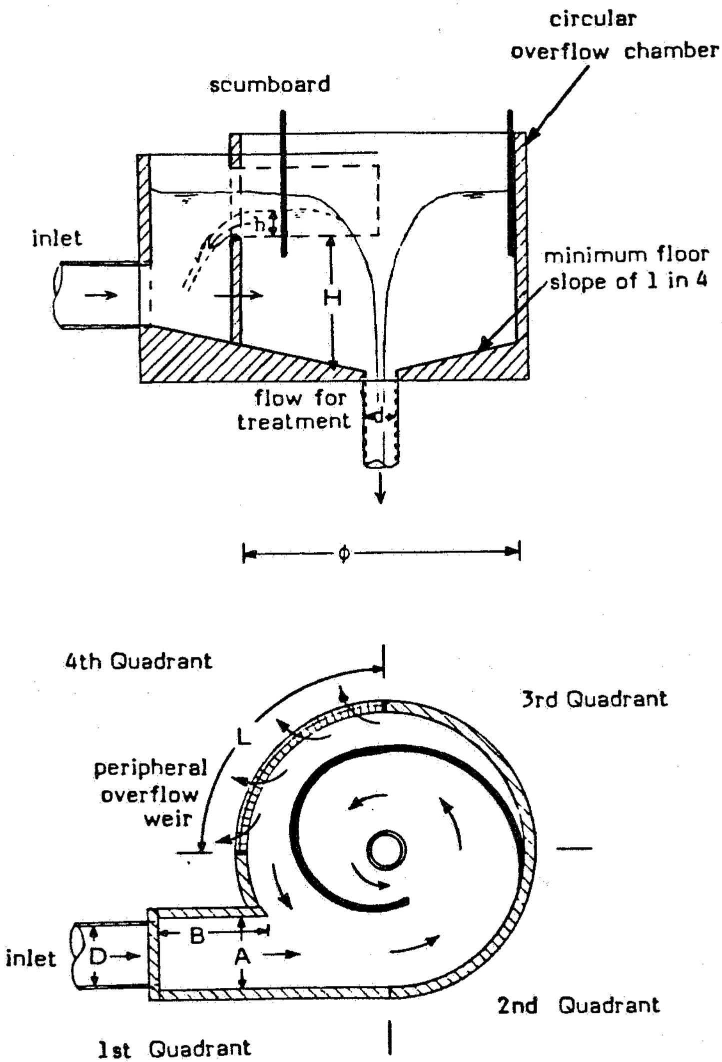

Figure 5 Vortex with Peripheral Spill

inlet velocity as would be found in the prototype.
Figure 6 shows the comparative performance of the chambers as obtained in the model tests. To obtain a more global picture of the comparative performance of the overflows in the field three sets of data on sewage solids reported by the Scottish Development Department (1970) have been used. This data is summarised in Figure 7 and has been used to calculate the overall separating efficiency of the chambers for the three different configurations given in Table 3.
The High Side Weir is the least efficient of the three types, but it may be designed to have only a small drop in invert. Since it does not require the upstream sewer to be surcharged it is particularly suited to existing sewers with relatively flat gradients.
The Stilling Pond requires a moderate drop in the invert and also some surcharge of the upstream sewer. Separating efficiency is more sensitive to changes in upstream conditions so that it would not be recommended when the discharge Q_{in} was significantly greater than $1.5D^{2.5}$ for long periods. The extended stilling pond performs marginally better than the original version.
The Vortex gives the best separating efficiencies of the three and is also the least sensitive to varying inlet conditions. It does however require a considerable drop in the invert of the sewer and is therefore more suited to hilly regions, particularly as it can readily be constructed in a circular shaft.
The efficiency of all three types decreases if the inlet flow increases, or if the diameter of the inlet sewer is reduced. From the point of view of separating pollutant solids, therefore, as much flow as possible should be passed forward to treatment and the inlet sewer should be as large as possible.

6 Conclusions

A short list of criteria for effective overflow operation has been given to help the engineer identify existing structures that require replacement as a matter of priority. These criteria may also be used as a guide in the design of new structures.
Three designs are recommended, the High Side Weir, Stilling Pond, and Vortex with Peripheral Spill. The choice will depend largely on

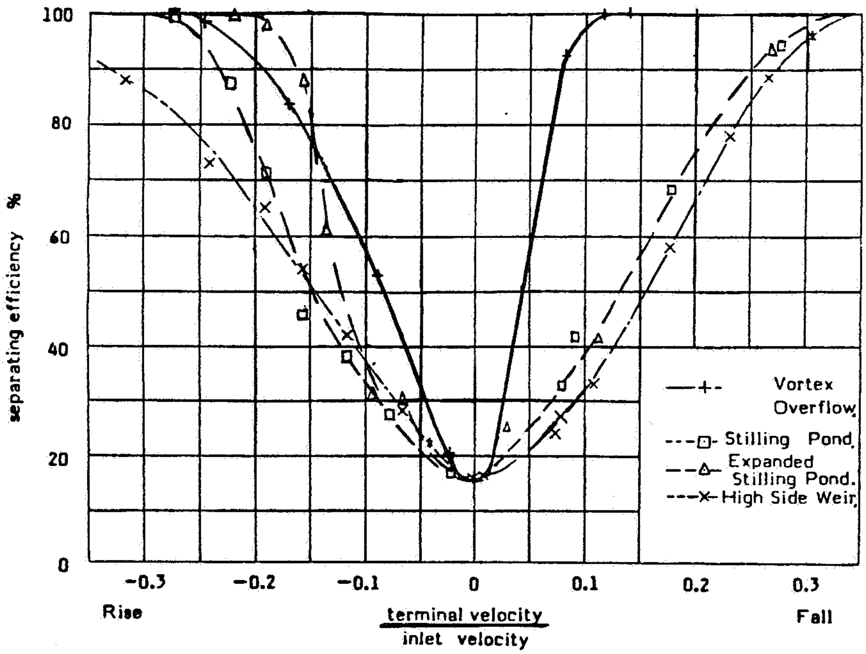

Figure 6 Comparison of Separating Efficiences

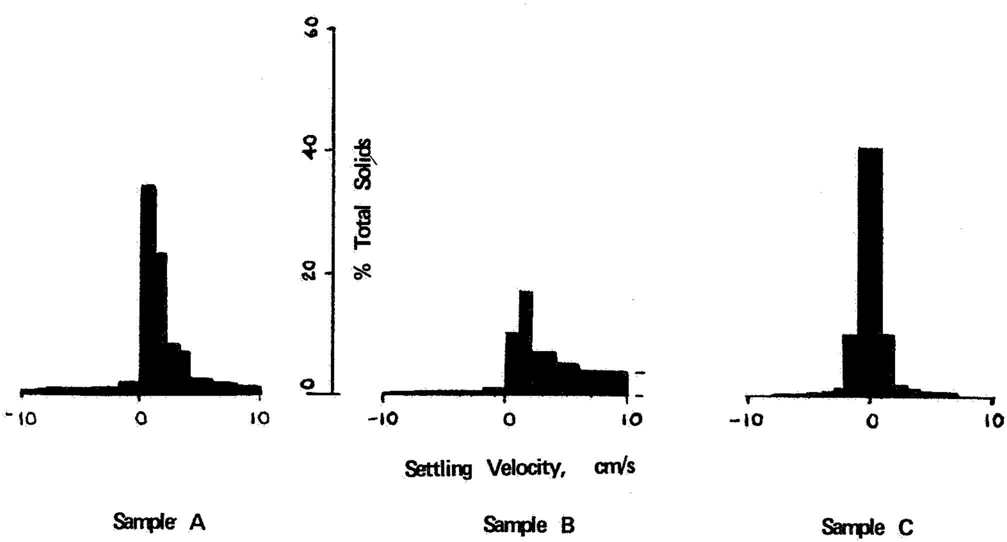

Figure 7 Storm Sewage Characteristics

topography and the layout of the existing system.
As much of the inflow as possible should be passed forward to treatment and the diameter of the incoming sewer should be as large as possible.

Even though the bulk of pollutants consist of very fine suspended solids the results demonstrate that the separating capabilities of the recommended structures can make a very real impact on the quality of urban water courses.

Table 3

Comparison of stilling efficiencies

Configuration		Sewage Sample	Separating efficiency: Vortex	Stilling Pond	Ext. stilling Pond	High Side weir
Q_{in}	= 300 l/s	A	55.5	42.0	43.1	35.8
V_{in}	= 0.35 m/s					
Q_s	= 60 l/s	B	75.5	62.7	60.2	62.8
Q_s/Q_{in}	= 0.20					
D	= 1050 mm	C	28.0	25.5	27.5	21.8
Q_{in}	= 400 l/s	A	44.7	26.9	31.3	28.2
V_{in}	= 0.46 m/s					
Q_s	= 60 l/s	B	72.9	45.1	52.2	47.2
Q_s/Q_{in}	= 0.15					
D	= 1050 mm	C	20.3	15.4	17.4	17.2
Q_{in}	= 300 l/s	A	44.0	39.1	40.3	34.5
V_{in}	= 0.47 m/s					
Q_s	= 60 l/s	B	69.1	59.2	60.1	48.6
Q_s/Q_{in}	= 0.20					
D	= 900 mm	C	24.5	24.4	26.6	14.8

7 Acknowledgements

The author is indebted to Mr Stephen Lea for his assistance with the figures.

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DETENTION BASINS AND THEIR CAPABILITIES FOR THE RETENTION OF POLLUTANTS

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Abstract

The objective of a project supported by the Deutsche Forschungsgemeinschaft was to determine the extent of water pollution arising from plants for combined sewage treatment and from clarification plant effluents during rainfall events.
The results can be summarized as follows: Substantial first flushes occur in those sewer sections in which a larger accumulation of pollutants can be expected during periods of dry weather run-off and in which, however, high flow velocities are found during storm water run-off. With increasing capacity of the detention basin the amount of pollutants discharged in each case with the overflow into the receiving water also increases. On the other hand, with increasing capacity the frequency of overflow events decreases, thus reducing the amount of pollutants discharged into the receiving water on a yearly basis. The more substantial the first flush in the combined water run-off, the less the extent of the pollution arising from the overflow.

1 Introduction

During rainfall events high pollution loads are discharged from paved and sealed surfaces together with the storm water run-off. This is especially the case, when rainwater is mixed with sewage in a combined sewer system, when surface pollutants are flushed off of paved or sealed areas or when the deposits in the sewer system are discharged. Natural bodies of water can thus only be succesfully protected, when the treatment of storm water is given the same priority as wastewater treatment. Therefore storm water and wastewater treatment must be looked upon as being of equal importance in respect to their effects on receiving waters.

Already in the seventies in the F.R.G. it was recognized that during storm water run-off very high amounts of pollutants are flushed out of sewer networks. In order to prevent these pollutants from entering receiving waters the construction of detention basins was implemented. Thus the first highly polluted part of the storm water run-off is stored and detained before it enters the sewage plant. The more intensive efforts in the F.R.G. to equip sewer networks with detention basins led, for example, after Göttle (1984) in Bavaria to the construction of about 600 000 m^3 of usable storage capacity by the end of 1983. Krauth (1984) reported, that at the beginning of 1984 detention basins with a total usable capacity of 1 Mio m^3 were in operation in Baden-Württemberg. About 2/3 of these basins are designed as storage basins.

The dimensioning and design of detention basins and overflow structures for combined sewage is determined by the guidelines of the ATV working paper A 128 (1983).

According to these guidelines larger drainage areas are to be divided in catchment subareas (A_{red}= 10 - 15 ha). In principle the first, highly polluted portion is diverted to the sewage works and the less polluted combined sewage is passed as soon as possible into the receiving water. The guidelines in the working paper are based on global assessments and require as an objective the containment of a yearly average of 90% of the biodegradable and settleable substances.

2 Types of storm water detention basins

Detention basins are necessary when the critical combined sewage run-off cannot or should not be treated in the sewage plant. After the rainfall event the contents of the basin are conducted to the clarification plant. Storm water detention basins are constructed either as storage basins or as detention-sedimentation basins.

A combination of both basin types is also possible. As a rule basins are dimensioned so as to allow storm water run-offs r_{eff} to be discharged to the clarification plant at a rate of 1 - 3 l/(s.ha).
Detention basin, Type A (Figure 1): Storage basins of this type detain the first portion of the combined run-off completely and, on the whole, discharge the major portion of the combined run-off, with the exception of the portion diverted to the sewage plant, directly into the receiving water.

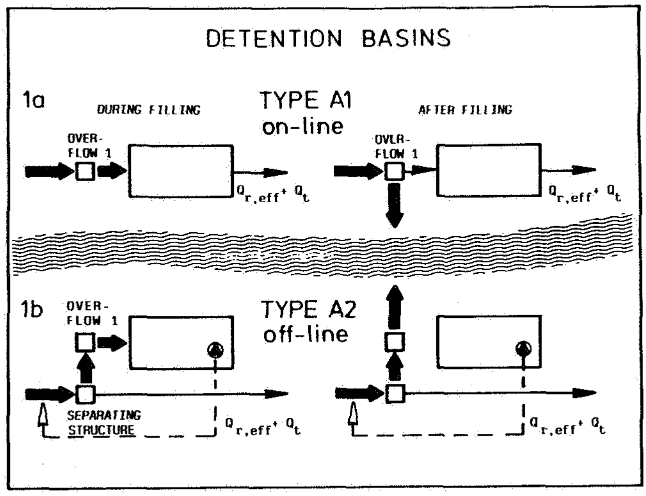

Figure 1 Principle of operation of the detention basin, Type A

If the storage basin is arranged on-line (Figure 1a), then its effluent is discharged to the clarification plant. If the quantity of combined sewage run-off is so large that it exceeds the capacity of the detention basin, the overflow is discharged directly into the receiving water.
If the basin is arranged off-line (Figure 1b), then the combined sewage is divided in two streams, one of which bypasses the detention basin and goes directly to the clarification plant, the other of which is passed into the basin. The basin overflow only goes into operation when the basin is full.
Detention basin, Type B (Detention-sedimentation-basin, Figure 2): Detention-sedimentation-basins do not differ from storage basins until they are completely full. At the beginning of storm water run-off events all of the combined sewage is stored in the basin. After they are

completely full, however, the inflow is limited to the maximum overflow discharge rate (max $Q_{kÜ}$). Thus, an additional elimination of suspended solids of the combined sewage by sedimentation is given. If the storm water run-off exceeds the critical value for max $Q_{kÜ}$, then the overflow of the detention basin goes into operation.

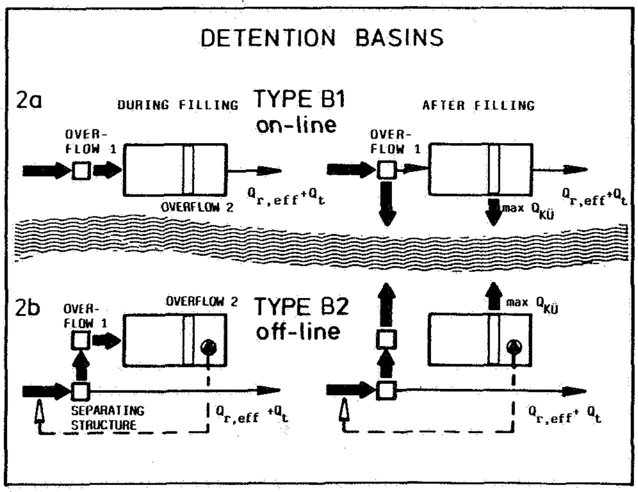

Figure 2   Principle of operation of the detention basin, Type B

Detention basins should be so planned that they are connected to flat sewer sections, in which the velocity of the calculated dry weather flow is lower than 0,50 m/s.
The arrangement of detention basins at several points, especially at points where pollutants accumulate, is much more effective than a system with larger basins at less points. The total catchment area should therefore be divided in catchment subareas consisting of 10 - 15 ha of sealed surface.
The catchment sub-areas should be homogeneous and the flow times to the detention basins should be nearly equal. When choosing a site for a detention basin one should also determine, whether or not a technically and economically feasible control of the discharge rate of the detention basin is possible. A nearly constant discharge rate should be achieved and the maximum allowable influent rate of the clarification plant must not be exceeded.
The optimal layout, as shown in the ATV working paper A 128, should be

the objective.

3 Flushing behaviour of sewer deposits

From time to time solids can be deposited in sewer systems. Comprehensive investigations carried out on sewer sections in Stuttgart-Büsnau and in an experimental channel showed that the deposition of solids and thus their scouring and discharge are influenced by several factors. Most important for the formation of deposits are the shear stress or flow velocity during dry weather run-off and the population density. The duration of the dry weather period between two rainfall events plays a significant role for the accumulation of solid substances. Measurements in Büsnau showed that during comparable dry weather periods approx. 8 times as much of solids are deposited in a relatively flat sewer section (I_s= 0.8%) than in a more steeply inclined one (I_s= 2.2%). The flushing behaviour of sewer networks differs as a rule significantly from that of the individual sewer sections. In Figure 3 the relative pollution loads F_s/F_{tot} that are discharged with the first 30% of the total run-off volume (Q_s/Q_{tot}= 0.30) are plotted against the sealed surface areas A_{red} in ha for different catchment areas. A_{red} was chosen as an idependent variable because it is a suitable parameter that implicitly comprises the effects of events resulting in weaker first flushes, when the average bed inclines and the homogeneousness of the networks are comparable.

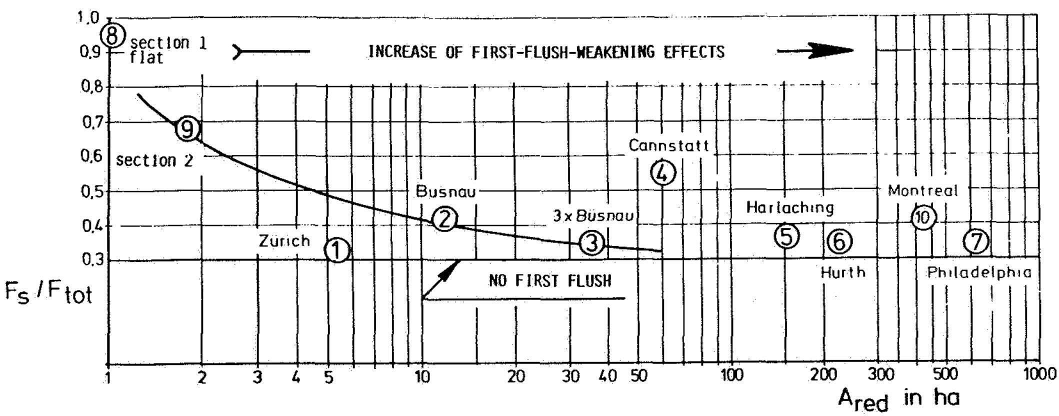

Figure 3 F_s/F_{tot} as a function of the sealed surface area A_{red}

The interdependence, as illustrated by the plottet curve, shows unequivocally that with an increase in the size of the sealed surface area A_{red} the influence of factors weakening the first flush also increases so that the first flushes lose more and more of their intensity. Obviously, first flushes seldom occur when the size of the sealed area lies above 100 ha, independent of the average bed slope.
In the following, calculations involving the indirect and direct pollution load for the receiving water are delt with. The basis for these calculations are the data measured during a two-year continuum of combined sewage run-offs. Furthermore they are based on the model assumption of an off-line storage basin (Type A2) being used as a storm water detention basin.

4 Influence of the storm water run-off

The data measured show that a direct interaction exists between the amount of pollutants discharged and the amount of the combined sewage run-off (average value x_M) as shown in Figure 4.

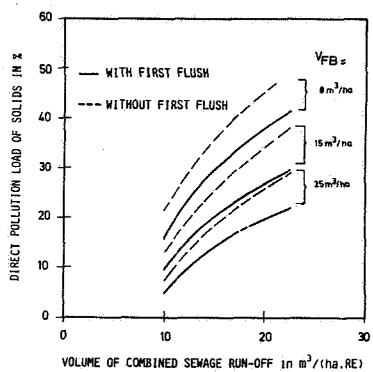

Figure 4 Direct pollution load of solids in % vs. the volume of combined sewage run-off for different basin volumes

Figure 4 also illustrates that more water and pollutants of all types flow to the receiving water, when the amount of combined sewage is larger. Thus it is obvious, that every storm water run-off leads to an other, different direct pollution of the receiving water when the overflow of a detention basin goes into operation. On the other hand it is also clear, that the flushing behaviour of the corresponding catchment area also exercises considerable influence on the direct pollution of the receiving water.

5 Influence of the size of the basin

For known rainfall events the overflow frequency and the corresponding overflow duration can be influenced by the detention basin size (Figure 5).By dimensioning a detention basin with 8 m³/ha the yearly overflow frequency of 241 events at V = 0 m³/ha can be reduced to a value of 120. The yearly overflow duration of 241 h/a is then also reduced by 44% to 135 h/a. This results in a decrease of 54% of the amount of water discharged via the overflow. By means of a storage basin the amount of solids discharged on a yearly basis during first flush events (Büsnau) decreases from 497 kg/(ha.a) to 241 kg/(ha.a).
The extent of the direct water pollution, as ascertained in terms of the yearly averages for solids and BOD_5, is not only dependent on the overflow frequency and thus the specific basin size but also on the flushing behaviour of the deposits in a sewer network. This means that it is of great importance for the extent of the direct water pollution whether or not the deposits in the sewer network are discharged by a first flush.

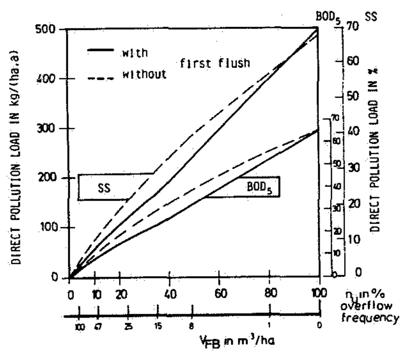

Figure 5 Direct pollution load in kg/(ha.a) and in % rsp. vs. basin volume and overflow frequency rsp.

In the case of a basin volume of 15 m³/ha, a yearly average of 167 kg/(ha.a) solids is directly discharged into the receiving water during a first flush event. This corresponds to a value of 23.2%, relative to the pollution load during combined sewage flows > $2Q_s + Q_f$. If no first flush - however the same yearly discharge of pollutants out of the sewer network - took place, then 215 kg/(ha.a) of solids would be carried into the receiving water. This would amount to 29% more than in the case of a first flush discharge, as found in Stuttgart-Büsnau. Analogously the same results are found in respect to the BOD_5-load, however less intense than in the case of the discharged solids.
Besides the BOD_5-value and the NH_4^+-concentration that are detrimental to the oxygen balance of a body of water, the substances that form sediments and adhere to surfaces capable of supporting biological films are also highly important for the quality of water. For soil flora and fauna not only the kinds of substances discharged into the water are important but also the frequency of pollution events and the absolute amount of pollutants. According to the statements of several hydrobiologists, a single strong event that takes place seldomly appears not to be so relevant for the water quality as the interaction of several pollution events taking place within a certain period of time. If this period of time is designated as being the interaction time and if all overflow events during this interaction time are combined to one single event, then this event is looked upon as being relevant for the water quality when the discharge of pollutants exceeds 8.4 kg/ha solids or 6 kg/ha BOD_5 during an interaction time of 16.7 hours. The results of such a calculation are shown in Figure 6. The graph in this figure shows the course of the decrease in the pollution frequency η_n, that ist relevant for the quality of the water, as well as the course of the directly discharged solids $\tilde{x}$ (50%-value), that results from a statistical evaluation of the individual results as a function of the size of the storage basin V_{FB} for events with first flushes and without first flushes.

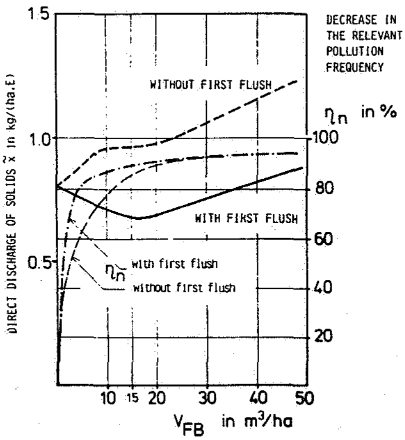

Figure 6 Direct discharge of solids x and decrease in the relevant pollution frequency η_n resp. vs. basin capacity V_{FB}

As shown in figure 6, a storage basin with a capacity of 15 m³/ha can reduce the pollution frequency η_n by 90%, when the combined sewage run-off is discharged with a first flush and by 84% in the case of no first flush. Basins with a capacity smaller than 15 m³/ha lead to a drastic increase, basins with a capacity larger than 15 m³/ha to a relatively insignificant decrease of the pollution frequency, as compared with basins with a capacity of 15 m³/ha. On the other hand, the statistical evaluation of each overflow event shows that the direct discharge of solids into the receiving water $(\tilde{x})$ for each overflow event is always higher without a first flush than with one. With increasing capacity of the storage basin (> 20 m³/ha) the direct discharge of solids generally increases. This means that when the overflow of large storage basin goes into operation, more solids are discharged into the receiving water than from a smaller basin. In the case of combined sewage run-off with a first flush effect the storage basin with a capacity of 10 - 20 m³/ha thus results in the smallest direct discharge of solids into the receiving water and also in a decrease of the relevant pollution frequency by 90%. These figures prove that it is much more purposeful to contain the first flush of combined sewage run-off in small-capacity storage basins than to attempt to treat a larger quantity of homogeneously polluted combined sewage in a large basin.

6 Interaction between direct and indirect water pollution

The extent of the direct and indirect yearly pollution of a receiving water ist related to the storm water detention basin size through the overflow frequency, $n_{ü}$, and the time, t_{ind}, that is necessary for emptying the basin (Figure 7). By increasing the size, V_{FB}, the yearly overflow frequency and duration and thus the combined sewage and pollutant amounts that are discharged directly into the receiving water decrease. In the same measure in which water is detained by storage at the basin overflow, the duration of the inflow of combined sewage into the clarification plant and thus the amount of the effluent from the clarification plant into the receiving water increases.

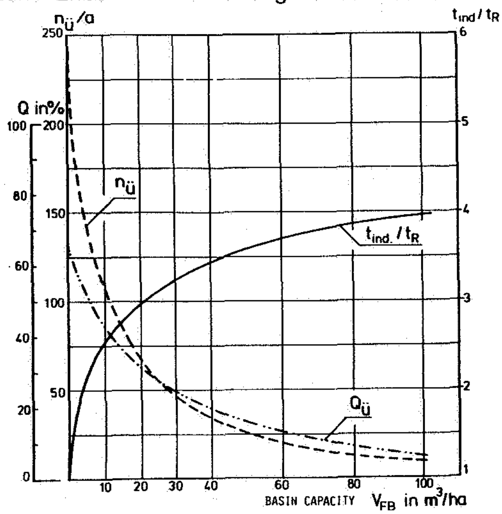

Figure 7 Overflow volume in %, overflow events per year and relative duration t_{ind}/t_R of the combined sewage run-off vs. basin capacity V_{FB}

The extent of the direct and indirect water pollution on a yearly basis is shown in Figure 8, from which the following conclusions can be derived:

The discontinuous direct pollution load is significantly diminished when using detention basins with a capacity of 8 m³/ha. When no first flush takes place, then larger detention basins are required than when a first flush occurs. Therefore it is purposeful to locate detention basins at those points in the sewer network where an intensive first flush can be expected. In this way it is possible to rapidly discharge relatively pollutant-free storm water into the receiving water. The indirect discontinuous pollution of the receiving water increases with increasing detention basin capacity. The increase is, however, insignificant when using detention basins with a capacity of 20 m³/ha or more. This indirect pollution is strongly dependent on the efficiency of the secondary clarifier during combined sewage run-off. The total yearly pollution of a body of water consists of the discontinuous pollution loads during combined sewage run-off and the pollution load originating from the dry weather run-off. A minimum for the total pollution load does not exist. Storm water and wastewater treatment must be looked upon as being of equal importance in respect to their effects on receiving waters. Therefore it is not purposeful to differentiate too strongly in respect to the extent of the directly and indirectly discharged pollutants during combined sewage run-off.

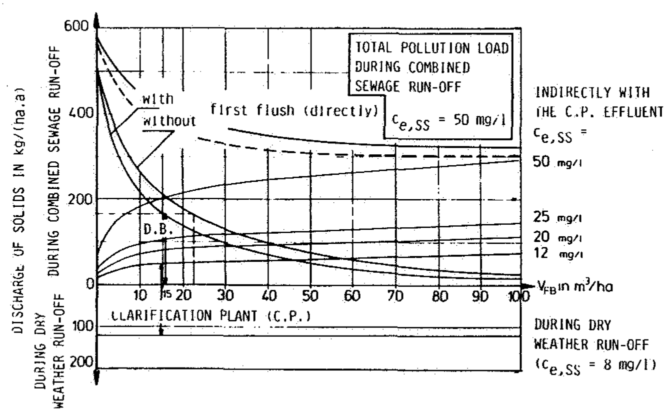

Figure 8 Yearly direct, indirect and total discharge of solids vs. basin capacity V_{FB}

7 Examination of the efficiency of a storage basin

During a 13-month period the quantities and the pollution loads in the combined sewage run-off and in the detention basin overflow were measured. The object of the investigations was a detention basin of the storage basin type with a specific capacity of 24 m^3/ha_{red}, located on-line. The effluent of the basin was throttled to a discharge rate of Q_{rab} = 24.5 l/s. The catchment area had a sealed surface area of 60.25 ha. The data do not comprise any extreme cases of combined sewage run-off. The basin was dimensioned according to ATV working paper A 128. Per year there were 43 overflow events; 37% of the combined sewage, 18% of the incoming BOD_5-load and 6% of the incoming settleable solids were discharged into the receiving water. Thus the results show that the efficiency of the detention basin was very high.

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EFFICIENCY OF STORMWATER OVERFLOW TANKS

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Abstract

The efficiency of two existing stormwater overflow tanks has been investigated. To obtain the necessary results, experimental investigations during six months have been performed. For the analysis of an annual load balance during an average year as well as the pollution load during extreme single events the simulation model SASUM (EAWAG, Munz) has been applied. The experimental data have been used for the calibration and verification of this model.
The most important factors affecting the pollution load from combined sewers overflow are the characteristics of the rainfall and the amount of pollutant on the ground surface and of deposits in sewers. Thus the transfer of generalized overflow data between different drainage areas is not reliable. An open question remains with regard to the effects of the short term pollution load from combined sewage overflow on receiving waters.

1 Introduction

In Switzerland combined sewers dominate over separate sewer systems. The hydraulic capacity of wastewater treatment plants (WWTP) is generally twice the daily peak dry weather flow. "Overflow tanks" are generally used to reduce the pollution load from combined sewage overflows (CSO). Depending on the local conditions these tanks are designed

a) to store the first flush and to return it after the end of the rain to the treatment plant,
b) to provide clarification (and some storage) before discharge into receiving waters; the sediment is treated at the WWTP after the rain event,
c) as a combination of 1) and 2).

Some hundred overflow tanks have been built in Switzerland since the sixties. However, experimental information on the efficiency of these tanks is not available.
The insufficient information about the efficiency of existing overflow tanks has lead to the systematic investigation of two combined system drainage areas with emphasis on the experimental investigation of two overflow tanks (Lake Thun area in Canton Berne).
The main purpose of this investigation is to obtain more information on the benefits gained from the use of overflow tanks in combined sewer system in relation to the high costs associated with construction of these tanks.
The project has been managed by the Federal Office for Protection of the Environment in Bern and the experimental investigations were conducted by the Consulting Engineers Prantl in Thun. The wastewater and its sediments were analyzed by the Cantonal Laboratory for Water Pollution Control in Bern, and the interpretation of the experimental data and the modeling has been performed by the Swiss Federal Institute for Water Resources and Water Pollution Control (EAWAG) in Dübendorf/Zürich.

2 Experimental investigations

The two investigated combined sewer areas Matten and Hilterfingen differ significantly. Matten (a suburb of Interlaken) is a flat area of 0.8 km^2 between Lake Thun and lake Brienz with residential, trade and business activities. Hilterfingen is a developing residential suburb of Thun (in 1982: 0.4 km^2) lying on steep slopes, above Lake Thun. Both of the investigated overflow tanks are operated parallel to the main sewage flow. The inflow into these tanks (= combined sewage overflow) occurs if the flow of sewage exceeds the capacity of the interceptor (Figure 1).

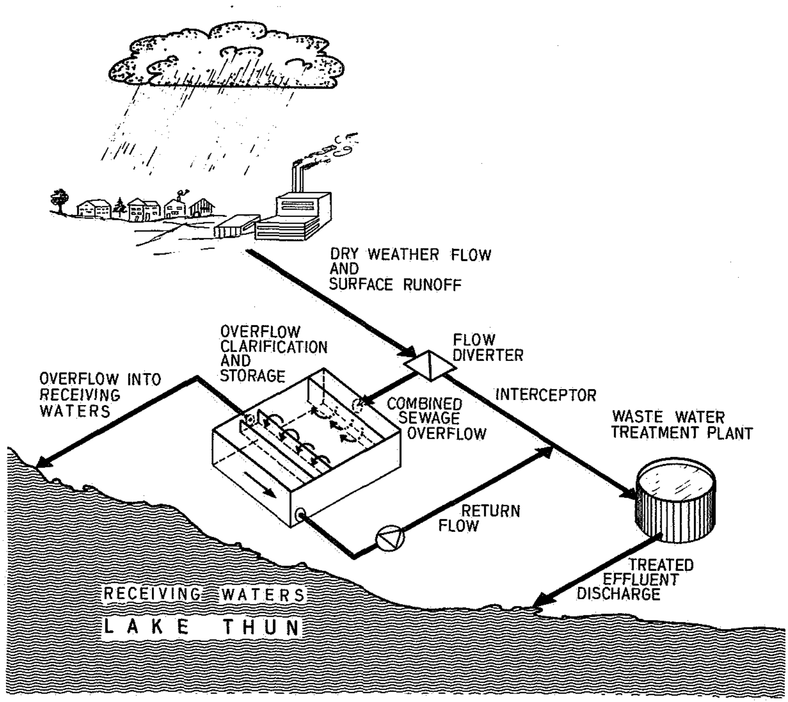

Figure 1 Flow scheme of overflow tanks in the study areas

Both of the tanks are designed primarily to provide clarification for CSO. The storage effect of these tanks (especially in Matten) is for many rainfall events only small. Further information about the study areas and their overflow tanks is shown in Table 1.

To obtain the necessary results, hydrologic, hydraulic and chemical investigations were conducted between May and October 1982. Figure 2 shows the sketch of both tanks with the sampling stations for chemical analysis of the wastewater and of the sediments. Table 2 summarizes the rainfall and the overflow events during the investigated period.

Table 1 Characteristics of the study areas

	Units	Study area Matten	Study area Hilterfingen
Catchment area			
- total, A_{tot}	ha	86	41
- impervious, A_{imp}	ha	32.6	10.5
Number of rainfall gages in catchment area	-	3	3
Max. full flow runoff time, t_F	Min	20	10
Number of inhabitants	-	4300	1500
Dry weather flow			
- peak, QTW	l/s	43	16
- annual averages, jQTW	l/s	17	6
Interceptor capacity			
- absolute, Q_{an}	l/s	90	40
- specific, $r_{an} = (Q_{an} - jQTW)/A_{imp}$	l/s.ha	2.2	3.3
Overflow tanks			
- absolute volume, I_t	m^3	250	330
- specific volume $i_t = I_t/A_{imp}$	m^3/ha	8	31
In-line storage in sewers			
- absolute volume, I_{sew}	m^3	210	-
- specific volume, $i_{sew} = I_t/A_{imp}$	m^3/ha	6	-
Total storage in drainage system			
- absolute volume, I_{tot}	m^3	460	330
- specific volume, $i_{tot} = i_t + i_{sew}$	m^3/ha	14	31

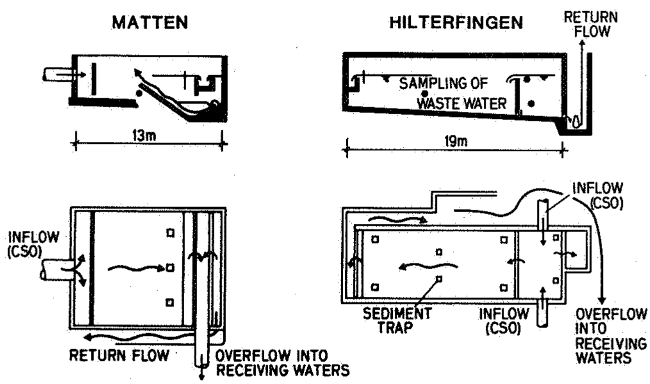

Figure 2 Sketch of overflow tanks in Matten and in Hilterfingen with the sampling stations for wastewater • and the sediments ▫

Table 2 Overview of the rainfall and overflow events in Matten and in Hilterfingen during the investigated period

Month	Rainfall (average of 3 gages)			Runoff	Overflow (recorded)			
	Number	Duration	Volume	(estimated volume)	Number of CSO (inflow into overflow tanks)	Overflow into receiving waters Number	Duration	Volume
	[-]	[mm]	[hrs]	[mm]	[-]	[-]	[mm]	[hrs]
Matten:								
May	11	70	73	56	4	2	15.5	9.2
June	19	158	129	126	11	8	46.4	33.5
July	13	175	88	140	10	9	62.7	30.8
August	16	206	120	165	12	10	84.2	53.2
September	7	58	24	46	6	4	15.4	11.4
October	13	96	85	77	8	8	23.2	29.7
May - October	79	763	519	480	51	41	247.4	167.8
Hilterfingen:								
May	9	81	50	40	1	-	-	-
June	21	150	100	80	13	4	17.5	4.1
July	16	183	72	58	13	6	15,7	4.7
August	14	152	112	90	12	3	5.1	0.6
September	6	58	25	20	4	2	2.3	1.6
October	10	68	66	53	6	-	-	-
May - October	76	692	425	341	49	15	38.6	11.0

It appears that from 70 - 80 investigated events, only about 20 % deliver suitable and reliable data for the planned evaluation due to the following reasons:

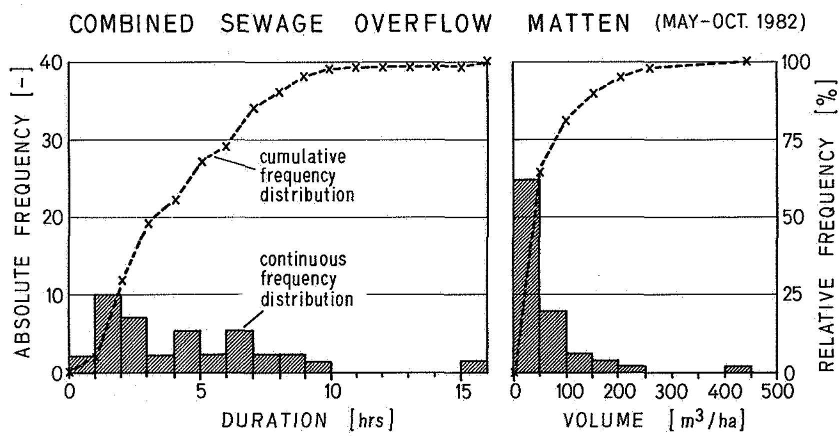

Figure 3 Frequency distribution of recorded overflow events in Matten between May and October 1982 (combined sewage overflow = inflow into overflow tank)

- the lack of combined sewage overflow (about 40 % of rainfalls were not intensive enough to cause an overflow),
- repetition of events with similar character of rainfall and overflow (see frequency distribution Figure 3),
- significant variation in character of the same rainfall within the study area (Figure 4), and
- errors in measurments and in sampling.

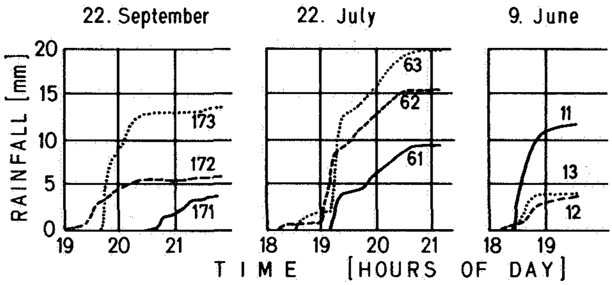

Figure 4 Examples for the variation of the rainfall in 3 rain gauges within the investigated area in Matten. The average distance between the single rain gauges was about 600 - 700 m

These experimental investigations are very expensive and involve a great deal of time. The estimated costs for the performance and documentation of these experiments are in the order of 100.000.- $ (without costs for chemical analysis, and excluding costs for the evaluation of results and for the modeling efforts).

3 Evaluation of results

3.1 The method

To study the influence of CSO on receiving waters both annual load and single event loads are important. Since it is too expensive to obtain sufficient results frome experimental investigations a well balanced ratio between experiments and the application of mathematical modeling is required.

Our approach was the following:

1) to perform a statistical analysis on rainfall series (see "catalogue of rainfalls" in Figure 5):

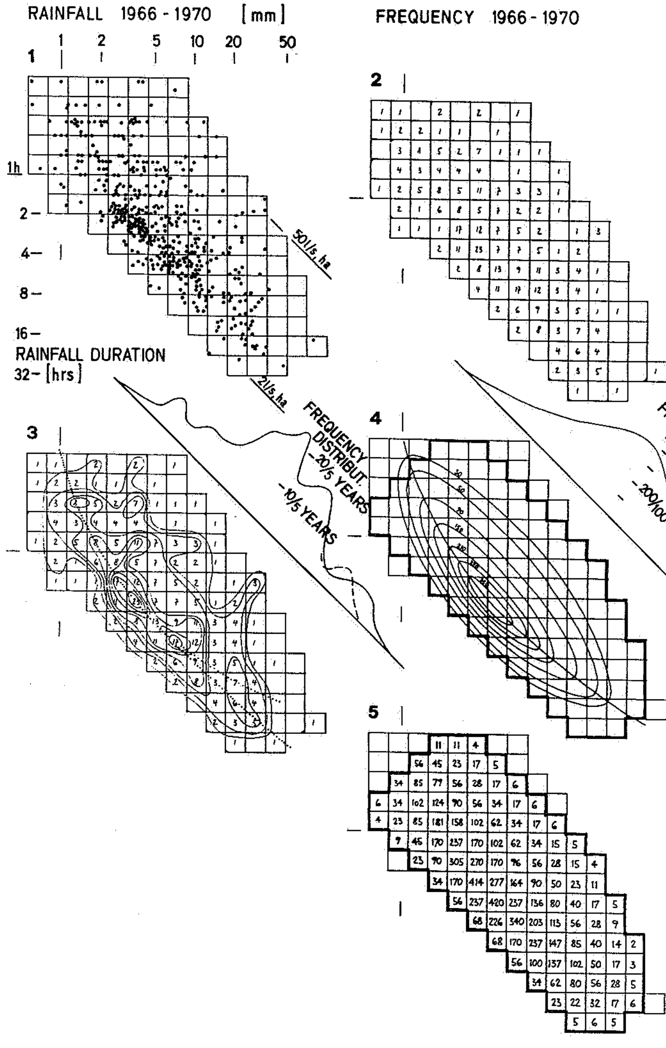

Figure 5 The catalogue of rainfall in Uster/Zürich. 1 + 2: frequency distribution of single events 1966 - 1970. 3+4+5: extrapolation to a 100 years period

- each event in the catalogue is characterized with a duration, volume, and its annual frequency,
- a hyetograph is generated as a function of duration and of average intensity,

2) to transfer a rain event to an overflow event by means of a simulation model (Munz (1984)). The catalogue of rainfalls makes it possible to easily simulate a group of events or the whole annual precipitation cycle,
3) to use the experimental investigations for the calibration and verification of the runoff overflow and treatment model,
4) to base final data analysis on the results of mathematical simulation.

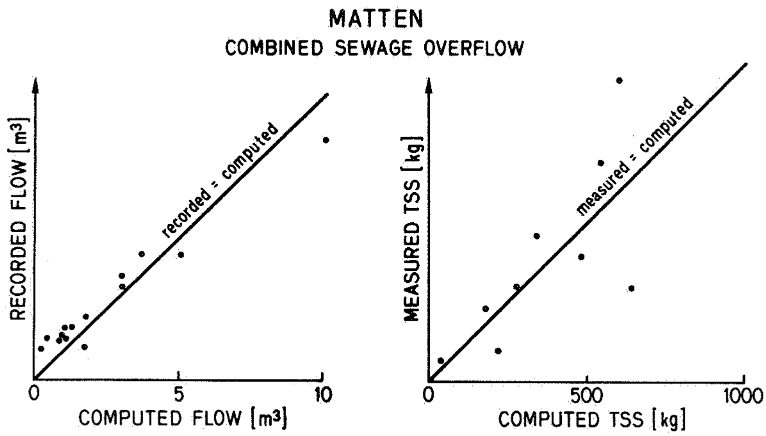

Figure 6 Agreement between observed and computed sewage overflow (= inflow into overflow tank) in Matten

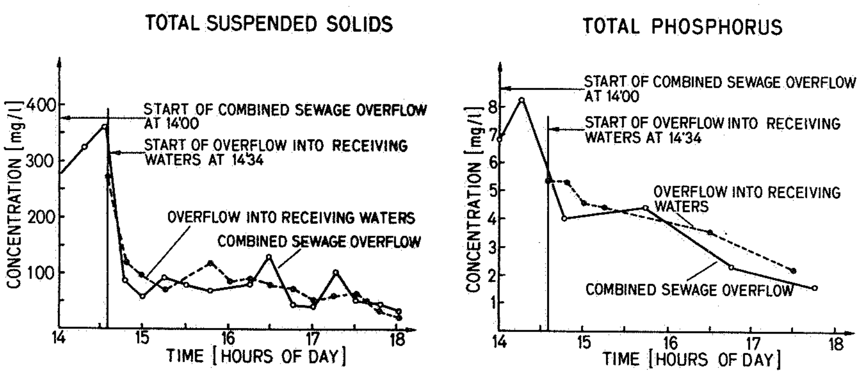

Figure 7 Change in concentration of total suspended solids and of phosphorus in the context of retention and treatment in the overflow tanks at Matten (CSO = inflow into tank, overflow into receiving waters = overflow from tank)

Thus, all results presented are based on mathematical predictions with the aid of calibrated and verified simulation program. Figure 6 shows the comparison between computed and observed combined sewage overflow (= inflow into overflow tanks). The behaviour of the overflow tanks can be simulated very easily: Figure 7 indicates that for the pollutants considered, overflow tanks retain the material contained in the first flush, whereas overflowing water seems to be barely affected by the tanks.

3.2 The results

Table 3 summarizes the information about wastewater and TSS loads from combined sewage overflow during an average annual period. The significant differences between Matten and Hilterfingen are primarily caused by the large specific volume of the overflow tank in Hilterfingen.

Table 3 Average annual balance of wastewater and TSS load in drainage area Matten and Hilterfingen during wet weather period (exclusive of WWTP outflow)

	Wastewater (m^3/ha_{imp})		Total suspended solids (kg/ha_{imp})	
	Matten	Hilterfingen	Matten	Hilterfingen
Total Flow (surface runoff + dry weather)	6350	6400	1330	1070
In-line storage in sewers	410	-	117	-
Flow to WWTP (interceptor capacity)	2890	4220	670	760
Combined sewage overflow	3040	2170	550	307
Return flow to WWTP (Effect of overflow tanks)	470	1220	240	270
Overflow into receiving waters	2560	930	307	37

Similarly, differences in frequency and duration of overflow into receiving waters are shown in table 4.

The efficiency of overflow tanks is frequently expressed as:

η = 1 - overflow into receiving waters/inflow.

However, this ratio is not satisfactory for the characterisation of this problem. The significance of overflow tanks as a water pollution control measure also varies with the capacity and the efficiency of the WWTP. A

Table 4 Average annual frequency and duration of overflow events in Matten and Hilterfingen.

	Matten	Hilterfingen
Frequency $[year^{-1}]$		
- CSO	66	69
- Overflow into receiv. waters	50	17
Duration [hrs/year]		
- CSO	153	117
- Overflow into receiv. waters	119	36

realistic analysis of efficiency of the overflow tanks has to include all pollution sources in the drainage area. Hence, balance of the load from all important sources during single events and during an average year give more information about the environmental impact of overflow from combined sewers and about the cost/benefit ratio of the overflow tanks.

For instance, in Matten the total annual overflow into receiving waters from urban area during dry and wet weather is 28 t TSS without tank.

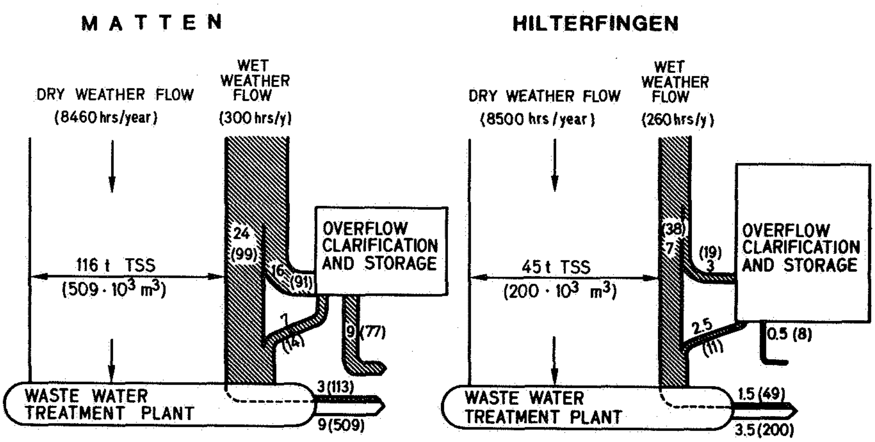

Figure 8 Total average annual balance (dry + wet weather period) of wastewater and TSS load in drainage area Matten and Hilterfingen (WWTP outflow is included)

This load is reduced by the existing tank to 21 t TSS/year. In Hilterfingen the load is reduced from 7.5 t TSS/year without tank to 5.5 t TSS/year with tank. The actual efficiency of overflow tanks in reducing the TSS annual load to receiving waters is in both cases about the same: ~ 25 % (Figure 8).

For the evaluation of water pollution control in rivers and small streams the knowledge of the load of single overflow events is equally important. This is illustrated in Figure 9 showing the estimated distribution of overflows into receiving waters in Matten and in Hilterfingen in relation to the rainfall distribution.

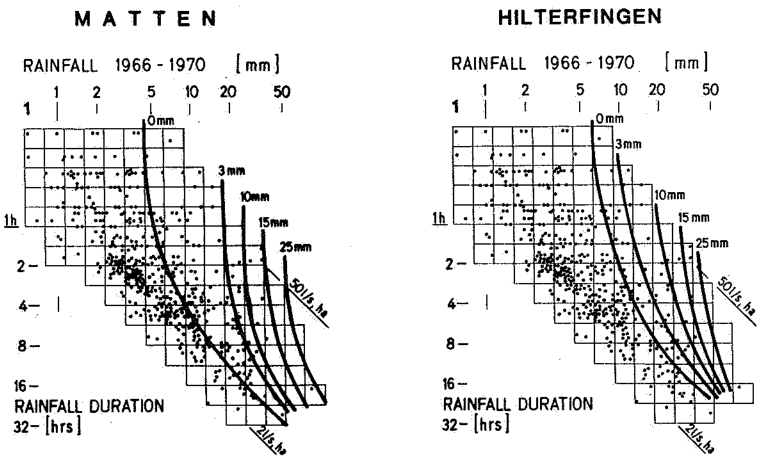

Figure 9 Estimated distribution of overflow events into receiving waters in Matten and Hilterfingen in relation to the rainfall distribution. Solid lines indicate the total amount of water discharged to the receiving waters

4 Discussion

4.1 Significance of results for other applications

Figure 10 shows the comparison between the results of this investigation and some earlier predicted values for the "Swiss average area" (residential area, full flow time = 15 minutes, no significnt deposits in sewers, no backwater effects, catalogue of rainfall in Uster/Zürich).

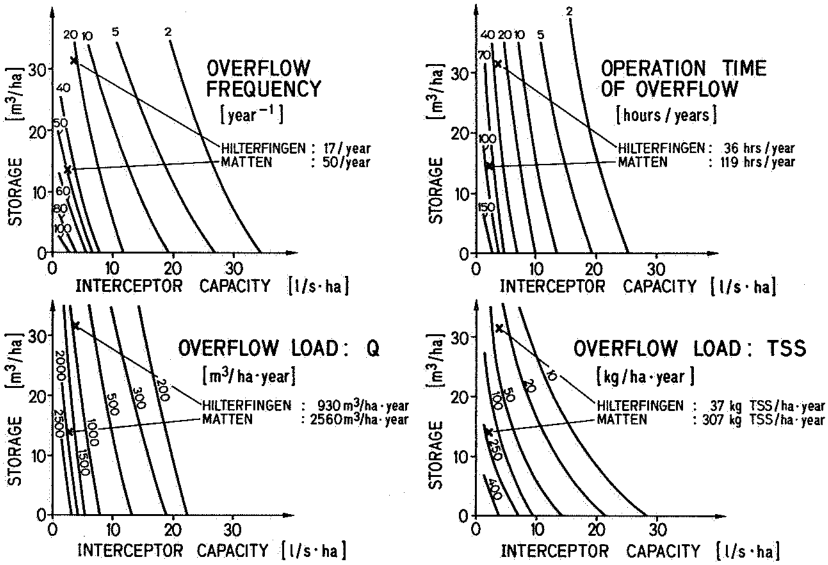

Figure 10 Comparison between the results of the investigation in Matten and in Hilterfingen and some earlier predictions for "Swiss average conditions"

The comparison gives a better agreement for Hilterfingen than for Matten, especially for the TSS load. The higher amount of pollutants on the ground surface in Matten and some deposits in sewers are the main reasons for the differences.
However, the comparison of the results obtained in Matten and Hilterfingen with some similar data from Denmark (Johansen et al. (1984)), Germany (Krauth (1973)) and Belgium (Berlamont and Smits (1984)) results in more important discrepancies. The main differences are related to entirely different characteristics of rainfall and therefore differences in overflow (volume, frequency and duration). Thus, uniform application of a combined sewage overflow treatment strategy based on a theoretical "typical Swiss situation" (Hörler (1978)), Munz (1979)) is not feasible. In view of the variation of the precipitation characteristics in different areas of Switzerland (Figure 11) reliable design information must be generated specifically for each region, even in a small (however mountainous) country such as Switzerland.

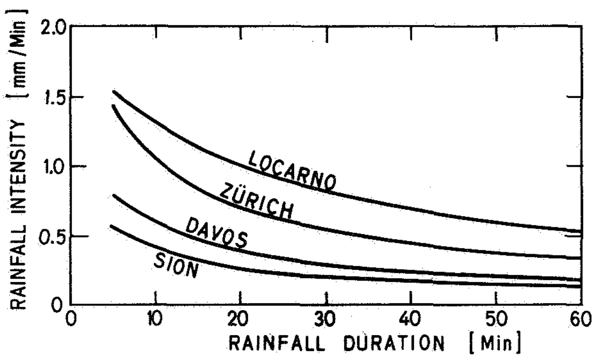

Figure 11 Rainfall intensity - duration curves of 1 year frequency for some station in Switzerland (Hörler and Rhein (1962))

4.2 The significance of overflow tanks for water pollution control in Switzerland

Standards for discharge into receiving waters and receiving waters quality regulations in Switzerland are not explicitly stated for wet weather situations. The goals of water pollution control during rain events are therefore ill-defined. In many countries dissolved oxygen demand and coliform organisms serve for justification of investment in stormwater treatment - both aspects are of limited significance in Switzerland.

Ecological problems related to CSO's are barely characterized for rivers, whereas for lakes CSO's are usually considered in yearly budgets for nutrients (predominantly phosphorus). Today, stormwater overflow tanks are primarily viewed as a solution for local aesthetic problems caused by sediments and floating coarse materials.

From the comparison of earlier model predictions and the results of this study (Figure 9) it appears that the efficiency of the utilisation of overflow tanks is limited by the definition of the goals of their application, rather than by our understanding of their operation. Thus, we are in urgent need of ecological criteria for the design of combined sewage overflow structures, whereas further refinement of mathematical models appears to be rewarding.

4.3 Cost/benefit ratio of overflow tanks in a wastewater network

Stormwater treatment is in operation only a small percentage of the year whereas WWTP's are operated continuously. The following direct comparison of specific costs per pollutant removed is therefore valid only if yearly pollutant budgets are considered.
The specific annual retention of TSS is 27 and 7 kg TSS/m^3 of tank volume per year in Matten and Hilterfingen, respectively. This retention is further reduced by approximately 10 - 20 % if the TSS in the WWTP effluent are included in the balance. With estimated yearly costs of SFr. 50.- per m^3 of tank volume for investment and operation and additional costs of SFr. 1.- per kg TSS for treatment in WWTP's the resulting costs for the net removal of 1 kg TSS with the aid of overflow tanks are in the order of SFr. 3.20 - 9.40 (US $ 1.50 - 4.50) for Matten and Hilterfingen, respectively. This simple analysis puts overflow tanks in the cost region of advanced wastewater treatment and could easily be matched by secondary effluent filtration. A similar analysis for phosphorus (where yearly budgets are most relevant) indicates specific costs of SFr. 200.- - 600.- per kg P removed, which is far beyond the cost of the most advanced technologies.

5 Conclusions

Efficient utilization of stormwater treatment must be based on two criteria:

1. The goal of the treatment must be defined from the perspective of the environment.
2. Information and models must be available to evaluate different strategies in view of the goals.

This study indicates that our understanding of the behaviour of sewer systems during storm events is far better than our grasp of the ecological significance of stormwater overflow.
In view of the costs of stormwater treatment, research on the ecological aspects of CSO's is urgently needed and possibly more rewarding than research into technical aspects of stormwater treatment. In this field natural sciences lag behind engineering.

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EFFECTS OF DETENTION BASINS (DISC) ON THE COMPOSITION OF STORM WATER AND COMBINED SEWER OVERFLOW

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Abstract

It has been recognized that combined sewer overflow is also responsible for receiving water pollution. In general storm water control regulations and ordinances for combined sewer systems require detention facilities for the storage of peak flows. Their realisation as designed inline storage capacity (DISC) takes into account both constructional and economical aspects. An increase of inline storage capacity is easily obtained by choosing large pipe diameters. Thus a reduction of the drag force will result in a larger amount of deposits during dry weather flow. The performance of DISC facilities with respect to quality of storm runoff and discharges is obviously affected by these accumulations.

Investigations are ongoing in the semi-urban catchment of Ense - Bremen (near Dortmund). Within this combined sewer system six DISC facilities are realized. For rain events the composition of in-, out- and overflow of two DISC tanks has been determined. The results of these investigations will be discussed in this paper.

1 Introduction

For water quality management attention has to be focussed not only on waste water but also with emphasis on storm water. The joint responsibility of combined sewer overflow (CSO) for receiving water pollution is commonly respected. Within the Federal Republic of Germany the call for the establishing of detention facilities in sewer systems for the storage of peak flows is pursued. State of the art are the regulations and ordinances of the Abwassertechnische Vereinigung A 128 (ATV, 1977). Concerning these directives the storm water tanks have to be realized as an intercepting or a run through system. The necessary storage volume can be achieved by

- seperate tanks
- designed inline storage capacity (DISC) or
- using the volume of the existing sewer system.

In principle the application of one of these facilities is not restricted. The selection between these given alternatives depends on the local circumstances.

2 Application of DISC facilities

Constructional and economical aspects promote the realization of DISC tanks. Such facilities can be designed by the overflow at the beginning (type I) or at the end (type II) of the basin. These implementation feasibilities are taken into account by the design criteria A 128 (ATV, 1977) as shown in Figure 1. The larger volume of DISC tank type II is explained by the fact that the total storm water runs through the tank.
The creation of storage capacity is easily obtained by choosing large pipe diameters or narrow rectangular sections. Thus, the existing trace of the sewer can be used and no additional area is necessary. However, the application

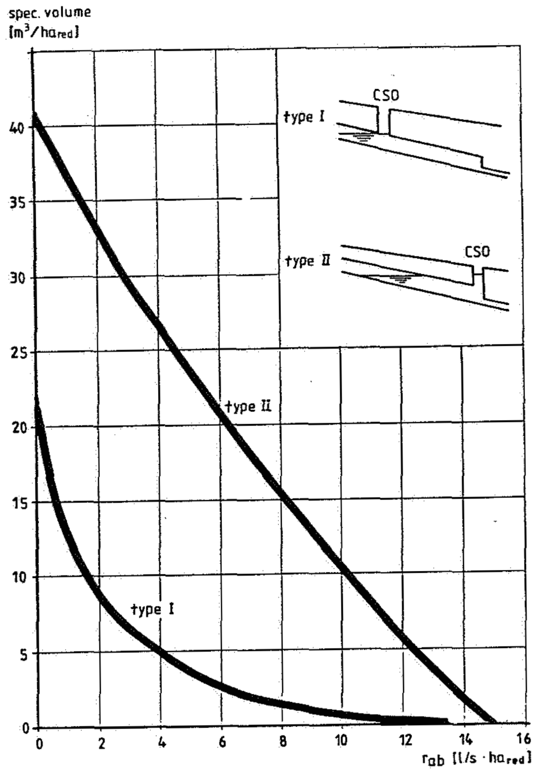

Figure 1 Specific volume for DISC tank type I and II a critical rainfall of 15 l/s•ha provided

of an enlarged cross-section will result in a reduction of velocity and drag force during dry weather conditions.
To meet the problem of a reduced drag force different cross sections of DISC tanks are used to prevent an accumulation of deposits (Figure 2). Other types like egg shaped sections are conceivable. (The lay-out of any cross-section type is subjected to no limitations). The installation of a trough for dry weather flow is considered to be necessary for the rectangular types and recommended for the conduit ones.

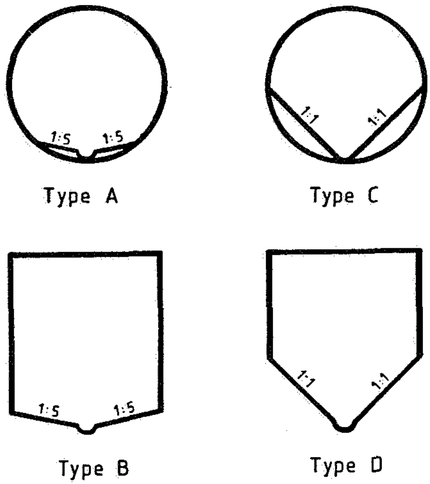

Figure 2 Cross-section types of DISC tanks

Statistics on storm water tanks carried out in the early eighties in the federal state Baden Württemberg reveals an interest of about 30 percent DISC tanks (Brombach 1979, MELUF 1980). Meanwhile the share of DISC facilities is still increasing. This is certainly due to cost savings. Comparing relative costs of covered seperate tanks with DISC facilities the economical advantage of the latter one is evident. Based on cost inquiries by Kaufhold (1981) relative cost dependencies are derived supplemented by cost determinations of the authors (Figure 3). The given costs represent the average value of a certain mean variation.

3 Investigations on DISC facilities

Information about the pollution reduction efficiency of DISC facilities in fact has not been published so far. Results of quantity measurements and analysis of combined sewer concentrations are hitherto available only for seperate tanks (Krauth 1984, RP Tübingen 1980, 1982). The effects

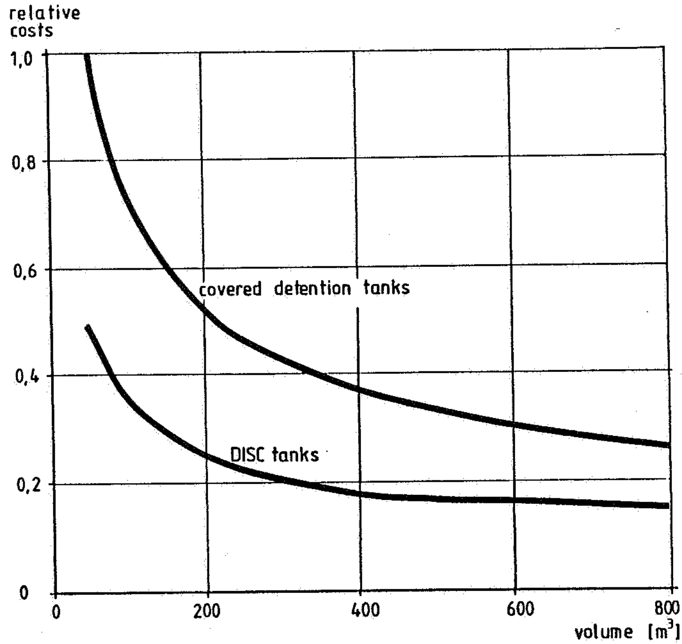

Figure 3 Relative costs versus storage volume for seperate and DISC tanks

of the DISC tanks on the composition of storm water and combined sewer overflow are being investigated in the semi-urban catchment of Ense-Bremen (near Dortmund). Within this combined sewer system six DISC facilities are realized. Cross-section Type C and D in Figure 2 and egg shaped section as well give a wide range of alternatives in observing the DISC tanks efficiency. Quantity of runoff and of each basin is being continuously monitored at a centralized computer based station.
For certain rain events the composition of in-, out- and overflow of the DISC tanks is determinated. Samples are taken by automatical sample devices. All sample devices are being controlled by external signals. The two sample devices at the inlet and the outlet of the DISC tank starts

running after exceeding a certain rain intensity checked by a conductivity gauge. The operation of the other one sampling the CSO depends on the DISC tank water level. The sampling intervalls vary from 6 minutes at the beginning of the rain event up to 2 hours during the draining of the DISC tank.
The measurement results will be exemplarily discussed at two existing DISC tanks of cross-section Type C (Figure 2). Table 1 contains the significant data of the two DISC tanks and their subcatchments. The great slope difference of 0.5 % resp. 4.0 % should be noticed.

Table 1 Characteristic data of two presented DISC facilities

		Tank 1	Tank 2
Population	E	2,110	1,360
Q_t	l/s	5	4
Q_{rab}	l/s	12.5	27.5
r_{ab}	l/s·ha	0.42	0.95
V	m^3	654	855
V/A_{red}	m^3/ha	21.5	29.5
Type of the tank	—	DISC,Type I,C	DISC,Type I,C
Slope	%	0.55	4.0
Draining time	h	14.3	8.6

4 Results

For statements of general validity a sufficient extent of investigations doesn't exist up to now. Therefore a single significant event caused by a 9.6 mm rain over about 4 hours will be presented to discuss exemplarily the effects of DISC tanks on the composition of storm water and CSO.
Maximum terms of storm water conditions - defined as time elapsed between beginning of storage and total draining of

the tank - were approximately 22 hours for DISC tank 1 and 14 hours for DISC tank 2 (Figure 4 and 5). The time variation depends on different design criteria especially on the various DISC tank outflow. CSO occured at DISC tank 1 for 111 minutes discharging 429 m^3 CSO resp. a COD load of 99 kg and at DISC tank 2 for 26 minutes discharging 352 m^3 CSO resp. a COD load of 4 kg (Figure 6). Load balances of this certain event are presented in Figure 7 and 8 as cumulative load curves versus cummulative runoff and in Figure 9 and 10 as cumulative load curves versus time.

The results of both investigated DISC tanks vary in a wide range. The hydograph of the cumulative COD inflow load reveals a distinct difference. Whereas at DISC tank 2 a faint first flush effect occurs (Figure 7) none can be ascertained at DISC tank 1 (Figure 8). 10 percent of cumulative runoff are related to a cumulative load of 30 percent at DISC tank 2 resp. 10 at DISC tank 1. Another characteristic is the percentage of the COD inflow load at the start of CSO. A share of 68 percent at DISC tank 2 stands against a share of only 30 percent at DISC tank 1. These facts are evidently affected by the difference in shape and the kind of their catchments.

The characteristic of the COD outflow load is similar in both DISC tanks. The initial pollution is straight passed on to the following interceptors. After attaining 1/3 of the DISC tanks storage capacity a sedimentation effect occurs. Comparing the COD outflow loads with the COD inflow loads it is conspicious, that inside DISC tank 1 25 percent of the inflow pollution is retained as deposits. On the contrary an approximate load balance takes place in DISC tank 2. An explanation for this effect might be given by the different constructional and operational features. DISC tank 1 is executed in a slope of 0.55 percent DISC tank 2 in one of 4.0 percent. As a distinct operational aspect the draining time of the DISC tanks has to be considered. The comparatively long lasting drainage of 18.75 hours for DISC tank 2 will result in a high level of sedimentation

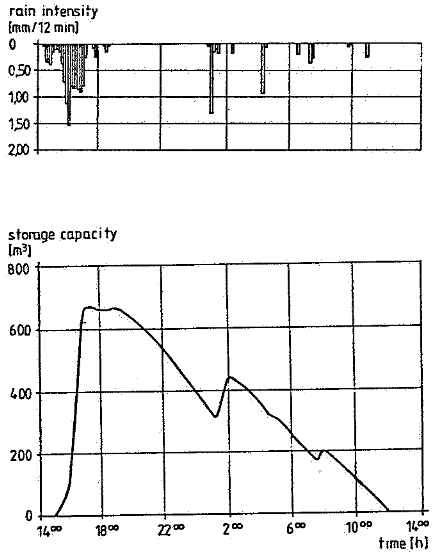

Fig.4 Storage hydograph and rainfall intensity of DISC tank 1

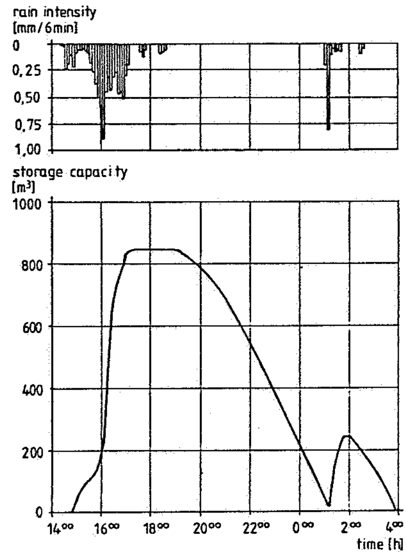

Fig.5 Storage hydograph and rainfall intensity of DISC tank 2

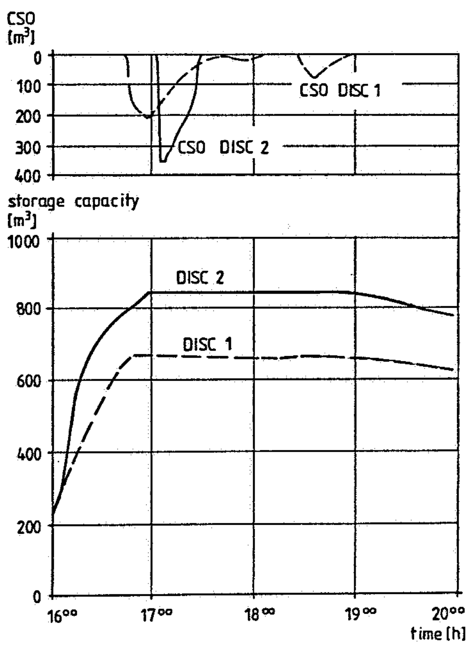

Fig.6 Course of CSO at both DISC tanks

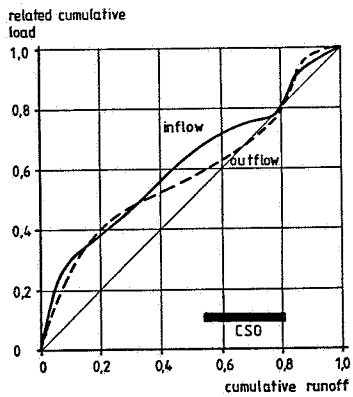

Fig.7 Cumulative COD load versus cumulative run-off for DISC tank 2

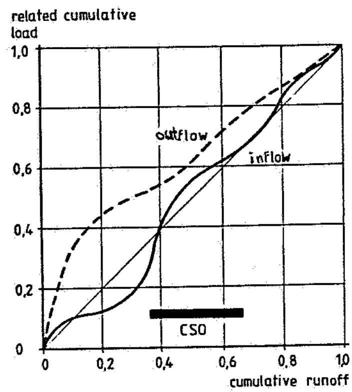

Fig.8 Cumulative COD load versus cumulative run-off for DISC tank 1

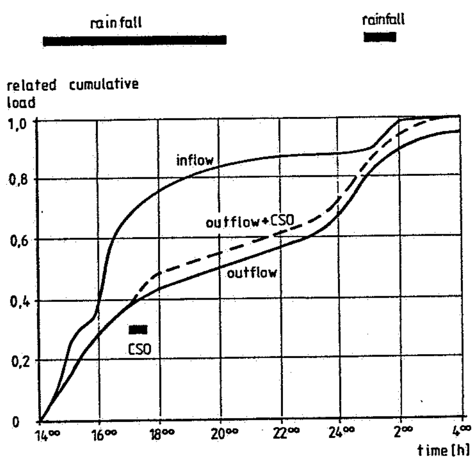

Fig.9 Cumulative COD load versus time for DISC tank 2

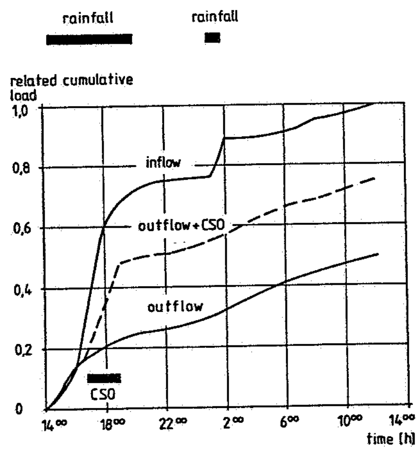

Fig.10 Cumulative COD load versus time for DISC tank 1

and a low self - cleaning efficiency. To give a survey on the efficiency of different detention facilities one can only refer to few investigations concerning this subject (Krauth, 1984, RP Tübingen 1982). Therefore a comparision of the results presented in this paper and these ones of the abovementioned investigations is limited. It should be considered that the other investigations pursue various aims. The different constructional and operational features have to be regarded as well. Significant data of the investigated detention facilities and their catchments are given in Table 2. A relation between inflow and overflow concerning a single event is cited in Table 3.

Table 2 Significant data of detention facilities and their catchments

Place of investigation	Previous CSD	A_{red}	Q_{rab}	r_{ab}	V	V/A_{red}	Tank type	Draining time
	-	ha	l/s	l/s.ha	m^3	m^3/ha	-	h
Stuttgart[1]	no	60.2	24.5	0.41	1452	24.1	circular	16.5
Rüb. tank 1[2]	yes	15.6	10.5	0.67	400	24.6	rectangular	0.9
Rüb. tank 2[2]	yes	33.5	37.8	1.91	120	3.6	circular	3.0
Ense tank 1	no	30.0	12.5	0.42	645	21.5	DISC	14.3
Ense tank 2	no	29.0	27.5	0.95	855	29.5	DISC	8.6

[1]Adapted from Krauth (1984) [2]Adapted from RP Tübingen (1982)

Table 3 Inflow - overflow relationship at investigated detention facilities

Place of investigation	Inflow	Overflow	Overflow / Inflow	Load of inflow	Load of overflow	Overflow load / Inflow load
	m^3/ha	m^3/ha	%	kg COD/ha	kg COD/ha	%
Stuttgart[1]	80.6	59.2	73.4	17.0	5.23	30.8
Rüb. tank 1[2]	385.8	96.7	25.1	50.67	11.66	23.0
Rüb. tand 2[2]	42.0	16.3	38.9	6.17	2.10	34.0
Ense tank 1	71.9	14.3	19.9	13.48	3.30	24.5
Ense tank 2	72.3	12.1	16.8	3.23	0.14	4.2

[1]Adapted from Krauth (1984) [2]Adapted from RP Tübingen (1982)

Except of DISC tank 2 the other investigated detention facilities reveal a relative high discharged share from 23 to 34 percent of the total load. A correlation between the ratio of CSO and the inflow isn't evident. However, the influence of the individual constructional and operational parameters doesn't permit any detailed comparison.

5 Conclusions

The impact of combined sewer overflow on receiving waters is uncontradicted. Therefore one objective of water management has to be the detaining of storm runoff as far as feasible. In general there's no doubt about the benefit of detention facilities but details on their efficiency are missing. Existing results don't permit any conclusions of general validity. However, it is ascertained that DISC tanks even with a great storage capacity are equal to the other detention facilities.

Acknowledgement

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Appendix - Notation

The following symbols are used in this paper:

A_{red}	=	drain effective share of the catchment
COD	=	chemical oxygen demand
CSO	=	combined sewer overflow
Q_{rab}	=	storm water share of the tank outflow
Q_t	=	dry weather flow
r_{ab}	=	ratio between Q_{rab} and A_{red}
V	=	storage capacity
V_{spec}	=	necessary storage capacity related to the drain effective share of the catchment
ha	=	10^4 m^2

CONTROL OF WASTE WATER DISCHARGES WITH FIXED CONTROL ALGORITHMS, AND THE EFFECT ON RECEIVING WATERS.

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Abstract

The water quality of a river is unsteady even during periods with steady flow rates. If good water quality conditions downstream of a waste water discharge are wanted, its flow-rate including the time of the waste water discharges have to be controlled.
Water quality forecasting for the receiving water are useful to assess the moments, that waste water discharges may occur without threatening the water quality of the receiving water. In this paper the theoretic effect of storm water discharges on a river is shown. The effects of storm water discharges on the river Leine have been recorded and the effects of fixed control algorithms in the river are computed by a water quality simulation model.

1 Introduction

After long periods of dry weather large deposits of polluted matter have accumulated on impervious surfaces.
Even during short storms a considerable amount of pollutants can be discharged from storm sewers of separate systems, if the rainfall intensity is high enough, to wash off the polluted matter (street dust and dirt/polluted deposits).

The effect of stormwater discharges on the quality of the receiving water can be reduced by controlling the discharges. In the United States examples exist of such controlled sewer systems, where the peak run off is retained in the sewer system. Control actions of waste water discharges have to be based on the actual water quality of both the receiving water and the stormwater. These can be recorded by on-line measuring instruments. On-line instruments for COD, BOD, TOC and other water quality parameters are available on the instrument market.
In this paper control algorithms are used to control sewers of separate sewer systems. By a computation example based on measurements in the storm sewer and in the receiving water, it is shown that the water quality of the receiving water can be improved by storing the storm water for some time (for example in a retention basin).

2 Effective control of waste water discharges by water quality forecastings

During periods of poor water quality conditions in the receiving water, highly polluted discharges have to be avoided.

Stormwater discharges can be controlled by:

storing the waste water in retention basins or the sewer system.

With only one water quality measurement station in the receiving water at the sewer outfall into the receiving water it is only possible to react on the actual water quality conditions. Future changes in the water quality are not considered and, as a result, the available storage capacity is not used.

Suppose the pollutant concentration in a receiving water is decreasing, it is not known whether the better water quality conditions are of short or long duration. In the first case it would be sensible to **retain** all the waste water over this short time. In the second case the retention basin could be emptied slowly. The basin has to be empty again at the moment that the next storm has to be stored.

With a second water quality measurement station upstream of the sewer outfall and water quality forecastings, information on how the water quality will develop over the next hours is available and a more optimal and sensible control of waste water discharges is possible.

The water quality forecasting can moreover be used to change the control algorithms. It is useful to choose a control algorithm with high sewer discharge in case a waste water wave is forecasted.
In this paper the forecasts are only made for the receiving water. It is presupposed that the storm water is highly polluted and that it is necessary to control it.

3 Control algorithms

A simple way to control waste water discharges is by using control algorithms. In assessing control algorithms a number of sometimes conflicting aims have to be considered.
For example, the waste load in the river should be as low as possible, therefore the waste water is retained as much as possible. On the other hand the retention capacity is limited.
In the subsequent algorithms a compromise between these aims is searched.

The waste water discharge is low, if the concentration of the pollution (expressed as BOD) in the receiving water is

high, but if the retention capacity is exhausted the out-flow nevertheless increase.
In table one and two control algorithms are shown. If the water quality conditions in the receiving water are high the waste pipe is open (100 %), but if the water quality is decreasing the opening rate is reduced. Under periods with poor water quality conditions the outlet opening of the retention basin is closed (0 %). If the retention basin is filled, the outlet opening of retention basin is closed and the inflow rate is greater than zero, the inflow goes through a separate spillway.

BOD concentration in the receiving water \ degree of filling of the retention capacity		25 %	50 %	75 %	100 %
BOD	2,7 mg/l	100	100	100	100
BOD	3,5 mg/l	30	35	40	50
BOD	5,0 mg/l	10	15	20	25
BOD	5,0 mg/l	0	0	0	0

Outlet opening of the retention basin (%)

Table 1 Algorithm 1 for the control of waste water discharges

In the control algorithm 2 the water out-flow is higher for the same pollutant concentrations in the receiving water.

BOD$_5$ concentration in the receiving water \ degree of filling of the retention capacity		25 %	50 %	75 %	100 %
BOD	2,7 mg/l	100	100	100	100
BOD	3,5 mg/l	65	70	80	90
BOD	5,0 mg/l	45	50	55	60
BOD	5,0 mg/l	25	30	35	40

Outlet opening of the retention basin (%)

Table 2 Algorithm 2 for the control of waste water discharges

Control algorith 2 is chosen if waste water waves in the receiving water are forecasted. It is wanted to have so much empty volume in the retention basin as possible, when the waste water waves in the receiving water are passing the outlet.

In the model the differences between the forecasted water quality parameters are sumed up. If the sum is greater than zero a waste wave is to expect.

In the model algorithm two is chosen, in dependence of a limit of the sum of the sumed up water quality parameters. It is possible to enlarge the model to three or even more control algorithms. In the model a new outlet opening of the retention basin is computed when new information about the water quality of the receiving water is available.

4 Computational example for controlled sewer overflow water discharges and their influence on the water quality of the receiving water

The computational example shall show how the influence of

storm water discharges on a receiving water can be soothed by using control algorithms. In this simple example no waste wave is forecasted, so only algorithm 1 is used.
In the city Hildesheim the impervious surface drained by separate sewer systems encloses 450 ha. No real control devices or retention basins are installed. In the model we have five fictitious retention basins with the sumed up volume of 3100 m³.
The duration of rainfall was one hour. Measurements were taken simultaneously in the storm drainage system and in the receiving water, the river at Hildesheim (Germany). In the storm drainage system NH_4 = 1 mg/l, BOD_5 = 15 mg/l and COD = 80 mg/l were measured. In the model these are the water quality parameters of the water which is stored in the retention basins.
These figures are higher than the means measured in the storm drainage in Hildesheim but they are not extremly high. Measurements in the river show that the water quality is worsened by the discharge of polluted matter washed off from impervious surfaces (see table 3).

Measurement station	time	BOD_5 (mg/l)	COD (mg/l)	NH_4 (mg/l)	pH	conductivity (µS)
1	9^{15}	2,7	24	0,6	7,5	6130
2	10^{15}	3,2	28	0,5	7,55	6230
3	13^{00}	11,2	40	0,6	7,4	6050

Table 3 Measurements in the river

Measurement station 1 is located 20 m upstream of the first sewer outfall, measurement station two 300 m downstream of that point and the third station is 400 m downstream of the last sewer outfall. The time difference between the measurements equals the flow time between the measurements stations, so the measurements are taken from the same water

plug.
Between the measurement stations no more outfall sites occur so there are only small differences between computed water quality parameters "with storm discharge" and observations in the case "with storm discharge" in the receiving water (see figure 1).

The influence of a controlled stormwater discharge on the water quality of the receiving water can be shown only theoretically.

The effect of the discharged pollutants in the river is computed by an enlarged Streeter-Phelps-model with equations for BOD_5, N, COD and O_2.

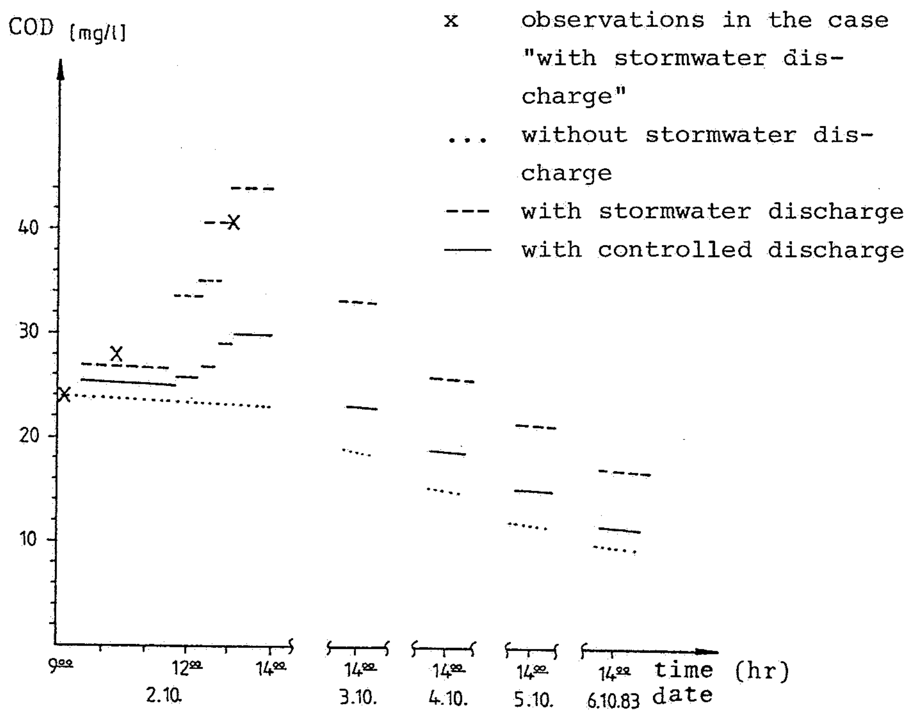

Figure 1 Computed and measured COD-concentrations in a water plug moving with the stream

The concentration jumps on the 2nd. October are the result of stormwater discharges from the 5 catchment areas. The curve "without stormwater discharges" shows the slowly reducing concentration of pollutants in the case no stormwater discharges occured. The difference between the curves "without stormwater discharges" and "with stormwater discharges" is caused by the stormwater discharges.
The effect of BOD_5, NH_4 and COD oxidation on the oxygen concentration is shown in figure 2:

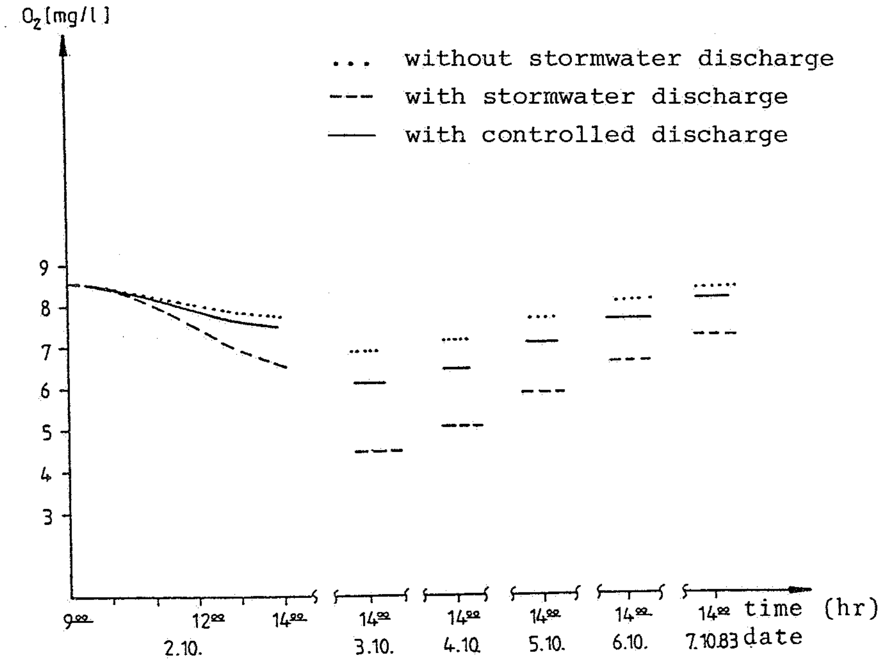

Figure 2 Computed oxygen level in water body moving with the stream

Approximately one day after the stormwater discharge the minimum of the oxygen level is computed.

When control algorithm 1 is applied the stormwater discharge is throttled, therefore the COD concentration is

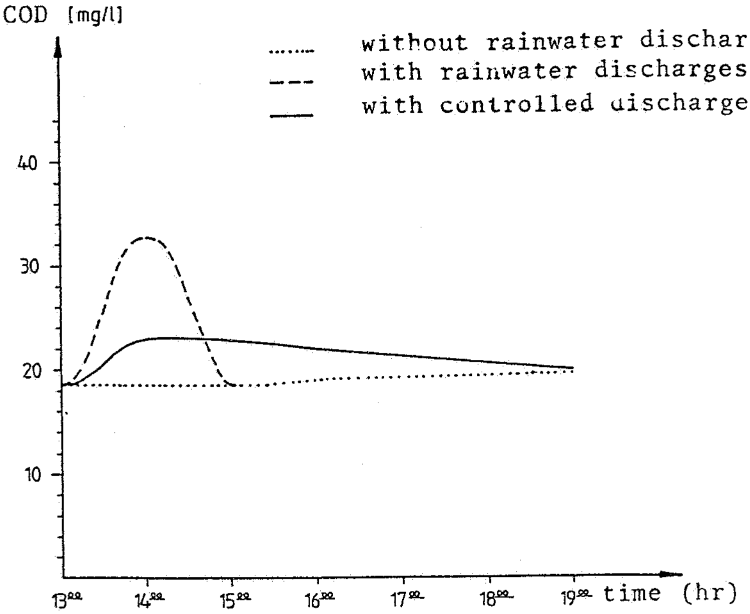

Figure 3 Computed COD-concentrations at a cross section located 24 h flow time behind the last storm sewer outfall

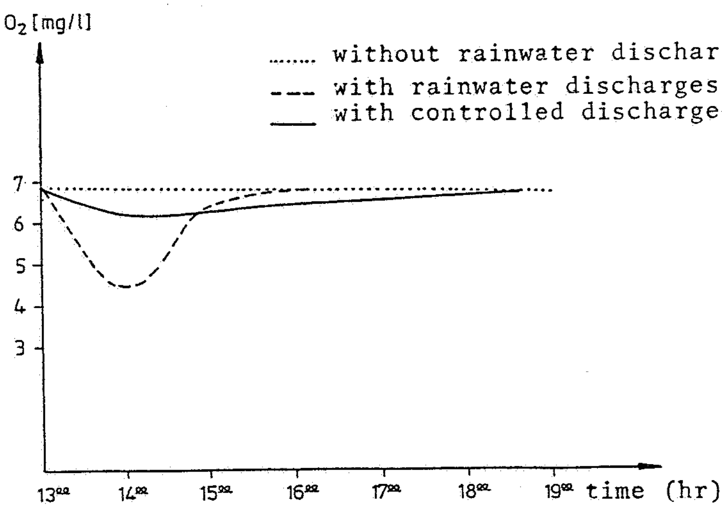

Figure 4 Computed O_2-concentration at a cross section located 24 h flow time after the last storm sewer outfall

lower (fig. 1) and the oxygen concentration higher (fig. 2).

The overall result of the control of stormwater discharges can not be shown by looking at only one plug of water. Therefore let us consider the curve of the COD and O_2 concentrations at a cross section downstream of the last storm sewer outfall (see fig. 3 and 4).

At this cross section the COD-concentration is rising from one hour; in the meantime the oxygen concentration is decreasing due to BOD_5, COD and NH_4 oxidation.

With controlled stormwater discharges vehement concentration changes do not occur, but the effect of the discharge is distributed over a longer period of time.

Conclusion

Storm water discharges from impervious surfaces are threatening the water quality of the receiving water. They cause waste waves in the receiving water. With controlled stormwater discharges vehement concentration changes do not occur.

The effect of the discharges is distributed over a longer time. Periods with low oxygen levels in the receiving water can be avoided.

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THE EFFECTS OF COMBINED SEWER OVERFLOWS UPON RECEIVING WATERS

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Abstract

A fieldwork programme of research at the University of Manchester, funded by the Science and Engineering Research Council, Water Research Centre and North West Water Authority as part of the Water Industry River Basin Management Programme, is currently being undertaken to examine the effect of the discharge from four individual combined sewer overflows on their respective receiving watercourses. The field sites are located at Bolton and Hyndburn and have been operational since April 1985. This paper presents results from two storm overflow spillage events at the Great Harwood CSO. A tentative analysis of combined sewer overflow impacts on receiving watercourses relates the degree of effect to storm profile, antecedent dry weather period, prevailing river characteristics and chamber hydraulic performance.

1 Introduction

In recent years the problem of watercourse pollution in the U.K. has become a cause for concern. The situation was reviewed by the Scottish Development Department (1977) and it was recognised that many storm sewage overflows had deleterious effects on receiving watercourse quality. Elsewhere the Water Research Centre/Water Authorities Association Sewerage Rehabilitation Manual (1984) identified the pollution of watercourses by storm sewage overflows as a significant problem.

Aspinwall and Ellis (1986) have described aspects of the impact of combined sewer overflows (CSO) and urban run-off on river quality in terms of the pollutant load of such spillages. Severe impairment of long term river quality objectives as well as more immediate shock effects were highlighted.

Table 1 outlines a brief summary of the National Water Council (1980) river classification system in terms of the chemical characteristics and river uses which is at present used to assess the quality of British watercourses. At present the mechamisms and processes by which storm sewage overflows affect watercourses are imperfectly understood. It has been recognised within the U.K. Water Industry that if future capital investment in sewerage upgrading works is to be efficient and effective, this understanding should be extended.

Table 1 Summary of National Water Council River Classification

Class	Chemical characteristics	Potential Uses
1a	Mean BOD ⩽ 1.5 mg/l Mean ammonia ⩽ 0.4 mg/l DO ⩾ 80% saturation	Supports Game Fishery Suitable for potable supply with conventional treatment
1b	Mean BOD ⩽ 2 mg/l Mean ammonia ⩽ 0.5 mg/l DO ⩾ 60% saturation	High amenity value
2	Mean BOD ⩽ 5 mg/l DO ⩾ 40% saturation Non-toxic to fish	Supports Coarse Fishery Suitable for potable supply with advanced treatment Moderate amenity value
3	BOD not greater than 17 mg/l DO ⩾ 10% saturation	Suitable for low grade industrial abstraction Limited amenity value
4	DO ⩽ 10% saturation Likely to be anaerobic at times	Grossly polluted river Negative amenity value

In the North West Water Authority (NWWA) area the subject is a particular matter for concern. Some 28% of the 3500 km of poor or bad quality (NWC Class 3 or below) inland waters in England and Wales lie in the NWWA region. NWWA published a discussion document (1983) which outlined the scale of the problem and the cost of upgrading watercourses in the region to an acceptable (Class 2) standard. A figure of £3.7 billion over a period of 25 years was estimated. The document identified storm sewage overflow spillage as a major contributory factor to the unsatisfactory state of many of these watercourses.

The UK Water Industry Research Programme on River Basin Management described by Clifforde et al (1986) has been instigated in response to the problem. The aim of the programme is to provide the necessary tools and methodology to allow the rational and objective upgrading of deficient sewer systems. The main products of the programme are to be:

a) increased knowledge of the rainfall inputs to sewer flow simulation models (time series rainfall);
b) to develop a sewerage sub-model to simulate quality for use with the existing WASSP-SIM procedure (1981);
c) a simple CSO river impact model;
d) to improve the existing U.K. river classification system to take into account transient and short-term changes in river quality due to storm events.

A fieldwork programme of research at the University of Manchester has been operational since April 1985 and is currently investigating the effect of four CSOs located and Bolton and Hydburn, Lancashire, on their respective receiving watercourses. The research is funded by SERC, WRc and NWWA as part of the Water Industry Programme. Existing quality models such as SWMM (Huber et al 1981) are not applicable to such discharges due to the transient nature of the spillage. The object of the research is to formulate a simple model to predict the impact of CSOs on the quality of the receiving watercourse.

2 Description of Field Sites

The overflow under investigation is located at Great Harwood, Hyndburn in North Eastern Lancashire. The chamber discharges into the Hyndburn Brook. At the point of discharge the watercourse is of Class 4 standard. Geometric details of the overflow chamber and details of the sewer catchment characteristics are outlined in Table 2.

Table 2 Sewer catchment and chamber characteristics of the Great Harwood CSO

Chamber characteristics		Catchment details
Dimensions	30 x 2.7 x 1.7m	Population 12500
Inlet dia.	2100 mm	
Storage volume	$140m^3$	Total area 120.9ha
Peak inflow*	4.5 m^3/s	
Flow to Treatment	0.27 m^3/s	Impervious area 55.7ha
Overflow	4.23 m^3/s	

* 2-year design storm

3 Sampling and monitoring system

A schematic representation of the overflow sampling and river monitoring system is given in Figure 1.

The location of the sampling sites has been selected to provide representative samples of effluent and river water. In particular, the river sites are situated at points where adequate mixing of any overflow effluent should have occurred.

The water level in the overflow chamber is monitored by means of a WRc-designed swingometer (a metal rod with a float at one end and a wire wound potentiometer at the other) which sends a varying angle-dependent voltage to an on-site Environmental computer. The microcomputer (programmed in the real-time language polyFORTH), stores the overflow level information. When a predetermined level is exceeded, the microcomputer triggers the overflow sampler. CSO inflows are also sampled and monitored in a similar manner, as described by Thornton et al (1986). Sirco and Manning discrete automatic samples are used, each taking a maximum of 24 x 250 ml samples.

The microcomputer is programmed to trigger a series of samples at predetermined time intervals. The time and number of each sample is stored by the computer. The overflow level is also read and logged at the same moment. This level is subsequently used to compute the magnitude of the flow discharged to the watercourse. Samples and stored data are retrieved and taken to the laboratory for analysis.

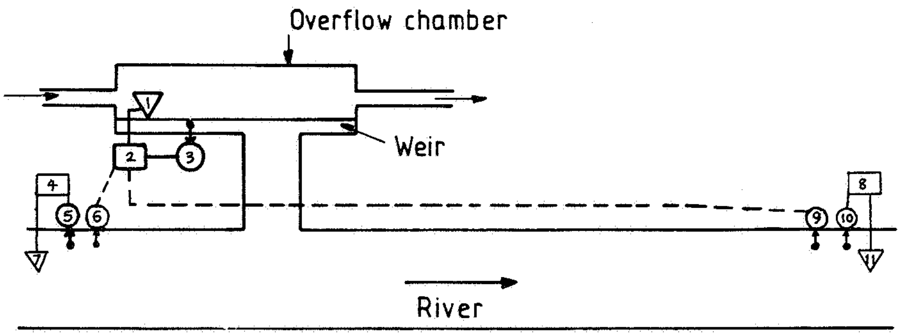

Figure 1 Schematic diagram of a sampling/monitoring system at a combined overflow

1. Overflow level monitor
2. Overflow microcomputer
3. Overflow sampler
4. Upstream river microcomputer
5. Upstream river level-triggered sampler
6. Upstream river overflow-triggered sampler
7. Upstream river ultrasonic level monitor
8. Downstream river microcomputer
9. Downstream river overflow-triggered sampler
10. Downstream river level-triggered sampler
11. Downstream river ultrasonic level monitor

The river samplers are activated by a signal from the overflow microcomputer via landlines or by a back-up system triggered by ultrasonic river level sensors (Figure 1). River levels are monitored by an ultrasonic level transducer mounted on the underside of each of two bridges. River flows are to be computed using data from a flow gauging survey. Rainfall in each catchment is measured by a raingauge, and associated data logger, located in the centre of the catchment.

4 Parameters

The samples collected are at present analysed to determine chemical oxygen demand (COD), suspended solids (SS) ammonia (NH_3) and conductivity. Analysis is carried out by NWWA. It is also proposed to determine organic and inorganic suspended solids for all storm events, and biological oxygen demand (BOD) for selected storms. An investigation into any correlation between COD and BOD will then be undertaken. Continuous monitoring of dissolved oxygen (DO), conductivity, pH and temperature is to be introduced at all sites in the near future.

5 Results

Two storms events of 5 December 1985 and 17-18 January 1986 are presented to illustrate initial results. A primary objective of the programme is to determine whether the chosen research methodology is the correct approach to the problem. A further aim is to assess the impact of individual events on the Hydburn Brook in terms of pollutant concentration. (In future, pollutant loads will also be studied).

Salient rainfall data and climatic and temporal hydraulic characteristics of the two events are presented in Table 3.

Table 3 Details of storm events

	Event	
	5.12.85	17-18.1.86
Max. precipitation intensity (mm/h)	11.5	3.8
Total precipitation (mm)	3.2	4.8
Overall intensity (mm/h)	5.7	1.3
Delay time prior to overflow* (min)	34	219
Antecedent dry weather period (h)	27	4

* Time from first stormflow entering chamber to time of first spillage

For the storm event of 5 December, Figure 2 shows the concentration of pollutants in the river upstream of the Great Harwood CSO, Figure 3 illustrates the concentrations in the CSO discharge and Figure 4 records

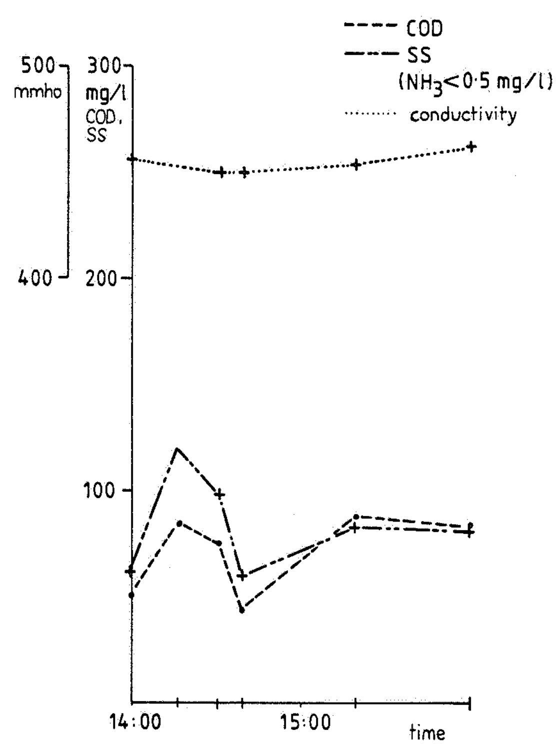

Figure 2 Upstream river site above Great Harwood CSO 5.12.85

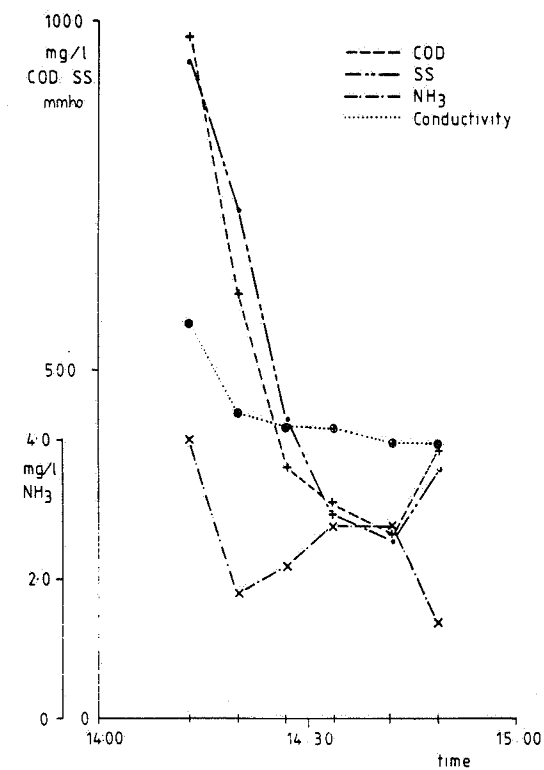

Figure 3 Great Harwood overflow 5.12.85

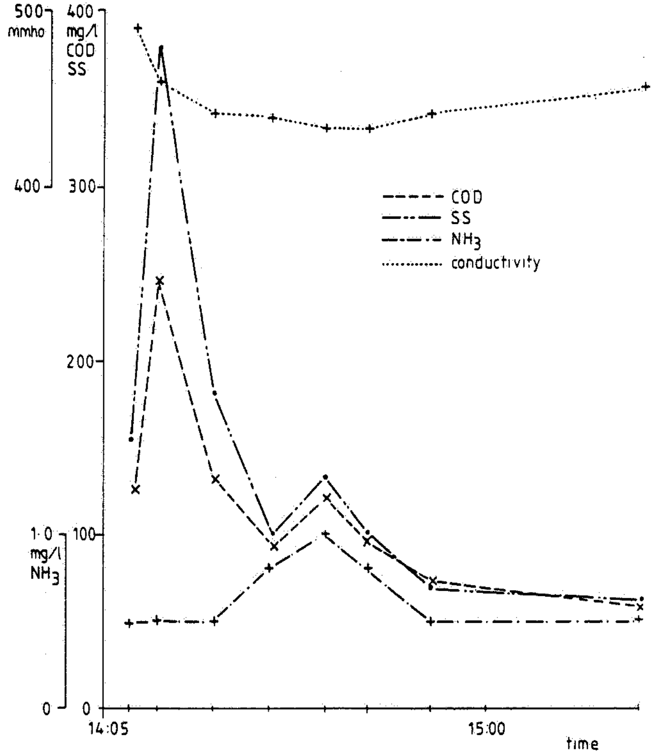

Figure 4 Downstream river site below Great Harwood overflow 5.12.85

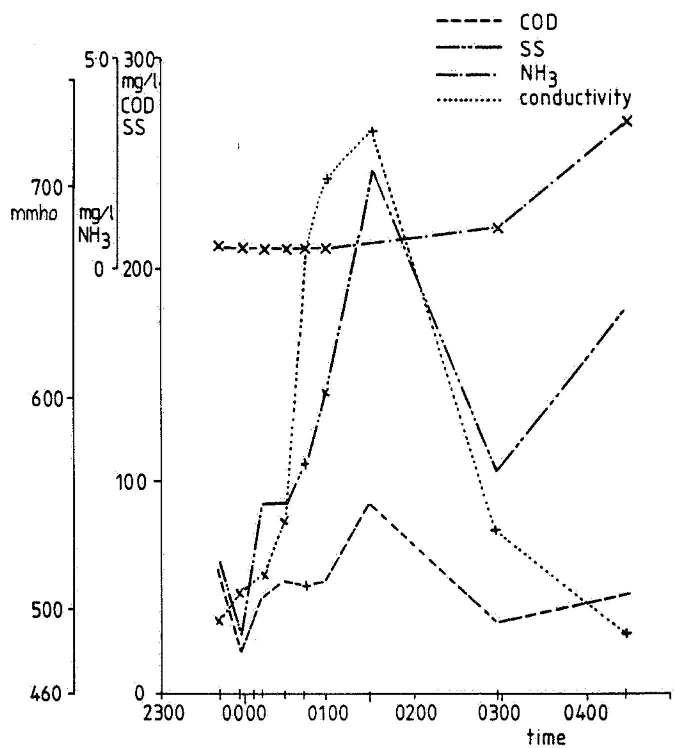

Figure 5 Upstream river site above Great Harwood CSO 17-18.1.86

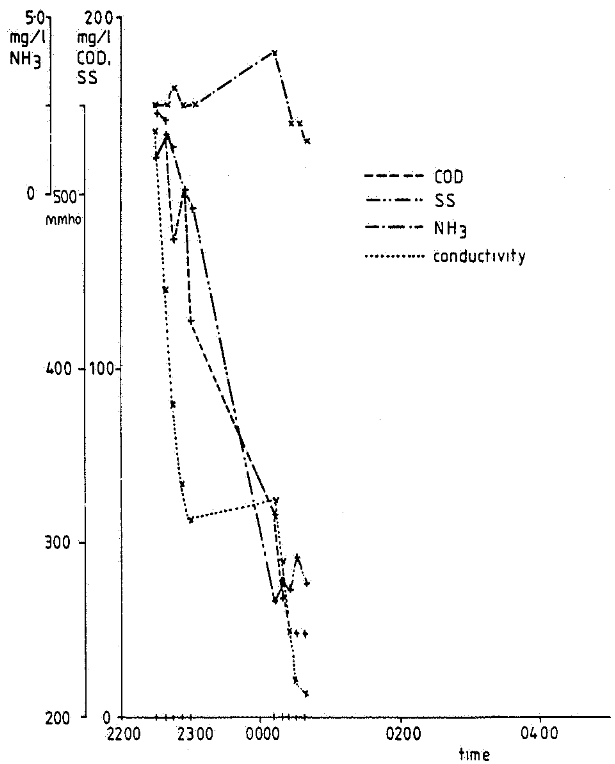

Figure 6 Great Harwood overflow 17-18.1.86

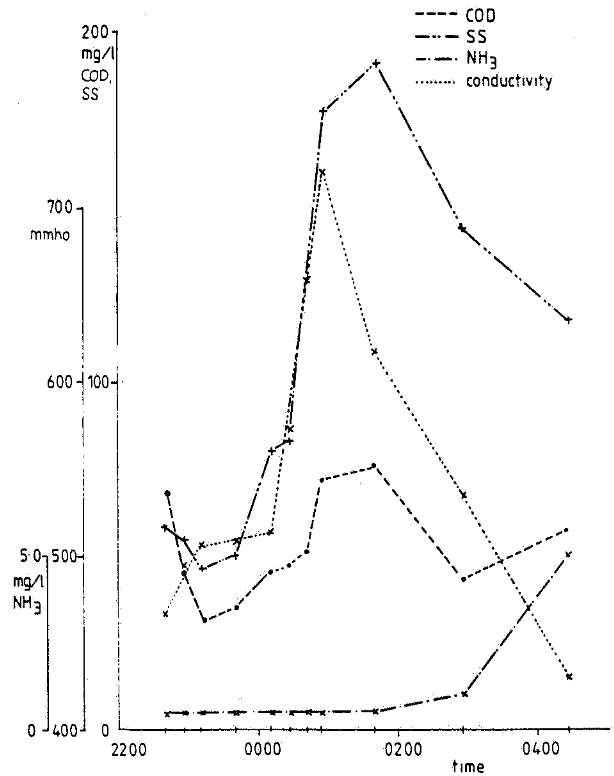

Figure 7 Downstream river site below Great Harwood overflow 17-18.1.86

the resultant concentrations in the river downstream of the point of discharge. Similarly, Figures 5, 6 and 7 outline the corresponding results for the storm event of 17-18 January 1986.

For the first storm event, outlined in Figure 2, pollutant concentrations in the river upstream of the discharge are low; SS and COD are less than 120 mg/l, NH_3 less than 0.5 mg/l, conductivity is less than 450 mmho. Concentrations in the overflow (Figure 3) are significantly higher; SS and COD around 1000 mg/l, NH_3 as high as 4.0 mg/l, conductivity as high as 550 mmho. The impact of this overflow discharge has a considerable effect on the quality of the watercourse monitored at the downstream river site. With reference to Figure 4, the peak concentrations of SS is increased to 380 mg/l, of COD to 250 mg/l and of NH_3 to 1.0 mg/l. This represents a major increase in pollutant concentrations downstream of the point of discharge as a result of the spillage.

In comparison, during the second storm peak, overflow spillage pollutant concentrations are of the order of 180 mg/l SS and COD (Figure 6), a considerably weaker discharge than produced during the first storm event.

The degree of impact may be explained in terms of the differing storm profiles of the two events. Pearson et al (1986) suggests that overflow discharge quality may be adversely influenced by an increased antecedent dry weather period (ADWP) and high intensity rainfall (Table 3). Hydraulic performance of the chamber is also a factor affecting overflow quality (Thornton et al (1986)), particularly in terms of the delay time prior to spillage (Table 3) and the proportion of any first foul flush of pollutants (which the increased flow has scoured out of the sewer system in the initial part of the storm) stored by the chamber.

The second storm event is characterised by a low maximum precipitation intensity (1.3 mm/h) and a shorter ADWP (4h), as shown in Table 3. In comparison, examination of the rainfall data for the first storm shows an overall precipitation intensity of 5.7 mm/h and an ADWP of 27h, and it is clear that these differences are sufficient to create an effect.

In addition, a short ADWP of 4 hours also indicates a difference in river conditions between the two events. The river on 17-18 January was exhibiting storm conditions with high flows and poor quality, possibly resulting from resuspension of river sediments and the effects of other upstream CSOs.

A difference can also be seen between the delay times prior to overflow for the two events. The first storm has a delay time of 34 minutes, whereas the second storm has a delay time of 219 minutes (Table 3). In the case of the second storm it is likely that the chamber has stored the majority of any first foul flush, thus ameliorating the quality of the overflow spillage.

6 Conclusion

Initial results suggest that the research methodology adopted provides an appropriate assessment of overflow impacts on watercourses.

Storms that exhibit relatively long antecedent dry weather periods, high precipitation intensities and short delay times prior to overflow spillage result in discharges with an increased degree of impact on the watercourse. Overflow spillage impact would be much greater were it not for the ameliorating effects of chamber storage, withholding the major part of the first foul flush and increasing the delay time before overflow operation.

River behaviour at Hyndburn is complicated by other storm overflow discharges affecting the degree of impact of the Great Harwood overflow. The area of study has been expanded to investigate the impact of other overflows in order to provide a more complete understanding of the effects of CSOs on watercourses.

7 Acknowledgement

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COMBINED SEWER OVERFLOWS, STATISTICS ON OXYGEN DEPLETION

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Abstract

Procedures for the design of combined sewer overflows are based on performance evaluated on the basis of compliance with exceedence statistics on required oxygen concentrations in a river. Design is based on simulation of the runoff process, the overflow and the oxygen depletion in the river. A historical rain series is used as input and a corresponding series of oxygen concentrations is compared with the required oxygen concentration.

1 Introduction

Combined sewer systems in the cities is a century long tradition. With these systems are inherited an incredibly large number of combined sewer overflow structures, which release unpurified sewage to the environment with a frequency that is not tolerable by todays yardstick. It is equally a tradition that these overflow structures are designed on the basis of rules of thumb, which have little relation to the real problem, even though they represent established practice, engineering codes or regulatory decrees.

While design of treatment facilities based on the ultimate goal: water quality in the environment, has been advocated and implemented in Denmark for more than a decade it is not until recently that this concept has been accepted for the design of combined sewer overflows. In this article a number of novel approaches are combined to establish new procedures for design of combined sewer overflows.

2 Statistical concept

In a series of results from measurements of water quality, those affected by rainfall constitute a rather small fraction. Accordingly, the data in a series of water quality measurements have to be divided into no less than two groups:

a) Data from periods without rain
b) Data which are affected by rain events

These two groups of data will constitute two distinctly different statistical populations. A bend is expected on the relative frequency curve for the measurements at the frequency of the occurrence of overflows. The bend is to be expected at 1-5% frequency, depending on the meteorology of the locality, the catchment, structures, detention basins, etc. It takes 100 - 1000 reliable data to determine the transition. This feature of the relative frequency curve makes extrapolation of the relative frequency curve to small frequencies potentially erroneous and always inaccurate.
The occurrence of severe oxygen depletion in a river due to combined sewer overflow is a relatively rare event, compared to the majority of results from rutine monitoring. In statistical terms it is sufficiently rare to be judged by extreme event statistics rather than relative frequency curves at small frequencies. Extreme statistics will tell you, e.g. that a given detrimental effect will be exceeded every 5 years. From extrapolation of relative frequency curves it may be determined that a given detrimental effect will be exceeded in 0.01% of the time. On a yearly basis that amounts to approximately 50 minutes. That information does not tell you how frequent the event is. It may be twice a year of 25 minutes duration or every 5 years with an average duration of 25o minutes.
For the purpose of this discussion pollutional effects can be grouped broadly in three categories, Harremoës (1982).

a) Pollutional effects lasting for the duration of the event only.
b) Pollutional effects lasting for an extended period past the occurrence of the event.
c) Accumulation of pollutants through extended periods before the effects become detectable.

An example of the first category is bacterial pollution affecting the health of swimmers in receiving waters. That risk occurs only as long as the water is contaminated. It is established practice to base water quality standards on permissible frequency, interpreted as percentage of time and illustrated with relative frequency curves.

To the third category of pollution belongs e.g. eutrofication and pollution from heavy metals and persistent organic compounds. Because of the accumulating effects the relevant figure for the pollution is the load over extended periods, like annual load. It is irrelevant whether this load is generated by a daily stream or by periodic events like overflows.

The subject of this article: oxygen depletion due to combined sewer overflows, belongs to the second category. The overflow event may be of short duration, but the effect may have a prolonged impact on the river. A fish kill of very short duration will have exterminated the fish population for as long time as it takes to build up a new population. The concern is: How infrequently does the event occur, or how frequently can it be allowed. That is judged by extreme event statistics, not through percentage time of occurrence.

In recent years several papers have advocated the use of relative frequency curves and percentage time of occurrence as the proper basis for the evaluation of the performance of combined sewer overflows. It is encouraging that a recent publication from USEPA (1983). introduce exceedence statistics with the words: 'Recurrence interval is a very useful definition'.

3 Water quality standards

Standards for required oxygen concentrations in rivers have been discussed for years, in so far as the daily events are concerned. There are also statistical methods for the evaluation of the compliance of a given

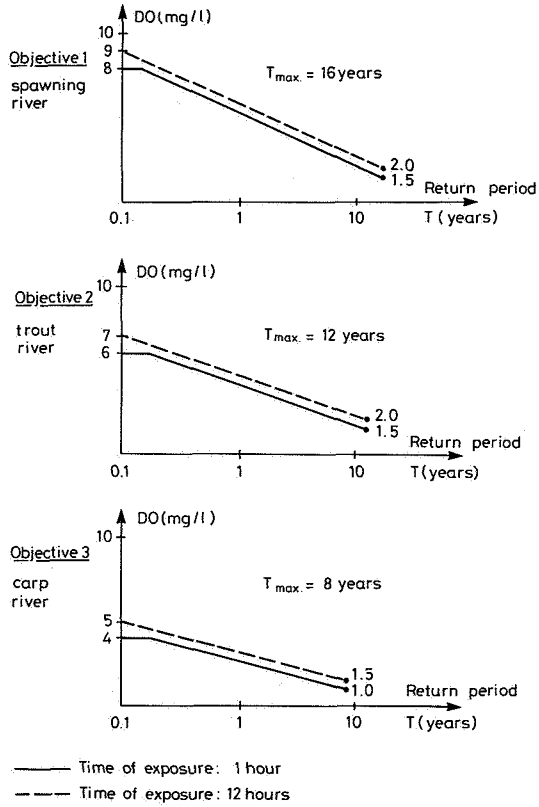


Figure 1 Recommended standard for extreme event statistics on required oxygen concentrations in rivers, as affected by combined sewer overflows

set of data with such standards. However, the counterpart to the concept of extreme event statistics is an adequate formulation of a new standard. Figure 1 shows the new standard recommended by the Danish Water Pollution Control Committee. The standard expresses the required oxygen concentration as a function of return period (or recurrence interval). Such curves have been selected for two durations of the DO depletion, 1 and 12 hours, and for three types of water quality, corresponding to habitats for spawning fish, salmon and carp.

These standards have been derived from extensive literature studies on the effects of low oxygen concentrations, referenced in the basic report, Hvitved-Jacobsen (1984). The criterion selected is that half

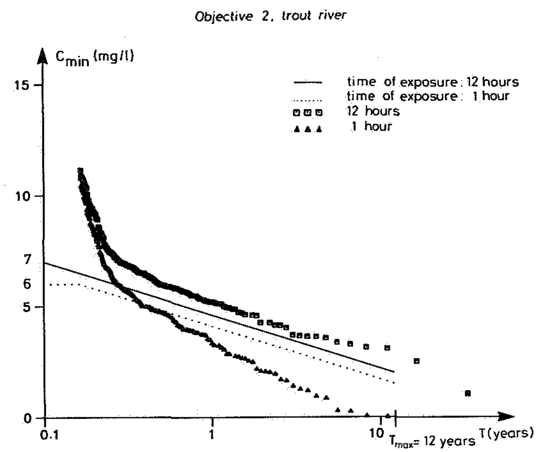


Figure 2 Predicted extreme event statistics for oxygen concentrations in a river, compared with the standard for trout

the fish population may be killed at the concentration and duration indicated for the rarest events (from an 8 to a 16 year return period).

4 Overflow series

It is not possible to judge the performance of combined sewer overflows on the basis of extreme event statistics with the traditional methods applied to urban storm drainage design. The alternative is to use historical rain series directly as input to models for calculation of runoff and pollution and to apply the statistics to the calculated concentrations in the river. Such statistics are illustrated on Figure 2, where a result is compared with the standard.

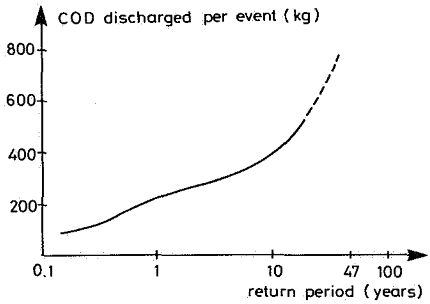
Available for this approach is a 33 year long rain record with a 2-5 minutes time resolution, a total of close to 6000 rain events, out of which were selected 1571 events with a rain depth greater than 3 mm. For the study in question the whole catchment contained 85 overflow structures. It is easy to imagine the number of computer calculations to be made and the cost involved. Accordingly, the following approach was used to simplify the calculations without sacrificing the basic principles.

For each catchment above an overflow structure the catchment is modelled with a kinematic wave approach model. From this is generated a time-area-curve for a rain event of mean intensity. That time-area-curve is taken as sufficient expression for the catchment characteristics. The overflow structure is characterized simply by the flow capacity of the interceptor. All flow exceeding that capacity is discharged to the river. All 1571 rain events are routed to the overflow structure according to the time-area-curve. The time of travel is adjusted according to the flow velocity in the partly filled pipes. This is done according to a standard relationship between mean flow during the event divided by the full pipe flow and the time of travel divided by the full flow time of travel. Details of this approach are described in Johansen et al (1984a), Johansen (1984b) and Johansen and Jensen (1981). However, this model is only a convenient way of cutting the cost of the main approach, but the details of this particular model of the runoff are not important to the basic principle. In principle any model of the runoff process can be applied to the 1571 rain events to generate a series of overflow events.

5 Series of pollution discharges

The study incorporated measurements of transport of pollutants past overflow structures during rain events over a period of one year for selected structures. From these brutto transports were subtracted the corresponding transports of sewage free of rain runoff, evaluated from daily flow records past the structure. From this was calculated the concentration in the rain runoff, which is a fictitious figure describing the contribution from other sources than regular sewage. This approach, described by Johansen et al (1984a) and Lindholm (1974), has the virtue that it is used in reverse to calculate the overflow for each event of a historical rain series.

Investigations were made of the variations of concentration from event to event and during events. Like many other researchers we found no adequate deterministic description of these variations. Equally, it was found that though variations during events could be detected to a minor extent (like 60% of matter discharged by 50% of the water) it was judged to be of little significance. The use of average concentrations is



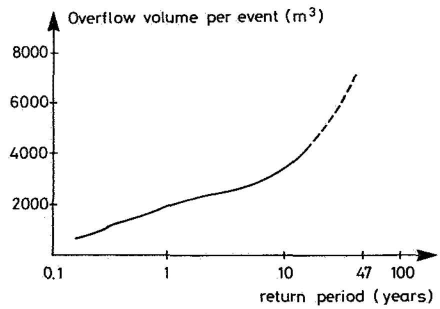


Figure 3 Predicted extreme event statistics for overflow volume and for pollutional load of COD pr. event of a 47 year long rain record

justified further by the mechanism of oxygen depletion in the river, from which it can be deduced that it is the total quantity discharged during an event that determines the depletion, irrespectively of the concentration distribution with time.

It is recommended to use the event mean concentration and to take into account the variability between events by multiplying the over-all average event mean concentration by a factor 2.0-3.0, depending on return period. In this simple way the combined extreme statistics of rain and concentration is accounted for, Harremoës (1986).

Figure 3 shows an example of extreme event statistics for the maximum overflow volume over a structure and for the maximum quantity of a pollutant discharged during an event.

6 Oxygen depletion by overflows

From the historical rain series is produced a series of discharges into the receiving water. This series is subsequently used as input to models of the oxygen depletion caused by the overflows.

Recent studies of oxygen depletion in small rivers have shown that the organic matter in the overflow is dominated by particulate and colloidal matter to the extent that most of the matter is adsorbed to the solid surfaces in the river much faster than it is degraded, Harremoës (1982), Hvitved-Jacobsen (1982) and Hvitved-Jacobsen and Harremoës (1982). This

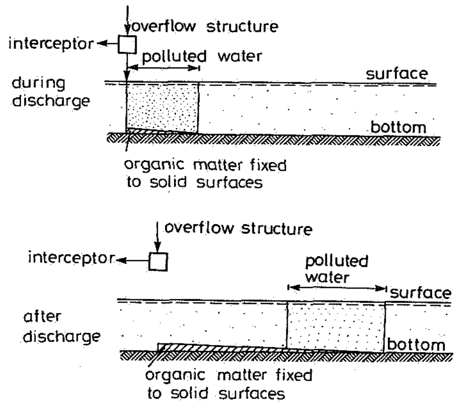


Figure 4 Illustration of the removal of organic matter from a water volume under transport down a river and the equivalent accumulation of the matter at the surfaces of the river, where it will degrade subsequently and cause delayed oxygen depletion in the river

causes a delayed depletion, i.e. oxygen depletion not only in the water volume receiving the overflow and moving down the river with the overflow, but dominantly in the water that flows past the overflow structure in the period just after the rain events. That water is exposed to the oxygen demand of the organic matter fixed to the solid surfaces. The effect is illustrated on Figure 4.

It has been shown for Danish rivers that the delayed effect is significantly greater than the immediate effect in the water volume actually receiving the pollution, Harremoës (1982), Hvitved-Jacobsen (1982) and Hvitved-Jacobsen and Harremoës (1982). This mechanism is enhanced by the fact, that the removal rate by sedimentation and adsorption is approximately one order of magnitude greater than the rate of actual degradation. The result is depletion concentrated in reaches close to the structure, where tributaries may have less smoothening effect on the depletion. Another result of the mechanism is the fact that only the totally discharged quantity of organic matter is of importance while the distribution in time is immaterial. These mechanisms can be illustrated for the simplest case of a uniform river polluted at time 0 from an overflow. In this case the classical oxygen sag curve can be modified to account for the adsorption and the delayed degradation.

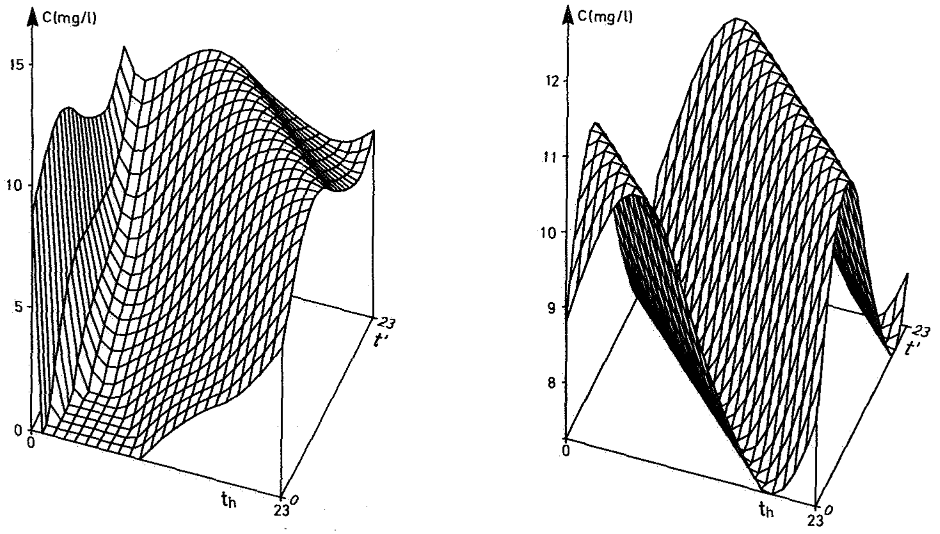


Figure 5 A three-dimensional illustration of the superimposition of diurnal oxygen fluctuations on the delayed oxygen sag in the river. C is oxygen concentration in mg/l, t_h is the distance down the river as time of travel in hours and t' is hours after passage of the overflow volume

Based on this concept the oxygen depletion at any point of a river can be calculated for each of the 1571 historical overflow events. However, it has been found that diurnal oxygen fluctuations have to be accounted for, because they can be very pronounced, Simonsen and Harremoës (1978). Accordingly, it is of great significance whether the oxygen depletion caused by an event occurs during the day, when photosynthesis in the river has created high oxygen concentrations to deplete from; or whether the event occurs during the night, when respiration has depleted the oxygen ressources of the river already. Accordingly, it has been found essential to account for the time of day for each event and to combine the two phenomena by superposition of the oxygen deficits. A three dimensional illustration of the oxygen concentration as a function of the location in the river and time after the overflow event is shown in Figure 5.

This constitutes the simplest of the models applied. In principle there is no limit to the complexity that can be built into these models, but the model described above is considered the basic, indispensable approach, that can be considered the stepping stone to more complex models in cases where the problem warrants an increased complexity and an increased cost of local data acquisition.

7 Statistical evaluation

The ultimate result of running the oxygen depletion model is a series of 1571 events of oxygen depletion in the river. The statistical approach is to rank the events according to level of concentration and to produce a series of oxygen concentration versus return period. An example is illustrated in Figure 2. The curve can be tested with respect to compliance with the standard described previously, as shown on Figure 2 with two curves: One that complies and one that does not. It is the experience that the one-hour duration standard is always the critical one.

In case of non-compliance remedial measures have to be taken, e.g. by increasing the capacity of the interceptor or better by introduction of a detention basin to cut down the frequency and quantity of overflows. Compliance can be depicted as the concentration difference between the standard and the calculated curve at the worst point, with the sign convention: positive for compliance and negative for non-compliance. An example is illustrated on Figure 6, where the compliance as mg/l of oxygen is shown as a function of the capacity of the interceptor: flow capacity pr. unit surface of the paved catchment in um/s (= 10 l/s pr. ha). The curves are shown for three volumes of basin pr. unit of paved catchment: 0, 3 and 10 mm. The curves are valid for a river with a dry-weather flow of 100 l/s. It can be seen that a volume of 3 mm for a capacity of the interceptor of 0.9 um/s will be required.

Figure 2 is an illustration of the ultimate use of the approach to design of combined sewer overflow structures based on water quality standards expressed statistically. However, the Danish Water Pollution Control Committee has found it expedient to publish graphs as shown on Figure 6 for a variety of cases in order to provide the basis for a first primitive evaluation of the expected performance of a particular

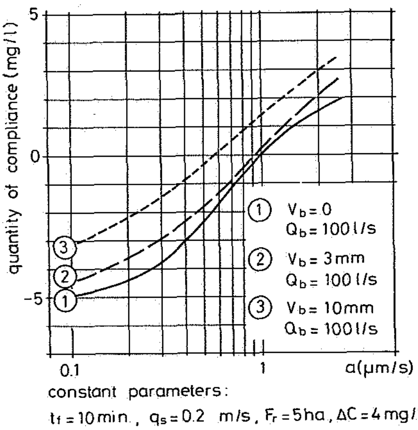


Figure 6 An example of design curves showing degree of compliance with the oxygen standard for a trout river measured as oxygen concentration difference for overflow from an urban catchment characterized by area: F_r, sewage flow: q_s, and full flow travel time in sewer system: t_f, as a function of capacity of the interceptor: a, for three volumes of detention basin: V_b, discharging to a river characterized by flow: Q_b.

structure, Hvitved-Jacobsen (1984). In case the structure does not comply or is close to non-compliance more refined methods must be applied and more information provided.

The recommendation includes a staged approach, Harremoës (1984), involving four levels of complexity - The first one being the use of graphs as in Figure 6. The second is individual computer calculations for the particular case, but with models and parameters as used for the basic approach. The third stage is the use of the same models as described above, but with parameters chosen individually based on investigations of the catchment and the river in question.

Such approaches have been introduced in Denmark as the SVK-system, which is a package of programmes available to the municipalities on their central mainframe computer. The same programmes have been converted to PASCAL-programmes for microcomputers, using the user-friendly facilities of the microcomputer. The system is called MOUSE. The basic idea is that the engineer concerned with the problems of discharge should have a system in front of him that allows him to concentrate on the problems of

rain runoff, rather than on the problems of communication with the computer. Accordingly, all communication with the microcomputer is based on menues guiding the user through the calculations.
The final stage is the use of individual models and parameters. The ultimate is the use of detailed simulations of historical, tracked rain series as input to continuous hydrological modelling and simulation of runoff based on the dynamic wave approach, combined with a detailed model of river pollution involving both immediate and delayed oxygen depletion due to organic matter degradation, nitrification, oxygen fluctuations, etc. to the point where the benefits of further refinement becomes meaningless.
In view of the uncertainties involved with respect to our knowledge of the processes and the parameters involved, and the uncertainty involved in the setting of the standard it is hardly ever warranted to engage the full orchestra of models. It is left to the engineer in practice to choose the level of complexity judged to be consistent with the seriousness of the problem, the quality of the input data and the uncertainty of the standard.
The basic principle is that the performance has to be judged on the basis of compliance with statistics on water quality in the receiving water - as an alternative to the traditional rules of thumb - without necessarily forcing upon the practitioner complicated computer models to be applied to even the simplest case, but leaving him with a range of approaches from simple to complex. The staged approach is based on confidence in the good judgement of the practitioner, when given the proper options.

8 Conclusion

Combined sewer overflows can be designed on the basis of water quality standards. The proper approach is to use extreme event statistics and to introduce water quality standards variable with frequency of recurrence. Standards for oxygen depletion in rivers caused by combined sewer overflows of organic matter are proposed.
The practitioner should be provided with means for calculation of performance of combined sewer overflow structures with respect to complian-

ce with a water quality standard as described above. This should include approaches from the simple use of graphs to complex use of models of the runoff process and the processes in the receiving water.
All approaches should be based on the use of historical rain series as input to models, through which to produce series of oxygen depletion events to be ranked and compared with the standard.
Simple graphs are produced by applying simple models to typical catchments and receiving waters, characterized by selected parameters. A first impression of compliance with the standard can be reached for the catchment and the receiving water in question by mere interpolation.
More advanced models involving individual parameters and more advanced modelling techniques are available and should be applied if warranted, when considering the seriousness of the pollution, the reliability of the available information and when considering the uncertainty of the water quality standard.

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THE USE OF WATER QUALITY MODEL QUAL II ON MICROCOMPUTER

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Abstract

The US-EPA Water quality model QUAL-II is a well known planning model for river systems and has been used extensively in many countries. In Belgium it was used for the first time in 1984 for evaluation of the effects of alternative wastewater treatment schemes in the Velpe, a small river system of 144 km^2. The main frame computer used for this application was CDC (Cyber 170/175) of the University Computer Centre. However the practical use of such a planning model, especially in developing countries, but also at subregional level in developed countries is limited when a mainframe computer is not available or not readily accessible. The recent personal computer boom makes it possible for virtually any local office to implement calculation programmes, and thus planning model, of relatively high complexity. The aim of the paper is to discuss the advantages, disadvantages and problems of implementing the QUAL 2E version for microcomputer (NCR Decision Mate V and Olivetti M24). Graphs adapted from the original version are part of the implementation exercise.

1 Introduction

As it is a phase where mathematical models are overtaking physical models, many attempts have been made to build mathematical river models so that behaviour of the river water quality could be depicted at any location and time. Effects of certain advancements like treatment or discharge regulation into the river system can be known in advance by using the model to forecast/simulate the quality parameters. QUAL-II is a well known planning tool for stream water quality, which was developed by Water Resources Engineers for Southeast Michigan Council of Government (SEMCOG) under the sponsorship of the US-Environmental Protection Agency (EPA). The program is written in Fortran IV, (Roesner et al. 1984 a,b) and is compatible with the UNIVAC 1108, CDC 6400, and IBM 360 and 370 computer systems. The SEMCOG version of QUAL-II requires an average 51000 words of core storage. It uses the system's 80 columns format for its input and 136 columns for output. This model has been applied by many people in different countries. To mention some, Van der Beken and Vanouplines (1985) applied the model to compare the water quality in the river Velpe using different treatment techniques, and Crabtree et al. (1985) applied the model to aid resource allocation. A modified QUAL-II to QUAL2E version for microcomputer was obtained from EPA Centre for water quality models, Athens, Georgia. Included on the diskettes from EPA are source program, executable program and three data set examples. The executable program was used by us to run the Velpe data on both Olivetti (M24) and NCR Decision Mate V (DMV). These data have been retrieved from CDC and filed for microcomputer. In this paper the results obtained from CDC will be compared with those from microcomputer. Time involved, advantages and disadvantages for CDC and microcomputer will be discussed.

2 The model on CDC

The original program of QUAL-II is constrained to 90 point loads, 75 reaches, and 500 total computational elements. The program uses unit 5 for its input and unit 6 for its output. Van der Beken et al. (1984) modified the program to allow a maximum of 170 point loads to be accomodated. Figure 1 shows Velpe layout. It contains 106 point loads, 14 headwaters, 74 reaches and 301 computational elements.

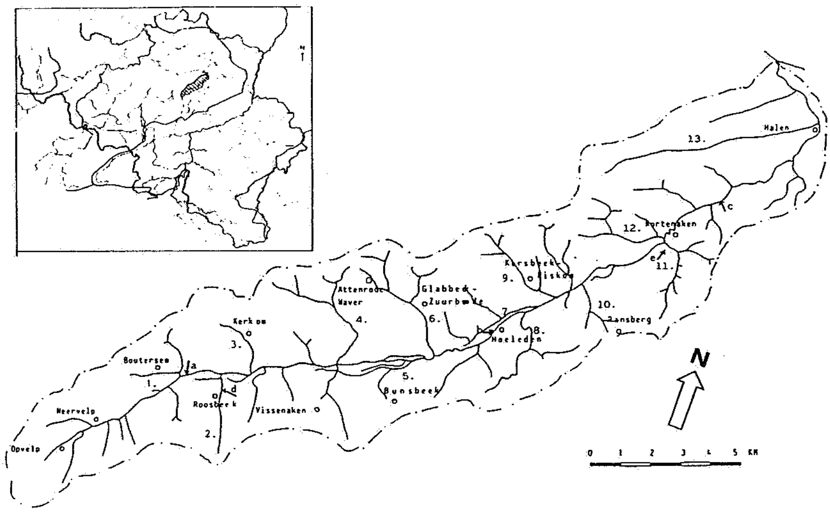

Figure 1 The Velpe watershed. The numbered tributaries are modeled, the others are considered point loads

The modified program was adapted again to include a unit called 7 for outputing interested quality parameters. A program which uses this unit 7 as its input was developed. This prepares Dissolved Oxygen (DO) and Biochemical Oxygen Demand (BOD) for any chosen sequency of reaches. These data are plotted using another developed plot program by the help of a graphic package FPL83 (Figure 2).

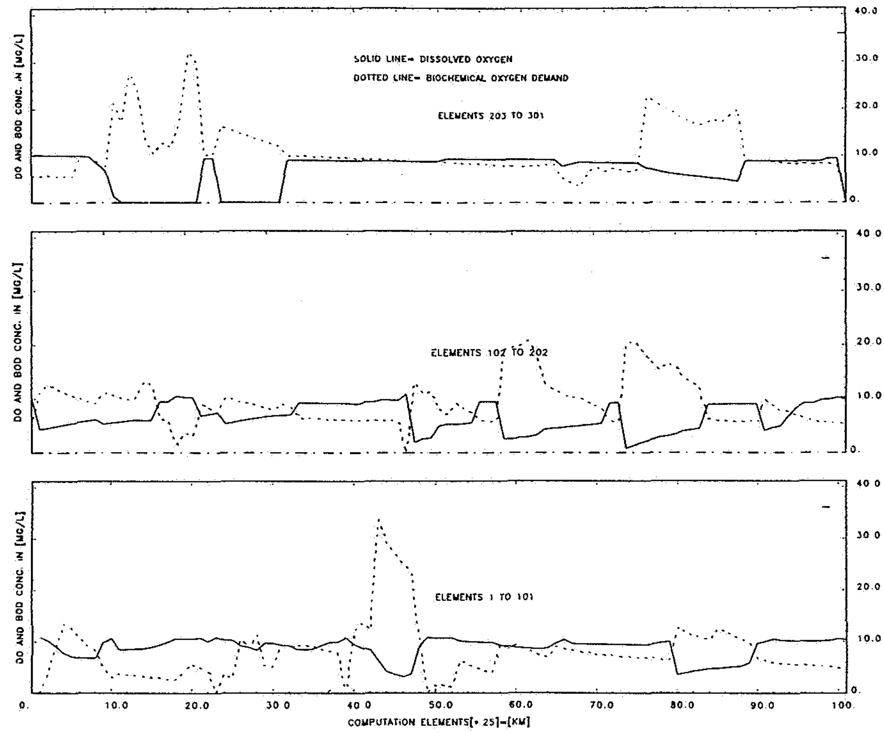

Figure 2 Plot of DO and BOD for Velpe River using plot 83 package on CDC

Execution of the Velpe data is done by a batch job. Compilation and execution time of the programs involved are summarized in table 1.

Table 1 Compilation and execution times in sec. on CDC for River Velpe

Program	Compilation	Execution
Main program (QUAL II)	51,10	50,06
Graphic data program	0,87	2,73
Plot program	0,64	6,50

3 The model on microcomputer

The program was run on both Olivetti (M24) and NCR (DMV). In both cases the data transferred from CDC were to be converted so that they can be run by use of QUAL2E. The program to convert the data from QUAL-II to QUAL2E data which was provided by EPA needed a hard disk to be able to run it. Since this was not available at the moment of application and in any case is an expensive extension of the personal computer, a new program to convert these data was developed. This uses normal diskettes to run. The source program consists of 44 modules. Some of these modules were modified and all were compiled in two steps : FOR 1 and PAS 2 creating object modules. Linking of these object modules to make an executable program was not possible because of limited space of the diskette. An executable version from EPA was used instead to run the data. This is constrained to 25 reaches, 25 point loads, 6 headwaters and 250 computational elements. The Velpe data with 106 point loads was divided into 4 parts and the extra 6 point loads were merged in two's which are similar and close together. These four parts were run separately starting with the upstream one. Concentrations of its most downstream element are taken as headwater concentrations for the next part. A program was developed to use the output from QUAL2E as its input. This program extracts DO and BOD and prepares a data file for plotting. Graphs (Figure 3) were obtained by using the graphic package Supercalc 3. Since the output format uses 136 columns, with EPSON FX-80+ printer under normal point mode, the output is unreadable. EPSON printer is capable of printing 136 characters in one line under a CONDENS mode. Using this mode enables the output to be readable. Compilation time and execution time for both M24 and DMV are presented in table 2.

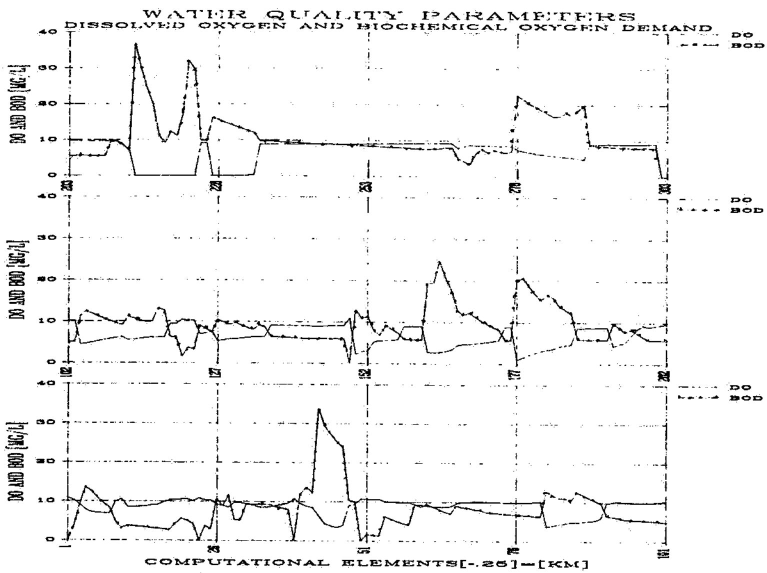

Figure 3 Plot of BOD for Velpe River using Supercalc3 package on microcomputer

Table 2 Compilation and execution time in min. on M24 and DMV for Velpe River

Program	OLIVETTI M24		NCR (DMV)	
	Compilation[1]	Execution[2]	Compilation[1]	Execution[2]
Main	85	15	120	70
Graphic and Plot data	5	15	7	22

1 Only compilation to create object modules without linking

2 Execution of the executable version of EPA

4 Comparison of OLIVETTI and NCR

Olivetti is about $1\frac{1}{2}$ times faster than NCR. It has a memory capacity of 640 Kbyte while NCR has only 256 Kbyte. Once Olivetti is initialized using a system diskette, one can work till the end. With NCR re-initialization is necessary before another command can be given because at the end of the execution of the program the system is lost.

5 Comparison of the overall results

Comparing figures 2 and 3 show that the results from the mainframe computer are almost the same as those obtained from the microcomputer. A slight difference is due to the fact that the model on the mainframe computer used QUAL-II and that on the microcomputer used QUAL2E which is a modified model. Blosser (1985), and Whitemore and Brown (1984) summarizes the modifications which have been made to QUAL-II. These modifications include Algal, Nitrogen, Phosphorus, Dissolved oxygen interactions, algal growth rate, temperature, Dissolved oxygen balance, arbitrary non-conservative minerals, hydraulic and downstream condition options. Also deviations of the results might be due to the point loads which were merged, in so doing the system configuration is changed. For dam rearation, Van der Beken et al. (1984) used equation (1). This was used by BUTTS and EVANS (1983) to compute oxygen input to the system.

$$r = 1 + 0{,}38 \times a \times b \times h\ (1-0{,}11\ h)(1+0{,}046T) \tag{1}$$

where r = deficit ratio

a = water quality factor

$a = 1{,}0$ was used

b = factor depending on the type of aerating structure

$b = 0{,}8$ for free overfall was used

h = height through which water falls

T = temperature °C

Reach and element containing a dam are specified

But QUAL2E uses equation (2) described by Zison et al. (1978) and attributed by Gameson

$$D_a - D_b = \left[1 - \frac{1}{1+0,11ab(1+0,046)h}\right] D_a \;\cdots\cdots \qquad (2)$$

with a = 1,25 in clear to slightly polluted water
= 1,0 in polluted water (a = 1,10 was used)
b = 1,0 for weir with free overfall
= 1,3 for step weir or cascade (b = 1,0 was used)
D_a = oxygen deficit above dam, mg/l
D_b = oxygen deficit below dam, mg/l

Since $r = D_a/D_b$, rearanging equation (2) gives,

$$r = 1 + 0,11 \times a \times b \times h(1+0,046T) \qquad (3)$$

The reach containing the dam and element below the dam are specified.

6 Conclusion

It is clear from the results obtained that even with small computers, one can attain the same results as obtained from mainframe computers. When comparing OLIVETTI and NCR (DMV), it is seen that the former is the best of the two. It is faster and more friendly. Mainframe computer is by far faster and time saving. If time is money, CDC is preferable. The use of the graphic package for CDC is more flexible since one can plot in any format and size desired. Packages for microcomputers are already prepared and no modification is possible. The only thing that can be changed is the input data. For places where mainframe computers are not available, microcomputers are appropriate. For big programmes which cannot be compiled and linked, an executable program can be purchased from elsewhere and used.

Notes

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MODEL-BASED ASSESSMENT OF EFFECTS OF STORMWATER OVERFLOW MANAGEMENT ALTERNATIVES UPON LOW DISSOLVED OXYGEN OCCURENCE IN THE RECEIVING STREAM

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Abstract

This novel method, based upon the decomposition-aggregation principle, aims at an engineering comparative assessment of the effects of storm water management alternatives (larger sewage treatment capacity, larger sewerage storage) upon the dissolved oxygen concentration (DO) in the receiving stream. A moving-cell plugflow-with-dispersion model is first used to estimate the effective model parameters from field data during stormwater overflows. Next, the model is applied in an off-line fashion to generate relationships between overflow duration and intensity, and maximum extra DO deficit in the stream. This is done for a limited number of characteristic environmental conditions, ensuring a minimum data requirement in contrast to the usual long term simulation procedure. The relationships derived are then used together with figures on treatment capacity and sewerage storage to construct equi-minimum DO lines for each of the alternatives in a plot of rainfall intensity versus rainfall duration. From this aggregated graph the expected annual frequency of occurrences of DO levels in the stream below a preset value can rapidly be determined, thus allowing for a comparison between various management options. The method developed from a case study of the River Vecht, Utrecht, the Netherlands, and the results are being used for actual decision support.

1 The River Vecht DO problem

The R. Vecht is a moderate size stream in the central part of the Netherlands (Figure 1). It originates in the city of Utrecht, and receives most of its water indirectly from the Rhine River. Its discharge rate can be controlled by manipulating a sluice in the centre of the city. Present policy is to maintain a low flow of about 2 m^3/s during the day, and a flushing of about 8 m^3/s during the night. The low flow value is dictated by the desire not to cause nuisance to the intensive recreation shipping during summer.

The most important pollution load originates from the sewage treatment plant of Utrecht (STP), which has a capacity of 390,000 inhabitant equivalents. The average dry wheater supply is 63000 m^3/d. The present pumping capacity is 9000 m^3/h, but the effective full-treatment capacity is limited to 6000 m^3/h due to insufficient hydraulic capacity of the post-clarifiers. As a consequence in case of rainfall the surplus of 3000 m^3/h has to be let off to the R. Vecht with not more than primary treatment. In the near future this undesirable situation will be terminated after completion of the post-clarifier expansion now under construction.

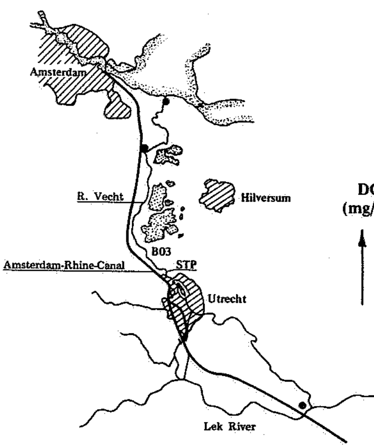

Figure 1 The River Vecht region, the Netherlands

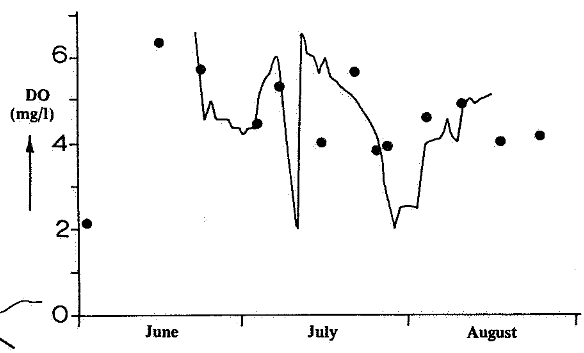

Figure 2 Dissolved oxygen observations (at B03) (dots: weekly monitoring; line: continuous record)

The average dissolved oxygen concentration in the stream is between 4 and 5 mg/l in summer, but lower values have frequently been observed. Figure 2 gives an impression of the regular weekly monitoring values, and how they compare to continuous records at the same location (B03). Incomplete observations in the summer of 1983 showed anaerobic conditions for five times, and in addition about twice as much occurences of values below 2 mg/l.

2 Causes of low DO

An analysis based on field measurements and a plug flow model with stationary inputs for average summer conditions showed that the moderate oxygen condition of the stream was due to low DO input from upstream, a considerable sediment oxygen demand (up to 100 mg/m^{2}h), and nitrification of N-BOD originating from the STP. The average condition is taken as background and will not be discussed in detail herein.

The occasional occurrence of low DO values turned out to be strongly correlated to heavy rainfall events, although not all occurences could be traced back this way. It is likely that the discharge of the surplus water at the STP during moderate rain periods also plays a part. The present storage capacity of the sewerage system of the city of Utrecht as a whole is about 85,000 m^3 (5.7 mm). However, in the sewerage area around the treatment plant (209 ha) the storage is less than average (4.6 mm). Moreover, since the pumping capacity of the local area pumps exceeds the capacity of the STP pumps by 2600 m^3/h, the effective pump overcapacity (Poc) is negative. Under the assumption of a storage of 1.5 mm in the streets, these figures lead to an expectation of a theoretical storm water overflow frequency directly on the R. Vecht of roughly 40 times a year under Dutch rainfall conditions. A large proportion of these must be expected to cause DO problems in the stream, as will be discussed below.

3 Management options

In this particular case study, several management options are available to cure the situation. The three most important are:

- option 1: expansion of the hydraulic capacity of the STP to 15000 m^3/h, which allows the treatment of the dry wheather supply plus a rainwater supply of 30 l/inhabitant h (a common value).
- option 2: expansion of the storage in the sewerage system of the city.
- option 3: the combination of both measures. The average storage to reach a stormwater overflow frequency of 5 per year (the standard in force) with the expanded hydraulic capacity of the STP in effect should be 8.7 mm for the city as a whole, and 9.85 mm for the district around the STP. These values are hold on for option 2 as well.

Other additional measures related to the hydraulic regime of the stream will not be discussed in this paper.

Considerable investment costs are associated with each of the options. Options 1 and 2 are about even in this respect. An interesting detail is that the measures come to the expense of different authorities: the STP is the competence of the provincial water authority, whereas the sewerage system is under responsibility of the municipality of the city of Utrecht. So, prior to any decision making it is important to assess the effects of each of the options, not only in terms of expected annual stormwater overflow frequency, but also, more importantly, in terms of the expected dissolved oxygen conditions in the stream.

4 Methodology

Prediction of expected DO levels in the stream requires some form of modelling, because present observational data patterns can not simply be extrapolated to future conditions. An immediate decision in stream flow modelling is whether longitudinal dispersion should be included or not. In this case strong longitudinal gradients occur because of stormwater overflows, so dispersion can not be neglected. Also, stormwater overflows are highly dynamic. As a consequence, a fully dynamic one-dimensional model with longitudinal dispersion is needed. The problem with application of this type of model over a prolonged period of time, which is of interest here, is that it requires a long time series of input data. Such extensive data are usually not available, and even if they were, the creation of the corresponding input files and, especially, similarly detailed input files for each of the management options, would be a cumbersome task. So, another solution would be most welcome.

The engineering method developed here is based on the principle of decomposition and aggregation. First, the problem is split into two parts: one related to average summer conditions, the other to overflow events. This is possible because the oxygen deficit model is linear. The average can be treated with a simple non-dispersive, stationary model, requiring a minimum of data. The more demanding fully dynamic model with dispersion then needs to be used under overflow conditions only. The second step in the decomposition is to run the model (after calibration) for a synthetic representative set of possible events, rather than for a complete series of historical events. The resulting maximum extra oxygen deficit due to overflow is plotted as a function of the characteristic parameters 'overflow duration' and 'overflow size'. Thus, detailed information from the model is aggregated in the form of this graph. In the final step, the aggregated graph is combined with the classical rainfall-duration plot to yield equi-minimum-DO lines, from which the annual frequency of occurrence of stream DO below a preset level can easily be evaluated.

Summarizing, decomposition, off-line model use and aggregation all are essential elements in reaching the desired savings in time and data needs.

4.1 Model

In an engineering study, little time is available for additional field experiments for model identification. So, modelling must largely be based on generally accepted basic process descriptions, and parameters have to be evaluated from literature and by calibration on field data.

The model is a classical DO-BOD model. State variables are DO, carbon-BOD, organic N, and NH_4-N. The processes modelled are BOD-decay, reaeration, ammonification, nitrification and sediment oxygen demand (SOD). The influence of algae on DO could be neglected.

The BOD-decay coefficient depends upon the type of pollution. With a formula suggested by Wolf (1974) a value of 0.15 d^{-1} is estimated for average summer conditions, and 0.35 d^{-1} during overflow events. The reaeration is estimated with the well-known O'Connor-Dobbins formula, but allowance is made for an additional constant contribution due to wind. The value of ammonification of organic N is taken from literature as 0.1 d^{-1}. The model is not sensitive to this value. Sensitivity analysis shows, however, that the model is sensitive to the nitrification rate coefficient. Under average conditions, a value of 0.35 d^{-1} is obtained by calibrating the model against a detailed longitudinal DO profile. The SOD has been measured in especially designed experiments on sediment cores. Typical values are in the range of 40-100 mg/m^2 h. This is responsible for a permanent deficit of about 1-1.5 mg/l in the stream.

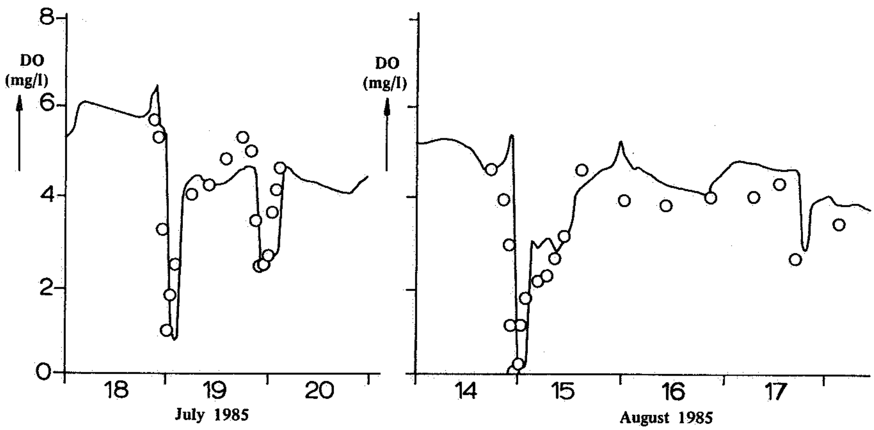

Figure 3 Calibration of the dynamic model with dispersion (dots: measurements; line: model results)

The dynamic model used under overflow conditions (GELQAM) is based on the moving-cell principle, described by De Boer (1979). The value of the longitudinal dispersion coefficient has been derived from an empirical formula presented by Rutherford (1981). Since nitrification during storm water overflows is probably low, its value was arbitrarily set to 0.1 d^{-1}. Possible deviations from this are aggregated in one single total-BOD decay coefficient, which accounts for all decay contributions. Calibration for two different stormwater overflow events leads to a value of 0.37 d^{-1} (Figure 3). The BOD-input was reconstructed from overflow duration records of some of the overflow points, and from the BOD measured in the influent of the STP.

The performance of the model is satisfactory; the time shifts are mainly due to deviations in hydrological transport.

4.2 DO effects of a stormwater overflow

The GELQAM model package still requires a carefully constructed input data set, and considerable computer time on a mainframe computer. Therefore, prior to using the model for further predictive calculations, a tailor made Pascal version running on a PC was developed. This version differs from the original one because a uniform cross-section is used, but is otherwise based on the same moving cell principle. Also the parameter values are maintained. This model produces similar plots for the calibration case. Each stormwater overflow event of a certain size (mm overflow water) and duration (in minutes) creates a slug of polluted water in the by-passing stream. The model calculates the BOD and DO in this slug, and a series of adjacent water segments, during the time of travel downstream. Dispersion spreads out the BOD originally contained in the polluted slug, and thus mitigates the occurrence of oxygen depletion. Figure 4 gives a typical example of the output. The DO deficit in the centre of the slug, at the edges, and at half the slug length outside the slug is shown. The effect of dispersion is directly apparent from this. In the top view the movement of the low oxygen slug through the river is recorded, as well as the boundaries of the area which is below the preset oxygen level of 2 mg/l. The deficit profile in the centre is rather flat in time due to the day-night rithm of the river discharge and the dispersion. Generally, the maximum deficit is reached at some 10 km downstream of the outfall.

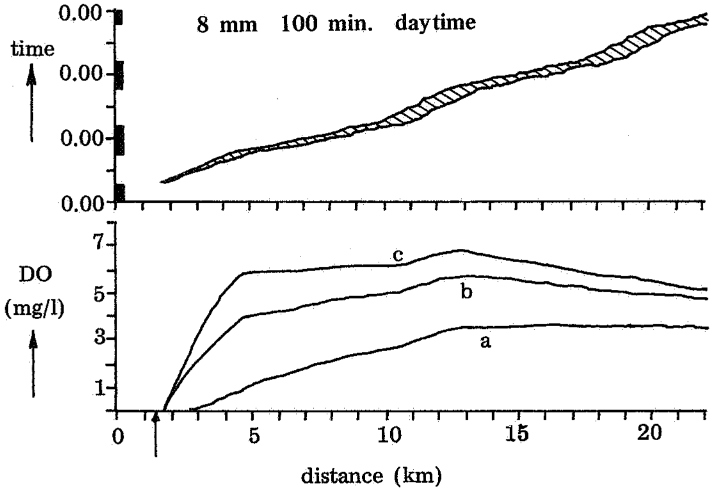

Figure 4 Example of DO predicted by the model
Top : travel through the system and area of DO below 2 mg/l
Bottom: DO in the centre (c), at the edge (b), and half-way outside the polluted slug (a)

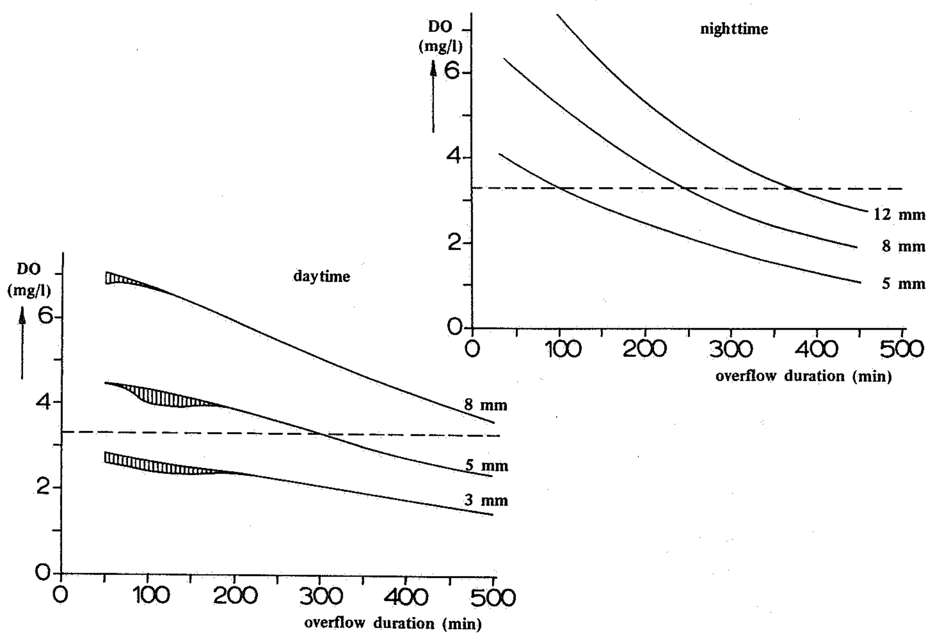

Figure 5 Maximum deficit contribution due to stormwater overflow in summer

The model is run for a number of size and duration combinations. Figure 5 summarizes the maximum extra deficit experienced somewhere in the river as a function of duration for a number of overflow sizes. A certain amount of mm overflow represents a fixed pollution load, which is calculated by multiplying the associated amount of water with a BOD-infinite of 135 mg/l, in agreement with an average of 95 mg/l BOD-5 measured in the STP influent during overflows. When the same amount enters the river in a shorter time, the BOD increase in the polluted slug is larger, which explains the more severe effects of short overflow times in Figure 5.

The lower part of Figure 5 holds for overflows occurring during the day and with an average summer temperature of 20oC. During the night the river discharge is larger, and so is the initial dilution. Therefore, the calculations had to be repeated for nighttime conditions (Figure 5, upper part). Also, renewed calculations were done for winter conditions, assuming a BOD-decay coefficient of 0.17 d^{-1} (i.e. 10°C;(not shown).

4.3 Frequency of occurrence of low DO concentrations

The next task is to determine how frequently DO concentrations below 2 mg/l will occur during a year for each of the management options. A starting point for this analysis is an intensity-duration plot of rainfall events over a large number of years (Figure 6). In these plots the amount of available storage in the sewerage system (including storage in the streets) represents a horizontal line. Theoretically, rainfall events below this line can be stored completely, and will not cause a stormwater overflow. In addition, due to the pump overcapacity at the STP, during the rain a certain amount of water can be pumped away. This is represented by a line with a slope proportional to the Poc.

Thus far, this is the classical 'dot-graph' frequently used in sewer system design. The novelty of the method, however, is that this graph can also be used to construct equi-DO-minimum lines ("iso-doxes"), with the help of Figure 5. As an example the iso-dox for option 3 is shown in Figure 6. Once this line has been drawn, it is a simple task to count the number of rainfall events above the iso-dox. These are the events causing DO problems in the stream. The annual amount is obtained by dividing this figure by the number of years on which the dot-graph was based (here: 37). So, in the end, a very simple and transparent way of evaluating the effects of overflows on stream DO is achieved. The procedure is repeated for each of the alternatives; the differences are mainly due to differences in storage and Poc (not shown).

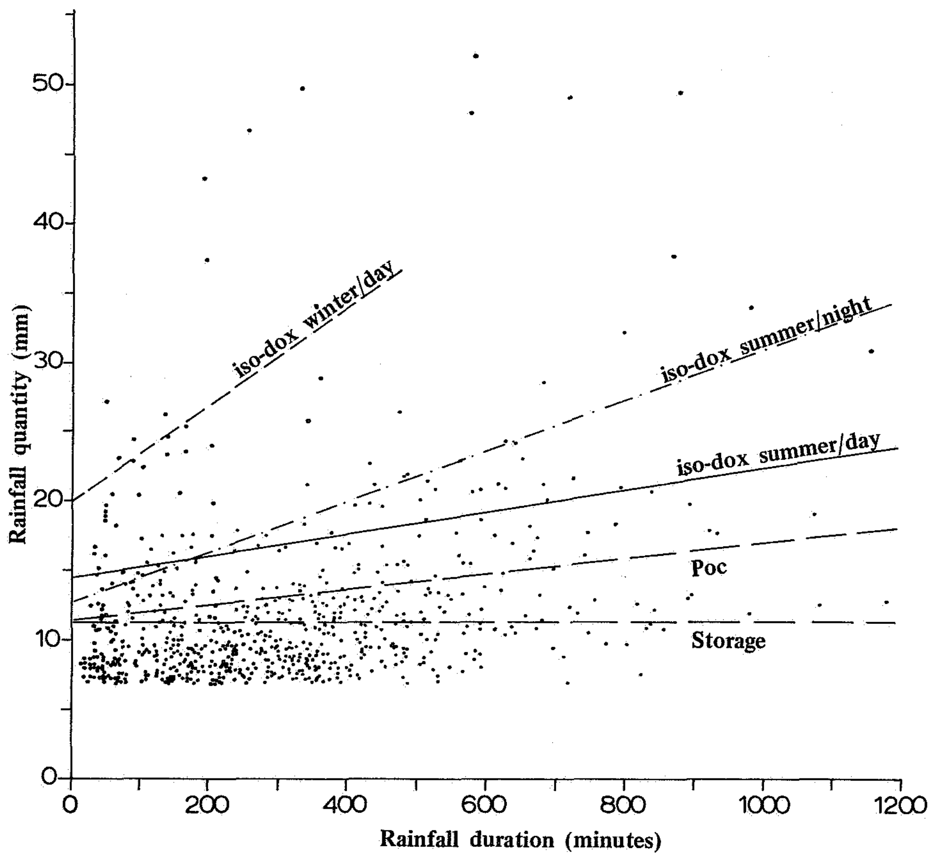

Figure 6 Equi-DO-minimum lines ("iso-doxes") in a rainfall duration plot ("dot-graph"). Rainfall events above the iso-dox cause DO's below 2 mg/l in the stream

4.4 Correction for day/night and summer/winter

Rainfall statistics for the Netherlands show that 30% of the stormwater overflow events on the R. Vecht fully occur during the period of high discharge rates at night. A limited amount of events take place during the winter half-year; the winter-summer ratio is about 25:75. Since the iso-dox obtained previously holds for day and summer conditions only, a correction is needed. This is achieved by constructing iso-doxes for night-in-summer and day-in-winter conditions as well (Figure 6; night-in-winter conditions are not shown; they appear not to cause any DO problems). Again, the number of critical events is counted. Each of these numbers represents the amount of critical events had all rainfall showers taken place during the conditions used to construct the iso-doxes. The corrected annual average can now easily be computed by adhering weights to each condition in proportion to its relative frequency of occurrence. So, summer-day conditions, which are contributing the largest amount of critical events anyway, have a weight of about 53 % (0.7 * 0.75), and so on. It appears that winter overflows rarely cause DO deficit problems in this case.

5 Results

The results for the R. Vecht can be read directly from figures like Figure 6 for each of the options. They are summarized in Table 1. Although expansion of the STP (option 1) leaves a few more overflow events than expansion of the storage in the sewerage system, the effects on the R. Vecht are slightly more favourable, because the increased pumping capacity reduces the severity of the remaining overflow events. Option 1 turns out to be more favourable also from the point of view of average summer conditions outside overflow periods, especially if an adequate nitrification is implemented at the STP. It is, however, clear that minimization of DO problems requires that both options be realized simultaneously. In view of the differences between low flow (day) and high flow (night) operational control of flushing could be an interesting additional possibility to reach even further improvements.

Table 1 Annual stormwater overflow frequency and frequency of occurrence of dissolved oxygen values below 2 mg/l in the stream

Option	annual average frequency of	
	stormwater overflow events	in-stream dissolved oxygen below 2 mg/l
0. no action	40	20
1. expansion of STP	18	4
2. expansion of storage	16	6
3. both 1 & 2	5	2

6 Discussion and conclusion

An engineering method as developed in this paper relies upon the adequacy of the knowledge about processes taking place in the receiving stream during storm water overflows. From a scientific point of view, our knowledge about nitrification, and about coagulation, settling and resuspension during an overflow load is still poor. Also, it is unclear to what extend the SOD will change in response to load reductions. Another factor for further consideration is the assumption that the pollutant load concentration in the overflow water remains the same irrespective of the measures taken.

It is, however, an advantage of the method that it allows the judgement whether a more accurate analysis or further research is really needed from the point of view of management. As the application to the R. Vecht shows, the kind of information required for decision making may prove not to be very sensitive to shortcomings in process descriptions. And if it does, the model-based tailor-made character of the method favours the easy implementation of additional knowledge.

Acknowledgements

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STORM EVENT IN THE GLATT RIVER VALLEY
I. DOCUMENTATION OF RESULTS

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Abstract

Samples collected in a small sewer system, in a waste water treatment plant, in a river and in ground water during an intense storm event allow the discussion of different processes which govern the dynamics of pollutants from their source to their sink. Differences between the behaviour of particulate and dissolved compounds are presented.

1 Introduction

The dry weather situation in rivers is characterized by relatively slow changes of state variables (flow rate and pollutant concentrations). This is in contrast to the situation during storm events:

- rapid changes of low rates result in large gradients of shear stress and therefore erosion and resuspension of sediments on drained surfaces, in sewers and river beds.
- additional pollutants from surface runoff reach sewers and receiving waters.
- sewer systems overflow and deliver untreated waste water to the rivers.
- increased hydraulic loads decrease performance of treatment plants
- elevated water levels and eroded sediments result in increased infiltration of river water to the ground water.
- etc.

All these phenomena are of interest to different research groups but it is very seldom that all aspects are of interest to one group. Due to a variety of activities at EAWAG (Swiss Federal Institute for Water Resources and Water Pollution Control) it was possible to direct the interest of some 40 collaborators from different research projects to a common goal:

To follow the pollutants during a rain event from their source to their sink and thereby gain insight into a whole series of processes as related to rain events in an entire catchment basin.

This report documents some of the results obtained in this project, a more detailed discussion is given by Gujer et al. (1982).

2 The study area

The catchment area of the river Glatt between the effluent of a lake (Greifensee) and its discharge into the Rhine river covers an area of 260 km^2 with a resident population of 240'000, 98 % of them are connected to "state of the art" secondary waste water treatment plants. Dry weather flow increases from about 3 $m^3 sec^{-1}$ to 8 $m^3 sec^{-1}$ along the river, up to 30 % of this flow is treated waste water. The dominant pollutant source is the treatment plant of the north side of the city of Zürich (~ 100'000 PE). In addition to 4 sampling stations on this treatment plant (Fig. 1), 3 sampling stations and several calibrated level gages in the river, 15 rain gages and flow information from other treatment plants are available in the area.

One research project provided a sampling station in a small combined sewer system upstream of the Zürich treatment plant (Fig. 1) and another field project allowed to follow the infiltration of river Glatt into the aquifer. All together a unique setup.

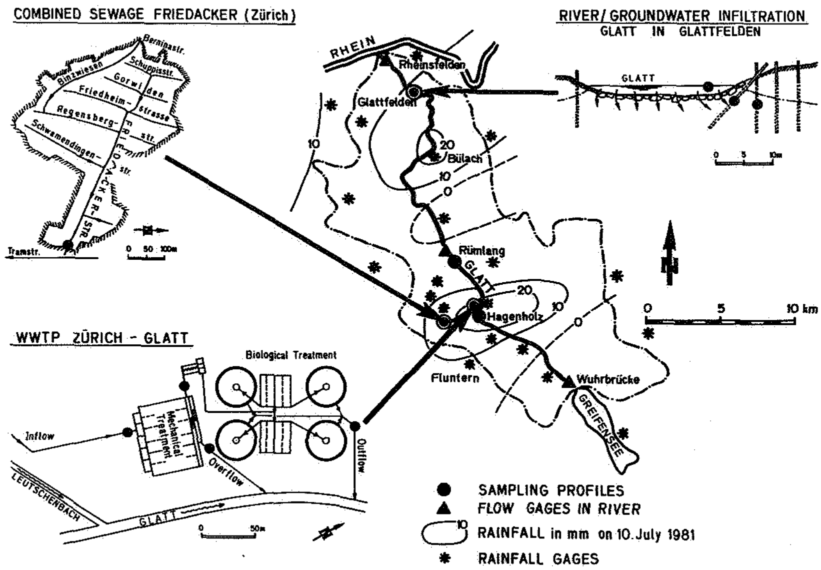

Figure 1 The study area: Glatt River Valley in Switzerland. Overview of sampling stations, cumulative rain fall, rain and flow gages, sampling stations

Table 1 characterizes the catchment areas of the most important sampling stations.

Table 1 Characteristics of catchment areas for the different sampling stations

	Units	Sampling station			
		Sewer system	Treatment Plant	Glatt River	
				Rümlang	Glattfelden
Total area	km^2	0.15	20	110	200
Sewered area total	km^2	0.13	15	33	43
impermeable	km^2	0.05	5.5	9.8	12
Combined sewers	km^2	0.13	12	23	32
impermeable	km^2	0.05	4.0	6.7	86
Inhabitants	-	1600	97000	200000	240000
Length of area	km	0.9	~ 10	~ 15	~ 30
Estimated concentr. time					
dry weather	hrs	0.4	~ 4	6 - 8	15 - 20
storm event	hrs	0.1	1 - 2	3 - 5	7 - 10

3 The rain event

After a dry period of six very warm days the river carried extremely low flow (Q < 5 % probability on a yearly basis, Table 2).

Table 2 Water flow in the River Glatt

Location	Catchment area km^2	Water flow in River Glatt, $m^3 \cdot s^{-1}$		
		Annual mean	Previous to event	Max. during event
Effluent	0	3.3	1.6	1.7
Lake Greifensee				
Hagenholz	70	4.8	2.2	9.0
Rümlang	110	6.0	3.5	18.5
Glattfelden	240	8.3	3.9	15.0
Rheinsfelden	260	8.5	3.9	15.0

On July 10, 1981 a storm occured over the Glatt river valley with variable intensity. Fig. 1 indicates the isohyets of the event, Fig. 2 demonstrates the variability of rain intensities in the area:

- The highest intensity was observed directly on the Zürich treatment plant (WWTP), with 233 l sec^{-1} ha^{-1} (~ 1.4 mm/min) during 15 min, an intensity which is expected only once every 10 years.
- Only 4 km away from the treatment plant (Fluntern) the rain was much less intense.
- In the lower part of the river valley (Bülach) the storm occured 1 hr. later, however this part of the storm resulted in a discharge of water in the river Glatt before the main wave reached the lower river valley (see Fig. 3, and 6, river in Glattfelden). The discussion in this paper is limited to the effects of the rain over the upper part of the valley only.

Most of the precipitation occured over densely populated areas of the City of Zürich whereas large parts of rural areas remained essentially dry: The results are therefore typical for runoff from urban areas only.

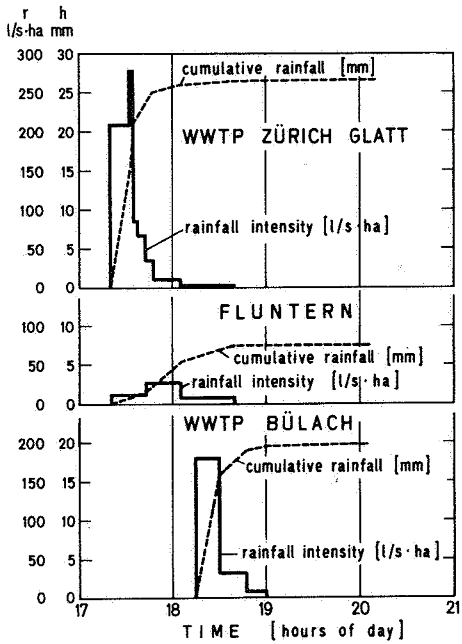

Figure 2 Rain intensities and cumulative rain as observed in three rain gauges (100 l s^{-1} ha^{-1} = 0.6 mm min^{-1})

4 Results

4.1 Water flow and ground water level

Fig. 3 documents the specific flow rate in different locations of the area. Variations are extreme and rapid. With increasing concentration time the peak specific discharge is reduced from > 5 m^3 sec^{-1} km^{-2} in a small sewered urban area down to 0.05 m^3 sec^{-1} km^{-2} for the entire catchment area by a factor of 100. The ground water level close to the river responded with only 1 hr. delay to the increased water level in the river (Fig. 7, Top Figure), indicating significant changes in infiltration rates. However, detailed interpretation of these results is difficult without the aid of mathematical modeling.
Separation of instantaneous flow rates from a base line (Table 3) indicates that 25 % of the additional discharge in the river can be cha-

racterized with samples obtained in the Zürich treatment plant, 88 % of the additional discharge are due to precipitation in the upper part of the river basin.

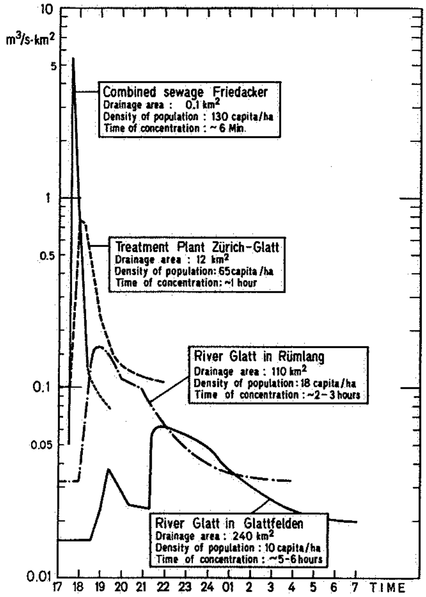

Figure 3 Specific runoff relative to the size of the drainage area during the storm event

Table 3 Characteristics of flood wave of the storm event in different sampling stations

Location	Characteristics				
	Beginning End hrs	Max. flow m^3s^{-1}	$m^3s^{-1}km^2$	Volume m^3 total	Peak[1]
Small sewer system	1726 - 1830	0.8	5.5	735	700
Influent treatment plant	1730 - 2200	7.5	0.63	48000	35000
River in Rümlang	1800 - 0400	18.5	0.17	268000	142000
River in Glattfelden	1830 - 0600	15.0	0.06	326000	161000

[1] After subtraction of base flow

4.2 Comparison of pollutant loads from different size sewer systems

The small sewer area Friedacker (Fig. 1, Table 1) and the catchment area of the Zürich treatment plant differ by a factor of 100 with regard to impervious area and 10-20 with regard to concentration time. Specific (per impervious area) mass flow rates of pollutants differ accordingly during the storm event (Fig. 4). However, integrated over the entire event, the total specific loads do not differ significantly (Table 4).

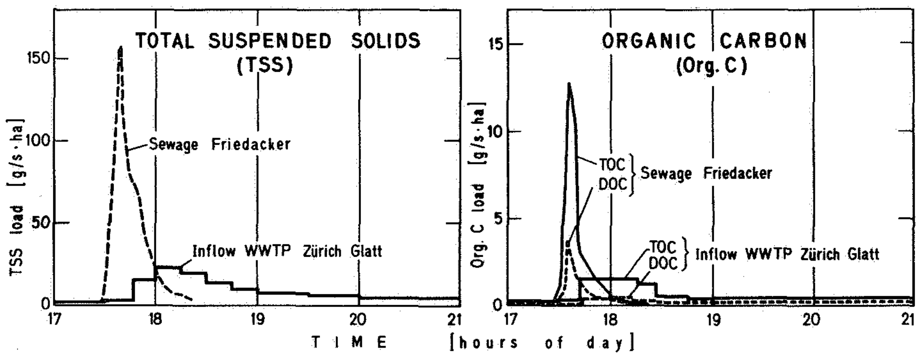

Figure 4 Comparison of pollutant runoff rate from a small sewered area and the entire catchment area of the waste water treatment plant of northern Zurich (1 ha = 10^4 m^2)

Table 4 Comparison of specific load in a small sewer system and in the entire catchment area of the treatment plant (load per 10^4 m^2 of impervious area)

Compound	Load in $m^3 ha^{-1}{}_{imp}$ or kg $ha^{-1}{}_{imp}$		
	Sewer system	Treatment plant	
	observed	observed	estimated total[1]
Combined waste water	137	119	170
Total org. carbon	6.6	5.8	8.2
Dissolved org. carbon	2.4	1.9	2.7
Suspended solids	37.2	27.5	39.3

[1] Corrected or 30 % loss over sewer overflows

The small Friedacker area which served as an experimental area for the investigation of the sources and dynamic behaviour of different pollutants is therefore reasonably representative for the entire sewered area of Northern Zürich. A further analysis of these results is given by Dauber et al. (1984).

4.3 Performance of the waste water treatment plant

A discussion of the observed phenomena in this plant requires a more detailed presentation of the hydraulic regime of this plant: The main sewer upstream of the plant has a large hydraulic capacity (9 m^3 sec^{-1}), about 10 times actual mean dry weather flow. Only an estimated 30 % of the combined waste water was lost during the observed event over overflows. Primary settling tanks have a dry weather capacity of 4500 m^3 of additional storage before their effluent overflows to the river. Biological treatment has a capacity of 1.6 m^3 sec^{-1} and operates under nitrifying condition during summer months.

Table 5 compares the different mass flows of pollutants and water during the storm event and an equivalent dry weather period. The increase of water flow and particulate pollutants during the storm event is substantial.

Table 5 Mass flux of waste water and pollutants during 4 hrs of dry weather and during the storm event in the treatment plant of northern Zürich

Compound	Load in the treatment plant (m^3 resp. kg)				
	Dry weather		Storm event		
	Influent	Effluent	Influent	Overflow	Effluent
Flow	9000	9000	48000	15000	33000
DOC	350	90	750	340	270
TOC	850	120	2300	700	400
TSS	1300	90	11000	2200	540
Pb (lead)	1.5	0.3	98	18	1.5
Cu (copper)	0.3	0.05	8.5	1.8	0.3

Fig. 5 shows the concentrations and the mass fluxes of dissolved as well as particulate pollutants in the waste water treatment plant. Since dissolved pollutants are diluted by storm water, the primary settling tanks contain waste water with higher DOC (dissolved organic carbon) concentration from the dry weather period before the storm event. The increased water flow pushes "older" waste water from the primary tanks over the overflow, which results in a net negative performance of the primary tanks for dissolved compounds. This is indicated by an elevated mass flux for DOC in the discharge of primary effluent to the river as compared to the influent. For particulate pollutants, the primary tanks do provide a net removal during the entire event. Biological treatment does not respond to the decreased concentration of dissolved pollutants in the influent - increased water flow results in significantly reduced percent removal of these compounds.

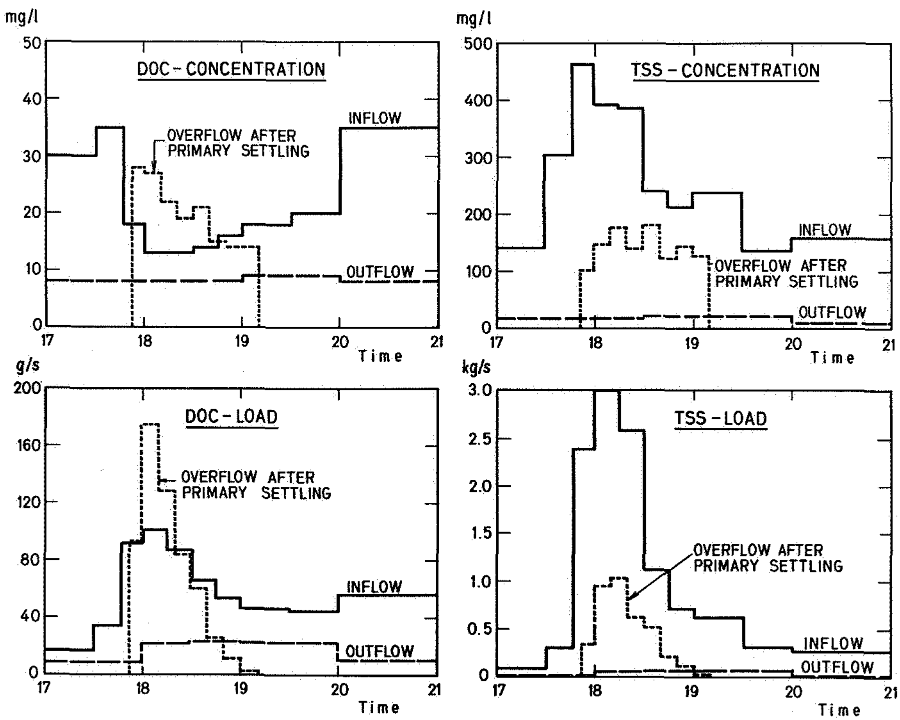

Figure 5 Pollutant concentration and mass flow rate in the waste water treatment plant of northern Zürich, in the inflow, the overflow after primary treatment, and the outflow from secondary treatment during the storm event

As a summary, the dynamics of the pollutants in a conventional treatment plant during a storm event indicates, that overall performance of the plant is poor for dissolved compounds and reasonable for particulate compounds. This is an interesting observation in view of the significant storage volumes generally planned in Swiss sewer systems. It is important to judge the performance of the entire system, storage in sewers together with the capacity and efficiency of the treatment plant, and not only storage in sewers by itself, otherwise the effect of storage may be significantly overestimated.

4.4 Pollutants in the river Glatt

Fig. 6 shows the variation of water flow, the load of suspended solids and ammonium at the different locations along the river Glatt.
The peak load of the suspended solids coincides with the discharge peak of water, which indicates that the river itself is the main source of these solids. The process controlling the solids load is resuspension rather than inputs during the storm event. The contribution of the treatment plant to the suspended solids is insignificant. Particulate organic carbon (POC) behaves similarly to total suspended solids. In Glattfelden (the lowest sampling station in the river) the POC peak load amounted to about 4.5 t of organic carbon. From this load, only about 1.5 t originate from direct input during the storm (0.5 t from the treatment plant surveyed). The difference (3 t POC) is due to resuspended solids that have been sedimented in the river during the dry weather period (6 d) previous to the event. During this period 1 t POC/d had been discharged in form of suspended solids by treatment plant effluent. The 3 t of POC are approximately equivalent to 15 g POC/m^2 of river bed or to a layer of not more than 0.5 mm. Other pollutants sorbed to particles, such as lead, lipophilic organic contaminants, etc. behaved similarly as POC with a tendency towards increased significance of contributions due to direct input during the event.

Ammonium is a dissolved pollutant which is not discharged during dry weather to any significant extent (since the most important treatment plants in the study area were operated in a nitrifying mode during the

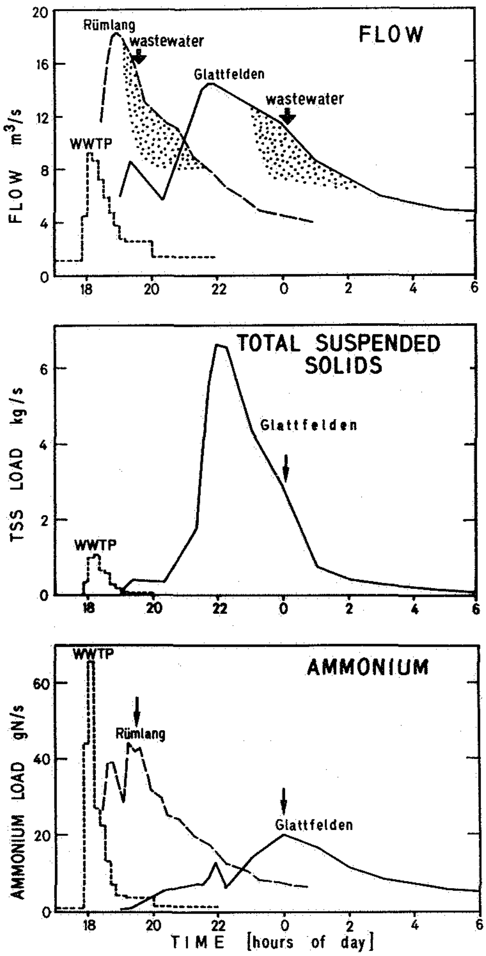

Figure 6 Water flow and mass flow rate of pollutants in the river Glatt and in the total effluent from the treatment plant

event. Ammonium is thefore a good tracer for discharged untreated waste waters. Indeed, the contribution from the treatment plant surveyed is significant. In addition, the peak discharge in Glattfelden is delayed relative to the peak water flow. Other dissolved pollutants such as DOC and volatile organic compounds behaved similarly to ammonium.
In summary, a survey of the dynamics of particulate and dissolved pollutants during a storm event in a river indicates that improved retention

of particulate pollutants during storm events would only marginally improve water quality in the river. The dissolved pollutants, which are ecologically more significant, but also more difficult to reduce, are to a significant extent contributed to receiving waters during storm events (a somewhat ironic situation). Improved waste water treatment during dry weather periods (e.g. filtration) could significantly reduce particulate pollutants due to resuspension during peak river flows. This effect should not be underestimated if dissolved oxygen is of concern, since resuspended sediments may effect a significant oxygen demand.

4.5 Pollutants in the ground water

Increased turbulence and erosion of the river bed together with increased water levels during peak discharge lead to higher permeability of the river bed and higher pressure gradients between the river and the ground water, which again results in accelerated infiltration of surface water to the ground water aquifer. Since dissolved pollutants are present at elevated concentrations in rivers during peak flow, significant amounts of these compounds may percolate into the ground water. The fate of organic contaminants during infiltration is controlled by a series of physicochemical and microbiological processes such as dispersion, adsorption, biological transformation and degradation, etc.

Fig. 7 illustrates the observed results for two different specific organic trace contaminants. 2-Heptanon, which has never been observed in the river before this event, and therefore does not have a history in the aquifer, appears with a delay of only 5.5 hrs in the ground water 2.5 m from the river sediment. Based on physicochemical information it is possible to state, that 2-Heptanon is only slightly retarded in sediment and aquifer material relative to water (Schwarzenbach and Westall, 1981). Contrary to 2-Heptanon, 1,4-Dichlorobenzene is significantly retarded due to adsorption, the response of the ground water concentration is therefore close to nil.

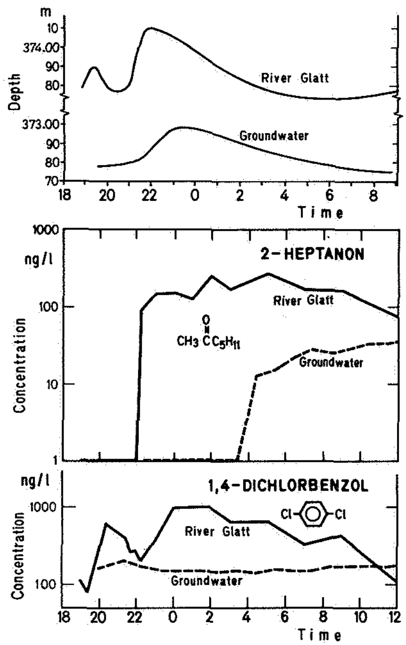

Figure 7 Variation of water depth and pollutant concentrations in the river Glatt and the nearby ground water (2.5 m from river bank)

5 Conclusions

Data collected during the rain event of July 10, 1981 in the Glatt river valley allowed to characterize the dynamics of particulate as well as dissolved pollutants between their source and the effluent of the river basin. Specific conclusions from this study are:

1) Information from small sewered areas may be used to estimate pollutant loads from larger, similar sewer systems.
2) Treatment of combined waste water may be efficient for the reduction of particulate pollutants, the treatment concept chosen in the Zürich treatment plant, however, resulted in a net increase of discharged dissolved pollutants when compared with the influent to the plant during the entire event.
3) The efficiency (percent removal) of biological treatment is signifi-

cantly reduced during storm events as compared to dry weather situations.

4) During high flow, pollutants carried in the river have three important sources:
   - Resuspended sediments from the river bed, which were deposited into the river prior to the storm event.
   - Suspended and dissolved compounds carried into the river due to storm water overflow and surface runoff.
   - Pollutants contained in the treatment plant before the storm event.

   The dominant source depends strongly on the characteristics of the pollutant. For many particulate compounds the first source is important, for many dissolved compounds the last source dominates.
5) Dissolved compounds which have a high mobility in ground water aquifers are subject to important variations in their concentration in the ground water near to the river.
6) The results of this study clearly indicate that the means to control pollution during storm events must be chosen very specifically for the compounds of interest. For some particulate pollutants additional waste water treatment during dry weather may be the most efficient mean for their reduction, whereas carefully planned retention storage and subsequent treatment at low flow rates may be efficient for some dissolved compounds.

As a whole it appears that the presented analysis of the reported results only touches the surface, a more sophisticated analysis requires a significant additional effort. The following paper indicates a possible route for such additional analysis.

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STORM EVENT IN THE GLATT RIVER VALLEY
II. DATA ANALYSIS BY USE OF A DYNAMIC MATHEMATICAL MODEL

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Abstract

A mathematical model designed to calculate the propagation of a flood wave and transport, transfer and transformation of various pollutants in a river is presented. It is used for the analysis of the storm event data of the Glatt river described in the first part of this report. Several compounds have been observed simultaneously; those with known behaviour serve to calibrate the model, which then is used to analyse the behaviour of other substances. This procedure leads to a consistent simulation of the dynamics of the observed pollutants and provides reasonable values for the parameters of transport and transformation processes.

1 Introduction

A lot of work has been done on water quality of rivers during dry weather periods, however, little is known about the effects of flood waves due to storm events on the ecology of a river. In order to gain insight into the dynamics of pollutants in a river under fast varying conditions, a local storm event in the Glatt river valley near Zürich (Switzerland) has been observed. A description of this event and a documentation of the results are presented in the first part of this report (Gujer et al. (1986)).
For the analysis of the extensive data and for the prediction of the

behaviour of pollutants in non-steady state situations a mathematical model is an indispensable tool. This part of the report describes data analysis by means of a mathematical model, which calculates the propagation of a flood wave and the transport, transfer and transformation of various physical, chemical or biological quantities in a river.

2 Description of the Model

Fast mixing over depth and width of the river is assumed and only longitudinal variations of the cross sectional averaged quantities are considered in the model. Experimentally determined transverse diffusion coefficients show that lateral mixing distances are short and that the one dimensional model provides an appropriate simplification of reality.

2.1 Hydraulics

The dynamics of unsteady flow in open channels with gradually varying geometry is described by the well known St. Venant equations (Henderson (1966), Strelkoff (1969)):

$$\frac{\partial y}{\partial t} = - \frac{1}{b} \frac{\partial Q}{\partial x} \tag{1}$$

$$\frac{\partial Q}{\partial t} = - \frac{\partial}{\partial x} \left(\frac{Q^2}{A} \right) + g\, A \left(- \frac{\partial y}{\partial x} + S_0 - S_f \right) \tag{2}$$

where

$$S_f = \frac{1}{K^2} \frac{1}{R^{4/3}} \frac{Q \cdot |Q|}{A^2} \tag{3}$$

and

x = coordinate in flow direction (L)
t = time (T)
y = water depth (L)
Q = discharge (L^3T^{-1})
A = area of cross section (L^2)

b = width at water level (L)

P = wetted perimeter (L)

R = A/P = hydraulic radius (L)

S_0 = slope of river bed (-)

S_f = friction slope according to Manning-Strickler (-)

K = friction coefficient according to Strickler ($L^{1/3}T^{-1}$)

g = gravitational acceleration (LT^{-2}) .

Equation (1) is the continuity equation which describes conservation of mass. Equation (2) is the equation of motion. It includes the effects of gravity as driving force by the terms S_0 and $\partial y/\partial x$ and it considers the friction between river bed and water body by the empirical formula (3) of the friction slope S_f.

Equations (1) and (2) hold as long as the geometry of the river bed and the hydraulic variables vary only gradually. Thus, the river has to be divided into reaches, bounded by discontinuities such as drops, weirs, tributaries or other sudden changes of geometry. For each reach two boundary conditions are needed to obtain a unique solution of the equations. Since in our case the flow is always subcritical, one boundary condition has to be specified at the upstream end and the other at the downstream end of each reach (a thorough discussion of boundary conditions and their numerical implementation is given by Liggett and Cunge (1975)). The upstream boundary condition is given by the inflowing discharge Q; the downstream boundary condition depends on the type of the discontinuity. For large drops or high weirs usual weir formulas (Henderson (1966)) provide a relation between water depth and discharge at the end of the reach. This relation yields a true boundary condition, which allows separate mathematical treatment of the reaches. Otherwise these formulas connect the hydraulic variables of adjacent reaches at their common boundary and represent internal boundary conditions which do not uncouple the reaches.

As initial condition for equations (1) and (2) an appropriate steady state solution is applied, but any measured spatial profile of the hydraulic variables could also be used.

The nonlinear system of partial differential equations (1) and (2) is solved by an implicit finite difference method. The chosen technique simplifies the method of Amein and Fang (1970). This reduction yields first instead of second order accuracy in time, but only requires a system of linear instead of nonlinear equations to be solved for each time step. The method is of second order accuracy in space.

2.2 Transport, Transfer and Transformation Processes

The river is divided vertically into two compartments: One represents the body of flowing water, the other the benthic zone. Variables in the upper compartment are the concentrations of dissolved or particulate substances which are determined by advection, dispersion, transfer and transformation processes. Variables in the benthic compartment are the concentrations of substances contained in dead zones of the river or in the water of porous sediment layers, or they represent areal densities of adsorbed substances or of fixed biomass. They are controlled by transfer or transformation processes only. As "transfer or transformation processes" we define the sum of all sources and sinks of a substance in a compartment, i.e. all its chemical reactions or biological transformations, but also its exchange between the compartments, with the atmosphere or with the groundwater.
These processes are described mathematically by the following system of differential equations:

$$\frac{\partial (Ac_i)}{\partial t} = - \frac{\partial (Qc_i)}{\partial x} + \frac{\partial}{\partial x} (AE \frac{\partial c_i}{\partial x}) + AS_i \qquad i=1,\ldots,n \tag{4}$$

$$\frac{\partial (Pd_j)}{\partial t} = Ps_j \qquad j=1,\ldots,m \tag{5}$$

where

c_i = concentration of substance i in the flowing water compartment (ML^{-3})

d_j = areal density of substance j in the benthic zone (ML^{-2})

n = number of variables in the flowing water compartment (-)

m = number of benthic variables (-)

E = coefficient of longitudinal dispersion (L^2T^{-1})

S_i = transfer or transformation rate of variable i in the flowing water compartment ($ML^{-3}T^{-1}$)

s_j = transfer or transformation rate of variable j in the benthic compartment ($ML^{-2}T^{-1}$) .

The hydraulic variables are defined in section 2.1.

The terms on the right hand side of the transport equation (4) represent advection, dispersion and transfer or transformation processes of substances in the flowing water compartment; the right hand side of equation (5) describes transfer or transformation processes of benthic substances. The rates S_i and s_j may be functions of any c_i, d_j, Q, y, A, b, P, x and t. The longitudinal dispersion coefficient E is calculated according to Fischer et al. (1979) by the formula

$$E = C \frac{Q^2}{A^2} \frac{b^2}{u_* y} \qquad (6)$$

with

$u_* = \sqrt{gRS_f}$ = shear velocity (LT^{-1})

C = 0.011 .

For each reach and each variable c_i two boundary conditions are needed. As an upstream boundary condition continuity of the mass flux is used; the downstream boundary condition is obtained by neglection of dispersion. As initial condition usually a steady state solution is applied.

To avoid well known numerical problems with advection dominated flow a third order accurate method is chosen to approximate the spatial derivatives (Leonard (1979)). For the time derivative a two step predictor-corrector technique is used, which is of second order. This combination leads to an implicit finite difference scheme, which is based on four spatial grid points.

3 Calibration of the Model

The model has been calibrated for the Glatt river, a small heavily polluted river with a discharge in the order of 10 m^3/s, a width of about 10 m and a bed slope of 0.03 - 0.4 %. First calculations have been performed with geometric data from the river bed which was available from construction plans of various corrections of the river and with estimated friction coefficients obtained by visual inspection of the river bed. A sensitivity analysis revealed that the precision of the calculation is limited mainly by the accuracy of the friction coefficients and the discharge data. Inaccuracies due to geometric data are one order of magnitude smaller.

To adjust the friction coefficients K in formula (3), to test formula (6) for longitudinal dispersion and to verify the assumptions of short lateral mixing distances, a tracer experiment was performed: For 13 minutes rhodamine B was added at a rate of 1.2 g/min in the middle of the river. The concentration in the river was measured with a fluorescence sensitive probe about one hundred meters below the injection point. A transverse profile showed almost complete cross sectional mixing. A recording of the passage of the rhodamine about 9 km downstream was used to adjust the friction coefficients. It appeared that besides a small loss, a second process, which indicates a reversible exchange of the dye with dead zones of the river was necessary to fit the data properly. The calculation uses equations (4) and (5) with the transfer rates

$$\begin{aligned} S_{RHO} &= -\frac{P}{A}(k_1 c_{RHO} - k_2 d_{RHO}) - \frac{P}{A} k_A c_{RHO} \\ s_{RHO} &= k_1 c_{RHO} - k_2 d_{RHO} - \frac{k_2}{k_1} k_A d_{RHO} \end{aligned} \qquad (7)$$

where c_{RHO} is the concentration of rhodamine in the flowing water compartment and d_{RHO} the areal density of rhodamine in the dead zones. The areal density d_{RHO} can easily be converted into the concentration in the dead zone by k_1/k_2, which can be interpreted as a mean depth of the dead zone. The rate constants k_1 and k_2 describe the exchange with dead zones. The parameter k_A quantifies the loss terms, which are taken to be proportional to the ratio of wetted surface to volume P/A and to the inverse depth of the dead zone k_2/k_1, respectively. The values of the rate constants $k_1 = 1.5 \cdot 10^{-5}$ m/s, $k_2 = 2.5 \cdot 10^{-4}$ s^{-1} and $k_A = 2.5 \cdot 10^{-6}$ m/s yield a very good fit between calculated and measured concentrations, as shown in Figure 1. It is also shown that a calculation without an exchange process cannot produce an asymmetry large enough to fit the "tail", which typically is observed in the presence of dead zones.

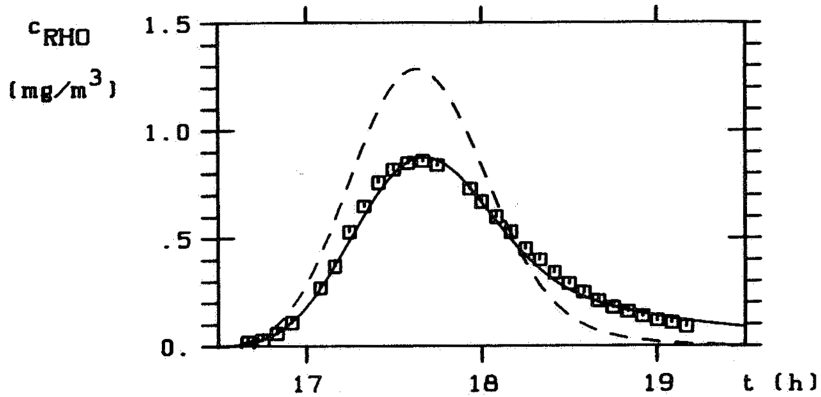

Figure 1 Comparison of measured and calculated concentrations of a rhodamine B pulse 9 km below the injection point. The calculation (solid line) indicates a loss of about 5% and an exchange of the dye with dead zones. The dashed line shows the same calculation without the exchange process

Infiltration into the groundwater, even if it might be locally important for the aquifers, was assumed to have only a minor effect on the hydraulics and pollutant concentrations in the river, and was not taken into account in the model.

4 Simulation of Flood Wave and Pollutants

4.1 Propagation of the Flood Wave

The hydraulic data base consists of four hydrographs of the Glatt recorded at Hagenholz (9.3 km below Greifensee), at Rümlang (14.8 km), at Glattfelden (30.6 km) and at Rheinsfelden (35.1 km). The measured discharge of the wastewater treatment plant in Zürich (9.6 km) is only of limited use for our simulation because a tributary discharging into the Glatt at the same location was not recorded. The contribution of this tributary was calculated from the hydrograph recorded at Rümlang. The discharges of four minor tributaries, which also have not been measured directly, were estimated from data collected in the catchment area. The hydrographs recorded at Rümlang and Rheinsfelden were used to determine the friction coefficients. It turned out that the friction was smaller for the flood wave than it was during the dry weather tracer experiment described in section 3. This may be due to the difference in water level or to the effect of plants in the river, the strong seasonal variation of which represent a well known phenomenon of this river.

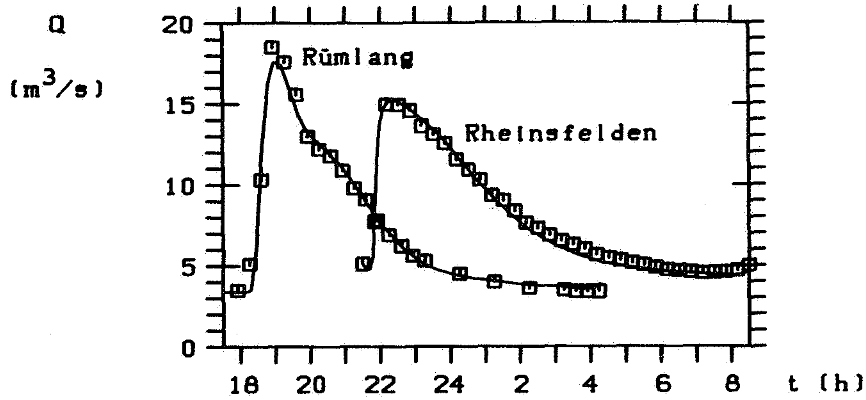

Figure 2 Comparison of measured and calculated discharge at Rümlang (14.8 km) and at Rheinsfelden (35.1 km)

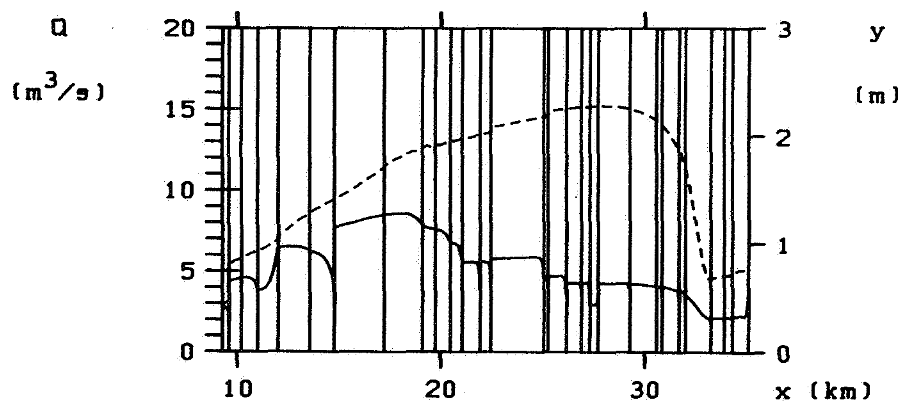

Figure 3 Calculated discharge Q (dashed line) and water depth y (solid line) at 9^{30} pm (July 10, 1981) between Hagenholz and Rheinsfelden

The correspondence of measured and calculated values of discharge at Rümlang and at Rheinsfelden is shown in Figure 2. Figure 3 illustrates the spatial expansion of the flood wave and the division of the observed river section into 28 reaches. Tributaries are shown as discontinuities of discharge, drops and weirs as discontinuities of water depth and corresponding backwater curves.

4.2 Transport, Transfer and Transformation of Pollutants

Since there are no essential contributions to the flood wave from Zürich below Rümlang, this section of the river is suited best for modeling the transport and transformation processes. At Rümlang (14.8 km) and at Glattfelden (30.6 km) volatile organic chloride compounds (FOCL), dichlorobenzene (DCB), perchlorethylene (PER), m-xylene (XYL), toluene (TOL) and ammonia (NH_4) have been measured and are modelled in the following subsections. The analysis of the data of lead and suspended solids is not presented in this report.

4.2.1 Volatile Organic Chloride Compounds

The dominant transformation process of volatile organic chloride compounds, dichlorobenzene and perchlorethylene is gas transfer to the atmosphere. This transfer forms a sink which is proportional to the concentration c_i of the compound and to the ratio of surface to volume b/A of the river. In addition an exchange of the substances between flowing water and dead zones is considered, analogous to equation (7) used for the evaluation of the tracer experiment. Gas exchange with the atmosphere is assumed to affect the concentration in the flowing water compartment only, as dead zones are assumed to occur near the bed of the river. This yields the following transfer rates for volatile compounds:

$$\begin{aligned} S_i &= -\frac{P}{A}(k_1 c_i - k_2 d_i) - \frac{b}{A} K_L c_i \\ s_i &= \quad k_1 c_i - k_2 d_i \end{aligned} \tag{8}$$

where K_L is the velocity of gas transfer. Figure 4 shows that a reasonable fit of the data is possible; systematic differences between measured concentrations and the calculations are thought to be due to a gas transfer mechanism via bubbles which depends on the respective Henry constants of the compounds.

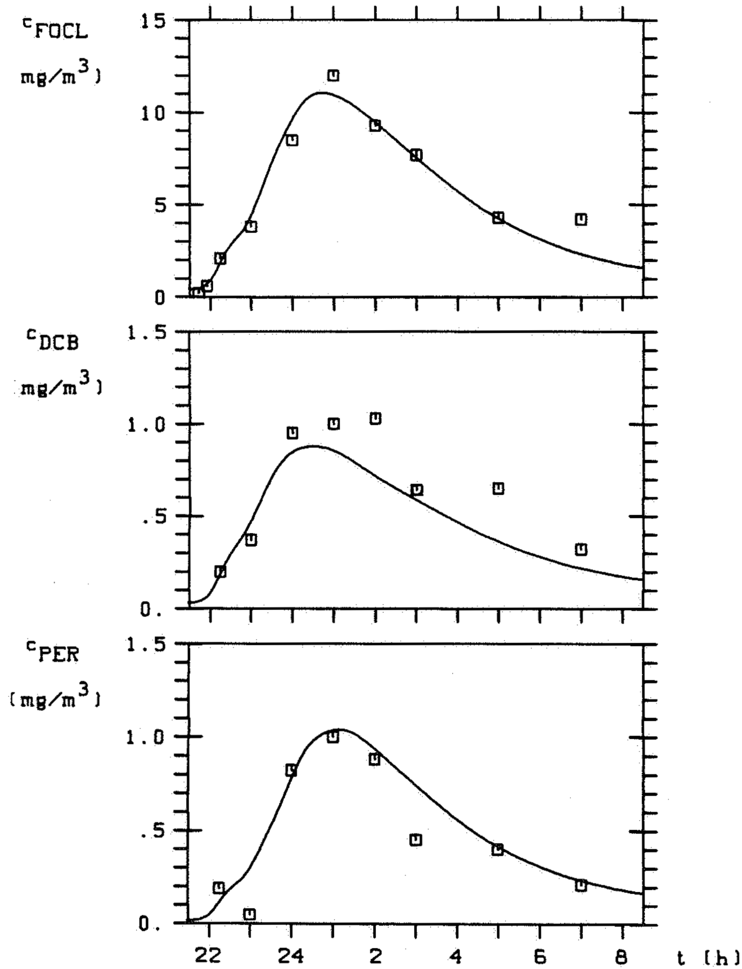

Figure 4 Comparison of calculated concentrations of volatile compounds with measured values at Glattfelden (30.6 km)

The parameter values used for the calculations are $k_1 = 1 \cdot 10^{-4}$ m/s, $k_2 = 5 \cdot 10^{-4}$ s^{-1} and $K_L = 3 \cdot 10^{-5}$ m/s. Comparison of the values of k_1 and k_2 with those determined for dry weather by the tracer experiment described in section 3, reveals that the delaying effect of dead zones seems to be even more important during flood waves. The value of K_L represents an average over all reaches between Rümlang and Glattfelden. It is smaller than the value of $K_L = 6.8 \cdot 10^{-5}$ m/s determined in a previous experiment with DCB and PER (Schwarzenbach (1983)). This difference is to be expected, because the previous experiment had been performed in a reach, where small drops increase the turbulence of the river.

4.2.2 Xylene and Toluene

These compounds are volatile and are assumed to be biologically degraded mainly by benthic biomass. Due to their high concentration in the flood wave, absorption from the atmosphere can be neglected. To account for the degradation process an additional sink term is included in equation (8). For the flowing water compartment this sink term is proportional to the ratio of benthic surface to volume of the water body P/A, and for the dead zones it is proportional to their inverse mean depth k_2/k_1. The resulting transfer and transformation rates of xylene and toluene are

$$
\begin{aligned}
S_i &= -\frac{P}{A}(k_1 c_i - k_2 d_i) - \frac{b}{A} K_L c_i - \frac{P}{A} k_B^i c_i \\
s_i &= \qquad k_1 c_i - k_2 d_i \qquad\qquad - \frac{k_2}{k_1} k_B^i d_i .
\end{aligned}
\tag{9}
$$

For the coefficients k_1, k_2 and K_L the values determined in section 4.2.1 are used; thus, the degradation rate k_B^i remains as the only parameter to be adjusted in order to fit the experimental data. Figure 5 compares measured concentrations of xylene with the calculation. No degradation was required to fit the xylene data, but it cannot be excluded that this is an effect of additional unknown sources, since the degradation rates of toluene and m-xylene are expected to be of the same order of magnitude (Kappeler (1976)). For toluene the degradation rate was determined to be $k^{TOL} = 2.5 \cdot 10^{-5}$ m/s. The calculation is shown in Figure 6.

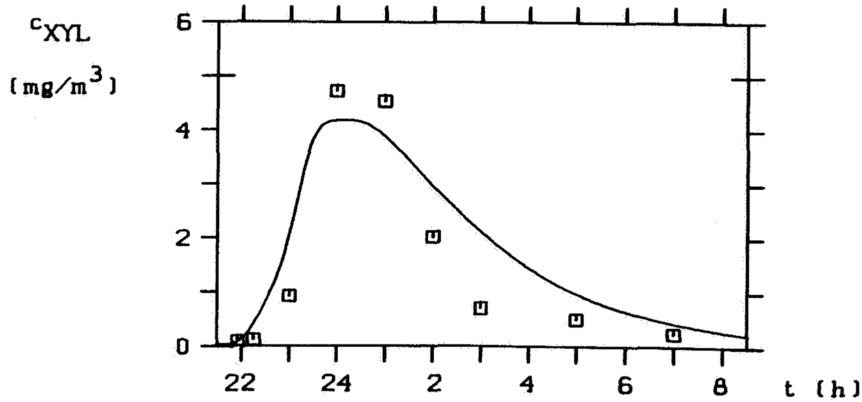

Figure 5 Comparison of concentrations of xylene measured at Glattfelden (30.6 km) with a calculation without biodegradation

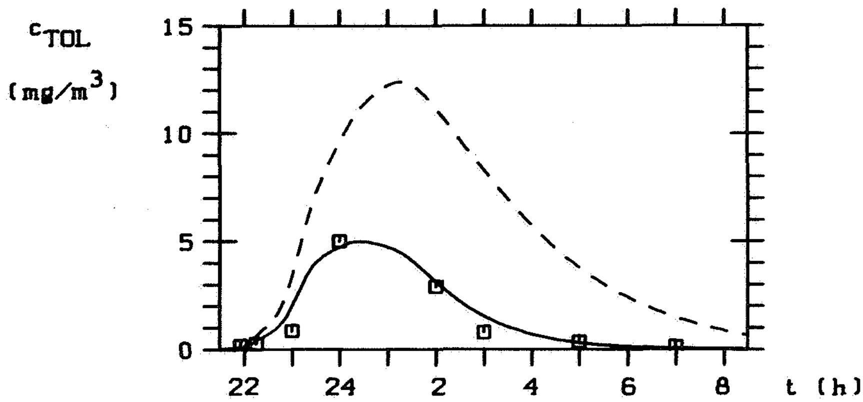

Figure 6 Comparison of calculated concentrations with biological degradation (solid line) with values of toluene measured at Glattfelden (30.6 km). The dashed line shows the same calculation without degradation

4.2.3 Ammonia

Since nitrite and nitrate have not been measured at Rümlang, we cannot establish a full model of nitrification and are restricted to the simulation of the degradation of ammonia. Elimination of gas transfer from equations (9) yields the following transfer and transformation rates for the case of ammonia-nitrogen:

$$S_{NH_4} = -\frac{P}{A}(k_1 c_{NH_4} - k_2 d_{NH_4}) - \frac{P}{A} k_B^{NH_4} c_{NH_4}$$
$$s_{NH_4} = k_1 c_{NH_4} - k_2 d_{NH_4} - \frac{k_2}{k_1} k_B^{NH_4} d_{NH_4}. \qquad (10)$$

The value of the degradation velocity of ammonia was determined as $k_B^{NH_4} = 6 \cdot 10^{-6}$ m/s. This value is of the same order of magnitude as $9 \cdot 10^{-6}$ m/s calculated from nitrogen mass balances which have been estimated for July 1974 by Gujer (1976). As can be seen in Figure 7 a good fit of the experimental data is possible even if a loss of benthic biomass triggered by the flood wave and supposed by Gujer et al. (1982) to be significant has not been included into the model.

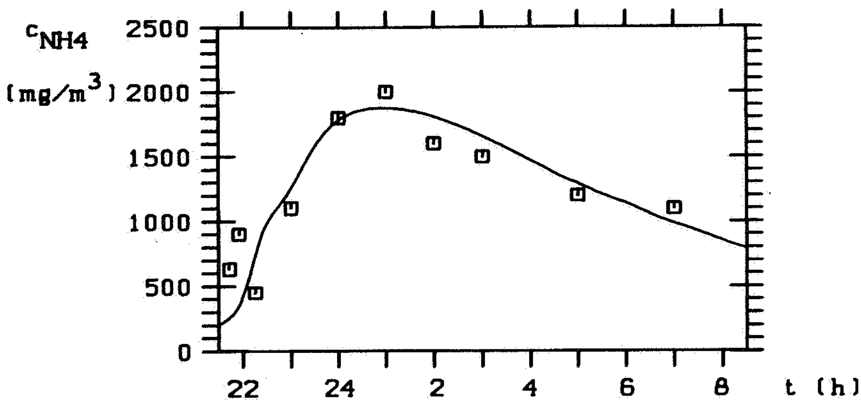

Figure 7 Comparison of calculated concentrations of ammonia-nitrogen with measured values at Glattfelden (30.6 km)

5 Conclusions

The developed dynamic mathematical model has allowed a thorough analysis of the data collected during the storm event described in the first part of this report (Gujer et al. (1986)). With a minimum number of adjustable parameters a consistent simulation of the observed phenomena and a good fit of the experimental data was possible.

The simultaneous observation of several quantities and compounds at various locations in the river provided a data base large enough to allow calibration of the model, analysis of the dominant processes and identification of their parameter values. The most important results are:

- The model revealed that dead zones strongly affect transport of substances in the river and allowed to quantify the exchange between dead zones and flowing water.
- For several compounds the model yielded volatilization and degradation rates; the obtained values are in good agreement with values measured in previous investigations or taken from the literature.

The presented model may now serve as a tool for investigations on pollutants of unknown behaviour and on the effects of storm events on the water quality of the Glatt river.

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We would like to thank our colleagues from EAWAG for their help and especially Prof. N.H. Brooks for his valuable advice.

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A DETERMINISTIC MODEL FOR DO AND SS IN A DETENTION POND RECEIVING COMBINED SEWER OVERFLOWS

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Abstract

A number of CSO-events and their effects upon the water quality in the receiving detention pond were measured and sampled intensively. A model was developed for the analysis of the effects on dissolved oxygen (DO) and suspended solids (SS). The model parameters were estimated from field data. Conductivity measurements served to calibrate the parameters controlling mixing. Settling and deoxygenation rates were subsequently based on observed concentrations of suspended solids and oxygen and additional measurements. Although it is difficult to model the complex mixing phenomena in the pond in detail, the overall description of SS and DO is satisfactory.

1 Introduction and Objectives

During the last century the quantitative aspects of stormwater runoff have been studied extensively. Several theories and models have been developed for the assessment of runoff flow, flow routing through sewer systems and the estimation of overflow frequencies, resulting in design criteria for sewerage systems. The attention for the quality effects of runoff overflows and the effects upon receiving waters is of a more recent date.

In the Netherlands in 1982 a comprehensive study was initiated by the NWRW (National Working Group on Sewerage and Water Quality) on rainfall, runoff, interception, overflows and effects on receiving waters.
The research presented here is a part of this research program and deals with the effects of CSO's on DO and SS in the receiving water. The objective is an analysis of the processes that take place in the receiving water. Other effects studied are:

- Hydrobiological effects as reflected by macrofauna (Willemsen and Cuppen, 1986) and by plankton and epiphytes (Roijackers and Ebbeng, 1986).
- Microbial effects (Lijklema et al., 1986).

2 Research site and sampling

The location studied is a detention pond of 120 x 35 m with a mean depth of 1 m, receiving CSO from the small village Loenen (Figure 1). The system drains about 16 hectares impervious area and serves a population of more than 2,000 people and a few local industries.
The pond is flushed by seepage (9.3 cm.day^{-1}), rich in iron. The average hydraulic detention time is about 10 days.

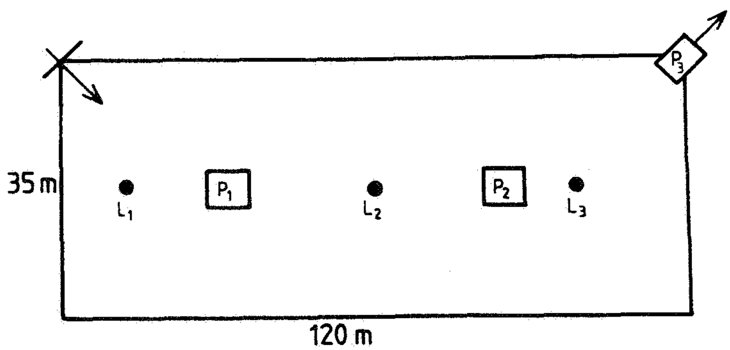

Figure 1 The pond in Loenen
P_1, P_2 and P_3: platforms
L_1, L_2 and L_3: sampling sites

During an overflow event the flow of the discharge was measured continuously and volume-proportional samples were taken. The automatic sampler in the overflow well also actuated samplers on platforms P_1 and P_2 in the pond and at the outlet P_3. At these locations also dissolved oxygen and conductivity were measured continuously. The samples taken were analysed for BOD_5 and suspended solids.
The water leaves the pond over a weir. The verified weir formula and the water level registered by a gauge served to calculate the water balance during an overflow.

3 Governing equations

The model is based on the two-dimensional advection-diffusion equation:

$$\frac{\delta C}{\delta t} = \frac{\delta}{\delta x}\left(E_x \cdot \frac{\delta C}{\delta x} - U_x \cdot C\right) + \frac{\delta}{\delta z}\left(E_z \cdot \frac{\delta C}{\delta z} - U_z \cdot C\right) + \Sigma S_i \qquad (1)$$

where

C	concentration	(M/L^3)
E_x, E_z	horizontal and vertical dispersion coefficients	(L^2/T)
U_x, U_z	horizontal and vertical velocity	(L/T)
x, z	horizontal and vertical dimension	
S_i	sources and sinks	$(M/L^3,T)$

The selection of a two-dimensional system was based on both visual observations and on measured effects during overflow events. An eye witness of an overflow event could clearly see that a polluted waterfront moved as a plug in the x-direciton. Measured profiles of temperature and conductivity over the z-direction showed that the polluted inflow moved as a layer over the relatively clean pond water before a gradual vertical mixing started. The stream velocity in the x-direction depends on the inflow and outflow rates. The inflow is a combination of the constant seepage and the accidental overflow events. The outflow is related to the waterlevel in the pond according to:

$$H_w \leqslant H \leqslant H_c: \quad Q_{out} = K_1 \cdot (H-H_w)^{3/2} \qquad (2a)$$

$H > H_c$: $$Q_{out} = K_1.(H_c-H_w)^{3/2} + K_2.(H-H_c)^{1/2} \quad (2b)$$

where

H	water level in the pond	(L)
H_w	level of outflow weir	(L)
H_c	level of upper boundary of outlet construction	(L)
Q_{out}	outflow rate	(L^3/T)
K_1, K_2	constants	

The constants K_1 and K_2 were estimated from the results of an experiment in which the waterlevel in the pond was raised artificially to 0,40 m above the outflow weir by blocking the outlet during 4 days. The observed drop of the water level after removing the blockage can be simulated using the overall water balance of the pond.
The best fit was obtained with values for K_1 = 3000 $[m^{3/2}/h]$ and K_2 = 630 $[m^{5/2}/h]$, as illustrated in Figure 2.

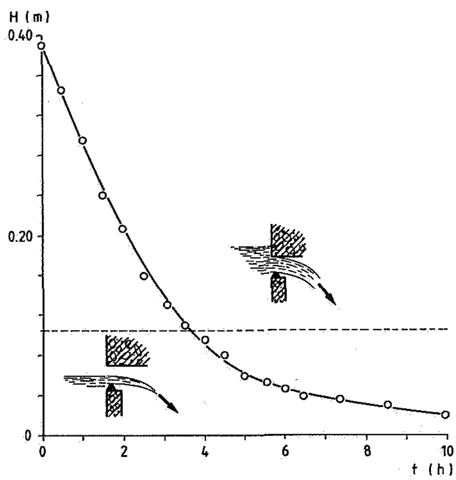

Fig. 2 Simulation of drop of the water level compared to measured waterlevel

In order to solve equation 1 numerically, the pond is modelled as a series of compartments, each divided into a lower part with constant volume and an upper part with variable volume. This results in a differential scheme. The relevant terms are summarised in Table 1.

Table 1 Contributing terms in differential scheme representing processes of advection (1,3), dispersion (2,4), sedimentation (5), decay (6), reaeration (7) and sediment oxygen demand (8).

	$C_i^{n+1} = C_i^n + \Delta C_i^n \cdot \Delta t$	ΔC_i^n	ΔC_j^n	ΔSS_i^n	ΔSS_j^n	ΔLP_i^n	ΔLP_j^n	ΔLS_i^n	ΔLS_j^n	ΔDO_i^n	ΔDO_j^n
(1)	$[U_{xi}^n \cdot C_i^n - U_{xi-1}^n \cdot C_{i-1}^n]/\Delta X$	-		-		-		-		-	
(2)	$[E_{xi}^n \cdot (C_{i+1}^n - C_i^n) - E_{xi-1}^n \cdot (C_i^n - C_{i-1}^n)]/\Delta X^2$	+		+		+		+		+	
(3a)	$[U_z \cdot C_j^n]/H_{upp}^n$	+		+		+		+		+	
(3b)	$[U_z \cdot (C_{seep} - C_j^n)]/H_{low}$		+		+		+		+		+
(4)	$[E_{zi}^n \cdot (C_j^n - C_i^n)]/\Delta Z^2$	+	-	+	-	+	-	+	-	+	-
(5a)	$[V_s \cdot (C_{si}^n - C_{ba})]/H_{upp}^n$			-		-					
(5b)	$[V_s \cdot (C_{sj} - C_{ba}) - V_s \cdot (C_{si} - C_{ba})]/H_{low}$				-		-				
(6a)	$RKD \cdot (LS_i^n - L_{ba})$							-		-	
(6b)	$RKD \cdot (LS_j - L_{ba})$								-		-
(7)	$RKA \cdot (Ox_s - DO_i^n)/H_{upp}^n$									+	
(8)	SOD/H_{low}										-

index i	number of upper compartment	
index j	number of lower compartment	
index n	number of time step	
ΔX	distance between midpoints of sequential compartments	(L)
ΔZ	distance between midpoints of upper and lower layer	(L)
Δt	time step	(T)
ΔC	value of $\delta C/\delta t$	$(M/L^3, T)$
C	concentration	(M/L^3)
C_s	concentration of settleable material	(M/L^3)
C_{ba}	background concentration	(M/L^3)
C_{seep}	concentration in seepage	(M/L^3)
SS	concentration of suspended solids	(M/L^3)
LP	concentration of settleable BOD_∞	(M/L^3)
LS	concentration of soluble and poorly settleable BOD_∞	(M/L^3)
L_{ba}	background concentration of soluble BOD_∞	(M/L^3)
DO	concentration of dissolved oxygen	(M/L^3)
Ox_s	saturation concentration of dissolved oxygen	(M/L^3)
U_{xi}	stream velocity from compartiment i to i+1	(L/T)
U_z	vertical stream velocity in lower layer	(L/T)
E_x, E_z	horizontal and vertical dispersion coefficients	(L^2/T)
H_{low}	thickness of lower layer	(L)
H_{upp}	thickness of upper layer	(L)
V_s	sedimentation rate	(L/T)
RKD	removal rate of LS	(1/T)
RKOX	oxidation rate of LS	(1/T)
RKA	reaeration rate	(L/T)
SOD	sediment oxygen demand	$(M/L^2, T)$

The lower part of the compartments are assumed to be stagnant with respect to horizontal flow and dispersion. Hence this layer is subject to flow with velocity U_z equal to the specific seepage rate q_s. Assuming that at all times the waterlevel is equal in all compartments (disregarding wave propagation), the flow rates through the boundaries between linked comparments i and i+1 can be computed by linear interpolation between the inflow rate Q_{in} at x = 0 and the outflow rate Q_{out} at x = L, according to:

$$Q_i = Q_{in}.(1-i).\Delta x/L + Q_{out}.i.\Delta x/L \tag{3}$$

where

Q_i	flow rate through boundary between compartment i and i + 1	(L^3/T)
Δx	length of compartment	(L)
L	length of the pond	(L)

The horizontal water velocity U_{xi} in the upper layer can be computed as the flowrate Q_i divided by the cross sectional wet area of the upper layer.

Horizontal dispersion can be modeled according to term 2 in Table 1. An additional dispersion is introduced as a result of the integration method used (Euler). This so called numerical dispersion can be substracted from the real physical dispersion and is approximately (Bella and Bobbins, 1968):

$$E_{num_i}^{\ n} = U_{x_i}^{\ n} \ . \ (\Delta X - U_{x_i}^{\ n} \ . \ \Delta t)/2 \tag{4}$$

For computional reasons term 2 was omitted from the differential scheme and the dispersion was represented by E_{num} only. A proper selection of the number of compartments (or the length Dx) in combination with the time step Δt allows to a certain extent the optimal representation of the actual horizontal dispersion.
Vertical mixing is accounted for by term 4 in Table 1, in which the

dispersion coefficient E_z is assumed to be the sum of a constant part E_{z0} attributed to the wind-induced turbulence in the pond, and a variable part proportional to the horizontal velocity and the depth of the upper layer H_{upp}. Hence, according to Fisher (1979):

$$E_{z_i}^{\ n} = E_{z_0} + \alpha . U_{x_i}^{\ n} . H_{upp}^{\ n} \tag{5}$$

4 Calibration and results

The overall calibration of the model was performed in several steps (Figure 3).

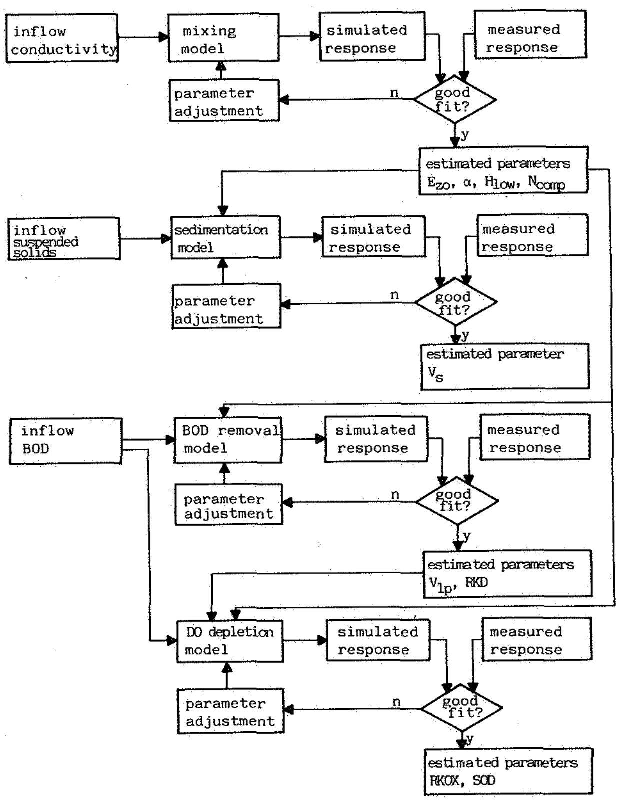

Fig. 3

The parameters describing the mixing in the pond were estimated independently from the conductivity profiles measured at location P_1 and P_2. Using these results the other parameters were estimated from observed concentrations at P_1 and verified with observed concentrations at P_2 (Table 2).
The estimation method used is based on the minimization of the sum of squares, using the Simplex method, as decribed by Nelder and Mead (1965).

4.1 Mixing phenomena

The optimal parameterset was found by varying the number of compartments N, independently followed by an estimation of the parameters H_{low}, α and E_{z0} (Table 2). In all cases the best fit was found when the pond was divided in 3 sections. The estimated depth of the dead zone (H_{low}) at P_1 is about 0.5 m after all events. The value found for location P_2 is about 0.7 m. This difference can be explained by the disturbance of the stratification by the overflow. A temperature gradient is induced by the cold seepage through the bottom and irradiance at the surface. The eroding effect of the overflow on the depth of the dead zone will decrease in the direction of the outlet.

Merely during the comparatively short period of the overflow the velocity dependent part of the vertical dispersion is in the same order of magnitude as the constant part E_{zo}. Moreover, the advective transport is of greater importance during this period. So the model is not very sensitive to the parameter α. This explains partly the great spread found in the value of this parameter and mitigates the significance of the variations found in α. Due to the small contribution of this term, the vertical mixing is described for the greater part by the background dispersion, which is caused by wind induced turbulence. The value of E_{zo} varied between 5.7×10^{-2} - 37.4×10^{-2} m²/day at location P_1. The low value was found after the event of 10/7, when wind velocity was low (1.6-2.0 m/s). During the period after the event of 21/5 wind velocity varied between 3.6 and 4.9 m/s.

Table 2 Estimated parameters affecting processes of mixing (H_{low}, α, E_{z0}, N), sedimentation (KV_s, V_{lp}) and oxygen depletion (RKD, RKOX, SOD)

event date		21/5/84	4/6/84	10/7/84	
H_{low}	(P_1)	0.40	0.59	0.45	(m)
	(P_2)	0.66	0.71	0.73	(m)
α	(P_1)	9.0×10^{-5}	5.3×10^{-5}	7.0×10^{-5}	
	(P_2)	4.8×10^{-5}	11.3×10^{-5}	12.7×10^{-5}	
E_{zo}	(P_1)	37.4×10^{-2}	18.7×10^{-2}	5.7×10^{-2}	(m^2/day)
	(P_2)	12.9×10^{-2}	12.9×10^{-2}	1.4×10^{-2}	(m^2/day)
N	(P_1)	3	3	3	
	(P_2)	3	3	3	
KV_s	(P_1)	34.6×10^{-3}	126.7×10^{-3}	93.6×10^{-3}	(m.m^3/g.day)
max. inflow rate		13.3	40	63	(m^3/min)
V_{lp}	(P_1)	1.4	53.3	43.2	(m/day)
RKD	(P_1)	0.85	0.86	1.93	(day^{-1})
RKOX	(P_1)	0.84	0.71	0.65	(day^{-1})
KSOD	(P_1)	0.13	0.14	0.12	(m/day)

Figure 4a shows the observed and predicted conductivity. The model gives a reasonable description of the mixing in the pond, but does not incorporate all the processes involved. For instance, the disturbance of the stratification by the overflow is a dynamic process, but the model uses a constant value for the depth of the dead zone. Hence the value found for H_{low} is an average over time.

The lack of detail limits the predictive capacity of the model with respect to the mixing phenomena in the pond. A more detailed description should include the dynamic behaviour of the depth of the dead zone as a function of the impuls of the discharge, irradiance, buoyancy and wind induced shear stresses. Although these processes are not addressed in the model, it can nevertheless be used as a basis for the characterization of the processes involved in the behaviour of suspended solids and dissolved oxygen, because the overall mixing is described sufficiently accurate.

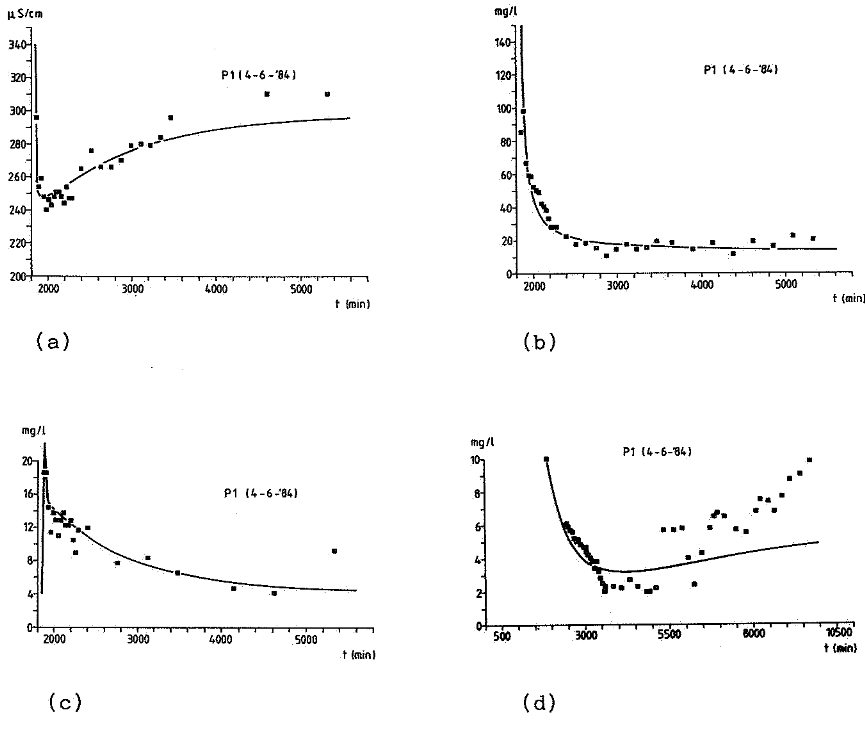

Fig. 4 Calibration results for the event of 4/6 compared to the measured data of conductivity (a), suspended solids (b), biochemical oxygen demand (c) and dissolved oxygen (d)

4.2 Suspended solids

The model was extended with terms 5a and 5b of Table 1, which account for the sedimentation. As result of the assumption of a stagnant lower layer, the effect of resuspension has been neglected. Initially sedimentation was modelled using a constant settling velocity (first order in SS-concentration). The settling velocity was estimated using the observed SS-concentrations at location P_1 after the event of 4/6. The optimal settling velocity found was 14.4 m/day. However, the agreement between measured and predicted SS-concentrations was poor. This is due to a decrease in the average settling rate because coarser particles tend to settle out first. The use of a second order description for sedimentation is a way to represent this and indeed resulted in a better fit (Figure 4b). The estimated second order rate constants (KV_s) are given in Table 2. The rate constants obtained vary considerably; especially the value of 21/5 is much lower. As indicated, the sedimentation rate will reflect the size distribution of the suspended particles and a high value would be connected with a predominance of coarse particles. Near the inlet the water is shallow and the sediment is soft. Therefore local resuspension is probable and high inflow rates will increase the proportion of particles with a high settling rate (Table 2). Also inherently there will be a variation in the particle size distribution of the CSO itself which is related to complex phenomena in the sewerage system during the preceding period. Hence one universal value for the sedimentation rate can not be expected.

4.3 Dissolved Oxygen

The input variable of the model for dissolved oxygen, the BOD-load, was divided into two fractions, based on the total BOD of the samples and the BOD after one hour settling. The latter represents the soluble and poorly settleable BOD_5 (LS). The particulate or settleable fraction (LP) can be calculated from the total BOD_5 and the BOD_5 after settling (Table 3). In order to simulate the oxygen depletion the BOD_∞ should be used, but considering the fact that the BOD discharged is readily decomposable, the BOD_∞ can be approximated by the BOD_5.

Table 3 Discharged load of total BOD_5, BOD_5 after settling and settleable BOD_5

event	total BOD_5	BOD_5 after settling	settleable BOD_5
date	(kg)	(kg)	(kg)
21/5	63.2	28.7	34.5
4/6	51.4	20.4	31.0
10/7	147.4	41.7	105.7
13/7	11.3	7.2	4.1

Due to the relatively short retention time of the settleable BOD in the water phase this fraction will have no direct effect on the dissolved oxygen concentration. So only a part of the BOD load (25-50%) is responsible for the oxygen consumption in the water phase.
The sedimentation rate (V_{lp}) of settleable BOD and the total removal rate (RKD) of the soluble and poorly settleable fraction were estimated using the observed total BOD concentration in the pond after an overflow and by incorporation of the terms 5 and 6 of Table 1 to model LP and LS. The removal of the latter fraction is caused by both oxidation and sedimentation. Formally the distinction of the BOD into two parts is a fractionation with respect to settling velocity. The oxidation rate constant (RKOX) was estimated subsequently from the observed oxygen concentrations after an overflow. For the simulation of the oxygen concentration DO and LS were calculated simultaneously, using the terms 1,3,4,6,7 and 8 of Table 1.

The sedimentation of LP was forced into a less appropriate first order term to allow comparison of the settling velocity with the first order removal rate of "soluble" BOD fraction. Except for the overflow of 21/5 the removal of settleable BOD is very fast, so neglecting the oxygen consumption of this fraction in the water phase seems to be justified. Similar as found for SS after the event of 21/5, the low settling velocity for LP can be explained by the low intensity of this overflow.
The settling velocity found for settleable BOD only indicates that the removal of this fraction from the water is fast. The accuracy of the

estimated velocities is low, because only the samples taken in the pond within one hour after an overflow contain settleable BOD. Hence, the estimation is based upon limited data.
An independent model based upon the transport equation for oxygen in the sediment showed that at the high infiltration rates observed in Loenen and with the low penetration depth, the sediment oxygen demand (SOD) is approximately linear to the oxygen concentration in the overlaying water. Parameter estimation resulted in a value for the proportional constant KSOD of ± 0.10 m/day resulting in a SOD of about 0.10 $g/m^2.day$ at the prevailing oxygen concentration of ± 1 g/m^3.

The mass transfer coefficient of oxygen through the air-water interface was not estimated, but measured directly with a hood floating on the surface. The increase of oxygen under the hood after flushing with nitrogen is a measure of the mass transfer rate. An averaged value of 0.185 m/day was found.

Figure 4c and d show the observed and simulated concentrations of BOD_5 and oxygen in the pond after the event of 4/6.
Initially the oxygen concentration increases after an overflow due to the high oxygen content of the storm water. The model gives a reasonable prediction of the minimal oxygen concentration after an overflow. Because the primary production was neglected, the model is not suitable to predict the recovery of the oxygen depletion. According to the model, within a period of one week a new equilibrium concentration will be reached in the order of 3 to 4 g/m^3. Growth of algae may increase this value. After the event of 10/7 the observed concentrations are below the predicted equilibrium concentration. This can be explained by the large proportion of settleable BOD discharged during this event (Table 5), resulting in a delayed oxygen consumption of the sediment. The SOD estimation however is based on the first part of the oxygen sag curve, where its role is of lesser importance as compared to the recovery phase.

5 Conclusions

- Although the mixing phenomena in the pond are not represented in detail, the model gives a reasonable description of the mixing in the pond for the purpose to study the short term effects on receiving water quality.

- The analysis of the processes produced a number of descriptions and characteristic rate constants that can be used in modelling the short term effects on SS and DO, but exhibit variability inherent to the properties of the CSO and may be blurred by local resuspension.

- The settling velocity of SS decreases after an event. A better performance of the model is obtained when using a second order description of sedimentation. Introduction of a settling velocity distribution would be interesting for the prediction of the distribution of the different fractions of the SS, but requires unusually detailed input data.

- The BOD-load can be subdivided into two fractions. The soluble- and poorly settling fraction has a direct effect on the oxygen-concentration. The settleable BOD will sustain the SOD and cause a delayed effect on the oxygen concentration.

- The organic matter discharged is readily decomposable, resulting in a fairly high value for the oxidation rate constant of BOD (0.6 - 0.8 day^{-1}).

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EFFECTS OF COMBINED SEWER OVERFLOWS ON THE MACRO-EVERTEBRATES COMMUNITIES IN A DETENTION POND

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Abstract

Macro-evertebrates were sampled monthly (February till November) at three sites in a detention pond receiving combined sewer overflows of the village of Loenen (the Netherlands). The results have been compared with samples from a similar pond without sewer overflows in the city of Apeldoorn. Special attention has been given to the short-term and long-term effects of the overflows on the macro-evertebrates coenoses. The results are mainly processed by cluster analysis and ordination techniques.
The macro-evertebrates coenoses in the detention pond at Loenen indicate a heavier organic pollution in comparison with the reference pond in Apeldoorn. The deeper sections of the detention pond are more polluted than the shallow parts and close to the inlet the effects of the overflows are also more pronounced than at some distance. Due to the numerous overflows in the past, no pronounced short-term or long-term effects were found during the sampling period.

1 Introduction

Effects of sewer overflows on macro-evertebrates communities have received little attention. In the Netherlands an extensive study by Roos (1983) can be mentioned. In the past (much) more attention has been given to the effects of raw sewage and effluents on the macro-

evertebrates communities, e.g. Moller Pillot (1971) and Waterschap Zuiveringschap Limburg (1985). Nowadays nearly all waste waters are treated in sewage treatment plants and the remaining discharges will be sanitated in the near future. After this, combined sewer overflows will be the main causes of direct organic pollution of surface waters.
The estimated number of sewage overflow constructions in the Netherlands is about 5500 (Nationale Werkgroep Riolering en Waterkwaliteit, 1984). The effects of overflows on the water quality will be different from (un)treated effluents from sewage treatment plants because overflows occur at irregular intervals (mainly in the summer in the Netherlands) in contrast to the regular and continuous flow of effluents. The immediate effects of an overflow will depend on the intensity of the rainfall, the intervals between overflows, the type and size of the receiving water, etc.
The present study is mainly descriptive and has the aim to review the main effects of the overflows on the quality of the receiving water ("quality" of the macro-evertebrates coenoses) in relationship with the amount and quality of the overflowing water. Special attention has been given to the short-term and long-term effects on the macro-evertebrates coenoses.
The research presented here is part of a comprehensive study in the Netherlands in the framework of the NWRW (National Working Group on Sewerage and Water Quality) on rainfall, runoff, interception, treatment and storage, overflows and effects on receiving waters. Macro-evertebrates communities in the detention pond are compared with communities in a similar pond without sewer overflows. Other investigations in the same ponds and in the same period compare physico-chemical parameters in water and sediments (Aalderink and Lijklema, 1985 and Aalderink et al., 1986), survival of bacteria and bacteriophages in the detention pond (Lijklema et al., 1986) and the plankton and epiphytic diatom communities (Roijackers and Ebbeng, 1986).

2 Description of the ponds and the sampling sites

The detention pond is a rectangular water body with a length of 120 m, a width of 35 m and an average depth of 1 m (Figure 1). The pond is situated near the village of Loenen (the Netherlands) on the eastside of the Veluwe and receives combined sewer overflows (CSO) from this village. The system drains about 16 hectares impervious area and serves a population slightly over 2000 people and a few local industries. The past four years the average frequency of CSO-events was about 15 times a year. The pond is flushed by iron-rich seepage with an average seepage rate (in 1983 and 1984) of about 9.3 cm.day^{-1}. Hence the nominal hydraulic detention time is about 10 days. The pond drains seepage over a weir into a brooklet.

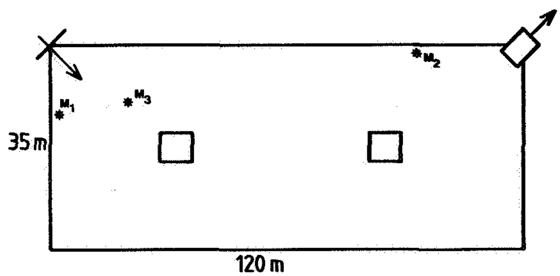

Figure 1 The detention pond at Loenen with sampling sites (M1, M2 and M3)

As shown in Figure 1, the sampling sites M1 and M2 are situated near the margins of the detention pond, respectively close to the inlet of CSO and at some further distance. At M1 there is a sparse vegetation of <u>Typha latifolia</u> L. and <u>Callitriche</u> spec. At M2 there is a dense vegetation dominated by <u>Typha latifolia</u> L.. The sediment at both sites consists of about 10 cm anaerobic mud on sand, covered with litter from the vegetation and leaves and twigs from the trees. Sampling site M3 is situated in the deeper part of the pond near the inlet of CSO. Here the sediment consists of a layer of 20-25 cm anaerobic mud (mainly from CSO) on sand; there is no vegetation.

The reference pond in Apeldoorn (Figure 2) has the same characteristics as the detention pond, except for a larger surface area (18.450 m^2) and the absence of CSO. The average seepage rate (also iron-rich) is about 8 $cm.day^{-1}$ and the hydraulic detention time is about 20 days.
The sampling site (Ap) is situated near the northern margin, where the vegetation is dominated by Myriophyllum spicatum L.. The sandy sediment is redbrown coloured by ironhydroxide deposits. The sampling sites in both ponds were chosen after an introductory study.

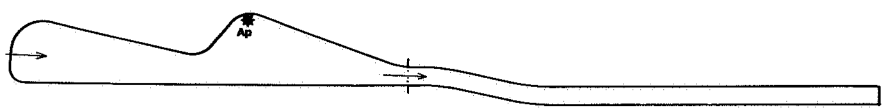

Figure 2 The reference pond at Apeldoorn with sampling site (Ap)

3 Material and methods

Monthly samples were collected at all sampling sites from February till November 1984. In May and October also samples were collected one week after a CSO-event to study possible short-term effects from CSO. All together 46 samples of macro-evertebrates were collected. The samples were taken with a standard macro-evertebrates net (aperture 20 x 30 cm; mesh-size 0.5 mm). The sampled area was approximately 1 m^2 at M1, M2 and Ap and 1/3 m^2 at M3. Samples were transported to the laboratory in plastic buckets, sieved and sorted in a white tray as soon as possible (but always within two days). The macro-evertebrates were preserved in 80%-ethanol, except for the Oligochaeta (4% formaldehyde) and Hydracarina (Koenike-solution). Triclads were not preserved but identified immediately. The nomenclature for the macro-evertebrates taxa follows Mol (1984).

The numbers of organisms, standardized for 1 m², were transformed according to an abundance scale from 1 till 9: 1 specimen (1); 2-3 specimens (2); 4-6 specimens (3); 7-12 specimens (4); 13-20 specimens (5); 21-40 specimens (6); 41-100 specimens (7); 101-500 specimens (8) and more than 500 specimens (9). The (co-)occurrence of taxa has been investigated with computer processing procedures as Detrended Correspondence Analysis (DECORANA) and Two Way Indicator Species Analysis (TWINSPAN), respectively ordination and cluster analysis techniques (Gauch, 1982).

4 Results

The results of the TWINSPAN-clustering for the macro-evertebrates are given in Figure 3.

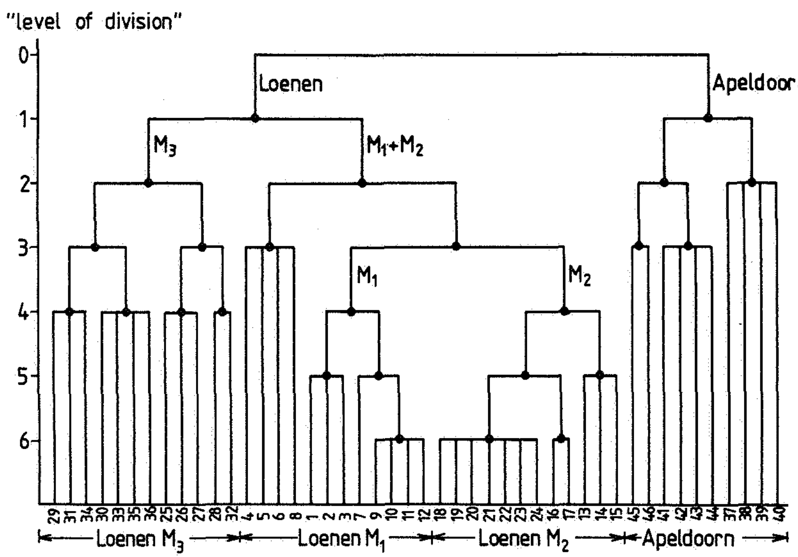

Figure 3 Macro-evertebrates. Dendrogram of the TWINSPAN-clusters for Loenen and Apeldoorn

The results from DECORANA for the samples from both ponds are presented in Figure 4, while the results for only the samples from the detention pond are presented in Figure 5.

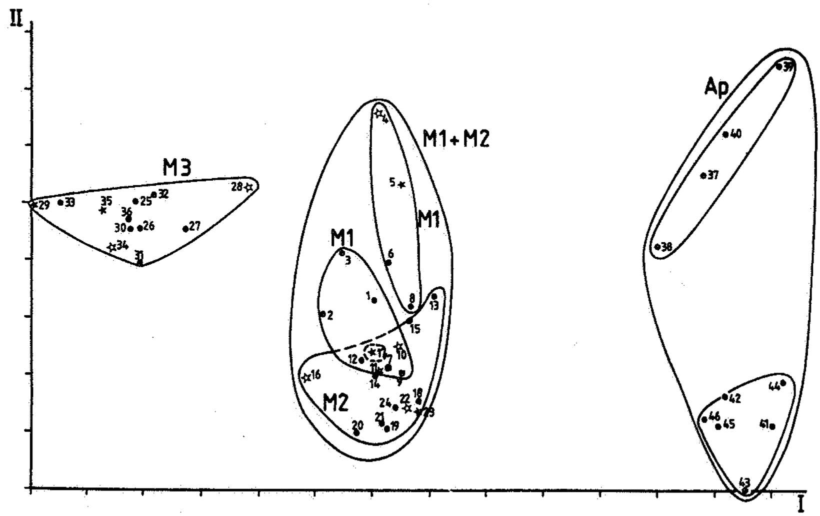

Figure 4 Macro-evertebrates. DECORANA-ordination plot from the samples from both ponds. The TWINSPAN-clusters (Figure 3) have been indicated.

1-12: samples at M1 from February (1) till November (12), with in May (4,5) and October (10,11) two samples; one week before (✫) and one week after (✶) a CSO-event

13-24: samples at M2 from February (13) till November (24), with in May (16, 17) and October (22, 23) two samples (as at M1)

25-36: samples at M3 from February (25) till November (36), with in May (28, 29) and October (34, 35) two samples (as at M1)

37-46: samples at Ap from February (37) till November (46)

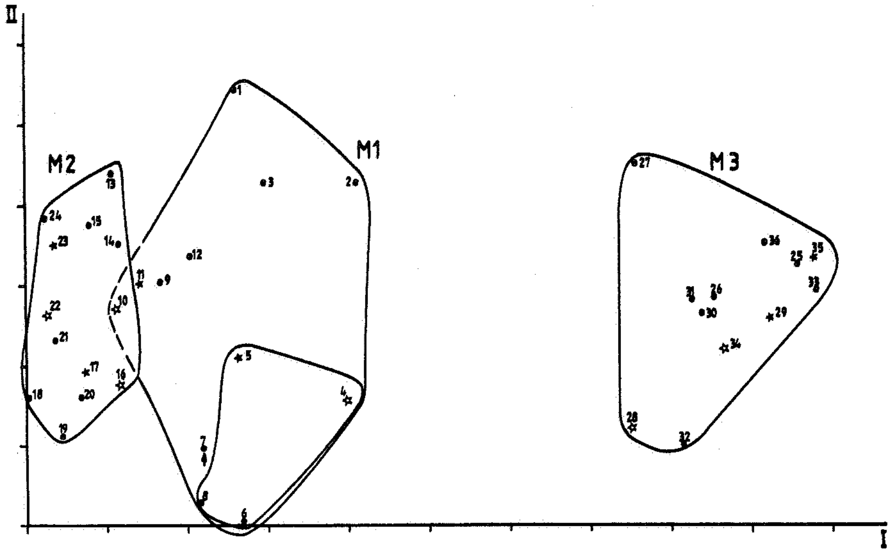

Figure 5 Macro-evertebrates. DECORANA-ordination plot of the samples from the detention pond at Loenen. The TWINSPAN-clusters (Figure 3) have been indicated.
Explanation see Figure 4

The meaning of DECORANA-ordination plots is that they reveal relative differences or similarities between the samples (the (co-)occurrence of the macro-evertebrates taxa). Ordination of a sample along a principal ordination axis (score) depends on the composition of the macro-evertebrates coenose of this sample in comparison with all other samples. Hence the scores along the axes in an ordination plot can be related to one (or more) important explanatory factor(s).
In Figure 4 the score along the first axis (I) can be interpreted as the degree of organic pollution or influence of CSO. A heavier polluted sampling site (M3) has a low score, while the samples from the reference pond have a relatively high score. On this first ordination axis no pronounced difference can be noticed between the samples from sites M1 and M2 in the detention pond. On the second axis (II) however, the M1-samples have a higher score than the M2-samples on the same date. This difference may reflect a difference in the structure of the vegetation of the sites; M2 has a dense vegetation, dominated by <u>T. latifolia</u> L.,

whereas at M1 only a sparse vegetation of T. latifolia L. and Callitriche spec. is present.

In Figure 5 the difference between M1 and M2 is more distinct, because interference with the Apeldoorn-samples is eliminated. (The Apeldoorn-samples tend to blur smaller differences between the Loenen-samples). The scores along the first axis in this figure may indicate a difference in the degree of organic pollution or influence by CSO. In that case it should be noticed that the scores along the first axis in Figure 5 are reversed with respect to those in Figure 4: a high score (M3) indicates a heavier organic pollution, and it can be concluded that M1 is disturbed more permanently than M2 due to the frequent CSO in the past. However, the scores along the first axis can be also interpreted as a difference in the structure of vegetation. Further, the clockwise arrangement of the consecutive samples from sites M1 and M2 in Figure 5 reflects a seasonal change in the abundance and composition of the macro-evertebrates communities. This pattern is more pronounced on the second axis, where the vegetation structure controls variations and a seasonal effect is obvious. However, to a lesser extent also along the first axis the seasonal cycle can be observed. This can be attributed (also) to variations in the quality and structure of the vegetation, which superimposes variations on the pollution background effect of CSO. Main organisms responsible for this seasonality are Chironomidae and Cloëon dipterum (emergence in summer), Anisus vortex, Lymnaea peregra, Gammarus pulex and Proasellus coxalis (appearance of juveniles in summer).

The ordination plot shows no remarkable differences between samples from sites M1 and M2 one week before, respectively one week after a sewage overflow (May and October). At M3 however, the plot shows a short-term effect of CSO, especially after the big overflow on the fifteenth of May. Samples, taken one week after an overflow, contain very few organisms and taxa. This decline is explained by wash out of most of the organisms present due to the strong current along the bottom of the pond near the inlet (passive transport) and/or because organisms seek shelter in the vegetation belt along the margins (migrations).

The ordination plots are appropriate for a rough distinction and description of CSO effects on macro-evertebrate coenoses. However, the presence of relevant indicator organisms is a further possibility to analyse mutual differences within the pond in Loenen or differences between the detention pond and the reference pond (see Table 1).

Table 1 Macro-evertebrates taxa with an abundance of ⩾ 4% on at least one of the sampling sites (+: abundance < 1%)

	Sampling sites			
Taxa	M3	M1	M2	Ap
Chironomus spec.	44.2	23.2	6.0	1.2
Psectrotanypus varius	40.1	5.6	+	+
Tubificidae	8.7	7.2	2.3	2.7
Cricotopus spec.	+	10.0	+	+
Proasellus coxalis	+	9.6	+	–
Lymnaea peregra	–	6.7	+	1.5
Anisus vortex	+	6.6	13.6	+
Erpobdella testacea	+	6.2	3.5	+
Helobdella stagnalis	+	5.6	1.9	+
Pisidiidae/Sphaeriidae	+	1.6	50.2	+
Gammarus pulex	–	+	4.5	4.4
Trichoptera	–	+	+	5.3
Asellus aquaticus	–	–	–	8.5
Stylaria lacustris	–	+	+	9.1
Dicrotendipes gr. lobiger	–	+	+	12.9
Proasellus meridianus	–	–	–	17.8
other taxa	5.5	16.8	16.5	34.9

Table 1 shows that site M3 is strongly dominated by Chironomus spec., Psectrotanypus varius (both chironomids) and Tubificidae; these three taxa (Chironomus-combination) represent more than 90% of the organisms. According to e.g. Moller Pillot (1971) and Moller Pillot and Krebs (1981) dominance of these benthic organisms is characteristic for highly dynamic situations, strongly fluctuating oxygen levels and transport of sediment. A strong decline in oxygen level in the detention pond after a CSO-event occurs frequently and can last several days (Aalderink and Lijklema, 1985). From the remaining organisms at M3 the most abundant species is Chaoborus flavicans (abundance of 2.7%; not in Table 1). According to Parma (1969) C. flavicans is typical for lakes with anoxic conditions or a low oxygen content beneath the thermocline. The food-chain at M3 is very simple: Chironomus and Tubificidae feed on detritus, while P. varius predates on both taxa. The anaerobic mud and the lack of vegetation structure prevent the occurrence of a better developed macro-evertebrates community on this site.

The mutual differences between the sites M1 and M2 near the margins of the detention pond are smaller than the differences between M3 and M1 or M2. The Chironomus-combination represents over 35% of the organisms at M1 and only about 8% of the organisms at M2. Other organisms occurring frequently at M1 generally have a great tolerance for organic pollution and low oxygen levels, e.g. Cricotopus spec. (Moller Pillot, 1984), Proasellus coxalis and the leeches Erpobdella testacea and Helobdella stagnalis (Dresscher and Higler, 1982). Especially snails as Anisus vortex and Lymnaea peregra indicate some structure in the vegetation.

The most abundant organisms at M2 are the bivalve molluscs (Pisidiidae/Sphaeriidae), which represent just over 50% of the organisms. These organisms are present in the superficial mud layer, provided that the mud is not heavily polluted (Janssen and De Vogel, 1965). Another good indication for the absence of major disturbance at M2 is Gammarus pulex, which can be seen as an indicator organism for a relatively good water quality (Moller Pillot, 1971). In total 734 specimens of G. pulex have been sampled at M2, whereas at M1 only in the summer period some juvenile specimens, migrated from less polluted parts of the pond, were found. Also at Apeldoorn G. pulex was found regularly. In conclusion it can be said, that the organisms (Table 1) indicate a difference in degree of organic pollution or disturbance between the sites near the inlet and at some distance. Further, the macro-evertebrates coenoses in the detention pond indicate a heavier organic pollution than the reference pond. M2 resembles the reference pond more than M1. The most important indications for an undisturbed environment in the reference pond are the relatively low presence of the Chironomus-combination, the abundant presence of Proasellus meridianus (Moller Pillot, 1971) and, to a lesser extent, G. pulex (Moller Pillot, 1971) and several species of Trichoptera (Lepneva, 1966).

5 Discussion

During this investigation no pronounced effects of individual CSO-events on the macro-evertebrates coenoses could be found. One observation is that immediately after a CSO-event the number of taxa and specimens in the deeper part of the detention pond is reduced. This is caused by passive transport of organisms and/or migration to the vegetation belt near the margins. Also no change in a certain direction in the composition of the macro-evertebrates communities during the year could be found, which means that the coenoses are rather stable and adapted to the occurrence of overflows. A further impoverishment of the macro-evertebrates coenoses is, considering the frequency of overflows and the quality of the overflowing water, not to be expected. A further increase of the number of CSO-events and/or a higher number of inhabitants in the drained area respectively an increase of industrial activities may result in a further impoverishment of the macro-evertebrates communities. Ultimately, there will be no life at all in the deeper parts of the detention pond and a reduction of species near the margins. Species of the Chironomus-combination will be the last to disappear. An improvement of the quality of the overflowing water or a strong reduction of the number of CSO-events will result in a better developed vegetation and macro-evertebrates community. Certain species, now occurring only near the margins of the pond, will be able to colonize deeper parts and a more differentiated and complex ecosystem will develop. It is also possible that some species from outside the pond will be able to colonize this ecosystem. The presence of a thick anaerobic mud layer, however, will prevent the development of a more differentiated ecosystem such as in reference pond. This would require careful dredging of and termination of overflows in the Loenen pond. Dredging of the detention pond without the prevention of CSO-events will be useless, as the improvement of the sediment conditions will only last till the first big overflow.

Acknowledgements

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STRUCTURE AND FUNCTIONING OF PLANKTONIC AND EPIPHYTIC COMMUNITIES IN A POND RECEIVING COMBINED SEWER OVERFLOWS

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Abstract

In 1984 a whole-year investigation has been done on the effects of combined sewer overflows on the water quality of a receiving detention pond (near Loenen). A pond in Apeldoorn was chosen as reference. Both ponds are highly influenced by seepage. This paper deals with the effects of CSO-events on phytoplankton, zooplankton and epiphytic communities. Phytoplankton biomass was low (Max. 40 $\mu g.l^{-1}$ in Loenen and 4.7 $\mu g.l^{-1}$ in Apeldoorn). The number of species was also very low, both for the plankton and the epiphytes. This sets bounds to the use of quotients characterizing the degree of eutrophication or organic pollution. Structural characteristics of both planktonic and epiphytic communities showed clearly the effects of CSO-events. Short-term effects observed are the plankton being washed out, resulting in a high transparency, thus enabling the phytoplankton to develop large populations. The inocculation function of the littoral and sediments was important. Long-term effects are reflected by the species diversity and composition: short-living organisms were found in Loenen and long living species in Apeldoorn. The structure of the epiphytic diatom communities revealed a more stable developing community in Apeldoorn and a more or less pioneer state development in Loenen.

1 Introduction

The use of planktonic and benthic (mikro)organisms as indicators for inorganic and organic pollution has a long and well-documented history (Kolkwitz and Marsson, 1908, 1909, Thunmark, 1945, Nygaard, 1949, Liebmann, 1951, Pantle and Buck, 1955, Zelinka and Marvan, 1961, Sládeček, 1963, 1973, Dresscher and Van der Mark, 1976). These methods are restricted to static characteristics of the communities and do not inform on their dynamic properties. As the structure of a community is the result of its functioning, it is obvious that methods are needed to reveal the functioning of communities.
In more recent studies the results of many years of monitoring and experimental studies in the field and laboratories on planktonic and benthic communities have been reviewed, resulting in new insights and also new lines of approach to the assessment of inorganic and organic pollution by means of planktonic and benthic organisms (Harris, 1978, 1980, Round, 1981, Reynolds, 1982, 1984). In these studies attention is drawn to the dynamic characteristics of the communities, such as life-cycle, growth rate, productivity, adapatation, mobility, etc.
This paper intends to compare the results of some of the classic methods of biological assessment of water quality with some of the recent insights based on the functional aspects of communities.
The present study is part of an overall study in The Netherlands in the framework of the NWRW (National Working Group on Sewerage and Water Quality) on rainfall, runoff, interception, treatment and storage, overflows and effects on receiving waters. The objectives of the study included an inventarisation of the effects and an analysis of the processes that take place in the receiving water. The effects studied are various and include:

- hydrobiological effects as reflected by macrofauna (Willemsen and Cuppen, 1986) and by plankton and epiphytes (this paper),
- microbiological effects (Lijklema et al., 1986),
- physico-chemical effects in water and sediments (Aalderink and Lijklema, 1985, Aalderink et al., 1986).

The structure and functioning of planktonic and epiphytic communities in a pond receiving CSO in Loenen are compared with the planktonic and epiphytic communities in a pond in the neighbourhood (Apeldoorn), closely resembling the pond in Loenen, except for the absence of CSO.

2 Description of the ponds

The location studied is a detention pond of 120 x 35 m with a mean depth of 1 m, receiving CSO from the small village Loenen (Figure 1). The system drains about 16 hectares impervious area and serves a population of more than 2,000 people and a few local industries. The pond is flushed by seepage (9.3 $cm.day^{-1}$), rich in iron. The average hydraulic detention time is about 10 days.

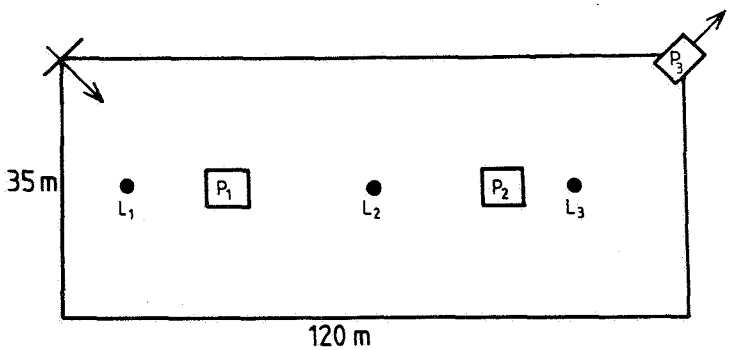

Figure 1 The pond in Loenen
P_1, P_2 and P_3: platforms
L_1, L_2 and L_3: sampling sites

The reference pond was located in Apeldoorn. It has a surface area of 18,450 m^2, a mean depth of 1 m and is also flushed by iron-rich seepage (8 $cm.day^{-1}$). The average hydraulic detention time is about 20 days (Figure 2).

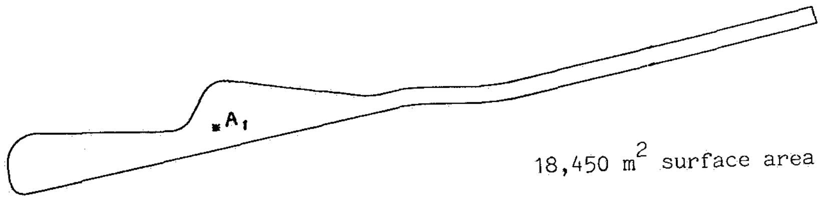

Figure 2 The reference pond in Apeldoorn
A_1: sampling site

3 Material and methods

Planktonsamples have been taken fortnightly at the sites L_1, L_2, L_3 and A_1. Phytoplankton was collected in the field according to generally accepted methods (Roijackers, 1985) and concentrated in the laboratory by membrane filtration (3 µm pore width). Zooplankton was collected with a planktonnet(60 litres; 60 µm gauze width). Identification of the taxa was carried out as soon as possible on living specimen. The abundancy was estimated using an abundancy scale 1 (rare) to 5 (abundant) (Roijackers, 1985). Epiphytic diatoms were collected from wooden sticks placed at the sampling sites. The relative abundancy of the diatoms was determined by counting 300-400 valves and calculating the percentage presence of each taxon.
Phytoplankton biomass was estimated weekly at Loenen and fortnightly at Apeldoorn; chlorophyll-a was selected as measure of biomass. The biomass data of the three Loenen sites have been averaged.
The co-occurrence of taxa has been investigated using clustering (TWINSPAN) and ordination (DECORANA) techniques (Gaugh, 1982).

4 Results and discussion

4.1 Nutrients and phytoplankton biomass

Both ponds are eutrophic regarding the concentrations of dissolved nutrients. The pond in Apeldoorn is less eutrophic than the one in

Loenen. The chlorophyll-a content is very low in both ponds: max. 40 $\mu g.l^{-1}$ in Loenen and 4.7 $\mu g.l^{-1}$ in Apeldoorn (Figure 3). The high concentrations of colloidal iron causes light limited phytoplankton growth. The low phytoplankton biomass is also the result of the rather high flushing rate.

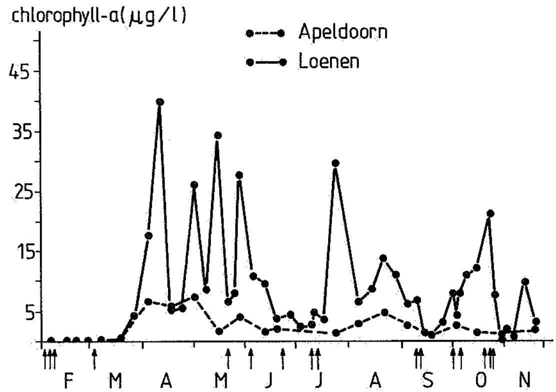

Figure 3 The phytoplankton biomass (chlorophyll-a in $\mu g.l^{-1}$). Arrows indicate CSO-events

The fluctuations in the phytoplankton biomass in Loenen indicate an immediate response of the phytoplankton organisms to a fluctuating environment. Partly these fluctuations coincide with CSO-events. It is obvious from Figure 3 that the overflows in February and March had no effect at all on the phytoplankton as the biomass was very low; the species succession in that period followed the pattern that is known from slightly eutrophic non-influenced ponds in that region. All other overflows result in an immediate decline of biomass. This is due to the physical effect of overflows on phytoplankton: it is flushed out of the main waterbody. The water then becomes very transparent as also the colloidal iron particles are flushed. In period of sufficiently high irradiance phytoplankton can grow very fast as it is not light limited up to 3 or 4 days after the CSO. The observed biomass increases in such periods are mainly du to fast growing taxa like *Chlamydomonas* spp. and *Cryptomonas marsonii*.

4.2 Comparison of the plankton samples from Loenen and Apeldoorn

In Figures 4 and 5 the ordination plots of the data are presented for the phytoplankton and zooplankton respectively. Differences between the points are the result of differences in the similarities of the species composition in the samples. It is clear from Figures 4 and 5 that the Loenen samples are quite different from the Apeldoorn samples, particularly the zooplankton samples. This reflects that complete different communities are present in both ponds throughout the year. The differences are less well pronounced in the phytoplankton, which reflects the trophic level, and very well pronounced in the zooplankton, which reflects the saprobic level. It is obvious that the first principal component in the ordination can be associated with the level of perturbation, particularly the organic pollution.

4.3 Trophic and saprobic level

The trophic level, primarily determined by nutrient content, will be reflected by the structure and functioning of the communities of autotrophs. So the high trophic level in both ponds can be illustrated by their species composition (Table 1).

Table 1 Contribution of the different algal groups to the total phytoplankton communities in Loenen and Apeldoorn

	Number of taxa		Relative abundance (%)	
	Loenen	Apeldoorn	Loenen	Apeldoorn
Cyanobacteria	3	2	11	5
Chrysophytes	9	3	9	13
Diatoms	2	4	12	33
Euglenophytes	2	3	7	11
Chlorococcales	7	1	6	1
remaining groups	7	3	55	37

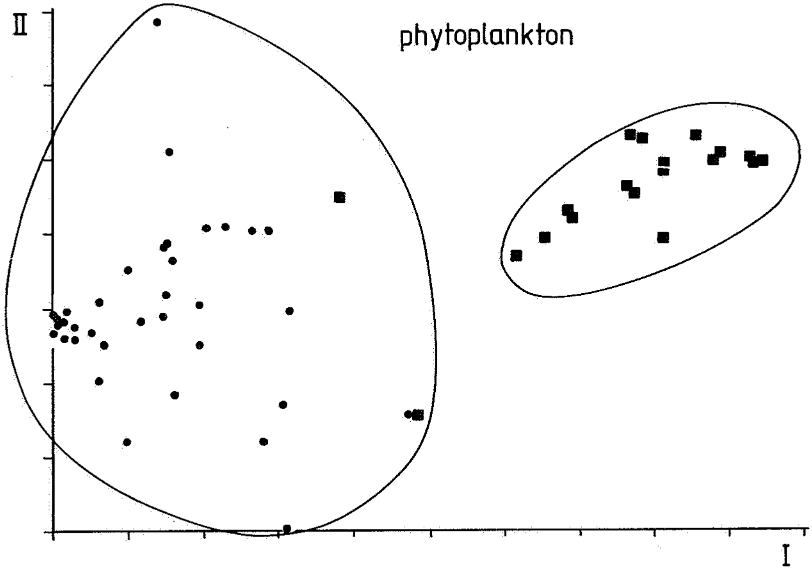

Figure 4 Phytoplankton. DECORANA-ordination plot of the sampling-data. The TWINSPAN-clusters have been indicated

● : Loenen data; ■ : Apeldoorn data

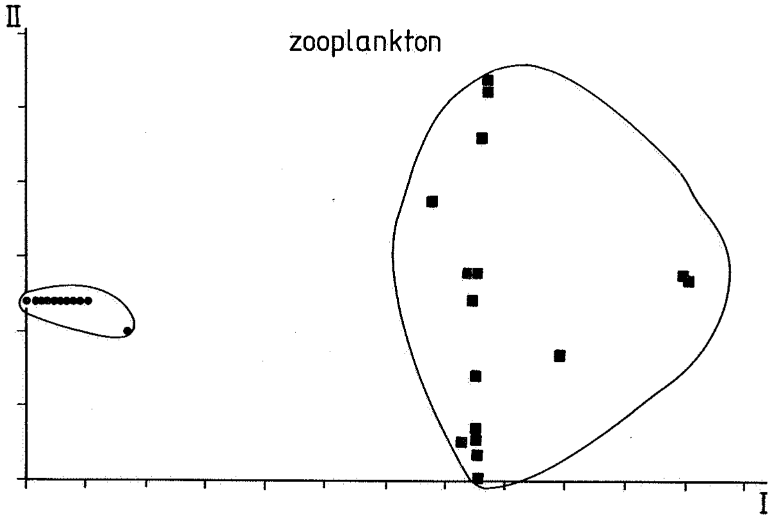

Figure 5 Zooplankton. DECORANA-ordination plot of the sampling-data. The TWINSPAN-clusters have been indicated

● : Loenen data; ■ : Apeldoorn data

The use of quotients as introduced by Thunmark (1945) and Nygaard (1949) is inadequate as the total number of taxa in both ponds is too low and the important group of Desmids has been sampled only once in Loenen. Judging from Table 1, particularly the presence of eutrafent Cyanobacteria and Chlorococcales, the pond in Loenen is extremely eutrophic and the pond in Apeldoorn only slightly eutrophic. However, more important is the abundancy of the 'remaining groups'. In Loenen many specimen of the taxa *Chlamydomonas* spp. and *Cryptomonas marsonii* are found, taxa with high growth rates; in Apeldoorn these taxa are less well represented.

The saprobic level is reflected better by the zooplankton communities than by the phytoplankton communities. In Loenen 29 zooplankton-taxa have been identified, in Apeldoorn 13. These values indicate the higher saprobic level in Loenen. Dresscher and van der Mark (1976) developed an index based upon the presence of several algal and zooplankton groups in a water body, all indicative of a certain degree of saprobity (Figure 6). From Figure 6 the following can be infered:

- The saprobic level is higher in Loenen than in Apeldoorn.
- The fluctuations in the saprobic level in Loenen are more frequent and of greater amplitude than in Apeldoorn.

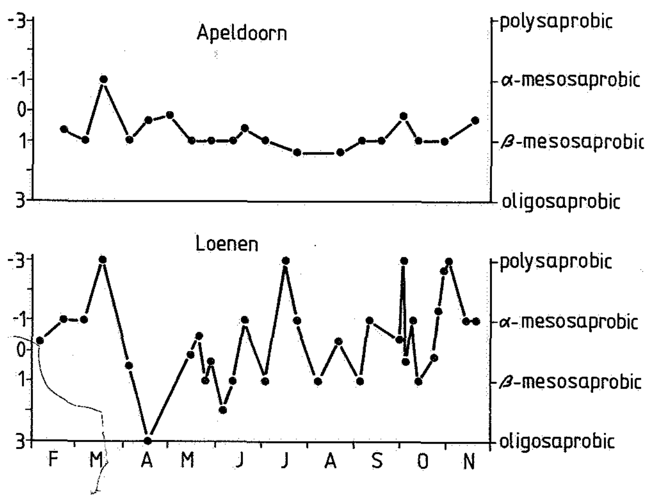

Figure 6 The saprobic level according to the method of Dresscher and van der Mark (1976) for the ponds in Loenen and Apeldoorn

Application of Sládeček's (1973) method to assess the saprobic levels is appropriate for the discrimination between the different biological components, such as phytoplankton, zooplankton and epiphytic diatoms (Figure 7). From Figure 7 additional conclusions can be drawn:

- The saprobic level is reflected most clearly in the zooplankton; also the immediate reaction of the communities upon changes are seen.
- Long-term effects are reflected by the saprobic level as indicated by the epiphytic diatom communities.

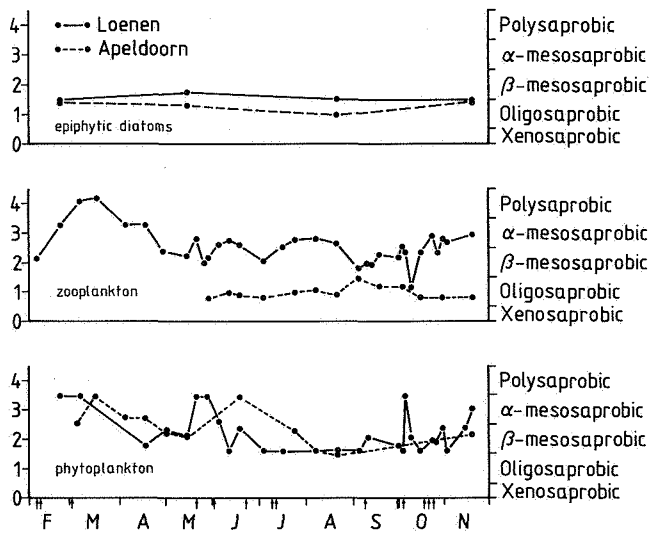

Figure 7 The saprobic level according to Sládeček (1973) for the ponds in Loenen and Apeldoorn

4.4 The effects of CSO on planktonic and epiphytic communities

4.4.1 Short-term effects

Immediately after a CSO the planktonic organisms are flushed out of the open water area; zooplankton species take refuge near the vegetation in the littoral or near the bottom, whereas the phytoplankton organisms

are washed out in proportion to the volume of the CSO. In the open water some benthic mikrofaunal species are observed after an event. This is due to resuspension and they will disappear after a few days. The water will be transparent for a few days as together with the plankton the colloidal iron is washed out. This enables the phytoplankton to increase its biomass, especially through the fast growing taxa. The water mass becomes turbid again and the phytoplankton growth becomes light limited. Inocculation of phytoplankton occurs from the littoral and sediments. Also after a few days zooplankton will leave its shelter and move towards the open water. This combined mikrofaunal succession by benthic and planktonic species is reflected by the saprobic level as illustrated in Figure 7.
So the short-term effects upon microflora and microfauna are mainly saprobical. The normal succession pattern will be disturbed by CSO as long living organisms do not get a chance to develop. Another clear effect is the appearence of large algal beds on the water surface after an event, they originate from the sediments of the pond. These algae are blue-greens, adapted to low light intensities. After a CSO these algae will be very productive, due to the increased irradiance at the sediment surface. The oxygen produced than causes flotation of the algal mat and at the surface bleeching of the algae is followed by decay due to the high light intensities.

4.4.2 Long-term effects

The long-term effects of CSO are reflected in all communities by a higher saprobic level and in the phytoplankton by a higher biomass. The saprobic level in the pond in Apeldoorn is more constant than that in the pond in Loenen. In the structure of the epiphytic diatom communities a clear long-term effect can be seen. The number of taxa in both ponds is restricted to a basic association of 10 species. In Loenen five additional species are found. Most of them indicating a higher trophic and/or saprobic level. The eight extra species in Apeldoorn on the contrary are indicators for a lower trophic and/or saprobic level. It is striking that all eight extra diatom species in

Apeldoorn are larger species, which are not attached to the substratum at their full length, but only by their apexes and in fact they are loosely attached. Hence they are in a favourable position to utilize more space for their nutrition. We suppose that this type of organisms has no chance to colonize the pond of Loenen as the frequent CSO-events impair their development. Therefore epiphytic diatom communities in Apeldoorn thus have developed more diversely, including several species which have the opportunity to specialize in many directions (nutrition, dispersal, etc.).

5 Concluding remarks

As illustrated the frequent CSO-events in the pond in Loenen result in a diverging structure and functioning of both planktonic and epiphytic communities as compared to a pond not affected by CSO's. However, the differences are limited because both ponds are constantly influenced by seepage, which also can be seen as a perturbation of the ecosystem. This perturbation results in a less stable ecosystem: low species diversity and niche-diversification. The use of quotients that indicate the trophic degree of the pond is practically impossible, due to the low species diversity in the autotrophic component of the ecosystem. Other ways to illustrate the high trophic level of both ponds are to be detected in the other structural but particularly functional (dynamic) characteristics of the communities: species composition, growth rate (productivity), and space utilization (niche-diversification). Among the methods used traditionally to characterize the degree of perturbation, in our study only the quotients of Sládeček (1973) and Dresscher and Van der Mark (1976) are usefull to indicate the degree of saprobity. The quotient of Sládeček permits us to trace the short-term effects of CSO as reflected by the zooplankton and the long-term effects as reflected by the epiphytic diatoms. Application of such quotients, characterizing static properties of the ecosystem, are still insufficient to understand the complete reaction of (parts of) the ecosystem. Penzhorn (1976) and Caspers and Penzhorn (1976) therefore have used productivity measurements in their studies on the effects of CSO upon planktonic communities in the surface waters in Hamburg. In this way they actually could measure the change in productivity as caused by the succession of slow growing phytoplankton species and fast growing species. They also showed that due to the changing environmental conditions resulting from a CSO (light penetration), productivity changed but the species composition remained the same.

In fact the pond in Loenen is intended as a buffering system to reduce the effect of CSO upon the actual receiving surface water. Seen from the present investigations, we may expect the pond to fulfil its function very well, as the succession of organisms shows a regular pattern. It remains, however, to be seen whether this succession pattern will be similar over a longer period of years, as the accumulation of organic and inorganic material continues. There will be a threshhold somewhere, over at which the system can not longer process the extra material originating from CSO. Then the system will become biological dead and the receiving surface water will be charged in an unacceptable way.

Acknowledgement

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BIOLOGICAL EFFECTS OF STORMWATER DISCHARGES IN URBAN CANALS

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Abstract

In this paper a study is reported on the effects of stormwater discharges on the aquatic biota of the receiving waters. The study area concerns the separate sewerage system of the newtown Lelystad, a low level pollution area with respect to stormwater discharges. The biological effects are evaluated by means of biological water quality assessment.

Quite remarkably, the storm sewer discharges caused a limited but significant disturbance in the composition of the aquatic ecosystem. In the vicinity of storm water outlets, the β-α-mesosaprobic state of the canals in Lelystad tends to the more polluted α-mesosaprobic state.

1 Introduction

As opposed to the combined sewer system generally applied in The Netherlands, the separate sewer systems collect wastewater and storm-water in separate sewer lines; the wastewater is transported to a treatment plant, while the stormwater runoff directly flows into receiving surface waters.

Major advantages of separate sewer systems are that the hydraulic capacity of the treatment facility is minimized and that a higher treatment process stability is attained, thanks to less fluctuations in hydraulic loading. In separate sewer systems, stormwater runoff causes high incidence low level pollution, whereas combined sewer systems incidentally discharge high pollutional loads, when overflowing during intensive rainfall.
The annual pollutional loads for receiving waters are not necessarily less in a separate system, while the costs are generally higher (Hogendoorn-Roozemond et al., 1981).

1.1 Quality of stormwater runoff

In former days, stormwater runoff was considered to be rather clean. Besides the hydraulic advantages, separate sewer systems were propagated for this reason. However, it has been found that significant pollution of stormwater can be caused by:

- atmospheric fall-out/precipitation; in Finnish cities 10 to 30% of organic matter, phosphorus and heavy metals present in urban runoff has been found to be caused by atmospheric fall-out (Melanen, 1981);
- corrosion of buildings and other facilities in urban areas; this has markedly increased by acid rain;
- street dirt, caused by e.g. traffic emission, animal faeces, litter;
- application of chemical detergents, insecticides and herbicides;
- vegetation.

In the past decade the water quality of storm sewer discharges has been investigated extensively (Krauth, 1979; Melanen, 1981; Brunner, 1975; O.E.C.D., 1982). The pollution load per ha of impervious area has been found to vary greatly from one area to the other, depending on land-use (Table 1), age of the city, street sweeping frequency etc. (Field and Struzeski, 1972).

Table 1 Concentrations of pollutants in street dirt by land-use characteristics (after Manning, 1977, and Sartor and Boyd, 1972).
Concentrations in ppm, unless indicated.

	Land-use		
	Residential	Commercial	Industrial
BOD	3370	7190	2920
COD	42000	61700	25100
Total-N	550	420	430
Soluble PO_4-P	58	60	26
Lead	1980	2330	1590
Zinc	280	690	280
Copper	73	95	87
PCB's (ppb)	810-2000	510-990	1000-2000

Significant decrease in the quality of stormwater discharges generally occurs because of the inevitable "false" connections between the wastewater and storm water system.
As opposed to the quality of stormwater runoff, little is known about the effect of stormwater discharges on the receiving waters. Because of the low concentrations of the different pollutants in stormwater runoff the classical water quality criteria are generally met and hence no mitigating measures are considered necessary.
Recently, the IJsselmeer Development Authority initiated long-term studies on this subject in the newtown Lelystad. Within the framework of this research programme, physico-chemical and biological effects of stormwater runoff were investigated.
Lelystad can be characterized as a low level pollution area, because of its rural surroundings, limited size and its intensive street cleaning (Greiner and Jong, 1976). Also the incidence of false connections is extremely low due to strict control measures during construction of the sewerlines.
Because of this low level pollution, Lelystad represents an ideal situation with respect to stormwater discharges. In this paper special attention is directed towards the biological effects on the aquatic

biota in the receiving waters in Lelystad. The minimum biological effects due to stormwater runoff could thus be evaluated in this study.

1.2 Biological water quality assessment

The urban canals receiving the stormwater runoff serve ecological and recreational purposes besides their hydrological function. Ecological water quality criteria are now being developed for different water types. These criteria will not only imply physico-chemical, but also hydrological and biological characteristics.
Biological water quality assessment, as one of the tools in water quality monitoring, differs fundamentally from the classical physico-chemical methods. The most pronounced differences are shown in Table 2. Being mutually complementary, the two methods are to be applied simultaneously in in-depth studies.

Table 2 Main differences between classical and biological water quality assessment methods (after Anonymus, 1981).

Classical analysis	Biological analysis
- each analysis is time isolated ("snapshot"); even in an intensive monitoring programme, incidentally occurring extremes can be missed	- each sample gives a time-integrated indication; different periods are found for various organisms, i.e.: . algae: one to several weeks . hydrophytes: 3 - 6 months . macro-invertebrates: 1 - 2 years; extremes are time-integrated and indirectly measurable
- each analysis gives one factor, parameter	- each analysis gives an overall-view of the water as an ecosystem, including many parameters
- sampling and analysis are relatively time-extensive	- sampling and analysis are relatively time-consuming
- a direct quantitative indication is obtained	- still difficult to quantify; however quantitative criteria are being developed, generally the indication of pollution obtained is qualitative

Biological water quality assessment is more closely related to the original goal of water quality management: maintenance of a stable and diverse ecosystem. An aquatic ecosystem is defined as a complex of biotic freshwater communities, their functional, physical, chemical and biotic interrelationships.
The structure of an ecosystem can give information on the stability and diversity of the system, and can thus serve as pollution indicator as well. Freshwater communities are diverse. In case of the canals studied here, the most important inhibitants of the ecosystem are: fish, macro-invertebrates, benthic organisms, hydrophytes, planktonic algae, epiphytic algae (diatoms) and filamentous algae. Many methods have been developed for biological water quality assessment, using many different groups of organisms as indicators. One of the most widespread is the saprobic system, initiated by Kolkwitz and Marsson (1902) and modified by a.o. Sladeček (1973).

2 Materials and methods

2.1 Study area

In Lelystad, a newtown in the polder Flevoland, the stormsewers discharge into semi stagnant, shallow urban canals of limited size (5 - 10 m. wide, 1 m. deep). The canals also receive water from the subsurface drainage system containing infiltrated precipitation and upward seepage.
Upward seepage is a common phenomenon in a polder situated below the level of the surrounding lakes, and amounts to 0.12 m/yr (Uunk and Van de Ven, 1984). The water inputs of the canals are enumerated in Figure 1.

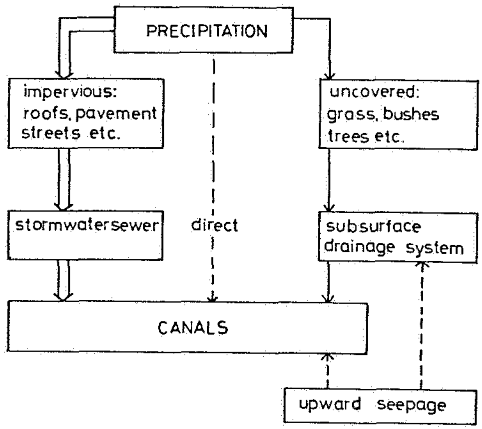

Figure 1 Hydraulic inputs of the urban canals in Lelystad

Two study areas were selected in Lelystad: "Bastion" and "Schouw". Both are primarily resendential areas. In Figure 2, the canals, the storm sewer outlets and the main flow direction are indicated. The sites B(astion) 1, S(chouw) 1 and S6 are not influenced by storm sewer discharges; S1 and S6 are situated in canals that at some distance are in open connection with a eutrophic lake. B3, S2 and S5 are situated in the vicinity of stormwater outlets.

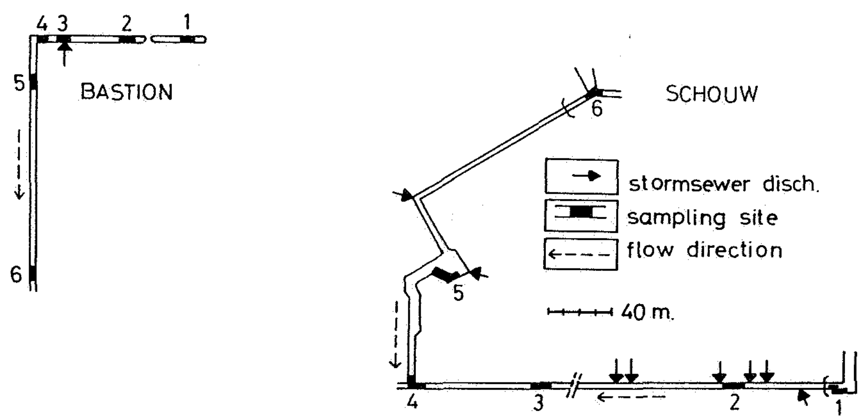

Figure 2 The study areas in Lelystad, The Netherlands

2.2 Sampling methods

Samples were taken every two weeks in the period May 15th - September 15th, 1983. At every sampling station, macro-invertebrates and epiphytic diatoms were sampled as well as the filamentous algae, present in mats floating on the water surface. Hydrophytes were identified on location.
Physico-chemical parameters were analysed in the laboratory. Macro-invertebrates were trapped by moving a net (mesh 500 μm) through the vegetation and over the bottom of the canal. For further identification, samples were taken to the laboratory after fixation with formaldehyde. Leeches, flatworms and water-mites were identified alive. Epiphytic diatoms were obtained from submerged reed-stalks (Phragmites) and fixated according to Werff and Huls (1957 - 1973).

2.3 Biological assessment methods

The biological water quality assessment applied in this study is based on the saprobic system, in which organisms are used as indicators for different degrees of organic pollution.
Sladecek (1973) listed in detail saprobic indices for approximately 2000 aquatic organisms.
The main saprobic classes are:
- oligosaprobic, almost no organic pollution;
- β -mesosaprobic, slightly polluted with organic matter;
- α -mesosaprobic, moderately polluted with organic matter;
- polysaprobic, very polluted with organic matter.

In order to obtain an overall picture of the effects of pollution, attention was also given to the more general characteristics of the ecosystem, such as diversity.

3 Results

In total, 96 samples were analysed, in which 176 species or other taxonomic entities were identified: 19 hydrophytes, 53 epiphytic diatoms, 99 macro-invertebrates and 5 filamentous genera of algae. The macro-invertebrates can be categorized as is shown in Table 3. The majority (110) of these 176 types of organisms were more or less specific for one or a few sampling stations, the others (66) were omnipresent. This phenomenon is illustrated in Table 4. In Table 5 the occurrence of aquatic organisms at the different sampling locations is shown.

Table 3 Number of species per subgroup of macro-invertebrates

Subgroup (larvae of)			Number of species found
Diptera	:	flies, mosquitos, midges	15
Trichoptera	:	caddis-flies	5
Ephemeroptera	:	mayflies	3
Anisoptera/Zygoptera	:	dragon flies/damsel flies	5
Hemiptera	:	water bugs	10
Coleoptera	:	water-beetles	25
Hydrachnellae	:	water-mites	7
Hirudinea	:	leeches	3
Mollusca	:	clams, mussels, snails	6
Crustaceae	:	water-fleas etc.	4
Restgroup	:	worms etc.	6

Table 4 Occurrence of aquatic organisms at the sampling locations

group	total number of species	number of species at sampling locations											
		B1	B2	B3	B4	B5	B6	S1	S2	S3	S4	S5	S6
macrophytes	19	13	9	9	9	8	8	13	10	12	11	13	10
filamentous algae	5	4	1	3	4	1	2	3	3	3	3	3	0
epiphytic diatoms	53	32	34	28	32	29	31	33	29	34	36	40	33
macro-invertebrates	99	61	36	47	46	54	55	63	52	54	51	50	31
Total	176	110	80	87	91	92	96	112	94	103	101	106	74

Table 5 Occurrence of aquatic organisms in relation to the presence of storm sewer outlets

group	number of species T	O	R	I	: species not or rarely found at storm sewer outlets	species, abundant near storm sewer outlets
macrophytes	19	16	1	2	: Lemna minor Myriophyllum spicatum	
filamentous algae	5	1	3	1	: Vaucheria sp.	
epiphytic diatoms	53	21	24	8	: *Amphora ovalis* Epithemia sorex Epithemia zebra Navicula radiosa Navicula simplex Nitzschia amphibia	*Nitzschia palea* Gomphonema parvulum
macro-invertebrates P = Planaria T = Trichoptera E = Ephemeroptera Z = Zygoptera H = Hemiptera C = Coleoptera M = Mollusca D = Diptera W = Worms	99	28	57	14	: Dugesia lugubris (P) Holocentropus picicornis (T) *Mystacides longicornis* (T) Oecetis Furva (T) Caenis horaria (E) Lestes viridis (Z) Notonecta lutea (H) *Haliplus obliquus* (C) Planorbis albus (M)	Chaoborus obscuripes (D) Chironomus plumosus (D) Glossiphonia complanata (H) Culex pipiens (D) Lumbriculidae (W)

T = total

O = omnipresent

R = randomly present

I = indicators

4 Discussion

The physico-chemical results are listed by Roos and Uunk (in press) and indicate that:

- there are no direct adverse effects of the storm sewer outlets on the oxygen balance of the canal water;
- indirect adverse effects are merely found in eutrofication stimulating activity;
- pollution of the underwater sediments is significant in the vicinity of the storm sewer outlets (phosphorus, some heavy metals and PCB's).

The biological results show that in general the canals can be classified as $\beta-\alpha$ -mesosaprobic, based on species composition. This is the most common saprobic state for Dutch eutrofic polderwaters, and indicates a light to moderate input of allochthonous organic matter (Caspers & Karbe, 1967) besides high nutrients-input.

Of the 176 (groups of) organisms identified, 76 could be classified as discriminative for a distinct saprobic state. In particular some Mollusca, Diptera, Hirudinea, Trichoptera and epiphytic diatoms were indicative.

The distribution of these discriminative organisms was:

- indicating slight organic input (β-mesosaprobic) : 50%
- indicating moderate organic input (α-mesosaprobic) : 35%
- indicating almost no organic input (oligosaprobic) : 15%

As is shown in Table 5, there are limited but distinct differences in species composition in the vicinity of the storm sewer outlets. This is quite remarkable, considering the low pollution level of the stormwater runoff in this study area. In particular some Diptera, Trichoptera and epiphytic diatoms were indicative for changes in the ecosystem. Species abundant near the storm sewer outlets are for the larger part (70%) indicators of the more polluted α-mesosaprobic state.

Species that are listed as not or rarely found near outlets represent the more undisturbed β-mesosaprobic state. An exemption to the above mentioned finding was sampling site S5, where species indicating the less polluted β-mesosaprobic state were even more abundant than at the other sites in the Schouw area. A closer investigation of the runoff area showed that only relatively unpolluted runoff from tennis courts was received in the S5 outlet. This illustrates the sensitivity of the aquatic ecosystem.
The highest species diversity of all sites was found at sites S1 and B1, which were not influenced by storm sewer discharges. In S6, however, the diversity was quite low. Wind-induced turbulence of the water and a finely textured clayey underwater sediment resulted in a high turbidity and poor conditions for hydrophytes and therefore also for the hydrophyte -related macro-invertebrates. Despite this low diversity of organisms, the species present at S6 were indicative for the β-saprobic state, the same as at S1 and B1.
Though situated upstream from a storm sewer outlet, site B2 shows a poor water quality. B2 is situated near a dead end of the canal, receiving accumulating duckweed and floating debris. Here, the stagnant conditions and covering of the water surface cause deterioration of the water quality and reduction of the available habitats for aquatic organisms.

5 Conclusions

In Lelystad, the storm sewer discharges cause a limited but significant disturbance in the species composition of the aquatic community in the receiving urban canals.
This is a remarkable phenomenon as the degree of pollution in runoff from these areas is reduced to a minimum.
Biological water quality assessment is a useful and sensitive instrument in assessing the impact of stormwater runoff.
As opposed to most hydrophytes, water-beetles and water-mites, some of the Diptera, Trichoptera and several epiphytic diatoms were particularly indicative for the saprobic state of the water.

In general, the canal water has a eutrofic and $\beta - \alpha$ -mesosaprobic character indicating an ample supply of allochthonous organic matter. In the vicinity of storm sewer outlets, the water tends to the more polluted α-mesosaprobic state, indicated by a shift in species composition.
Dead ends result in poor water quality and from a water quality management point of view, should be avoided as much as possible.

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POSTERS

MODELING TOTAL SEWAGE WATER DISCHARGE TO A REGIONAL TREATMENT PLANT

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Abstract
In the Netherlands, sewage water is often treated on a regional basis. In case of combined systems that are spread within a large region of several hundreds of square kilometers, reduction of the hydraulic capacity of the regional treatment plant seems possible, because of space-time variations in rainfall.

1 Introduction

In the Netherlands, sewage water, is usually treated on a regional basis by sewage treatment plants, that serve a number of combined sewage systems, spread within the region. Only a minor part of such regions consists of impervious area contributing to flow in combined sewage systems. In designing the regional treatment plant, the common design procedure is to set its hydraulic capacity equal to the sum of discharge capacities of the contributing sewage systems. In case of combined systems spread within a large area, however, it seems worthwhile to investigate the possibility of reduction in design capacity of the regional treatment plant, because of space-time variations in rainfall.

Space-time variations in rainfall and its effects on the hydraulic capacity of the regional sewage transport system and treatment plant are studied in the region of West-Brabant (450 km^2). In this region, rainfall is measured by nine recorders. Other components of the water-balance of the regional sewage transport system are measured as well.

2 Waterbalance of the regional collection system

For time interval j and combined sewage system i, the waterbalance reads:

$$Q(i,j) = cA_i\hat{X}_A(i,j) - O(i,j) - \Delta S(i,j) + \hat{F}(i,j)+Q_u \qquad (1)$$

where

Q : discharge to regional treatment plant

c : runoff coefficient for impervious area (c may be time-dependent)

A_i: size of impervious area

$\hat{X}_A(i,j)$: estimated areal rainfall over combined sewage system i

O : overflow

S : storage

F : estimated dry weather flow

Q_u: discharge to sewage system from "upstream" system(s)

In Equation 1, A_i and total available storage S are known, Q is being measured, $\hat{F}$ has been estimated from measurements of Q during long dry periods, and $\hat{X}_A$ is the estimate of areal rainfall by IRF-0 block-kriging.

The waterbalance (Equation 1) has been analysed for 37 rainfall events. These events occurred during the period July '83 to October '85, and have been selected by means of a peaks-over-threshold method. The rainfall events have been discussed in Witter (1986). It appears that without any monitoring and control of sewage flow to the collection system, there is already some reduction because of the variability of rainfall.

In Cusell (1985) two rainfall events have been extensively evaluated.

For those events it was concluded that both a local control policy, (reduction of local pump capacities at the start of an overflow) and a regional policy (reduction of local pump capacities in order to keep $\Sigma_i Q(i,j)$ below some threshold level) could have resulted in a reduction of $\Sigma_i Q(i,j)$ by 20%. However, without special provisions such as retention reservoirs, this would have resulted in an increase of the volume of overflows by 5-10% of total discharge, $\Sigma_j \Sigma_i Q(i,j)$, for that event to the treatment plant.

The poster that will be presented at the Conference on "Urban storm water quality and effects upon receiving waters" will give information on the selected rainfalls, and will demonstrate and evaluate some control policies of the regional collection system in terms of (i) possible reduction of hydraulic capacity of treatment plant, (ii) consequences on number and volume of overflows, (iii) costs.

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THE POLLUTION OF STORMWATER RUNOFF FROM SPECIFIC PAVED AREAS AND ROOFS

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Abstract

The object of this study is to produce practical directives, which should be applied by sewerage authorities who intend to disconnect specific paved surfaces or roofs from the combined sewerage system.
An inventory of experiences in the Netherlands in separating the runoff from paved areas and roofs was made. Many roofs and paved areas have already been disconnected.
Up to now there have been reported very few analyses of stormwater runoff from specific paved areas in the Netherlands.
It is determined that a number of gaps in knowledge need to be filled.
Further investigations have started in april 1986.

1 *Introduction*

In the Netherlands most of the sewerage systems are combined systems; they collect and transport the rainfall runoff from paved areas and roofs as well as the wastewater.
During the last decennia the amount of paved areas and roofs is increasing. This has negative consequences for the hydraulic loading rate of the sewerage systems and wastewater treatment plants, and in consequence for the surface water quality.
Disconnection of the flow from certain paved areas and roofs is a possible solution for this problem. The stormwater runoff is then discharged separately to surface water or is infiltrated.

2 The rainfall runoff from specific surfaces

There is a vast number of factors which influences the quality of rainfall runoff from paved areas and roofs.
Most important are: -the length of the preceding dry weather period;
-the quality, quantity and intensity of the atmospheric deposition;
-the types, the function, the age and the intensity of use of the paved areas and roofs.
Substantial variations may exist from site to site, seasonally at a given site and even within a single storm event.
Untill now there have been reported very few analyses of stormwater runoff from specific paved areas in the Netherlands.

Stormwater quality does not always meet the water quality requirements, for example

- the corrosion of galvanized steel in drain pipes, gutters, and fences can be a source of (sometimes problematic) high zinc concentrations in the stormwater runoff.
- the sediment collected by runoff from asphalt roads contains high concentrations of polycyclic aromatic hydrocarbons (PCA) and heavy metals produced by vehicular traffic.
- loading and unloading of certain materials, for example meal, diary products, can lead to an increase in the organic matter concentration in stormwater runoff.

3 Inventory of experience in disconnecting paved areas and roofs

An inventory of experience in the Netherlands was made by interviewing seven water quality authorities and nine municipal authorities (sewerage-system authorities). The most important reason for the municipalities to separate paved area runoff are the quantity requirements set by the water quality managers for the sewer system. Many roofs and paved areas are already kept separate. Only very few storm runoff samples have been analyzed for water quality. The effects on the recipient water quality has not been considered by the authorities.

4 Research needs

A number of gaps in knowledge needs to be filled before practical guide-lines can be given on the separation of runoff from paved areas and roofs from combined sewer systems. In this respect the most important topics are an extensive and systematic sampling program of stormwater runoff from specific types of paved areas and roofs in the Netherlands and an inventory and assessment of simple separation techniques.

The second phase of the study started in april 1986. The following sampling program and analysis are planned: after this, flow weighted samples will be collected for the most important surfaces.

land use	surface		analysis [1]
	type	number	
residential	roofs	4	heavy metals
	streets	4	EOCl, PCA,
	parking-places	4	oil, BOD,
	market-places	2	Nkj
industrial			
- with organic micro-pollutants	-	10	EOX, PCA
- with heavy metals	-	10	heavy metals
- with organic macro-components	-	5	BOD, Nkj

1) only the most frequently measured components are noted

5 Acknowledgement

This study is part of a large-scale research programme of the Ministry of Housing, Physical Planning and Environment (Ministry of VROM) and the Foundation for Applied Wastewater Research (STORA).
For this research programme they established the Netherlands National Research Committee on Sewerage and Water Quality (NWRW).

DETERMINATION OF URBAN STORM WATER QUALITY

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Abstract

The french investigation method to evaluate urban storm water quality by experimental catchments has been tested. The results obtained enabled us to define the constraints and the limits of any urban runoff study.

1 Introduction

One of the most important problems encountered by swiss towns that have been equiped a long time ago with sewage systems is the overloading of combined sewers and wastewater treatment plants in rainy periods.

The eventual changing of sewer networks into separate systems in built-up areas, and the possibility of treating the urban stormwater runoff are discussed; the considerable expense involved in such work requires serious cost/benefit studies.

Information about pollution load in urban runoff is needed; but the differences in the results of many studies carried out all over the world illustrate the difficulties in applying them to another situation.

And thus, everytime important work is planned, measurements and samplings are required.

2 Aims of the study

As Federal Institute of Research and Teaching, our main aim has been to make available to the interested public bodies, a method of investigation for evalution "in situ" of urban storm water quality. To do this, we have carried out a bibliographic study, chosen the most suitable method suggested by the documentation then tested it under real conditions. The results obtained enabled the method to be corrected, and to define their constraints and their limits.

As well we have added a scientific aim to the teaching one and try to understand better the pollution mechanisms of runoff water using the accumulated information.

3 Method

The selected method was developped in France and described by Hemain (1981) and Ranchet and Deutch (1982); it consists in choosing and equiping some experimental catchments and in calculating the pollution load of runoff water at their outlets. The studies of Göttle (1978) and Gutteridge, Haskins and al (1981) helped us to define the characteristics of the experimental catchments. The sampling and flow measurement problems in separate storm drainage have been resolved thanks to Grange (1982), Gros (1982), De Heer, Holenweg and al (1984).

In our testing we measured and analysed as many different parameters as possible, to describe the rain events and the outcome runoff.

3.1 Selection and equiping of catchments

Two experimental catchments near the city of Lausanne and the Geneva Lake have been used as studies, both with an area of approximately 6 ha, one in a residential town (bungalows or small 2-3 storeyed buldings) and the other in the suburbs (large 5-10 storeyed buldings).

The following criteria determined the selection of the catchments .

- impervious surface larger than 30 % of the catchment,
- uniform ground occupation,
- no definite source of atmospheric pollution in the surroundings,
- separate storm drainage system.

The equipment of the catchments was :
- raingauge,
- water level recording equipment,
- three automatic samplers (one of them is specially equiped to micropollutants study) with 24 bottels each,
- threshold switch-on mechanisms,
- "chronologue" : equipment to record the starting time of sampling.

3.2 Regulation of the equipment, measurement, sampling and analysis.

The water level equipments were set to measure the level every minute. At the measuring section, the relationship between level and flow were obtained by chemical ganging (using the dilution method).
The automatic samplers and threshold switch-on mechanisms were regulated after study of :
- the water level variations in the sewers for some rainy periods,
- the statistical relationship duration/frequency of the rain from 1985 to 1980.

The filled bottels were grouped to form the samples, taking into account the evaluation on the discharge of runoff water.
18 pollutants were analysed, systematically or occasionnally (several compounds of nitrogen and phosphorus, suspended solids, COD, DOC, several heavy metals, hydrocarbons and other organic micropollutants, ...)

4 Comments

The quality of a runoff event was described by total load and time-variations of each pollutants. The most significant sources of imprecisions were quantified. The biggest problem was encountered in gauging of measurement sections and maintenance of measurement equipement in rainy periods.

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THE APPLICABILITY OF IMPROVED OVERFLOWS IN THE NETHERLANDS

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Abstract

Improved overflows are at present mainly applied in combined sewerage systems in the United Kingdom to reduce the pollutant load on surface waters. The applicability of improved overflows in the Netherlands has been investigated. In a first phase an inventory has been made of various types of improved overflows. In a second phase the performance of one selected type i.e. the high side weir chamber, will be tested under field conditions.

1 Introduction

In 1982 the Netherlands National Research Committee on Sewerage and Water Quality has been established. Main study object of this Committee is the influence of stormwater overflows on the quality of the receiving waters. The research programme is divided into 11 themes, one of which is: the efficiency of overflow devices with respect to the reduction of the pollutant load on receiving waters. Within this framework a study has been performed by BKH consulting engineers on the applicability of improved overflows in the Netherlands.

2 Improved overflows

Main objective of improved overflows in a combined sewerage system is to reduce the pollutant load. This reduction mainly consists of a decreased spill of suspended solids as a result of an improved lay-out of the overflow chambar. The processes involved are sedimentation and flotation. Choosing the optimal dimensions, the overflow chamber works as a stilling chamber. In case of a vortex overflow an additional advantage is the ability of vortex motion to separate settleable solids.

In a literature survey an inventory of the types of improved overflows has been made. From the literature it can be concluded that improved overflows are mainly applied in the United Kingdom. Visits have been paid to research institutes and municipal maintenance departments in the United Kingdom to exchange views on the improved overflows.

3 Applicability in The Netherlands

3.1 Differences in sewerage systems

The effect of improved overflows is highly dependent on the specific characteristics of a sewerage system. Main differences in characteristics of sewerage systems in the Netherlands compared with those in the United Kingdom are:

- a mesh structure of the sewerage systems instead of a tree structure;
- relatively large sewers and low fall;
- a pump is used to transport the sewage to a treatment plant instead of transport by gravity;
- overflows are preferably situated in the upper part of the sewerage system and therefore no throughflow occurs;
- during an overflow situation the sewers are filled completely;
- relatively many overflows, low overflow discharge, low overflow frequency and low total discharge are typical for the situation in the Netherlands;
- a pronounced first foul flush does not always occur.

These items have been discussed with the researchers in the United Kingdom. It was concluded that the differences should lead to an overall positive effect on the performance of an improved overflow in the Netherlands. However research is needed to determine the optimal dimensions of the improved overflow in the Netherlands.

3.2 Selection of the most suitable improved overflows

The most suitable improves overflows have been defined on the basis of several criteria. The selection procedure is presented in table 1.

++ = very positive + = positive o = indifferent - = negative -- = very negative	separating efficiency	sensitivity	reliability	land requirement	applicability	discharge regulation	environmental impact	operation and maintenance	capital costs	operational costs	availability of design criteria	design stage/experience
Leaping weir overflow	--	--	++	++	+	--	o	o	++	+	--	+
Low side weir chamber	-	-	++	+	++	-	+	+	+	+	-	+
High side weir chamber	+	++	++	+	++	+	+	+	+	+	++	++
Stilling pond	+	o	++	+	o	+	+	+	+	+	++	++
Central vortex overflows	o	o	+	+	o	o	o	o	o	o	o	-
Stilling pond with siphon overflow	+	+	+	+	+	+	o	o	o	o	++	+
Stilling pond with jet-siphon	++	o	-	+	o	+	o	-	o	-	-	--
Expanding stilling pond	+	o	++	o	+	+	+	+	o	+	-	o
Overflow chamber without drop in invert level	+	o	+	o	+	+	+	+	o	o	+	--
High side weir chamber incorporating storage	++	++	++	-	++	+	+	+	-	o	+	+
Shaft overflow	++	++	++	+	-	+	o	o	o	o	o	o
Vortex overflow with peripheral spill	++	++	++	+	+	+	o	+	+	+	o	-

Table 1 Evaluative matrix of the selection procedure

For the circumstances in the Netherlands the high side weir chamber, the stilling pond and the stilling pond with siphon overflow are the most suitable. The vortex overflow with peripheral spill, the high side weir chamber incorporating storage and the shaft overflow are still in the design stage but the characteristics of these overflows with respect to separating efficiency, sensivity and reliability are very promising.

4 Comparison of the selected improved overflows with a conventional overflow chamber

To get an impression of the costs of the selected improved overflow structures compared with the costs for a conventional overflow chamber designs have been made for a representative situation in the Netherlands. The cost-accounting has a global and indicative character. The results are summarized in table 2.

	capital costs (%)	volume of overflow chamber (%)
high side weir chamber	175	350
stilling pond	190	500
stilling pond with siphon overflow	230	500
conventional overflow chamber	100	100

Table 2 Capital costs and volume of overflow chambers

5 Conclusions and recommendations

The circumstances in the Netherlands compared to those in the United Kingdom seem to have a positive effect on the performance of improved overflows. To ascertain this, research is needed.

As the high side weir chamber is the cheapest improved overflow and is widely applicable, it is recommended to start research for this type. To gain more insight in the effect of the high side weir chamber on the reduction of the pollutant load on receiving waters, a field study has to be carried out followed by research at a laboratory scale. This should result in design criteria for the situation in the Netherlands.

Notes

Preparations for the field study have been made. The high side weir chamber has been designed based on the British design rules and is being constructed in Rotterdam.

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QUALITY MANAGEMENT OF SURFACE WATER, RECEIVING STORM WATER DISCHARGES IN AMSTEL- EN GOOILAND

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Abstract

Minimum quality standards of surface water receiving storm water discharges in Amstel- en Gooiland are met only in a minority of the investigated locations. Sewer storage capacity does not seem to keep up with growth of waste water supply. In sewer construction rules are handled to estimate allowable storm water discharge frequency. Predictive models to estimate DO indicate that these rules do not garantee acceptable transient water quality conditions during a storm water discharge.

1 Introduction

Anticipating the water quality management plan 1986-1991 an investigation program was started in 1984 to evaluate the effect of storm water discharges on the quality of receiving water. The program is focused on urban canal-pond systems. A regional system of reservoirs for superfluous polder water is included in the project.
Investigation objectives are to determine whether :

- standing water quality conditions meet the minimum water quality standards (VROM/V&W, 1985) ;
- higher water quality aspirations will be in conflict with the licensed storm water discharge frequency (3 - 10 times a year) ;
- urban water quality management needs adjustement.

In this communication preliminary results and possible management alternatives are presented.

2 Investigation

Knowledge of surface water receiving urban storm water discharges in Amstel- en Gooiland is rather poor. The decision has been taken to set up a reduced monitoring program allowing a greater number of locations to be examined. The collected data are processed and interpreted with the experience obtained in other investigations on this subject, especially regarding storm water discharge quality. In the subjoined table a general outline of the investigation is presented.

General data	Monitoring data *
Size of the catchment area	Temperature, pH, Eh,
Constitution of the catchment area	Conductivity, Chloride
(industrial/residential, paved/	Transparency, Suspended solids
unpaved, terrain slopes)	BOD, O_2, Chlorophyl-a
Sewer system (combined/seperate,	Phytoplankton composition
pumping capacity/frequency,	NH_4-N, $NO_{2,3}$-N, Kj-N
storage capacity)	Ortho-P, Total-P
Dry weather flow	As, Cd, Cr, Cu, Hg, Ni, Pb, Zn
Rainfall	PCA, PCB, Pesticides, Phenol
Licensed and theoretical storm	E.coli (bacteriological indicator)
water discharge	Storm water discharge frequency/
	duration (automatic registration)

* Surface water sampling frequency once a month;
Sediment sampling once during the investigation.

For some of the investigated systems predictive models are used with regard to the quantification of storm water discharge pollution and the oxygen depletion in waste receiving surface water.

3 Results and discussion

Standing water quality conditions only meet the minimum water quality standards at locations with a seperate sewer system. The observed storm water discharge frequencies in almost all cases are higher than the licensed discharge frequencies. Theoretically computed average discharge frequencies based on actual sewer storage capacity and waste water supply, agree fairly well with the observed frequencies. Sewer storage capacity therefore does not seem to keep up with growth of waste water supply. The pollution of surface water receiving storm water discharge is mainly determined by the parameters BOD, O_2, total-N, total-P and to a lesser degree by the parameters NH_4-N, PCA and phenol. Sediment can always be qualified as moderately to seriously polluted (VROM, 1983). Model calculations used to estimate acceptable storm water discharge frequencies indicate that rules handled to determine acceptable storm water discharge frequencies in sewer construction do not garantee acceptable water quality conditions (Verstraelen et al., in prep.).

4 Possible management alternatives

The most obvious measure to be taken in order to meet water quality standards is to limit the amount of storm water discharges into receiving surface water. Sometimes this can be achieved by simply altering the sewage pumping regime during periods of rainfall. However, often it involves enlarging sewer storage capacity. Economic alternatives in consideration are:

- flushing during periods of heavy rainfall;
- enlarging and/or deepening of urban waters;
- installation of tanks for waste water storage;
- transposition of storm water discharge:
- artificial aeration.

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THE U.K. RIVER BASIN MANAGEMENT (RBM) PROGRAMME

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Abstract

A programme of research which seeks to establish techniques for the effective management of river quality in the U.K. is described. The initial phases of the programme are concerned with the impact of combined sewer overflows on receiving streams; work is currently under way to quantify overflow impacts and to develop appropriate sewer and river quality modelling techniques.

1 Introduction

This programme has been established to review, and where possible fulfil, the needs of the Water Industry to effectively manage the quality of rivers in the U.K. Early phases of the programme are concerned with the impact of combined sewer discharges on receiving streams, which has been defined as an area of major concern in many regions of England. Concurrently, the more wide-ranging problems associated with river quality management are being reviewed with the objective of identifying modules of work which will be necessary in the longer term. The growing agricultural pollution problem, the significance of pollution "incidents" and the problems of effective consent setting for all types of discharges are likely areas for future consideration. This note restricts itself to describing the objectives, targets and benefits to be derived from the short-term programme of sewerage impact research which is currently in hand.

2 Definition of the problem

There are in excess of 3500km of rivers in England and Wales that are classified as being of poor to bad quality in terms of their chemical composition and the uses that can be made of them[1]. The effect of discharges from storm sewage overflows (SSOs) on combined sewer systems has been identified as a major contributory cause of this situation. It has been estimated[2] that there are some 10000 to 12000 SSOs in England and Wales and that by any reasonable standards approximately 40% may be regarded as unsatisfactory. Clearly then the problem is extensive, although it tends to be concentrated in the north of England. One Regional Water Authority serving part of the north has estimated that expenditure of £3700 millions is necessary to attain acceptable standards for all its inland surface waters. Of this figure some £1500 millions is directed towards the rehabilitation of sewer systems causing pollution. Thus the problem is of major importance to the future funding requirements of the U.K. Water Industry.

3 Present practice

In recent years SSOs have frequently been designed to "Formula A", a criteria devised by the Technical Committee on Storm Overflows[2] which is dependent on the dry weather flow, contributing population and industrial inputs. Such criteria approach the subject from the point of view of the sewerage system, potentially to the detriment of the receiving stream.
The Sewerage Rehabilitation Manual[3], whilst recognising the shortcomings of existing methodology in this area, was unable to offer any major advances to aid the effective rehabilitation of sewer system causing pollution. A more recent update of this Manual (published in April 1986) has embodied the preliminary output of the RBM programme and sets out an interim procedure to establish rather more rational criteria for SSO design.

4 Needs for the future

The vast majority of conurbations in the U.K. are served by sewer systems which are to some degree combined in character. Financial constraints on sewerage expenditure are such that these systems are likely to remain in service for the foreseeable future. Hence, the bulk of future expenditure will be directed towards rehabilitation of existing systems rather than towards major reconstruction works. This means that a methodology is required to enable realistic criteria for the setting of SSOs to be established and to allow design of engineering works which attain these criteria.

Four elements have been identified where improved tools are required to enable these objectives to be fulfilled:

(i) Appropriate rainfall inputs to sewer flow simulation models,

(ii) A sewer flow quality model,

(iii) A river impact model,

(iv) A comprehensive receiving water quality classification system.

Considering each element in detail:

4.1 Rainfall inputs

The use of design storms is only appropriate to the design of works where peak rate of flow is important. New sewer construction falls into this category. In looking at the performance of SSOs, the volume, frequency and duration of storm events is of interest. Hence a "time series" of historic storms has been synthesised which represents the complete spectrum of rainfall events occurring over a period of time. At present the series is rather crude but further development will enhance the statistical validity of the tool.

4.2 Sewer flow quality model

WASSP-SIM is the most widely used sewer flow quantity simulation programme in the U.K. Hydraulic Research (the authors of WASSP-SIM) have been contracted by the Department of Environment to develop, in collaboration with WRc, a complementary quality modelling capability for this programme.

4.3 River impact model

The effect of SSO discharges which is of most immediate concern is that of the short term impact in terms of DO sag and toxics. Hence the immediate program is intended to develop a river impact model which simply looks at the implications of an SSO discharge on the reach of river immediately below the discharge point. More comprehensive and sophisticated models may follow in the future.

4.4 Receiving water quality standards

Existing water quality classification systems[4] are based on statistical compliance of routinely taken samples. Such an approach is not appropriate to consideration of the impact of SSO discharges. A more comprehensive framework which considers the short term pollution effects of intermittent discharges is required. Present activity is directed towards the gathering of existing knowledge in this area and the setting of preliminary standards. Subsequent research will fill the gaps which are identified by the current review.

5 Conclusion

A great deal of data collection and fundamental research is required to enable the targets that have been described to be produced. Much of this activity is currently in hand and, in addition to feeding into the overall programme, these activities are frequently providing much useful "spin-off" information and knowledge. The bulk of the data gathering activity for the programme is being handled by the University of Manchester under the guidance of Dr A J Saul. The three papers presented to this conference by K Howard, L Pearson and R Thornton (in co-authorship with Dr Saul) are examples of the benefits that are being derived in understanding the fundamental performance of combined sewer systems and their interaction with rivers.

It is anticipated that the programme of research outlined in this note will be completed in approximately 3 years. At this time the products which have been described will be available to the U.K. Water Industry and the wider areas of RBM will be addressed.

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