

COMMISSIE VOOR HYDROLOGISCH ONDERZOEK TNO
TNO COMMITTEE ON HYDROLOGICAL RESEARCH

Verslagen en Mededelingen No. 33

Proceedings and Information No. 33

WATER IN URBAN AREAS



WATER IN URBAN AREAS



PROCEEDINGS OF
TECHNICAL MEETING AT
TNO

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SCIENTIFIC RESEARCH TNO, 1985

CIP DATA

Water

Water in Urban Areas: proceedings of technical meeting 42 (April 1985) / (ed. by J. C. Hooghart); with contributions of R. H. Aalderink . . . et al. – The Hague: TNO. – Ill. – (Proceedings and Information/TNO Committee on Hydrological Research; no. 33).

With references

ISBN 90-6743-075-7

SISO 649 UDC 556(492-21)

Subject heading: water; town/water pollution; town/hydrology; The Netherlands.

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APRIL, 1985

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INRODUCTION

M.K.H. Gast

Water is of great importance for a city. In many cities in Holland the sphere is characterized by water. What would Amsterdam be without its canals? The sight of Rotterdam as a world-port is characterized by the estuary of the river Rhine.

Control of the water is of vital interest for life in the city and for the functioning of the municipal society.

Inundations always cause great damage whether occurring in Amsterdam, in Jakarta or in Bangkok.

Water attracts people. Fishing, swimming and all kinds of recreation take place in, on or by the water.

Therefore the quality of water is of great importance.

General intention of the study-days

In 1977 the TNO Committee on Hydrological Research (CHO-TNO) spent a two-day study conference on this subject. In the period 1977-1985 different organizations had carried out sometimes costly research programs into aspects of municipal water. The findings of these various bodies and reports remained uncoordinated.

These study days were set up to review the results of the research executed during the period 1977-1985 and to determine the present state of experiences and opinions.

The following main subjects formed the basis of the program:

- quality and quantity of rain
- the inflow and discharge from the sewage system
- control of the water in the city
- control of the groundwater level.

The phenomenon rain

The international attention for "acid rain" won't have escaped anybody's attention. Also a large scale registration system has been developed to measure the effect of the so-called wet and dry deposition from the air on the pollution of water and soil. Central measurements are used to calculate the resources required to catch, drain and, if necessary, treat the rain water. Should local or regional statistical variations occur, these can form a basis for design optimization.

Inflow and discharge from the sewage system

Primary rain water is collected in the sewerage. How much of the rain water will reach the sewerage? Which factors play a determining part in this process? What is the influence of these factors on designing a sewage system?

The water quality is influenced by rain water discharging from the separated system or overflows from the combined system.

What is the rate of pollution coming from these discharges? What is the influence on the quality of the surface water and which processes occur?

By which methods can the discharges be counteracted?

Executed research in the past years give answers to the questions posed.

Control of the water in the city

The control of the water in the city has for centuries been of concern for the municipality. Water-boards in charge of the quantity control usually have vast experiences in function delimitation and in cooperating with the municipalities on this subject. Quality control has been a history of only decades. Several authorities are involved. Especially now that large sources of pollution have been brought to light, the teamwork between these authorities controlling the water quality in the city is once more seen as an important factor. It seems an optimum hasn't been found yet.

Control of the groundwater

The control of the superficial groundwater level is a point of great concern. On the one hand wooden foundations may not be allowed to dry out whilst on the other hand a drainage system of insufficient capacity will cause moisture problems in and around the houses. Which possibilities has a municipality in both the formal and the technical aspects to control groundwater? What is the relationship between a sewage system and groundwater? Which other factors play a part? These problems appear to be under urgent consideration in many cities.

The organization of the study-days

To achieve the greatest possible combination of knowledge and experience these study-days were organized by CHO-TNO in cooperation with the Netherlands Research Committee on Sewerage and Water Quality, IJsselmeerpolders Development Authority and the Municipality Lelystad. This new city recently built on and reclaimed from the IJsselmeer was also willing to act as a host for this conference.

QUANTITATIVE ASPECTS OF PRECIPITATION IN THE NETHERLANDS

T.A. Buishand and C.J.E. Schuurmans

ABSTRACT

A selection is made of quantitative aspects of precipitation which may be of interest for urban run-off problems. Although the country is flat and relatively small there are marked geographical differences in mean precipitation. The possible contribution to it by urbanization and other human factors is still unsolved. Annual and daily variations of precipitation can be understood in a general meteorological and climatological context. Precipitation data show a large interannual variability which makes it difficult to distinguish systematic changes from pure random fluctuations.

Depth-duration-frequency curves, precipitation profiles and areal reduction factors for short time-intervals are discussed in some detail. Results are compared with those in the relevant literature. Recommendations for use are given where appropriate.

Finally, the importance of the movement of individual rain storms for sewer system design is dealt with and a description is given of one of the largest single events which struck The Netherlands in recent times.

1 INTRODUCTION

The precipitation intensities in The Netherlands are on average not very large: an annual amount of approximately 750 mm is delivered in about the same number of hours, which means an average intensity of 1 mm/h. Urban run-off problems nevertheless exist, partly because very large positive departures from the average intensity can occur over short time-intervals. In order to solve such problems in an economic way detailed knowledge of the precipitation climate is needed.

In the paper presented here first some general characteristics of the distribution of precipitation in space and time are discussed in Ch. 2. Then Ch. 3 deals with precipitation amounts over short time-intervals. Finally, in Ch. 4 attention is paid to individual precipitation events. In the last chapter the emphasis is on very heavy events. The study of such events is important in view of flooding in the urban area and the impact on receiving waters (erosion of banks, flooding and violent pollution loadings).

2 THE PRECIPITATION CLIMATE OF THE NETHERLANDS

In this chapter characteristics of precipitation in The Netherlands are discussed mainly on the basis of average values. There are some marked geographical differences and there is also a pronounced annual cycle. At inland stations showers occur more frequently in the afternoon and early evening than on other times of the day.

Section 2.1 shows how the average annual amounts vary over the country. Differences between monthly averages are dealt with in Section 2.2. Daily variation is discussed in Section 2.3. In Section 2.4 attention is paid to long-term trends in precipitation.

2.1 Geographical distribution

Fig. 1 shows the distribution of the average annual precipitation amounts

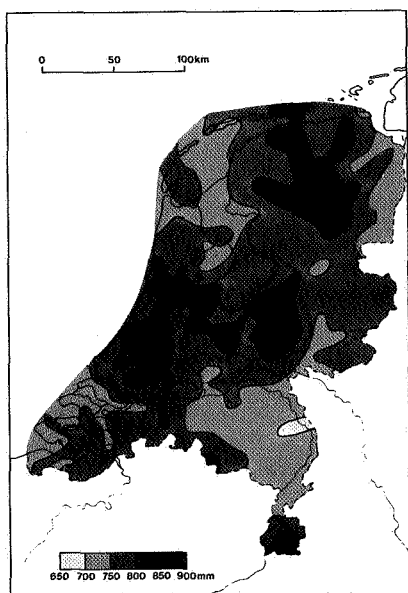


Figure 1 Average annual amounts
for the period 1951-80

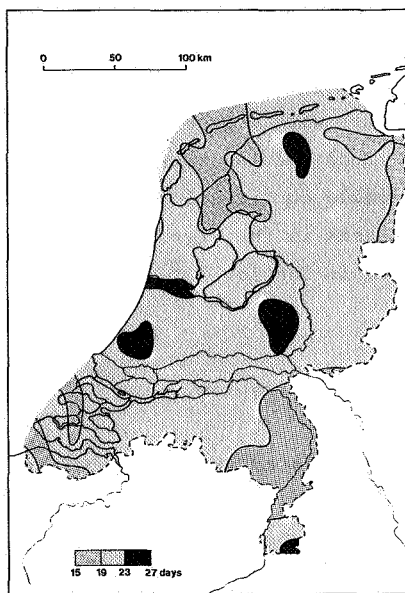


Figure 2 Average annual number of
days with a precipitation
amount ≥ 10 mm for the
period 1951-80

in The Netherlands. There are some marked differences, even though the country is flat and relatively small. Four locations, well separated from each other, have an average annual amount greater than 850 mm. The hilly areas (100 to 300 m above sea level) in the east-central part of the country and in the far south have the largest annual averages. Orographic enhancement of precipitation is of importance in these areas. The maximum in the west of the country is located in the so-called Randstad area, an urbanized area containing the four cities Amsterdam, Rotterdam, Den Haag (The Hague) and Utrecht. In the past ten years several studies have been made to relate the larger precipitation amounts in this area to urbanization and industrialization, see Witter (1984) for a review. Although some indications are found that cities like Amsterdam and Rotterdam might be the cause of changes in heavy precipitation

characteristics in their vicinity, the precipitation maximum in the west of the country is certainly not of urban origin alone. The maximum in the north-east of the country is not related to orographic features or to an urban effect.

The same geographical distribution is found for the annual average number of days with a precipitation amount ≥ 10 mm, which are given in Fig. 2. Such events occur on average nearly twice a month and give a considerable contribution to the annual amounts. The applicability of the patterns in the Figs. 1 and 2 for urban sewer design problems is discussed in Section 3.1.

2.2 Annual variation

Average monthly totals for five stations, well-distributed over the country, are given in Table 1.

Table 1 Average monthly totals in mm at De Bilt (52°06'N, 5°11'E), De Kooy (52°55'N, 4°47'E), Vlissingen (51°27'N, 3°36'E), Eelde (53°08'N, 6°35'E) and Beek (50°55'N, 5°46'E) for the period 1951-1980

Station	J	F	M	A	M	J	J	A	S	O	N	D
De Bilt	67	50	51	52	54	70	77	88	65	69	75	79
De Kooy	63	43	42	42	38	43	66	77	79	78	88	74
Vlissingen	61	49	46	44	44	58	71	77	67	73	77	70
Eelde	64	45	48	48	59	62	87	83	70	66	76	73
Beek	59	53	54	52	57	71	79	85	63	57	71	69

From the table the following conclusions can be drawn:

- (1) for all stations the period February till May is relatively dryer than the rest of the year,
- (2) the higher summer and autumn level of precipitation starts already in May and June at the inland stations De Bilt, Eelde and Beek; for the coastal stations De Kooy and Vlissingen the increase in mean precipitation begins about one month later,

- (3) inland stations have on average maximum precipitation amounts in July and August; coastal stations in late fall.

The properties of the annual variation of precipitation in The Netherlands find their explanation in a number of well-known facts concerning the annual variation of the general circulation over the area, instability due to surface heating in the summer season and the shift of the latter to the autumn in coastal areas due to the delayed cooling of the North Sea. For all seasons holds that independent of the precipitation causing systems the warmer the air at mean surface level the higher precipitation amounts one may get. The mere fact that warmer air contains more moisture is of importance here.

From the foregoing it follows that the annual variation is mainly a consequence of increased precipitation amounts and intensities at higher temperatures. For precipitation events of interest in urban sewer design the annual variation is therefore much more pronounced than for monthly averages.

2.3 Daily variation

There is an obvious daily variation in the occurrences of heavy showers. Buishand and Velds (1980) analysed the ten largest 15-minute events in De Bilt for a 72 yr. period and found that nine of these events occurred between 1700 and 2400 GMT.

During the summer period the largest average intensities on relatively warm days at De Bilt are found in the late afternoon and early evening. This is demonstrated in Fig. 3 where hourly averages on warm days are compared with overall hourly averages. During most hours of the day the overall averages are much larger than the warm day averages because warm days in summer have a low probability of precipitation, but the overall averages do not show a pronounced daily variation. Note further from Fig. 3 that hourly averages during the night following a warm day are much larger than those during preceding nights.

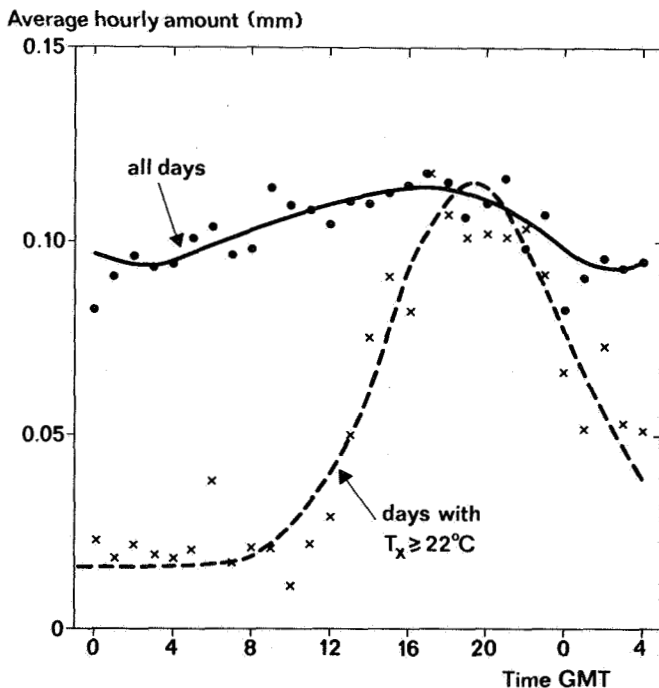


Fig. 3 Average hourly precipitation at De Bilt during summer (June-August) for the period 1921-70. The crosses and the dashed line refer to days with a maximum temperature $T_x \geq 22^\circ\text{C}$. After Können (1983)

In coastal regions there is a clear diurnal variation in autumn and early winter due to vertical instability over relatively warm water and coastal convergence. Maximum precipitation is found after midnight.

2.4 Interannual variability and trends

Fig. 4 shows that during this century an upward trend in the order of 50 mm has been observed in the 30-year average annual precipitation amounts at De Bilt. Parallel with this trend an increase of about 2 is found in the annual number of days on which the precipitation amount exceeded or equalled 10 mm. These changes are small in comparison with the geographical differences shown in the Figs. 1 and 2, but nevertheless it is often supposed that they have practical relevance. However, the

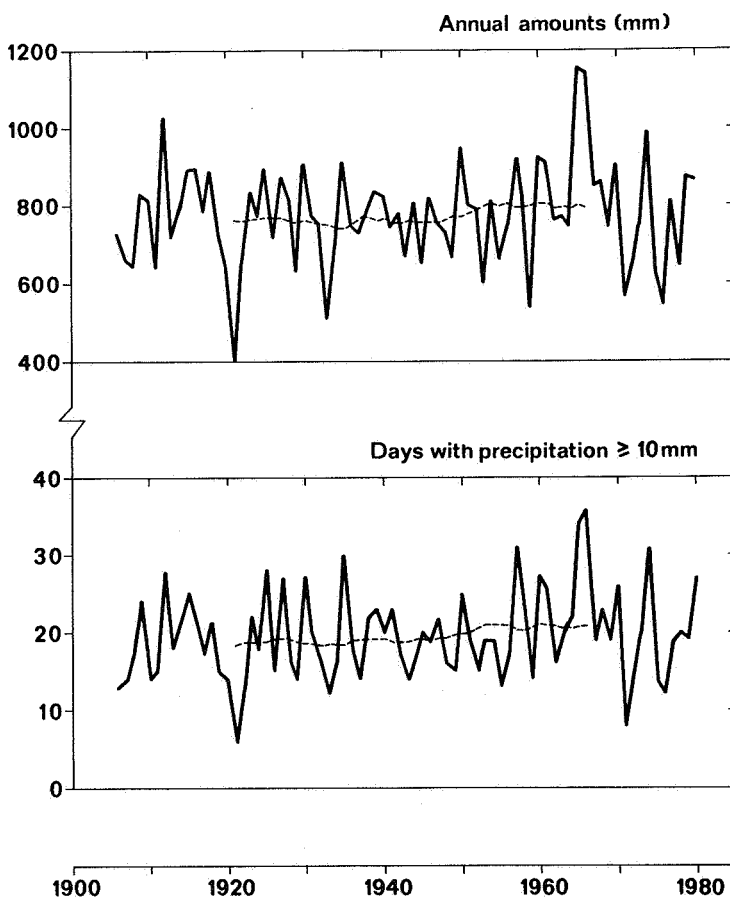


Figure 4 Annual amounts and number of days with a precipitation amount ≥ 10 mm at De Bilt for the period 1906-80. The dashed line refers to a 30-year moving average

reasons underlying these trends are rather obscure, which means that extrapolation is not justified.

From a practical point of view the magnitude of the interannual variability may be of more importance than the unexplained small trend. From Fig. 4 it is seen that annual precipitation amounts may vary from year-to-year by a factor of 2 to 3. For the annual number of days with a precipitation amount ≥ 10 mm the relative variability is even larger. As a consequence the standard deviations of a 30-year average are rather large: 24 mm for the average annual amounts, and 1.1 for the average

annual number of days with a precipitation amount ≥ 10 mm. This is about half the magnitude of the upward trend mentioned earlier. Because of these rather large standard deviations it is difficult to discriminate between systematic changes and random fluctuations.

From a purely theoretical point of view evidence has been presented that the present increase of CO₂-concentration of the atmosphere will cause higher temperatures and a more intensive hydrological cycle (Manabe and Stouffer, 1980). Sensitivity studies using three-dimensional climate models, with realistic topography, have shown that relatively large changes of regional precipitation patterns may take place, in case the CO₂-concentration is doubled or quadrupled (Manabe et al., 1981). A 10% real change in either direction of the annual or seasonal precipitation levels discussed above seems to be a realistic possibility for the next century.

3 PRECIPITATION AMOUNTS OVER SHORT TIME-INTERVALS

In urban sewer design precipitation amounts over short time-intervals are important. Often one is interested in the probability distribution of precipitation amounts for durations shorter than 1 day. Information about this probability distribution is usually presented in the form of depth-duration-frequency curves. Another topic is the selection of a suitable precipitation profile. Further the use of frequency information about precipitation amounts at a point may not be appropriate for precipitation amounts over an urban catchment area.

Section 3.1 deals with depth-duration-frequency curves for commonly required durations and return periods. In Section 3.2 some notes are given on the location of peak intensities within a precipitation event. Questions about areal precipitation are discussed in Section 3.3.

3.1 Depth-duration-frequency curves

To avoid confusion the idea behind depth-duration-frequency curves is

illustrated with an example. Fig. 5 gives average intensities of a shower in 5-minute intervals. For this shower the maximum precipitation amounts for a number of fixed durations D are determined. For instance, the maximum precipitation amount for $D = 30$ min is 16 mm in Fig. 5 which corresponds with an average intensity of 0.53 mm/min. It is allowed that there are dry episodes within the duration D . For each value of D the selected local maxima (or peaks) from a precipitation record form a so-called partial duration series or peak over threshold series. A depth-duration-frequency curve gives a percentage point of the probability distribution of these peaks as a function of duration D . The probability of exceedance is usually expressed in terms of the mean recurrence interval or return period T_p . Examples of depth-duration-frequency curves are given in Fig. 6. From the figure it is seen that the maximum 30-min amount in Fig. 5 is exceeded on average once in 2.5 yr.

In The Netherlands the discharge capacity of sewer systems has to be large enough that water in the streets may occur on average only about once in 2 yr. Return periods of 10 yr. and longer are less common but can be useful if one wishes to investigate measures to prevent serious flooding. For these design objectives one usually has to work with durations less than 1 hour. The precipitation events of interest are all high intensity storms that mainly occur in the summer season. Often these events are found in the second half of the day (Section 2.3). Since the critical durations are much less than one day it is dangerous to relate regional differences with respect to these design objectives to regional differences of heavy daily rainfalls. For instance, from the literature it is known that orography has a much smaller impact on the occurrence of high intensity storms than on the distribution of daily amounts.

Another demand on combined sewer systems in The Netherlands is that overflows to waters in the urban surroundings may occur only with a frequency in the order of 5 times per year. Overflows are not only caused by short heavy showers but can also occur during prolonged frontal rains. The diurnal variation in the occurrence of overflows is therefore much less pronounced than in flood occurrences. For problems concerning the frequency of overflows durations of several hours are

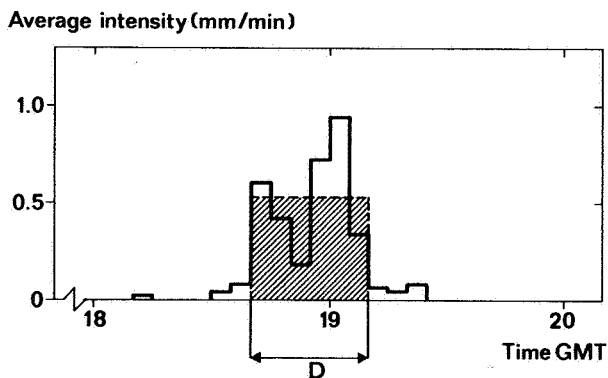


Figure 5 Average 5-minute intensities in a shower at De Bilt (27 July 1972). The hatched area denotes the maximum precipitation amount for $D = 30$ minutes

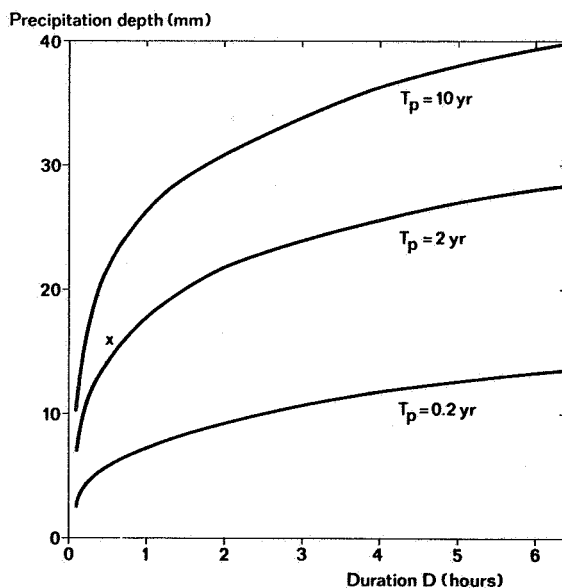


Figure 6 Depth-duration-frequency curves for De Bilt. The maximum 30-minute amount in Fig. 5 is marked with a cross. The curves for $T_p = 2$ and 10 yr. have been taken from Buishand and Velds (1980); to construct the curve for $T_p = 0.2$ yr. (5 times per year) use has been made of Buishand (1983) for $D \geq 1$ hour and KNMI (1968) for $D < 1$ hour

usually of interest. Because of these longer durations there is a rather strong relation between the frequency of overflows and large daily rainfalls. Regional variation with respect to this problem is therefore about the same as in the average number of days with a precipitation amount ≥ 10 mm (Fig. 2).

The discussion given above about rainfall-frequency information is far from complete. Other methods exist. It is however outside the scope of this paper to give further details. For more information about precipitation statistics in The Netherlands for sewer design the reader is referred to Buishand (1984).

3.2 Location of peak intensities and precipitation profiles

Besides frequency information about precipitation amounts for different durations knowledge is also needed about the shape of precipitation profiles in sewer design. It is not always justified to assume that the intensity is constant during the duration D . Although the depth-duration-frequency curve gives a profile, this is however an unrealistic one with a very sharp peak. Therefore the selection of precipitation profiles requires a further study of the original precipitation data.

Table 2 gives the position of the most intensive part within a duration D for precipitation data of the Emscher and Lippe catchment area (about 50 kilometres to the east of The Netherlands). For $D = 1$ hour there is a preference in summer of the most intensive part for the beginning of the precipitation event. Large 1-hour amounts are mainly due to showers from cumulonimbus clouds. In the beginning of the shower the cloud is in its mature stage giving the highest intensities. For The Netherlands the preference of peak intensities to the beginning of a shower has been confirmed by Levert (1958) from a study of a 2-year record of 5-minute amounts and by Buishand and Velds (1980) from an analysis of the ten largest 15-minute precipitation events in De Bilt for a 72 yr. period. For longer durations the position of the peak gets a more random character. Here heavy showers are less dominant and sometimes there might be two distinct rainy periods within the duration D .

Table 2 Location of the most intensive part within the duration D. The quantity p_i denotes the probability that the largest precipitation amount during an interval of length $D/4$ falls in the i -th quarter of the duration D. After Anderl and Stalman (1977)

		P_1	P_2	P_3	P_4
D = 1 h	summer ^x	0.37	0.29	0.22	0.13
	winter	23	26	18	33
D = 6 h	summer	33	23	29	15
	winter	23	23	25	30
D = 24 h	summer	27	18	26	30
	winter	22	24	19	36

^x summer = May-October; winter = November-April.

It should be noted that the maximum precipitation amount in a fixed duration D does not correspond with the total amount in a storm. This is for instance the case in Fig. 5. Only a few studies on precipitation profiles include precipitation outside the interval with maximum intensity. Information about this can be found in Normand (1971) and Arnell et al. (1983).

The selection of a suitable profile cannot be done without a sensitivity analysis. A sound approach is found in the UK Flood Studies Report (1975). Although this work deals with riverflow data the approach can also be extended to the design of sewer systems (Packman and Kidd, 1980). Additional results about sensitivity analysis can be found in the earlier cited publications of Anderl and Stalman (1977) and Arnell et al. (1983).

3.3 Areal reduction factors

The depth-duration-frequency curves in Fig. 6 refer to the distribution of precipitation amounts at a point. Especially for large urban areas it

may be necessary to adjust this distribution to give an appropriate distribution of areal averages. This can be done by multiplying the ordinates in Fig. 6 by the statistical areal reduction factor (ARF). The value of the ARF varies with duration D, size of the area A and return period T_p . For the 2-yr. event ARF can be obtained from the relation (Buishand and Velds, 1980):

$$1-ARF = 0.24 A^{0.23} D^{-0.32} \quad (1)$$

where A is in km^2 and D is in min. Table 3 shows that this relation gives good results for areas in Europe with a temperate climate.

Table 3 Comparison of areal reduction factors from Eq. (1) with those for the United Kingdom (NERC, 1975), Lund-Sweden (Niemczynowicz, 1982) and The Netherlands (Witter, 1984)

Duration D (min)	Area A (km^2)	Areal reduction factor ARF for $T_p = 2$ yr.			
		Eq.(1)	UK	Lund	Netherlands
5	8.4	0.77	0.78	0.79	-
5	21.0	0.71	0.70	0.69	-
30	8.4	0.87	0.89	0.90	-
30	21.0	0.84	0.85	0.85	-
60	100	0.81	0.79	-	0.76
60	250	0.67	0.72	-	0.66

For a 10-year event a smaller value for the ARF has to be used. Calculations by Bell (1976) and Witter (1984) suggest that for this return period the right-hand side of (1) should be multiplied by a factor in the order of 1.1 to 1.4. For $T_p = 0.2$ yr. the converse holds (Niemczynowicz, 1982).

It should be stressed that the statistical ARF has no other meaning than a ratio between the areal average and point amount with the same return period. It cannot be used to convert a point value of an individual storm to an areal value. Further in studies on areal variation of precipitation it is not always obvious to look at areal averages. For

instance, the differences over the region are of primary interest if one wishes to determine the optimal hydraulic capacity of a regional sewage treatment plant (Bakker et al., 1983).

4 CHARACTERISTICS OF INDIVIDUAL PRECIPITATION EVENTS

This chapter starts in Section 4.1 with a discussion on storm movement. Attention is paid to the distribution of storm speeds and to the significance of this subject for sewer design. Section 4.2 deals with the "Gouda storm", being one of the most spectacular precipitation events in this century in The Netherlands.

4.1 Storm movement

The direction and speed of a storm are associated with the upper air motion. The storm track can often be related to the wind at the 700 hPa level (this isobaric level has a height of about 3 km). A study on storm movement near London has been made by Shearman (1977). From a network of self-recording gauges in Surrey the direction and speed were derived for the 20 largest storms in every year. Information about the 700 hPa wind could be obtained from upper air measurements in Crawley (Sussex). Table 4 compares the frequency distribution of storm speeds with the distribution of the 700 hPa wind. There is a reasonable correspondence between the two frequency distributions. Further, the 700 hPa wind speed distribution at Crawley during heavy precipitation corresponds nicely with the 700 hPa wind speed distribution during rain at Trappes near Paris. Therefore, it may be assumed that the results in Table 4 hold for The Netherlands too.

Table 4 learns that the average storm velocity is in the order of 15 m/s (50 to 60 km/h). A typical example of the spatial distribution of daily precipitation amounts from rapid moving heavy showers is given in Fig. 7. During a period of about five hours the storms moved from Vlissingen in the south west of the country to Groningen in the north east. The isohyets are oriented in the direction of the storms which corresponds in

Table 4 Frequency distribution of storm speeds and wind velocities in m/s at the 700 hPa level. The table is based on data from Shearman (1977) and Misme (1980). The storm speed distribution refers to heavy storms of duration up to 2 hours. The 700 hPa wind at Crawley has been taken at times as close as possible to the storms in Surrey; measurements of the 700 hPa wind at Trappes have been considered during rainy periods

	Probability of non-exceedance (%)						
	1	5	10	20	30	40	50
Storms Surrey	2	7	8	11	14	18	-
700 hPa wind Crawley	4	5	7	10	12	14	15
700 hPa wind Trappes	3	5	7	10	13	15	17

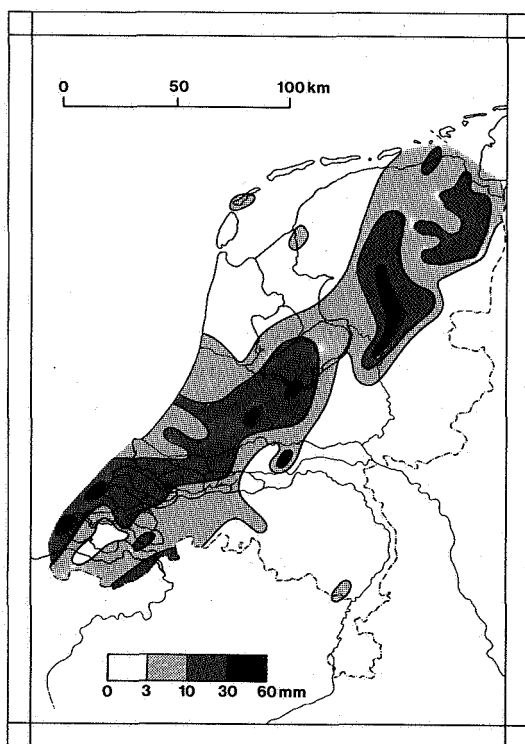


Figure 7 Spatial distribution of 24-hour precipitation amounts registered in the morning of 3 August 1967. After Buishand and Velds (1980)

this case very well with the 700 hPa wind direction.

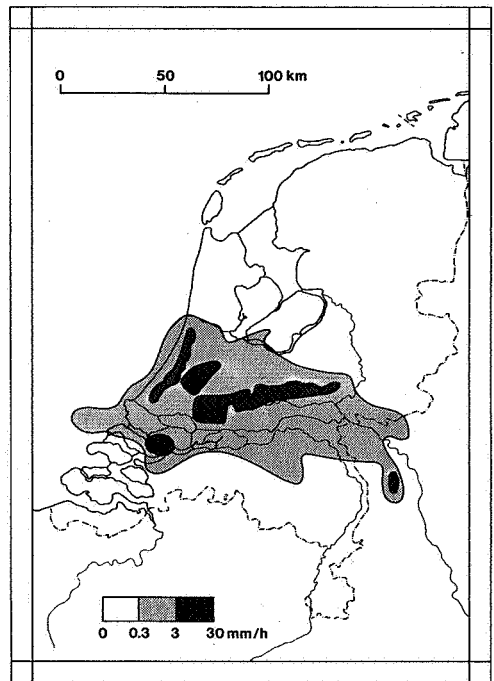
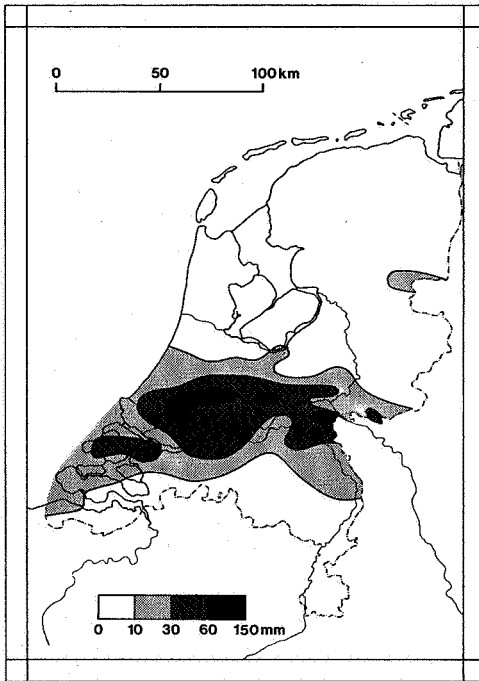
In most occasions the storm speed is much larger than the flow velocity in the sewer system. As a consequence lowering or reinforcement of peak flows due to storm movement may in general be disregarded in sewer design. This has been confirmed in a detailed study by Niemczynowicz (1984).

4.2 The "Gouda storm"

A remarkable precipitation event occurred on 23 June 1975. In the late afternoon and evening heavy showers developed over a large area. In Gouda, a small town 20 km to the north east of Rotterdam, a 24-hour precipitation amount of 145.5 mm was registered. At the moment of the present technical meeting this daily value is the 3rd largest in the history of precipitation measurements in The Netherlands.

The synoptic situation giving rise to the showers was very complex. A low pressure area over southern Germany brought warm air to The Netherlands. The maximum temperature in De Bilt was 27°C. Heavy thunderstorms occurred in an almost stationary trough (area with relatively low air pressure) over the central part of The Netherlands. The precipitation amounts were considerable due to the large humidity and long duration. Noteworthy is the fact that the air pressure was 1019 hPa during this precipitation event. The wind in the upper layers was light from the east-north east; at the ground mainly northerly winds were observed.

Fig. 8 shows the daily precipitation amounts, a radar picture and the hourly amounts at Zestienhoven (Rotterdam, airport). A precipitation amount of nearly 40 mm was registered between 1500 and 1600 GMT. The mean return period of this value is about 100 yr. The showers in the early afternoon mainly occurred in the west of the country. During the second period of high precipitation in Zestienhoven the showers were spread over the whole central part of The Netherlands. The region of the showers on the radar picture corresponds roughly with the area within the 10mm isohyets. The total 24-hour precipitation amount at



Precipitation amount (mm)

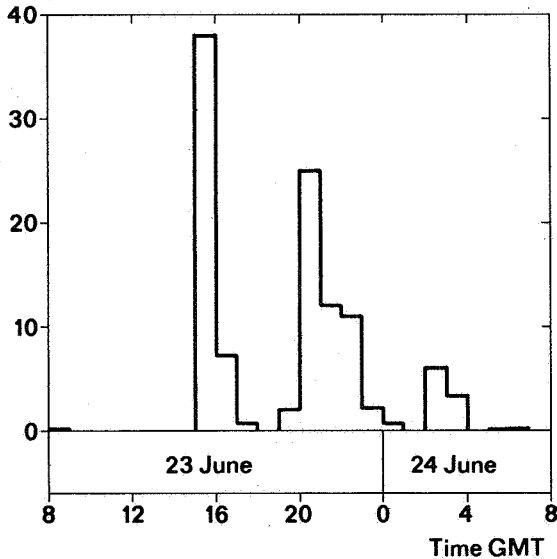


Figure 8 Heavy precipitation on 23-24 June 1975. The left map gives the measured 24-hour amounts on 24 June, 0800 GMT. The right map shows the 2100 GMT radar picture from Schiphol (Amsterdam airport). Precipitation intensities in this picture are very rough estimates. The graph gives the temporal variation of the hourly amounts at Zestienhoven (Rotterdam airport)

Zestienhoven was 109 mm and all precipitation stations in Rotterdam registered more than 50 mm. People in this area experienced a very unpleasant aspect of Water in Urban Areas.

REFERENCES

- ANDERL, B. and STALMANN, V., 1977. Temporal variation of precipitation intensities in design storms for the Emscher and Lippe area (in German). Eigenverlag Emschergenossenschaft-Lippeverband, Essen, 48 + 12 p.
- ARNELL, V., HARREMOËS, P., JENSEN, M., JOHANSEN, N.B. and NIEMCZYNOWICZ, J., 1983. Review of rainfall data application for design and analysis. In: Rainfall as the basis for design and urban run-off analysis, p 7-51. Preprints Specialised Seminar, Techn. Univ. of Denmark, Copenhagen.
- BAKKER, K., HARTONG, H. and WALTER, J.J.W.M., 1983. Variations in point precipitation and the capacity of sewage treatment works (in Dutch). H₂O, 16: 17-21.
- BELL, F.C., 1976. The areal reduction factor in rainfall frequency estimation. Rep. 35, Inst. of Hydrol., 58 p.
- BUISHAND, T.A., 1983. Distribution of D-hr precipitation amounts (D = 1, 2, 4, 6, 12, 24 or 48) in The Netherlands (in Dutch). Wetensch. Rapp. W.R. 83-5, Kon. Ned. Meteor. Inst., De Bilt, 58 p.
- BUISHAND, T.A., 1984. Precipitation data for storm sewer design (in Dutch). H₂O, 17: 195.
- BUISHAND, T.A. and VELDS, C.A., 1980. Climate of The Netherlands 1: Precipitation and evaporation (in Dutch). Staatsuitgeverij Den Haag, 206 p.
- KNMI, 1968. Detailed analysis of records of pluviographs A. Frequency distributions of the amounts of precipitation in periods of 5 to 660 minutes. De Bilt, 1928, 1933, 1951-1960. Publication 147, Kon. Ned. Meteor. Inst., De Bilt.
- KÖNNEN, G.P., 1983. The weather in The Netherlands (in Dutch), Thieme, Zutphen, 143 p.
- LEVERT, C., 1958. The morphology of a rainfall event (in Dutch). Hemel en Dampkring, 56: 183-193.

- MANABE, S. and STOUFFER, R.J., 1980. Sensitivity of a global climate model to an increase of CO₂ concentration. J. Geophys. Res., 85 (C10): 5529-5554.
- MANABE, S., WETHERALD, R.T. and STOUFFER, R.J., 1981. Summer dryness due to an increase of atmospheric CO₂ concentration. Clim. Change, 3: 347-386.
- MISME, P., 1980. Dimension of rainfall cells (in French). La Météorologie, VIe série (20-21): 299-301.
- NERC, 1975. Flood studies report (5 volumes). Natural Environment Research Council, London.
- NIEMCZYNOWICZ, J., 1982. Areal intensity-duration-frequency curves for short term rainfall events in Lund. Nordic Hydrol., 13: 193-204.
- NIEMCZYNOWICZ, J., 1984. Investigation of the influence of rainfall movement on runoff hydrograph. Part 1: Simulations on conceptual catchment. Nordic Hydrol., 15: 57-70. Part 2: Simulations on real catchments in Lund. Nordic Hydrol., 15: 71-84.
- NORMAND, D., 1971. Models for the investigation of runoff in towns (in French). La Houille Blanche, 26: 231-238.
- PACKMAN, J.C. and KIDD, C.H.R., 1980. A logical approach to the design storm concept. Water Resources Research, 16: 994-1000.
- SHEARMAN, R.J., 1977. The speed and direction of movement of storm rainfall patterns with reference to urban storm sewer design. Hydrol. Sci. Bull., 22: 421-431.
- WITTER, J.V., 1984. Heterogeneity of Dutch rainfall. Ph.D.thesis Agric. Univ. Wageningen, 204 p.

THE CHEMICAL COMPOSITION OF RAINWATER IN THE NETHERLANDS - WET DEPOSITION OF AIR POLLUTANTS

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ABSTRACT

The objectives of the National Rainwater Composition Measurement Network are discussed. The wet depositions of macrocomponents, some trace elements and organic substances measured in 1983 are presented. Finally, the composition of Dutch rainwater is evaluated.

1 INTRODUCTION

The term rainwater quality usually refers to the chemical composition. It means the condition measured by means of sampling and analysis, and subsequent interpretation of the data. The term quality, however, also means a subjective evaluation of the chemical composition, e.g., to judge whether the rainwater is suitable for a certain use. Such an ambiguous concept of waterquality has led to much confusion in other fields like groundwater, drinking water and surface water. With respect to rainwater the term chemical composition is generally, and internationally too, preferred to quality, unless one wants to designate the rainwater's functional aptitude.

The study of the chemical composition of rainwater is of interest for the evaluation of many scientific and practical problems.

Thus, a knowledge of the chemical composition makes it possible to evaluate the degree of air pollution, the sink terms in the atmospheric cycle of air pollutants, the corrosiveness of atmospheric waters and the effect of rainwater on the material balance of soils, waters and vegetation. For these reasons, many rainwater analyses have been done, mostly in the last three decades.

Figure 1 illustrates the atmospheric emission, dispersion and ultimate removal of air pollutants. Deposition at the earth's surface may lead to eventual effects.

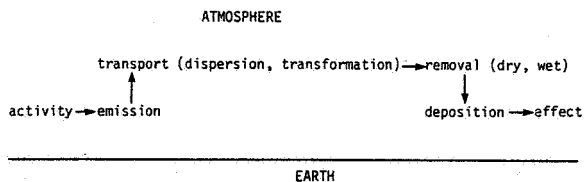


Figure 1 Atmospheric emission, dispersion and removal of air pollutants

Wet removal or precipitation scavenging processes are associated with precipitation. Both natural and pollutant trace constituents in the atmosphere are scavenged by a variety of hydrometeors: cloud droplets, raindrops, snowflakes, fog droplets, etc. The scavenging process depends on altitude (below-cloud or in-cloud), on precipitation type (rain, snow), on the nature of the pollutant material (particle, gas) and on the actual physical mechanism of uptake [Whelpdale, 1982a, b]. Dry removal processes take place without the mediation of precipitation and may take place continuously. Dry removal includes sedimentation of aerosols and direct absorption of gases. The term deposition commonly refers to the flux of material reaching the earth's surface as a result of the wet and dry removal processes.

Whereas the measurement of dry depositions is still in an experimental stage, wet depositions may be determined relatively easy by multiplying pollutant concentrations in precipitation and the amount of precipitation fallen at the same time. The outcome provides a practical approximation to the quantitative description of wet deposition fluxes [Whelpdale, 1982a, McMahon and Denison, 1979].

2 THE NATIONAL RAINWATER COMPOSITION MEASUREMENT NETWORK

The acidification of lakes in Sweden in the beginning of the fifties led to rainwater monitoring networks throughout Europe. In the Netherlands several networks were installed. Since January 1983 there is one National Rainwater Composition Measurement Network. It is co-operated by the Royal Netherlands Meteorological Institute (KNMI) and the National Institute of Public Health and Environmental Hygiene (RIVM).

The objective of the National Rainwater Composition Measurement Network is to monitor the chemical composition of rainwater in the Netherlands, i.e.,

- 1) to determine the spatial distribution (over the Dutch geographical area),
- 2) to detect temporal trends (of the annual means over the Netherlands) and
- 3) to verify air pollution models with respect to wet removal.

To this extent large-scale, background concentrations and depositions are measured every month at 21 'rural' sampling stations distributed over the Netherlands.

ad 1).

Very recently statistical optimization techniques have been applied by Van Egmond et al. [1985] in order to assess the relationship between the density of the Network (number of sampling stations) and the accuracy of the measurements of concentrations and depositions of inorganic components. As a result a network of 12 stations evenly distributed over the country instead of 21 would be sufficient to describe the spatial distribution accurately.

The relative interpolation errors at a point in between four stations for SO_4 , NO_3 and NH_4 would be 17%, 14% and 20% respectively for monthly concentrations. For depositions the errors are somewhat less. The errors for Na, K, Ca, Mg, Zn, F and Cl would be relatively large (about 30%).

The standard error in estimating the areal average concentration (and deposition) over the Netherlands would be 5%, 5% and 10% for monthly values of SO_4 , NO_3 and NH_4 , and 2%, 2% and 6% for the respective annual values.

It may go without saying that such a reduction will minimize the personnel endeavours with respect to sampling and analysis. For this reason, the reduction of the Network will be carried out soon. By then, the information gained by the local rainwater measurement networks will become even more important. At present local networks are operated by the provinces of Limburg, Noord-Holland and Drenthe, around Rotterdam (Rijnmond area) and Delft (one station); the province of Zeeland ceased monitoring in 1983 [KNMI and RIVM, 1985].

ad 2).

The statistical detection of temporal trends has been investigated in general by Buishand [1984], who reveals the following important points:

- The number of observations. A test on annual means over a relatively short period is not recommended because then one has too few degrees of freedom. On the other hand no gain in power can be expected by a higher sampling frequency than one sample per month since measurements refer to average concentrations over an interval.
- Variance reduction by using precipitation amounts (or other meteorological information) as an explanatory variable. For most components there is a negative correlation between concentration and amount of precipitation (high concentrations when there is little rain). The scatter of the concentrations decreases with increasing precipitation amount. Depositions are positively correlated with precipitation amount.
- Variance reduction by using areal averages of a number of stations. This works effectively for components which have little spatial correlation.

From a number of published series of measurements Buishand [1985] made further estimates of the magnitude of trends which can be detected in sequences of about 5 years (1978-1982).

Table 1 summarizes some of his results.

Table 1

component	<u>single stations</u>	<u>yearly averages</u>
	Van Egmond	Van Egmond
H ₃ O	14	10
NH ₄	16	13
F	15	7
NO ₃	12	9
SO ₄	11	9

Relative systematic change (in % per year) which can be detected with a probability of 80% in a series of monthly measurements of concentrations over 5 years (two-sided test, significance level $\alpha = 0.05$). The changes were calculated for every single station and subsequently averaged. The last column refers to changes calculated from monthly concentrations averaged over all stations. Adapted from Buishand [1985], data from Van Egmond et al. [1985].

The table indicates that the 'detectability' differs per component. The columns 'Van Egmond' are based on measurements at 12 stations of the National Network. They indicate that calculating the changes from monthly average concentrations (over 12 stations) instead of measurements at single stations, noticeably improves the detectability. With restriction to the components studied, this holds especially with respect to H₃O and F (cf. 3rd general point above). In the latter case, the Netherlands can be considered as one measurement field equipped with 12 samplers. For this reason the Network's 2nd objective relates to trends of yearly averages over the whole country. It should be noted that table 1 refers strictly speaking to concentrations.

In all, for the components given in table 1, a temporal trend of about 10% per year can be detected with 80% probability by means of a simple regression model applied to 5 years of measurements of a 12 stations

network. When 10 years of data would be available, the detectability of trends would be 3 times improved. The rough estimates may be improved by incorporation of seasonal variation and meteorological information (see above) into the regression model. In some cases a non-linear, e.g., exponential regression model might be more justified. In all it can be concluded that the detection of actual trends in the chemical composition of rainwater is still hampered by the shortage of the time series available (5 years). Nonetheless, further studies on trend detection are in progress.

ad 3).

In order to better verify the mathematical air pollution models run by the KNMI and RIVM together with other institutes, the 12 rainwater sampling stations remaining after optimization will be integrated if possible with other national environmental monitoring networks for air, soil and groundwater.

The integration should then contribute to more accurate and reliable source-receptor relationships.

Finally, concentrations and depositions are measured at 21 'rural' sampling stations. This means that local emission sources are avoided if possible, so that the station becomes representative of a larger area. Each sampling station of the Network is equipped with an open rainwater collector for the sampling of inorganic main components. Another collector is for trace elements, where a slight amount of perchloric acid is added to preserve the sample in the field.

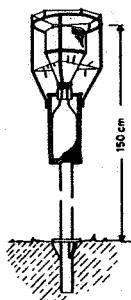


Figure 2 Open rainwater collector used in the National Rainwater Composition Measurement Network

The open rainwater collector (fig. 2) consists of a vulcathene funnel (orifice 400 cm^2) mounted on a 5 l sample container of opaque polyethylene. The sample container is protected from daylight to avoid the risk of loss of NH_4 and a proportional increase of H_3O ions, as well as changes in the K contents while the sample is standing in the field [Ridder, 1984 and Ridder et al., 1984]. A 450 μm sieve prevents insects from entering the container, and nylon wires around the funnel serve to scarify birds. The collecting orifice is at 1.50 m above station level. At every station a standard KNMI rain gauge at 0.40 m height records the amount of fallen precipitation (rain, snow and hail).

The methods of analysis of main components are summarized in the Network's annual report on 1982 [KNMI and RIVM, 1983], whilst those for trace elements are referred to in the annual report on 1983 [KNMI and RIVM, 1985].

The use of permanently open funnels is generally accepted for the sampling of falling precipitation, i.e., rain, hail and snow, though the latter are collected insufficiently and less. However, an unknown amount of dry deposition is collected at the same time, due to dry removal of gases and aerosol particles.

The dry contribution depends on the local situation and may lead to an overestimate of the wet and an underestimate of the total (wet + dry) deposition.

The contribution of dry removal is limited by the use of 'wet-only' collectors, where the funnel is closed by a lid during dry periods. Upon the onset of an precipitation event, the lid is automatically turned away. Comparative studies using open and wet-only collectors indicate that the dry contribution may account for 10-40% of the 'bulk' deposition of main components determined by open collectors [Ridder et al., 1984]. In 'clean areas' the differences are relatively small, e.g. 25% for SO_4 , NO_3 and NH_4 [Ridder and Frantzen, 1982, Ridder, 1984]. The measurements by means of permanently open collectors should best be regarded as approximations of wet concentrations and depositions. This holds in particular for the 'rural' stations of the National Network.

Table 2

component	median	lowest	highest
conductivity	4.6	2.0	31.3 (mS.m ⁻¹)
Na	7	0	96
K	0.5	0.1	2.1
Ca	1.2	< 0.3	4.2
Mg	0.9	< 0.1	10.7
Cl	8	0	111
SO ₄	4	1	12
SO ₄ correct	3.8	0.5	11.5
NH ₄	7	1	32
NO ₃	3	1	13
PO ₄	0.04	< 0.01	0.95
H ₃ O	2	0	12
pH	4.68	4.05	7.37 (units)
pot. acid	16.3	2.4	65.8
Zn	0.02	< 0.00	0.14
F	0.16	< 0.01	1.03

Wet depositions of main components (mmol.m⁻².month⁻¹) in 1983 determined by the National Rainwater Composition Measurement Network [KNMI and RIVM, 1985]. Median depositions are related to the mean monthly depositions calculated for every single station.

In the context of this 42th CHO-TNO technical meeting, the wet depositions of air pollutants determined by the National Rainwater Composition Measurement Network can be considered as basic influxes of these substances into the urban sewerage system. Surface run-off and other discharges into the sewers should subsequently be added to the basic influxes in order to calculate the complete loading of the system, as will be outlined in the following papers. For this reason the chemical composition of rainwater will be presented here mainly as depositions. On the other hand, it should be pointed out that concentrations may be related more directly to certain effects of rainwater, e.g. on vegetation, or to the corrosion of materials.

3.1 Main components

Table 2 summarizes the wet depositions of main components in the Netherlands in 1983.

The table presents annual medians of the monthly depositions, i.e. the medians of the mean monthly depositions calculated for every single station in 1983 [KNMI and RIVM, 1985].

Such a way of presentation eliminates the influence of extreme (high and low) results as can be seen from the 2nd and 3rd column. The variation between lowest and highest depositions (at some station and for some month) is rather large, both in space and time. Factors ranging from 10 to 100 are not uncommon. (The database was previously purged of outlying results caused by bird droppings, obvious procedural errors, etc.).

For the benefit of a discussion, which is largely based on the annual report on 1983 [KNMI and RIVM, 1985], the main components have been arranged to four topics of interest. The topics are: the influence of the sea, eutrophication, acidification ('acid rain') and industrial activities.

The annual median depositions of Na, Mg and Cl are almost entirely brought about by wet removal of seasalt aerosol. The depositions strongly decrease with distance from the coast. Figure 3 illustrates the typical spatial distribution and shows the isolines of conductivity that is mainly determined by the NaCl content. The K and Ca depositions originate from sea-spray to a lesser extent. The distributions of K and Ca would show additional enhanced depositions in the south-east of the country. The latter may be due to the collection of locally flown-up marldust. A minor part of the SO_4 -depositions (fig. 4) originates from sea-spray, preponderantly in winter-time. A correction based on the Na-content which is totally seaborn leads to corrected SO_4 -depositions (fig. 5). The corrected SO_4 -distribution, e.g.. in the south-east of the Netherlands, corresponds to industrial SO_2 -emissions.

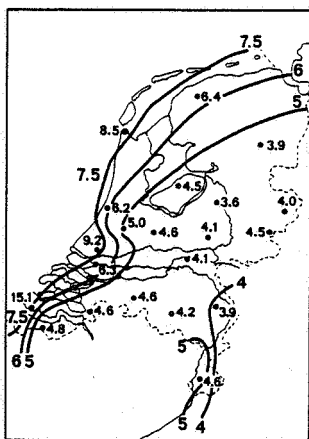


Figure 3 Spatial distribution of conductivity (mS.m^{-1}) in 1983

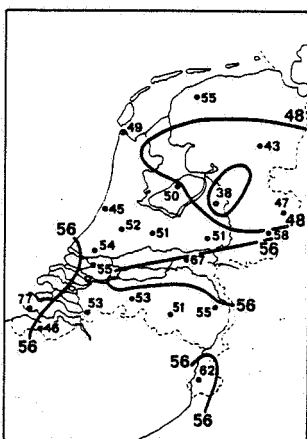


Figure 4 Spatial distribution of SO_4 -depositions ($\text{mmol.m}^{-2}.\text{month}^{-1}$) in 1983

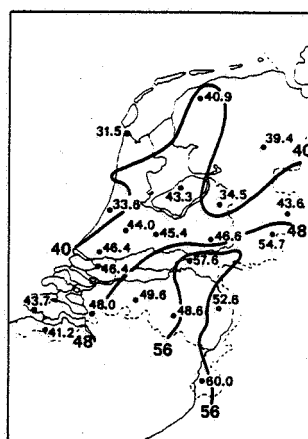


Figure 5 Spatial distribution of SO_4 -depositions corrected for sea-spray ($\text{mmol.m}^{-2}.\text{month}^{-1}$) in 1983

The eutrophying substances NH_4 , NO_3 and PO_4 show different spatial distributions.

NH_4 -depositions (fig. 6) increase landinward and reach high values in regions where intensive cattlebreeding takes place and NH_3 is emitted due to excessive dumping of natural fertilizer (manure) [buijsman et al., 1985]. NO_3 -depositions are relatively uniformly distributed as they originate principally from NO_x -emissions by automobile road traffic and space heating (fig. 7). The spatial distribution of PO_4 -depositions shows increased values around localised industries (fig. 8).

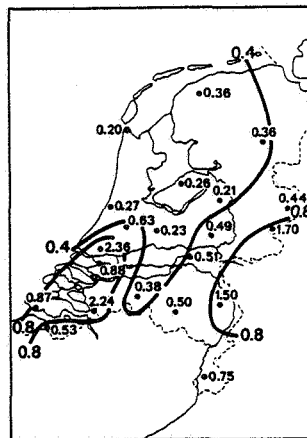
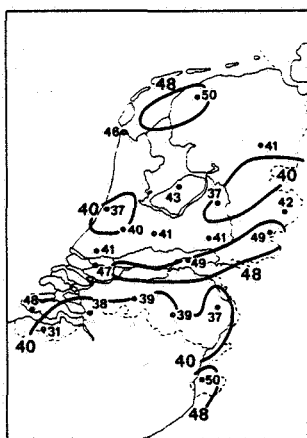
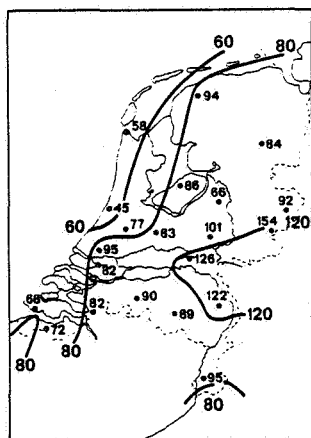


Figure 6 Spatial distribution of NH_4 -depositions ($\text{mmol.m}^{-2}.\text{month}^{-1}$) in 1983

Figure 7 Spatial distribution of NO_3 -depositions ($\text{mmol.m}^{-2}.\text{month}^{-1}$) in 1983

Figure 8 Spatial distribution of PO_4 -depositions ($\text{mmol.m}^{-2}.\text{month}^{-1}$) in 1983

The term 'acid rain' relates to acid and acidifying substances. The free acid content (base neutralizing capacity) is mainly due to the presence of strong acids like H_2SO_4 and HNO_3 . The depositions of free acid (H_3O , fig. 9) decrease up-country, and the pH-values increase accordingly (fig. 10). This phenomenon may be explained by neutralizing NH_3 -emissions there (see above). Upon deposition, however, bacterial oxidation of NH_4 may contribute to the acidification of surface water and soil. Complete nitrification of 1 mol NH_4 leads to 2 mol H_3O . Thus the total potential acid deposition reaches relatively high values up-country (fig. 11).

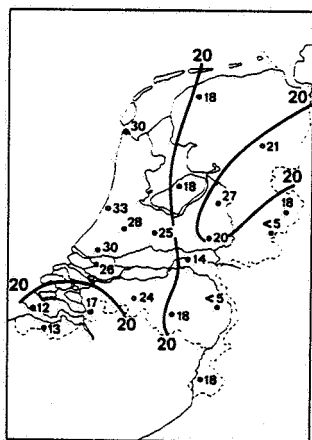


Figure 9 Spatial distribution of H_2O -depositions ($mmol.m^{-2}.month^{-1}$) in 1983

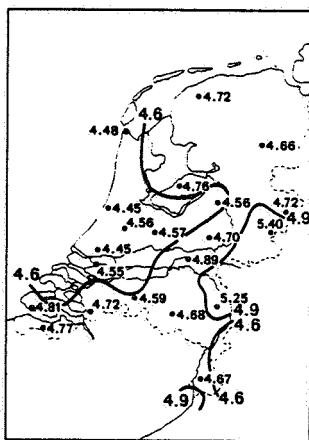


Figure 10 Spatial distribution of pH (units) in 1983

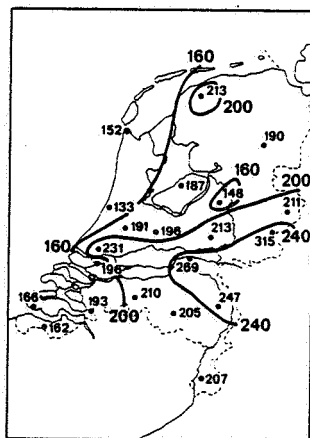


Figure 11 Spatial distribution of potential acid depositions ($mmol.m^{-2}.month^{-1}$) in 1983

The spatial distributions of Zn and F would reveal localised industrial emissions, e.g., in the IJmond area near Amsterdam.

3.2 Trace elements

Apart from main components, a comprehensive series of trace elements are measured by the National Rainwater Composition Measurement Network since 1983. The series includes Cd, Cu, Fe, Mn, Ni, Pb and V, as well as Al, Ba and Sr on an experimental basis. At the central station of the Network, De Bilt, As, Cr, Co and Se(IV) are measured in addition. Hg is not measured anymore because the sampling techniques are inadequate [KNMI and RIVM, 1983].

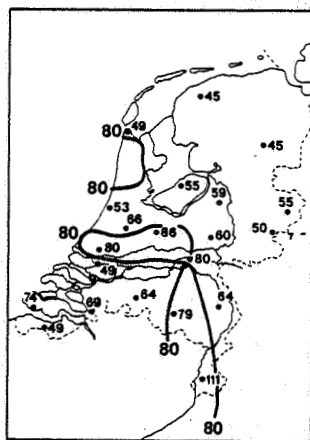
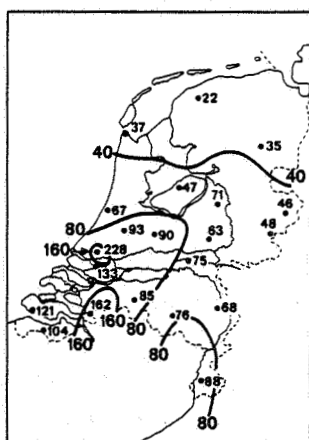
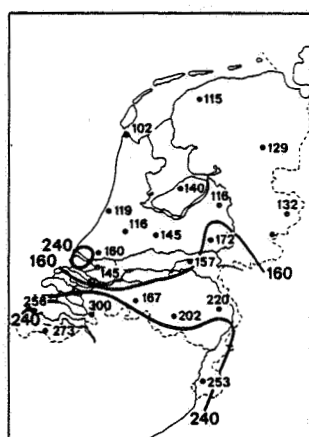
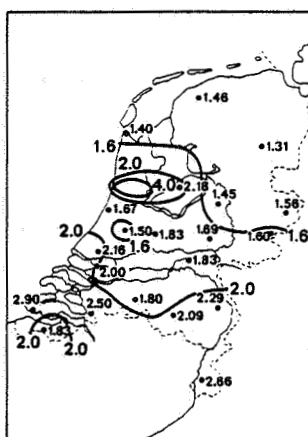
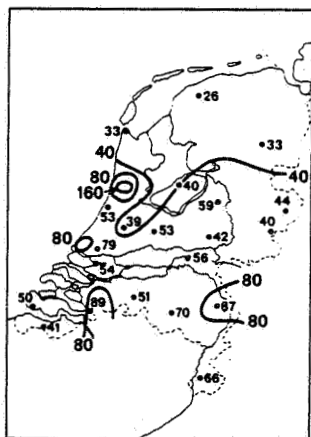
Table 3 summarizes the depositions of the former series of trace elements, based on the annual report on 1983 KNMI and RIVM, 1985. Again annual median depositions are presented, and the lowest and highest values indicated. The variation is in the order of 50 to 500. It should be noted that the detection of trace elements sometimes approximates the analytical limits. In such cases, notably Cd, inaccuracies may be large.

Table 3

trace element	median	lowest	highest
Cd	0.168	0.012	0.990
Cu	4.29	0.43	33.10
Fe	152.3	109.4	233.7
Mn	13.1	8.5	25.0
Ni	1.43	0.06	22.47
Pb	5.01	0.56	20.45
V	6.28	1.82	19.01

Wet depositions of some trace elements ($\mu\text{mol.m}^{-2}.\text{month}^{-1}$) in 1983 determined by the National Rainwater Composition Measurement Network [KNMI and RIVM, 1985]. Median depositions are related to the mean monthly depositions calculated for every single station.

The spatial distributions of the metal depositions mostly point to industrial sources, e.g., in the IJmond area near Amsterdam for Cu, Fe and Mn (figs. 12, 13 and 14), as is the case with Zn and F above. The deposition patterns of Ni and V indicate high values near Rotterdam that can possibly be attributed to petrochemical industries in the Botlek area (fig. 15). The isoline pattern of Pb-depositions is an exception to the above and shows a rather uniform distribution over the Netherlands (fig. 16). The high depositions in the west and the lower depositions in the north-east may be explained by the more or less intensive automobile road traffic in these parts of the country.



3.3 Organic substances

Organic substances are preliminary measured at 3 stations of the Network, including polynuclear aromatic hydrocarbons (PAHs), polychlorobiphenyls (PCBs) and organochlorinated pesticides. Sampling takes place in open glass rainwater collectors (orifice 600 cm²). The sampling of volatile compounds is being developed [KNMI and RIVM, 1985].

Sampling and analysis of PAHs were originally carried out in the framework of the National Coal Research Program (NOK-LUK) and has been reported by Van Moort and co-workers [1984, 1985] and Den Hollander, [1985]. Monthly concentrations of individual PAHs in 1983 were in the order of 10 to 500 ng.l⁻¹, whilst phenanthrene and fluoranthene showed the highest values [KNMI and RIVM, 1985].

PCBs have been investigated since 1979 using another type of glass collector at De Bilt. Table 4 summarizes the results of hexachlorocyclohexane (HCH) and PCBs in 1983 [KNMI and RIVM, 1985]. γ -HCH is the agricultural pesticide lindane, whilst the α -isomer is an impurity in the commercial product. The sum of PCBs was calculated by adding the amounts of 20 individual PCBs. Annual median depositions are presented as well as the range, but they are related to the monthly measurements of De Bilt only, and the unity is $\mu\text{g.m}^{-2}\text{.month}^{-1}$.

Table 4

component	median	lowest	highest
γ -HCH	1.25	0.40	21
α -HCH	0.88	0.25	1.2
Σ 20 PCBs	0.42	0.08	0.71

Wet depositions of some organic substances ($\mu\text{g.m}^{-2}\text{.month}^{-1}$) in 1983 determined by the National Rainwater Composition Measurement Network [KNMI and RIVM, 1985]. Median depositions are related to the monthly depositions at De Bilt.

Results from an earlier investigation by Hofstee et al. [1984], covering the years 1972 to 1982, revealed that from 1975 organochlorinated pesticides other than α - and γ -HCH could not be demonstrated anymore in rainwater. These pesticides include hexachlorobenzene, β -HCH, aldrin, dieldrin, endrin and telodrin, heptachlor and its epoxide, DDT, DDD and DDE.

3.4 Natural isotopes

The geographical distributions of the isotopic ratios $2\text{H}/1\text{H}$, $18\text{O}/16\text{O}$ and radioactive tritium (^3H) are expected to correlate with meteorological conditions during precipitation. For this reason rainwater from all stations has been analysed by the Isotope Physics Laboratory at Groningen since 1980. The results of the investigation will be reported in due time, after sufficient data will have become available. Meanwhile the data are available for hydrological studies and they have been reported annually [KNMI and RIVM, 1985].

4 DISCUSSION AND CONCLUSIONS

In this section the chemical composition of rainwater will be evaluated and some qualitative conclusions drawn. To this extent table 5, 2nd column summarizes the annual median concentrations (in contrast to the depositions above) in Dutch rainwater during 1983 [KNMI and RIVM, 1985]. The 1st column lists volume-weighted average concentrations of main components measured at a remote station in Alaska about 10 years ago [Galloway et al., 1982a]. For PO_4 only a H_2PO_4 value from Venezuela was available [ibidem]. The concentrations of trace elements are the median values compiled by Galloway et al. [1982b] of several different investigations in the Antarctic and Arctic. At the remote stations most of the precipitation was in the form of snow. The 3rd column of table 5 represents the Dutch standard for surface water quality taken from the Water Action Programme 1980-1984, i.e., the so-called basic quality for surface water [Rijkswaterstaat, n.d.].

Table 5

component	rainwater		basic quality surface water
	remote	Netherlands	
pH	4.96	4.68	6.5 - 9 (units)
SO ₄	3.6	64	1000
SO ₄ correct	3.6	56	-
NH ₄	1.1	101	70
NO ₃	1.9	48	700
PO ₄	0.6*	0.5	6
Cd	0.07	2	20
Cu	0.9	60	800
Ni	-	20	850
Pb	0.4	70	250

* H₂PO₄

Concentrations of some main components (volume-weighted averages, mmol.l⁻¹) and trace elements (medians, μmol.l⁻¹) in precipitation in remote areas of the world [Galloway et al., 1982a, b] and in the Netherlands (annual medians from tables 2 and 3). The last column denotes the Dutch standard for surface water [Rijkswaterstaat, n.d.].

From table 5 it appears that the pH of Dutch rainwater is about that in remote areas. The slight difference is due to neutralization by NH₃ emissions as discussed above. The concentrations of SO₄-corrected, NH₄ and NO₃ higher than in remote areas. Amazingly the concentration of PO₄ in Dutch rainwater is about that in remote areas. No explanation could be found for this phenomenon. It is also evident that rainwater in the Netherlands contains considerable amounts of trace elements. The latter amounts may be overestimated because open collectors were used.

Table 6

component	rain	Rhine
SO ₄	185.10 ³	6.10 ⁶
SO ₄ correct	175.10 ³	-
NH ₄	60.10 ³	60.10 ³
NO ₃	90.10 ³	1400.10 ³
PO ₄	2.10 ³	130.10 ³
Cd	9.1	35
Cu	130	760
Ni	40	400
Pb	520	660
γ-HCH	0.7	1.0
α-HCH	0.4	0.2

Loading (in tons per year) of the Netherlands with pollutants via rain (depositions from tables 2 and 3) and the river Rhine (annual mean loads at Lobith [RIZA, n.d.]) in 1983.

With respect to the 2nd and 3rd columns, Dutch rainwater versus Dutch surface water, it can be noted that the concentrations in rainwater are generally less. The NH_4 content is, however, an exception that may be explained by the fact that nitrifying bacteriae cannot be present in rain, but in surface water they may reduce the NH_4 content considerably. In all it may be concluded that rainwater is 'cleaner' than surface water (the standard), but the concentrations are not to be neglected. The NH_4 concentrations are remarkably.

A more tentative way of reaching qualitative conclusions has been followed by tabulating the loading of the Netherlands geographical area with pollutants via rain (wet depositions) and via the river Rhine (loads) (table 6). After the river Rhine, rain is the biggest water supply of the Netherlands. The flow of the Rhine is about $70 \cdot 10^9 \text{ m}^3$ per year and the annual amount of rain is about 750 mm, i.e. $30 \cdot 10^9 \text{ m}^3$. In spite of the enormous upgrading that leads to inaccuracies in the values (very low concentrations in the Rhine multiplied by enormous flows; low depositions times a large area of $4 \cdot 10^{10} \text{ m}^2$) both depositions and loads of main components and trace elements are of the same order of magnitude.

Thus the following general conclusion may be drawn, that wet depositions of air pollutants must be taken into account with balance studies. The general conclusion should perhaps also be borne in mind whenever influxes into urban sewerage systems are calculated.

ACKNOWLEDGEMENT

The authors wish to thank T.A. Buishand (Royal Netherlands Meteorological Institute) for his statistical contribution.

REFERENCES

- T.A. BUISHAND, 1984. Detectie van trends in de chemische samenstelling van de neerslag. (Detection of trends in the chemical composition of precipitation, in Dutch.) Royal Netherlands Meteorological Institute, De Bilt, memorandum FM-84-42.
- T.A. BUISHAND, 1985. Ruwe schatting van de grootte van te detecteren trends in de chemische samenstelling van de neerslag door middel van regressie-analyse. (Rough estimate of the magnitude of trends that can be detected in the chemical composition of precipitation by means of regression analysis, in Dutch.) Royal Netherlands Meteorological Institute, De Bilt, memorandum FM-85-7.
- E. BUIJSMAN, H. MAAS and W. ASMAN, 1985. Een gedetailleerde ammoniak-emissiekaart van Nederland. (A detailed map of ammonia emissions in the Netherlands, in Dutch with English summary.) Government Printing Office, The Hague, Publikatiereeks Lucht 41.
- N.D. VAN EGMOND, H. KESSEBOOM and D. ONDERDELINDEN, 1985. Statistische optimalisatie Landelijk Meetnet Regenwatersamenstelling. (Statistical optimization of the National Rainwater Composition Measurement Network, in Dutch.) National Institute of Public Health and Environmental Hygiene, De Bilt, report in prep.
- J.N. GALLOWAY, G.E. LIKENS, W.D. KEENE and J.M. MILLER, 1982a. The composition of precipitation in remote areas of the world. *Journal of geophysical research*, 87, 11, p. 8771-8786.
- J.N. GALLOWAY, J.D. THORNTON, S.A. NORTON, H.L. VOLCHOK and R.A.N. McLEAN, 1982b. Trace metals in atmospheric deposition: a review and assessment. *Atmospheric Environment* 16, 7, p. 1677-1700.
- A.W.M. HOFSTEE, R.C.C. WEGMAN, T.B. RIDDER and H.F.R. REIJNDERS, 1984. Organische microverontreinigingen in natte depositie. (Organic micropollutants in wet deposition, in Dutch.) National Institute of Public Health and Environmental Hygiene, Bilthoven, report no. 21781007.
- H. DEN HOLLANDER, D. VAN DE MEENT, P.C.M. VAN NOORT and E. WONDERGEM, 1985. Wet deposition of polycyclic aromatic hydrocarbons in the Netherlands. National Institute of Public Health and Environmental Hygiene, Bilthoven, publication in prep.
- KNMI and RIVM, 1983. Chemical composition of precipitation over the

- Netherlands, Annual report 1982. Royal Netherlands Meteorological Institute, De Bilt and National Institute of Public Health and Environmental Hygiene, Bilthoven, number of publication KNMI 156-5.
- KNMI and RIVM, 1985. Chemical composition of precipitation over the Netherlands, Annual report 1983. Royal Netherlands Meteorological Institute, De Bilt and National Institute of Public Health and Environmental Hygiene, Bilthoven, number of publication KNMI 156-6.
- T.A. McMAHON and P.J. DENISON, 1979. Empirical atmospheric deposition parameters, a survey. *Atmospheric Environment* 13, p. 571-585.
- D. VAN DE MEENT, J. VAN OOSTERWIJK and T. ALDENBERG, 1984. RID-VEWIN Meetnet Regenwater 1978-1982, deel 1, Samenvatting en statistische bewerking van de meetresultaten, deel 1a, De meetgegevens (RID-VEWIN Rainwater Measurement Network 1978-1982, part 1, Summary and statistical evaluation of the measurement results, part 1a, The measurement data, in Dutch, English translation available). National Institute of Public Health and Environmental Hygiene, Bilthoven, report no. ECOWAD-84-01.
- P.C.M. VAN NOORT and E. DE GREEF, 1984. The effects of in-cloud scavenging on the concentration profile of organic compounds in rain, vol. 1, polycyclic aromatic hydrocarbons. National Institute of Public Health and Environmental Hygiene, Bilthoven, report no. 84-4.
- P.C.M. VAN NOORT and E. WONDERGEM, 1985. Scavenging of airborne polycyclic aromatic hydrocarbons by rain. Accepted for publication in *Environmental Science and Technology*.
- T.B. RIDDER and A.J. FRANTZEN, 1982. Acid precipitation over the Netherlands. Proceedings CEC-workshop, concerted action physico-chemical behaviour of atmospheric pollutants, Berlin, 9 September 1982, Reidel, Dordrecht, p. 123-129.
- T.B. RIDDER, J.H. BAARD and T.A. BUISHAND, 1984. De invloed van monstermethoden en analysetechnieken op gemeten chemische concentraties in regenwater. (The influence of sampling and analysis techniques on measured chemical concentrations in rainwater, in Dutch.) Royal Netherlands Meteorological Institute, De Bilt, technical report TR-55.
- T.B. RIDDER, 1984. Vijf jaar metingen van de samenstelling van de neerslag. (Five years of measurements of the chemical composition of precipitation.) Proceedings symposium of acid rain, 's-Hertogen-

- bosch, 17 and 18 November 1983, Pudoc, Wageningen, p. 55-58.
- RIJKSWATERSTAAT, n.d. Indicatief Meerjarenprogramma Water 1980-1984, Water Action Program 1980-1984, in Dutch, English summary available), Government Printing Office, The Hague.
- RIZA (NATIONAL INSTITUTE OF WASTE WATER PURIFICATION), n.d.
- Kwaliteitsonderzoek in de Rijkswateren, verslagen van de resultaten van het 1e, 2e, 3e en 4e kwartaal 1983. (Quarterly reports 1983 on the quality of Dutch surface water), Government Printing Office, The Hague.
- D.M. WHELPDALE, 1982a. Removal processes, in: H. Rohde, A. Eliassen, I. Isaksen, F.B. Smith and D.M. Whelpdale, Tropospheric chemistry and air pollution. World Meteorological Organization, Geneva, technical note no. 176, Ch. 4.
- D.M. WHELPDALE, 1982b. Wet and dry deposition, in: H.W. Georgii and W. Jaeschke (eds.), Chemistry of the unpolluted and polluted troposphere. Reidel, Dordrecht, p. 375-391.

FROM RAINFALL TO SEWER INFLOW; A PROCESS WITH CONSEQUENCES

F.H.M. van de Ven

ABSTRACT

The conversion from rainfall to inflow is a process that can be separated in two parts: the runoff losses and the transformation of net rainfall to sewer inflow. The runoff losses can be separated in 4 parts. First the initial losses as the wetting loss and the loss due to the filling of the depression storage. Secondly we have the evaporation loss. When the pavement or roof is hot, the evaporation is governed by the heat fluxout of the underground, but normally this is not the case. The infiltration in semi-pervious pavement is the third and largest loss; the infiltration-rates are high. The fourth loss is the runoff from paved to unpaved areas. The runoff coefficient expresses the sum of all losses. The behaviour of this coefficient is discussed.

The rainfall is smoothed and delayed by the runoff process before it enters the sewer system. This transformation is quantified by the unit hydrograph.

The consequences of these processes for the groundwater recharge, the water balance, the design discharge for sewers, the amount of over-flowing water and for the storm profile are shown. Sewer inflow instead rainfall should be used as a basis for sewer design. The construction of a design inflow storm is discussed.

1 INTRODUCTION

The sewer-system is meant to drain the runoff from the surface of an urban area. The design standards for such sewer systems are normally based on rainfall data. However, between the falling of rain and the inflow in the sewer system, the so-called rainfall-runoff process converts the rainfall to sewer inflow. Now the question is: Is the conversion so rigorous that one should take it into account for the design of sewer systems? To answer that question we have to understand what goes on during the process.

In order to understand what subprocesses are involved, the pathways of the water in an urban environment with a separate sewer system are shown in figure 1. Paths of minor importance are not indicated.

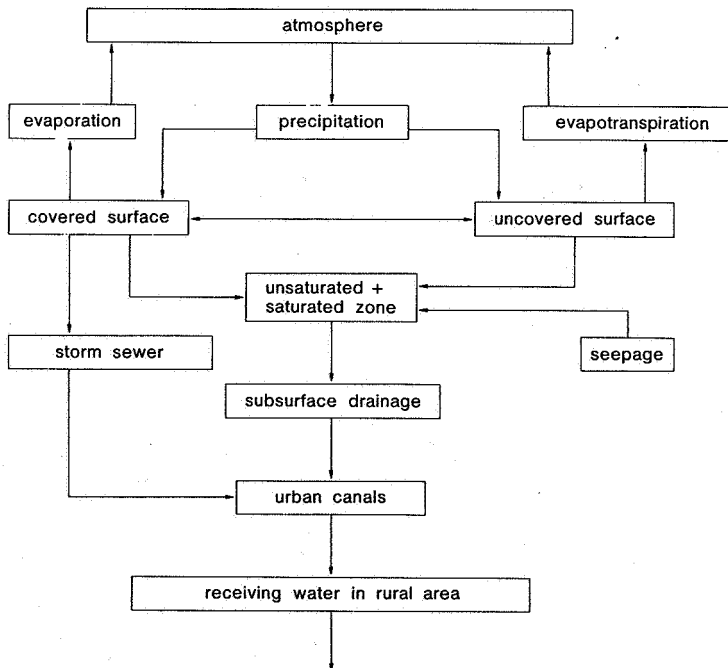


Figure 1 Pathways of the storm water in an urban area

Part of the precipitation falls on paved and covered surfaces and part

on unpaved surfaces. Most of the paved surfaces drain to the sewer system. Runoff from the unpaved area is not expected due to the generally high permeability of the topsoil. In extreme situations some runoff from the uncovered surfaces can occur, though. Paved or covered surfaces that drain to unpaved areas are taken into account as 'unpaved' because they do not contribute runoff to the sewer system.

During the time that the paved surface is wet evaporation occurs. The evaporation flux is strongly influenced by the temperature of the pavement and the vapour transport in the air.

In case the runoff water flows over a brick or tile covered surface, water will infiltrate through the joints and/or cracks into the subsoil. New asphalt is fully impervious however (except porous asphalt). The water that is stored on the surface, infiltrates and evaporates even after the end of a runoff event. The consequences of the infiltration are groundwater recharge, a raise of the groundwater level, the need for groundwater control measures and a reduced surface runoff.

The runoff enters the sewer system by the gully pots. In case of a separate sewer system, the sewer-pipes convey the water to the receiving surface water. When a combined sewer system is applied, the water in principle is led to the treatment plant. In order to prevent an exceedance of the capacity of that plant, spillways or "overflow structures" are installed. By these structures the excess water can be discharged.

On its way through the sewer system the flow is retained a little by the hydraulic resistance of the pipes and the manholes and by the storage available in the pipelines.

2 THE RAINFALL-RUNOFF PROCESS

2.1 A storm event as example

Now that we know the possible flowpaths of the water, let us look at the results of a measurement. In figure 2 the hyetograph and the inflow hydrograph are shown of an event that occurred on July 22nd, 1973. The

event was of short duration but the precipitation intensity was high: Within 10 minutes 5.9 mm of rainfall was recorded. The indicated inflow is recorded in a housing area in Lelystad. In annex 1 a short description of the basin and the inflow measurements is given.

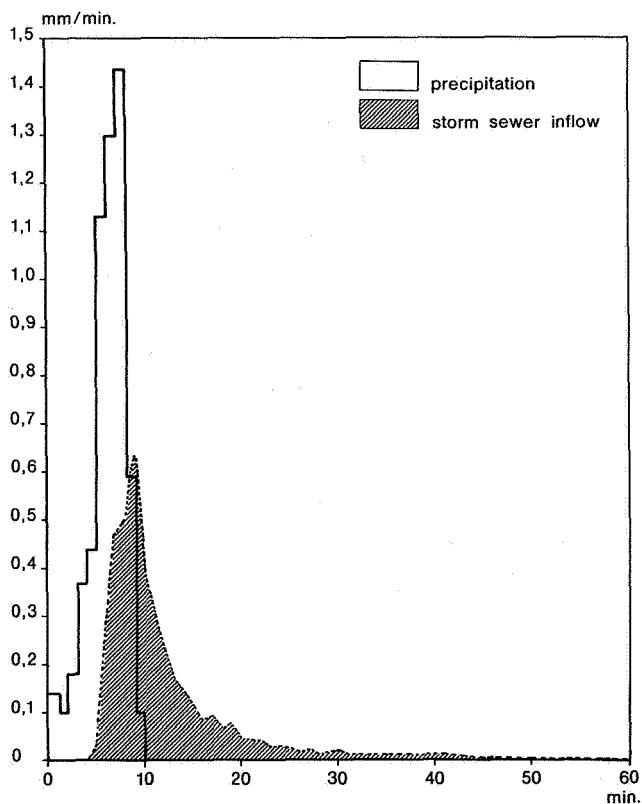


Figure 2 Hyetograph and inflow-hydrograph of an event in July 22nd, 1973 recorded in a housing area in Lelystad,

By comparing the hyetograph with the inflow hydrograph some striking differences are observed:

- The inflow starts 2 - 3 minutes later than the precipitation;
- The amount of inflow is smaller than the amount of precipitation. Only 69% (4.1 mm) of the rainfall reaches the gullypots;
- The maximum inflow is much smaller than the maximum rainfall intensity.

Taking the data on a one minute time-base, the peak inflow is about 56% smaller than the maximum precipitation;

- The maximum inflow occurs 1 - 2 minutes later than the maximum precipitation;
- A measurable inflow is recorded until 40 - 50 minutes after the rainfall occurred.

Although this is not so clear from figure 2, the inflow-hydrograph is much smoother than the hyetograph. The precipitation intensity is extremely variable when the data are taken on a time-base of one minute or less; high intensities can be interchanged with almost dry intervals.

The difference between the amount of inflow and the amount of precipitation is generally expressed in the runoff coefficient C:

$$C = \frac{\text{inflow or runoff of a storm event (mm)}}{\text{precipitation of the storm event on the covered area (mm)}}$$

So, in the runoff coefficient all the loss terms - initial loss, evaporation, infiltration, exchange with unpaved area - are summarized. Let it be notified that the amount of inflow is equal to the amount of runoff. The rainfall minus losses is called net rainfall.

For a better understanding of the importance of each loss term, an analysis is made of the mechanisms that govern these losses (§ 2.2 - 2.5). In § 2.7 the transformation of net rainfall to sewer inflow and discharge is discussed.

2.2 Initial losses

The basic flowpaths causing runoff losses are evaporation and infiltration, so the term initial loss only indicates an intermediate state. In practice two types of initial losses can be discerned. First we have the wetting loss: water is absorbed by or adsorbed to the surface cover in a way that it can only be removed by evaporation after the end of the event. And secondly we have the storage in depressions. By the end of an event that water is removed by infiltration and evaporation.

The wetting loss depends on the type of cover that is applied. Concrete bricks and tiles absorb about 0.5 mm within 10 minutes, when the surface remains saturated with water. A more porous brick type absorbs 0.7 mm in the first 5 minutes and 0.15 mm in the next 10, while asphalt absorbs or adsorbs only 0.07 mm within 15 minutes (Van Dam, Schotkamp, 1983).

About the depression storage a lot of figures exist. In many cases however, these figures do not represent only the depression storage, but include an infiltration and evaporation loss part, so the depression storage is overestimated. On a main road in Lelystad a depression storage of about 2 mm was found and on a flat roof the storage was about 4 mm (Voortman 1977 and 1984). In general the amount of depression storage ranges between 0.5 and 1.5 mm, according to the sources found in the literature (Pecher, 1969).

So in the initial phase of the storm event, at least 1 - 2 mm of precipitation is fixed in a position that it will only be removed from by evaporation and infiltration after the rainstorm. The runoff to the sewer system will start at the moment that the precipitation intensity exceeds the wetting loss demand plus the lowest infiltration capacity on any part of the covered surface. The runoff increases when the depression storage becomes more and more filled.

2.3 Evaporation

The evaporation from paved surfaces and roofs is a rather black spot in the knowledge on urban hydrology. Using some general theory, only a rough estimate of the amounts can be made.

When the surface pavement is wet, the evaporation is assumed to be equal to the potential open water evaporation E_o . Using the modified Penman equation E_o can be calculated as:

$$E_o = \frac{(1/L) A \left(\frac{R_n + H_s}{1 + A} \right) + B (e_{zs} - e_z)}{1 + A} \quad \frac{\text{kg}}{\text{m}^2 \cdot \text{s}}$$

where L = latent heat of evaporation for water at temperature T_s
 A = factor containing the gradient of the saturation vapour
 pressure vs. temperature curve evaluated at the temperatures
 of the water and air and the psychrometric constant
 R_n = net incoming radiation
 H_s = heat energy transferred from below the surface
 B = factor indicating wind effects
 e_{zs} and e_z = saturation and actual vapour pressure at 2 m above
 the surface.

In general H_s is assumed to be zero. Now suppose that on a hot summerday
 a thunderstorm occurs in the afternoon. The burning-hot asphalt has a
 surface temperature of 55 - 60°C at the beginning of the event, but is
 cooled suddenly to about 20 - 30°C by the falling rain. The sky is
 clouded by that time, so the net radiation R_n is only about 75 W/m².

In such a situation the heat flux from below the surface can be
 estimated by assuming that the process is comparable to a sudden surface
 temperature change on a theoretical layer of asphalt of infinite thickness
 with an initial temperature of about 55°C at the surface and about 40° C
 deeper in the asphalt. By using these estimates for H_s , table 1 could
 be composed. The instantaneous heat flux and evaporation flux is given
 in that table at several times after the beginning of the storm event.
 Moreover, the accumulated amount of evaporation is indicated.
 For calculating the evaporation in table 1, the term $B(e_{zs}-e_z)$ was
 neglected, because its magnitude was about 1 - 4% of the first term of
 the Penman equation.

Table 1 Surface heat flux, evaporation flux and total evaporation from
 asphalt during and after a thunderstorm on a hot summerday

time	H_s	<u>evaporation flux</u>		total evaporation
(h)	(W/m ²)	(x 10 ⁻⁵ mm/s)	(mm/day)	(mm)
1/60	3,400	97	87	≈ 0.06
5/60	1,500	43	37	0.27
15/60	870	25	22	0.46
30/60	620	18	15	0.69
1	440	13	11	1.0
2	310	8.9	7.7	1.4

The heat flux from below the surface by far exceeds the amount of net incoming radiation and therefore H_g controls the evaporation process in the sketched situation.

The conclusion is, that an evaporation of about 1 mm in the first hour of a storm event is possible, but only in rather extreme situations. In the normal situation the evaporation flux is negligible as loss term.

In general, the pavement will not be so hot. For investigating the amount of evaporation from a covered surface for the water balance, a more general approach was developed. To make estimations for the monthly evaporation of the covered surface in two experimental basins in Lelystad, Voortman (1983) assumed that the evaporation was equal to the open water evaporation during the time that rainfall and/or inflow was recorded. When a storm in summer was not followed by another storm within 1 hour, he assumed an extra evaporation of 0.2 mm - the amount for wetting the surface. The water of the very small rainstorms that didn't cause any measurable runoff was assumed to evaporate up to a maximum of 0.4 mm per event.

Using these assumptions and about 12 years of data he calculated an average evaporation from the covered surface of a housing area (see annex 1) of about 175 mm/yr. For a parking lot he found an average of 112 mm/yr. These amounts did fit well in the total water balances of the basins. The difference between the two amounts is caused by the total duration of inflow: 1445 versus 980 hours respectively.

So the yearly evaporation of the covered surface is certainly not negligible. It would be advisable to do more research in this field, including the heat flow in pavement and roof materials. But the contribution to the loss of rainfall in each individual event is limited. Only in extreme cases the loss will exceed 1 mm but in general it is less than 0,5 mm, including the evaporation of the wetting loss.

2.4 Infiltration

Only the last decade the hydrologists started to realize that the amount

of water that could infiltrate on brick or tile pavement is considerable. Amongst others Van den Berg (1978) found large amounts of runoff from the subsurface drains on a parking lot in Lelystad (see annex 1). According to his figures about 40 - 50% of all rainfall was discharged through the pavement. A similar result was found by Jacobsen and Harremoës on a parking lot in Denmark (1981). A more extensive analysis of the data over the period 1970 - 1980 showed that about 220 mm infiltrated water/year was discharged from the parking lot by the subsurface drains; that is 30% of the precipitation (Voortman, 1983).

The first field-study in Holland on the permeability of pavement was done by Bebelaar and Bakker (1981), not to investigate the infiltration losses but to check the watersupply for trees standing in or along paved area's. A more detailed analysis of the infiltration process was made by Van Dam and Schotkamp (1983). In table 2 the outlines of the recorded infiltration capacities are given for different types of pavement.

Tabel 2 The average infiltration capacity of different types of pavement recorded by Bebelaar and Bakker (B.B) and Van Dam and Schotkamp (D.S)

Type of pavement	source	<u>infiltration capacity(mm/h)</u>			no. of experiments
		median	minimum	maximum	
concrete brick (0.22x0.11x0.08)	BB	32	10	353	21
" " (0.20x0.10x0.07)	DS	14	7	24	6
" " (0.20x0.09x0.065)	BB	34	8	300	13
bricks (0.20x0.20x0.08)	DS	9	6	15	9
tiles (0.30x0.30x0.04)	BB	16	1	254	15
" (0.50x0.50x0.06)	DS	10	7	16	4
asphalt	DS+BB	0		0	4

The permeability of the pavement turned out to be highly dependent on the conditions of the joints between the bricks and tiles. If the joints were filthy, a low infiltration capacity was found; older pavement and pavement that is used intensively are in general less permeable. The

data of Bebelaar and Bakker were collected on all types of pavement, with various age, while Van Dam collected his data only on one parking lot, where the pavement was about 14 years old. That explains the wide ranges found by Bebelaar. Of course the pavement must be laid upon a highly permeable subsoil of e.g. coarse sand. In all experiments the subsoil had a coarse texture.

In general one must conclude that the infiltration capacity is surprisingly high and certainly not neglectable.

Van Dam and Schotkamp used their data to check several infiltration formulae to describe the process. The best model they found was the formula of Hillel and Gardner for infiltration in crusted soils.

$$I_{cum} = \sqrt{at + b} - c$$

I_{cum} = cumulated infiltration; a,b,c = parameters; t = time

In the urban hydrology the Horton infiltration formula is wide-spread used. This formula

$$i = f_e + (f_o - f_e) e^{-kt}$$

i = infiltration rate f_e = final infiltration rate

k = parameter f_o = initial infiltration rate

turned out to be second-best and better than the formulae of Philip, Kostiaikov, and the constant infiltration rate model.

The infiltration rate is the highest in the first part of the event. In figure 3 both the results of the field measurement on bricks (0.20x0.20x0.08 m) and the calculated average infiltration show the flexure in the relation.

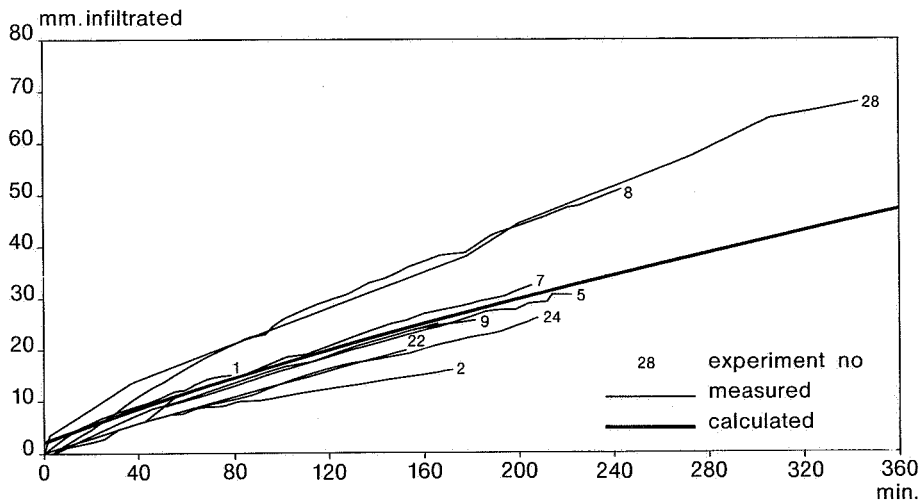


Figure 3 Recorded cumulative infiltration on brick (0.20x0.20x0.009 m) pavement and the calculated average modelled with the Hillel and Gardner formula (Van Dam, Schotkamp, 1983)

Because the infiltration rate decreases with time, it is easy to understand that the initial infiltration rate depends on the antecedent moisture conditions. But taking the average infiltration rate as a design standard is not too bad, as can be seen from figure 3, where a linear approximation seems allowable.

The infiltration in semi-pervious pavement is an important loss factor for the surface runoff. Most of the storm events have only a low to moderate rainfall intensity and in such a situation the infiltration process causes that large infiltration loss.

Now that we know the wetting loss and are able to simulate the infiltration loss, the amount of water on the street surface of an experimental basin can be simulated, when evaporation is neglected and when the non-covered surface does not contribute to the runoff. The amount of water that is still in the basin is found by subtracting the recorded cumulative inflow from the cumulative precipitation. When the wetting loss and the cumulative infiltration is subtracted from this curve, a

picture remains of the amount of water that is stored on the surface during the event. In figure 4 such pictures are given for two events, recorded at the parking lot basin in Lelystad.

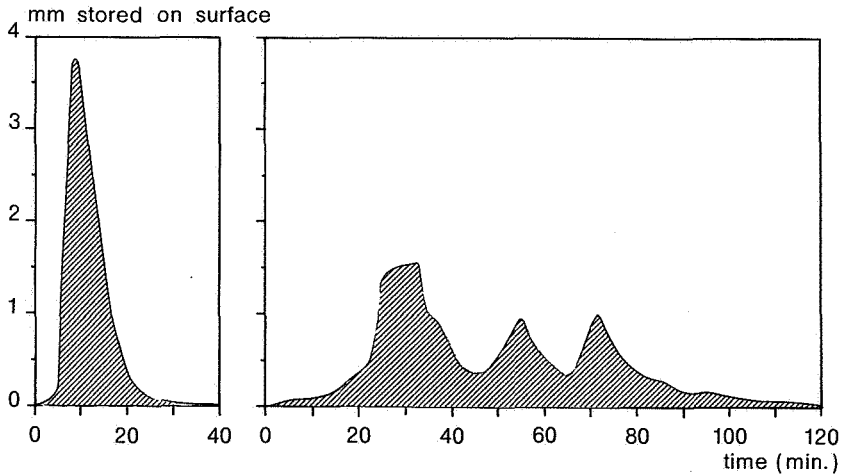


Figure 4 Amount of water stored at the covered surface of the parking lot during two storm events

As can be seen from figure 4, the amount stored on the surface is highly variable during the event. These variations are very important for the water quality of the sewer inflow, both for the soluble pollutant concentrations and the solids transport. The moments with a thick layer are very appropriate for the transport of particulates and adsorbed pollutants. So, for the interpretation of a pollutograph of storm sewer runoff information on the depth of the water layer can be useful.

2.5 Contributions from non-covered surfaces

In today's design practice in Holland generally the assumption is made that the runoff coefficient of non-covered surfaces is zero. In practice however this is not necessarily true in all situations. Figure 5 is taken from a paper by Ando, Takahasi and Kuan (1984) and shows the final

system will get surcharged and the water searches its way downhill over the street surface and via other depressions, often resulting in ponding of water downhill. Damage by this type of runoff can be prevented, e.g. by use of 'dual' drainage as in Canada. There the roads have an explicit function for conveying the water to a discharge point. Because they are designed for that function, preventive measures are taken to avoid basement flooding and damage when the streets get flooded.

In the Dutch design method the effect of storms, larger than the design storm, are not taken into consideration. Often a lot of damage could be avoided by taking relatively simple and cheap measures. Runoff from uncovered surfaces could be avoided in many cases by lowering the level of the non-covered surface to below the level of the covered surface. In case this solution is not applicable, a separation between the covered and non-covered surface should be made in such a way, that enough storage on the uncovered surface prevents runoff.

2.6 Runoff coefficient

Now that the processes that govern the precipitation loss are better understood, let us take a look at the runoff coefficients found in the experimental basins in Lelystad. All losses are incorporated in the runoff coefficient. The coefficient is defined *with respect to the covered area* so theoretically it can be larger than 1. Let it be notified that the definition of a storm event is an artefact. The event begins with the first measurable rainfall and ends with the last measurable runoff. The sensitivity and accuracy of measuring devices therefore is an important factor.

Now let us take a look at the runoff coefficients recorded in the three Lelystad basins and consider the probability that a certain coefficient occurs. In figure 6 the probability density function (pdf) of the coefficients is given for the three basins. Only events with a total runoff larger than 1 mm were taken into account. In figure 7 the same is shown for events larger than 5 mm.

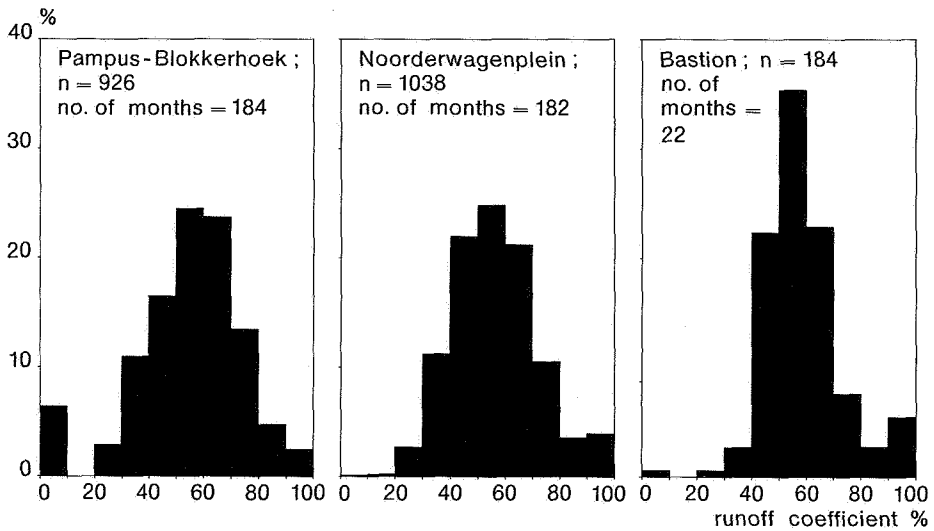


Figure 6 Probability density function of the runoff coefficients observed in 3 basins in Lelystad for storms with more than 1 mm runoff

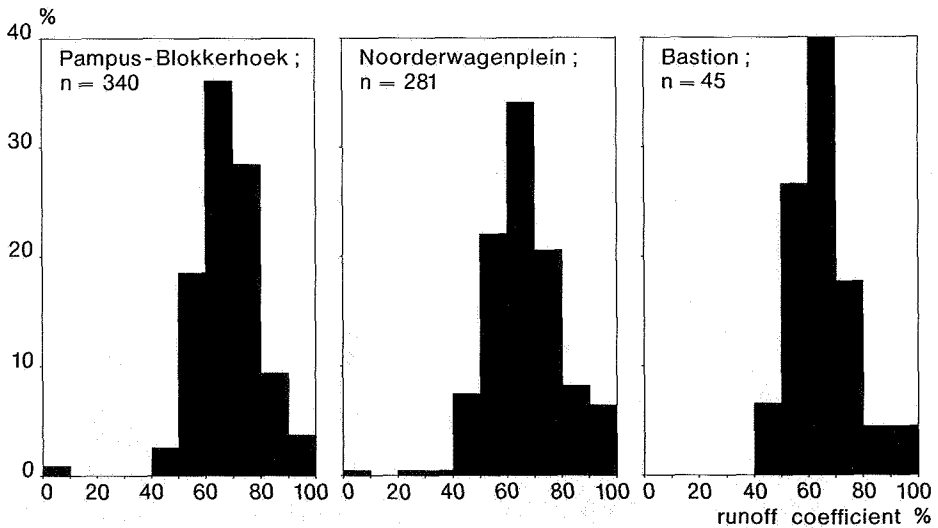


Figure 7 Probability density function of the runoff coefficients in 3 basins in Lelystad for storms with more than 5 mm runoff

Although the basins are quite different in their characteristics, the distribution of the coefficients is remarkably alike: in most events 20 - 60% of the rainfall on the covered surface is not discharged by the sewer-system. The runoff coefficients of the housing area Pampus-Blokkerhoek are slightly higher than those of the parking lot, mainly due to the smaller percentage of semi-permeable pavement. The number of data of the Bastion housing area is considerable less than of the other basins, so the accuracy of the pdf is less too.

An outstanding phenomenon is the smaller standard deviation (spreading) of the Bastion pdf. An explanation is found in the extensive use of tile pavement and the sloping roofs in the Bastion basin, so that the initial losses are relatively small. For the small events a higher runoff coefficient will be found, so the left tail of the pdf is 'pressed' somewhat to the right. A smaller standard deviation is the result.

Comparing figure 6 to figure 7 an increase of the runoff coefficient is found because 1. the initial loss is relatively less important in heavy storms and 2. the events are in general of longer duration and the infiltration rate decreases with time, resulting in relatively more runoff. The median values of the runoff coefficients in the figures 6 and 7 differ about 10 - 15%.

To investigate the seasonal fluctuation the pdf's of the runoff coefficients found in the summer and winter half year is shown in figure 8.

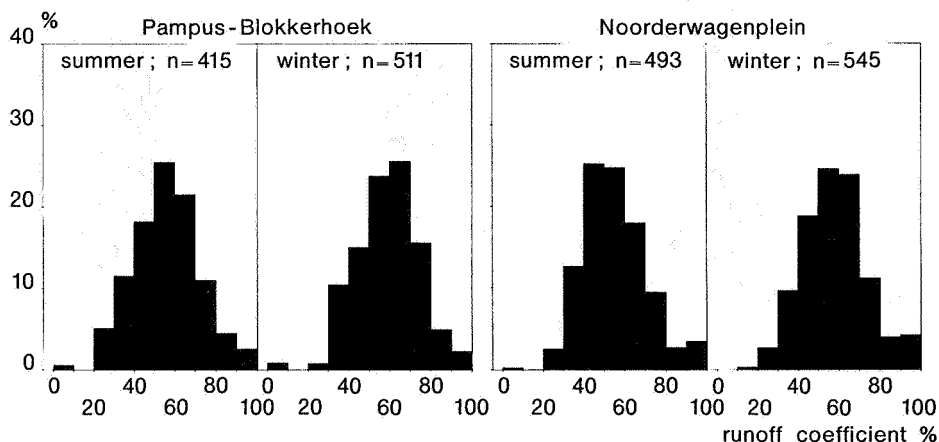


Figure 8 Probability density function of the runoff coefficients observed in summer (April-September) and winter in two basins in Lelystad. Storm events larger than 1 mm

The coefficients are larger in winter. For Pampus-Blokkerhoek the median coefficient is 56% in summer and 59% in winter. For the Noorderwagenplein basin these values are 54% and 57% respectively. Less initial and evaporation losses and a reduced infiltration due to the wet initial conditions and/or due to a frozen underground cause the difference in pdf's. In summer the storms have higher intensities, resulting in higher coefficients, but this seems not to counterweight the effect of the reduced losses.

2.7 Rainfall to inflow transformation

Apart from the losses, the rainfall-runoff process is characterised by another phenomenon: the delay and smoothing of the hyetograph to the inflow hydrograph.

Assuming that the relation between the net rainfall and runoff is linear, a unit hydrograph can be calculated for that relation. This unit hydrograph shows how 1 unit of rainfall in 1 time unit is transformed to inflow. By Van de Ven, Van der Kloet and Van der Wal (1981) is shown that a linear relation between rainfall and inflow is a justifiable assumption and that Laguerre functions are well suited to describe the unit hydrograph. The estimated unit hydrographs for the rainfall-inflow relation in the basins Pampus-Blokkerhoek and Noorderwagenplein and the rainfall-discharge relation for the basin Bastion are shown in figure 9.

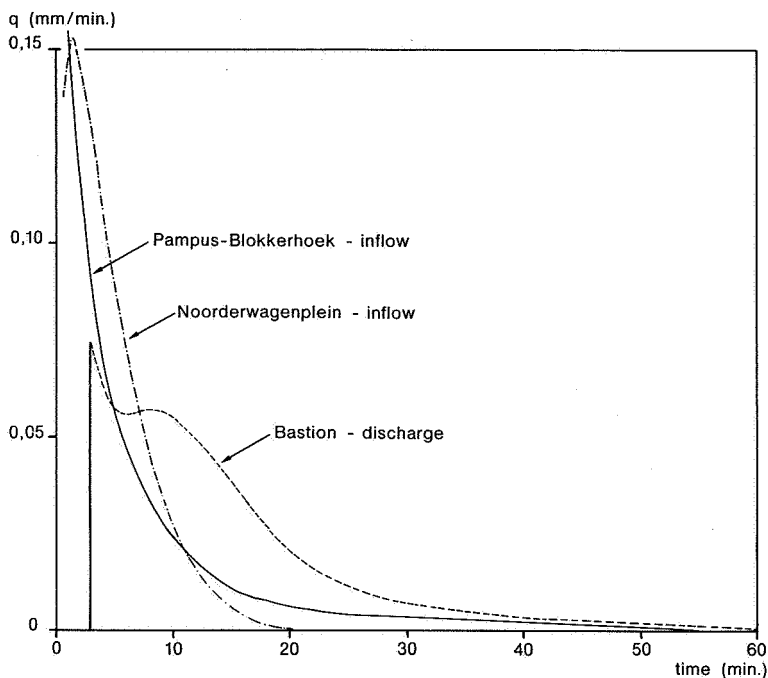


Figure 9 Unit hydrographs for the inflow on two basins in Lelystad (Pampus-Blokkerhoek and Noorderwagenplein) and for the sewer discharge from another basin (Bastion)

The unit hydrographs for both the inflow and the sewer discharge indicate that one unit input leads to runoff until more than 20 minutes after the input occurred. The inflow on the Noorderwagenplein parking lot is less prolonged than the runoff from the housing area. This is probably due to the less differentiated cover with a relatively quick responding surface. The Pampus-Blokkerhoek area e.g. gives long response times because of the flat roofs in the basin. Both *inflow* hydrographs are rapidly declining functions while the *discharge* hydrograph of the Bastion area is more smooth. The location of the centre of gravity of the curves illustrates that:

Noorderwagenplein parking lot,	inflow	:	3.5 min
Pampus-Blokkerhoek housing area,	inflow	:	3.7 min
Bastion housing area,	discharge	:	11.1 min

There is no reason why the centre of gravity of the inflow hydrograph of the Bastion area would not be between 3 and 4 minutes. So, the delaying effect of the stormsewers in this 4.5 ha large basin must be about 8 minutes. The flow in the sewer-pipes therefore leads to a considerable smoothing of the inflow. For medium to large sewer systems this smoothing should certainly be taken into account for the design of the main sewers.

The time scale of the rainfall-inflow relation is important because of the possibility to simulate the inflow with the aid of a model and rainfall data. As can be seen, the time interval of the rainfall data should preferably be less than 3 minutes to get a good simulation of the inflow. Problems with the collection of data at these time intervals can be avoided by using event-sense recording.

3 THE CONSEQUENCES

Now that we have investigated the rainfall-inflow process we can try to formulate the consequences for the output (= the runoff) and for the design of the water management system of urban areas.

3.1 The water balance

The average water balance of the parking lot over 1970 - 1980 is composed by

IN:	OUT:
precipitation : 737 mm/y	storm sewer runoff : 376 mm/y
vertical seepage : 101 mm/y	subsurface drainage runoff : 337 mm/y
lateral seepage : 20 mm/y	evaporation covered surface : 112 mm/y
	evaporation trees : 27 mm/y
	change of storage : 5 mm/y

Assuming that all seepage is discharged by the subsurface drainage system, the average groundwater recharge rate below the paved parking

lot comes to about 220 mm/y. The assumption that groundwater recharge on a paved area is negligible is therefore questionable. The groundwater recharge from covered surfaces can even be larger than from uncovered surface, because the evaporation from covered surfaces is limited: 110 - 180 mm/y for the covered part versus 455 mm/y for the non-covered part. Problems with high groundwater tables are often a result of the combination of semi-pervious pavement and a lack of subsurface drainage.

3.2 Maximum inflow and design discharge

The losses and the rainfall-inflow relation also severely influence the maximum intensity of the inflow. In figure 10 depth-duration-frequency curves are given for both the rainfall and the inflow recorded at the parking lot and in the housing area in Lelystad (Van de Ven, 1984).

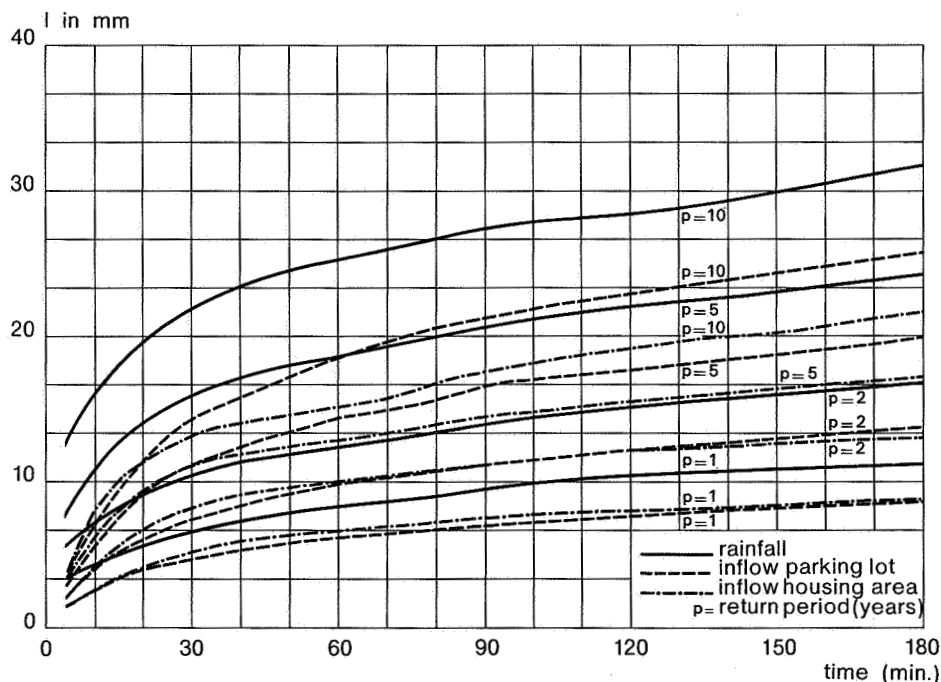


Figure 10 Rainfall and inflow depth-duration frequency curves from data collected on a parking lot and in a housing area in Lelystad in the period 1963 - 1980

As can be seen the inflow depths are 20 - 70% smaller than the comparable rainfall depths. The largest differences occur at short time intervals.

It is clear that not the rainfall but the inflow data should be used as basis for the design discharge for storm sewers. This selection can not be done using the depth-duration-frequency curves because they do not represent real time series. Using a simple model and the recorded inflow series, the storage-design discharge-frequency-curves can be calculated for different return periods. Based on such an analysis, Van de Ven (1984a) comes to a design discharge of 4 - 5 m³/s/km² (40 - 50 l/s/ha) for a situation as in a large part of Holland.

3.3 Overflow amounts

The amount of discharge out of the overflow structure of a combined sewer-system is often calculated for estimating the required magnitude of a storage basin and/or for estimating the pollutant load. This last function is important because 86% of the Dutch overflow structures discharge on small, semi-stagnant receiving water.

Using a 'tree' of different reservoir type elements the sewer-system of the villages Harderwijk, Ermelo and Putten was modelled by Van de Ven (1984) to simulate the amount of overflowing water from the main overflow structure. The total covered area is about 310 ha. The villages are located about 30 - 40 km south of Lelystad, so the 10 years series of both hourly rainfall data and inflow data from the parking lot in Lelystad were used as input for the model. To correct for the losses an extra calculation was made with a runoff coefficient of 0.80 on the precipitation. The calculated averages of the overflowing amounts depended on the input:

input	average overflow 1000 m ³ /yr
precipitation	212
precipitation * 0.8	112
inflow parking lot	72

The difference is considerable. Again it should be advised to use inflow data instead of precipitation data in this type of calculations.

3.4 Duration of runoff and location of peak in the storm profile

For some problems it is interesting to know how much time the street surface is wet or runoff occurs. These figures can be derived from the runoff measurements in the basins in Lelystad. The recorded total duration of runoff in the Pampus - Blokkerhoek basin is 1445 h/yr on the average, while on the Noorderwagenplein runoff occurs during about 980 h/yr. Because every storm event is characterised by a recession curve of the inflow after the rain has stopped, the peak in the inflow hydrograph will always occur relatively early in the event. In figure 11 from Van de Ven (1984a) this is illustrated. To make this figure, the duration of each rainfall an inflow storm was set on 100%. Now the expected depth of rainfall after, say 30% of the time, is 30% of the total depth. In practice the depth deviated from the expected percentage and the probability distribution of these deviations are shown in figure 11. The lines in the figure give the connection between the calculated percentage point of the distribution of the deviations. So the average deviation is indicated by the 0.5 line.

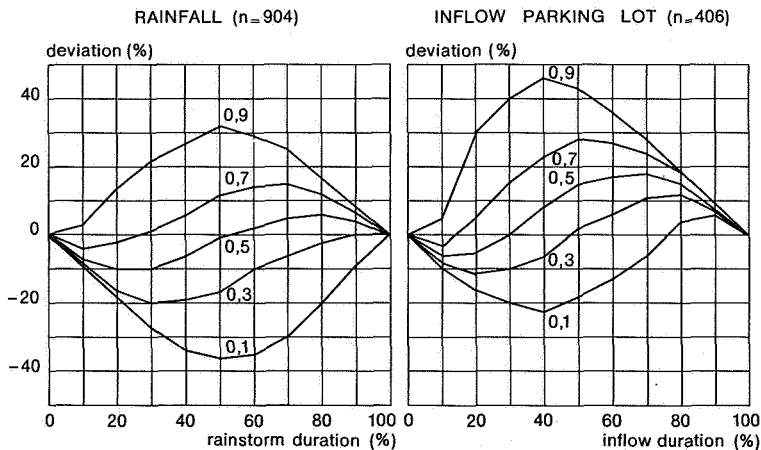


Figure 11 Deviation of the actual rainfall and inflow depth from the expected depth after a part of the event duration (all storms larger than 2.5 mm). In the figure the percentage points of the probability distribution of the deviations are shown for the different parts of the storm duration

As can be seen from the average the most intense part of the rainfall occurs between 25% and 75% of the storm duration, while for the inflow the peak generally occurs between 15% and 60%. Design storms - often based upon rainfall instead of inflow data - with uniform intensity or with the peak in the middle of the storm event or even later, therefore are less realistic. When selecting the design (inflow) storm for non-steady flow calculations in sewers, this variation should be taken into account.

4 CONCLUSIONS

The difference between rainfall and sewer inflow is large. The design parameters for the sewer system therefore should be based upon the inflow and not upon the precipitation.

The extremes in the sewer inflow are 30 - 70% smaller than comparable extremes in the precipitation, according to the depth-duration-frequency curves. These curves cannot be used to derive a design discharge.

By means of the storage-design discharge-frequency curves based on the inflow series a design discharge of 4 - 5 m³/s/km² (40 - 50 l/s/ha) is found when the return period for exceedances is set to 2 years.

Because of the permeability of the pavement, about 220 mm/yr of ground-water recharge was found below a parking lot in Lelystad. The evaporation from paved surfaces was about 110 - 180 mm/y. That is much less than the 450 mm/y for the non-covered areas. More research on the evaporation from covered surfaces is required.

In a case-study the amount of overflow from a structure in a combined sewer system was calculated with a model. The amount was overestimated with a factor 3 when rainfall data were used as input instead of inflow data.

The peak in a storm (inflow) profile tends to be situated between 15 and 60% of the total storm duration. Design storms should be adapted to that.

The process of conversion from rainfall to sewer inflow can be split up in 1. The losses and 2. the transformation of net rainfall to inflow.

The losses are separable in three basic loss processes:

- the initial losses consist of
 1. the wetting loss of say 0 - 0.8 mm which can only be removed from the surface by evaporation
 2. the depression storage of say 0,5 - 1,5 mm which is removed by both infiltration and evaporation
- the evaporation can amount up to 1 mm/h in extreme cases, the wetting loss excluded, but is negligible as loss under average conditions
- the infiltration through semi-permeable pavements as tiles and bricks ranges from 10 to 35 mm/h.

The magnitude and behaviour of the runoff coefficient can be explained very well from the knowledge of the loss processes: For heavier rain storms the coefficient increases; a smaller initial loss gives less variable coefficients. For modeling and design at least the initial loss plus the infiltration loss should be taken in account. In many cases the total loss will exceed 2 - 4 mm.

Runoff from uncovered areas generally does not occur under design conditions, except when e.g. the permeability of the soil is low. If that is the case it should be taken into account. When the design conditions are exceeded, some runoff can appear however.

To avoid problems in that situation, runoff from the uncovered to the covered area should be avoided e.g. by making a separation.

The amount of water at the covered surface fluctuates strongly during an event. This variation has important implications for the concentration of the transported pollutants and for the transport of sediments to the sewer system.

The centre of gravity of the unit hydrograph for inflow is located at 3 - 4 minutes. The unit hydrograph of the discharge from a 4,5 ha

housing area had its centre at about 11 minutes. The retardation of the runoff by the storm sewer itself therefore is considerable. In the sewer design technique the delay in both the inflow and the outflow should be taken into account.

By using the sewer inflow instead of the rainfall as the basis for the sewer design, a reduction in the sizing of the system can be achieved - and so in the costs.

5 RECOMMENDATION FOR THE DESIGN STORM

If possible, inflow instead of rainfall data should be used for determining the design storm. In many cases however, inflow data are not available, so the engineer is forced to work with the rainfall data. Basically he can apply two ways to derive the design storm; an approximate one and one based upon simulation.

The first is based upon the rainfall depth-duration-frequency curves. The step-wise process is as follows (also see figure 12):

1. Select a return period and design storm duration;
2. Read the rainfall amount in the depth-duration-frequency curve;
3. Assume a uniform distribution of the rainfall over the duration;
this gives the design rainstorm;
4. Subtract 1 - 2 mm initial loss;
5. Subtract the infiltration and evaporation loss. The evaporation loss can be neglected. The infiltration loss can be calculated as the weighted average infiltration capacity. The fraction of each pavement type of the total covered area serves as a weighting factor. Roofs are impermeable; other infiltration capacities can be derived from table 2, by taking the average of the indicated values for each pavement type.
6. Add the run-off contribution from the uncovered area, if necessary.
If so, subtract the infiltration capacity from the design storm intensity. The rest is stored in depressions (3 - 6 mm) and the surplus flows to the covered area. This flow has to be expressed in L/T (e.g. mm/h) with respect to the covered area;

7. The result now is the net rainstorm;
8. By convolution of the net rainstorm with the inflow unit hydrograph, the design inflow storm can be calculated. The unit hydrograph can be derived from figure 9. The design inflow is the maximum of this inflow storm.

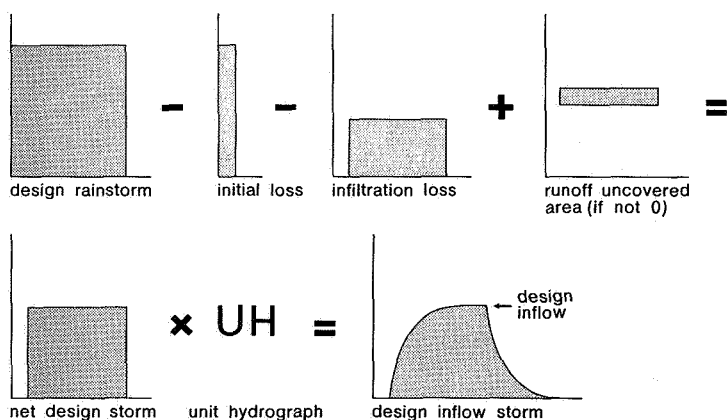


Figure 12 From design rainstorm to design inflow storm

As can be seen in figure 12 the maxima of the net rainstorm and the design inflow storm differ only when the storm duration is less than 20 - 30 minutes.

The second approach is based upon time series of rainfall and a simulation model. The method is as follows:

1. Convert the rainfall series into inflow series by subtracting the losses and by convolution with the unit hydrograph;
2. Route the inflow through a model of the sewer-system - in many cases in Holland a simple reservoir model. By doing this, series of amounts stored in the reservoir or a discharge by overflows can be calculated;
3. Repeat step 2 for different values of the controlling value(s), (e.g. design discharge, pump capacity, etc.);
4. Analyse the results of the steps 2 and 3 and select the design standards; for e.g. design discharge or pump capacity.

A prerequisite for applying this second approach is a sufficiently long

time series of rainfall data with short time intervals.

For calculating the required amount of (extra) storage in a system the application of depth-duration-frequency curves is basically wrong.

Instead, storage-design discharge-frequency curves should be calculated from the results of step 3. And these curves should be used for selecting the amount of storage.

REFERENCES

- ANDO, Y., TAKAHASI, Y., and KUAN, M.F., 1984. Relation between landuse and final infiltration rate in urban areas. In: P. Balmér, P.A. Malmquist, A. Sjöberg, Proceedings of the 3rd Int. Conf. on Urban Storm Drainage, Vol. 3 Planning and Control of urban storm drainage. Chalmers University of Technology, Göteborg.
- BEBELAAR, J.P., and BAKKER, J.W., 1981. Infiltration of rainwater through road pavement for the water supply of street-trees (in Dutch). Note 1247, Institute for Rural Engineering and Water Management, Wageningen.
- JACOBSEN, P., and HARREMOES P., 1981. Significance of semi-pervious surfaces in urban hydrology. In: B.C. Yen. Urban Storm Drainage; Proceedings of the second International conference. Volume 1. Dept. of Civil Engineering University of Illinois, Urbana-Champaign.
- PECHER, R., 1969. Der Abflussbeiwert und seine Abhängigkeit von der Regendauer. Eigenverlag des Instituts für Wasserwirtschaft und Gesundheitsingenieurwesen der Technischen Hochschule München.
- VAN DAM, C.H., and SCHOTKAMP, P., 1983. Infiltration in pavement (in Dutch). R.IJ.P.-report 1983 - 38 Abw., IJsselmeerpolders Development Authority, Lelystad.
- VAN DE VEN, F.H.M., 1984a. Design inflow intensity and design inflow profiles for storm sewers. In: Water Science Technology Vol. 16, Copenhagen, pp. 207-218.
- VAN DE VEN, F.H.M., 1984b. A model for estimating the duration of sewer overflows and the amount of overflow water, with the sewer-system of Harderwijk - Ermelo - Putten as example. R.IJ.P.-report 1984-22 Abw., IJsselmeerpolders Development Authority, Lelystad.
- VAN DE VEN, F.H.M., VAN DER KLOET, P., and VAN DER WAL, M., 1981. Some models and calculus methods for the relation between precipitation

and sewer inflow (in Dutch). Flevobericht 176, part A, B and C, IJsselmeerpolders Development Authority, Lelystad.

VOORTMAN, B.R., 1977. The drainage of some roads in Lelystad (in Dutch). Flevobericht 123, IJsselmeerpolders Development Authority, Lelystad.

VOORTMAN, B.R., 1984. The monthly water balance of a housing area and a parking lot in Lelystad (in Dutch). R.IJ.P.-report 1984 - 35 Abw., IJsselmeerpolders Development Authority, Lelystad.

Annex 1 THE MAIN FEATURES OF THE EXPERIMENTAL BASINS IN LELYSTAD

Lelystad is a new town, situated about 60 km east of Amsterdam in the polder Flevoland. The building activities started in 1967 and by now it has 60.000 inhabitants. The town is provided with a separate sewer-system and canals that convey the runoff from the storm sewers to the rural environment. Since 1968 a hydrological research project is going on in Lelystad. To obtain a better understanding of the urban hydrology, precipitation, storm water runoff, subsurface drainage runoff, ground-water levels and since 1982 in one basin also the quality of precipitation, storm water runoff and canalwater is monitored continuously in 3 basins by a central automatic datalogging and controlling system. Some characteristic data of the basins are given in table 1.

The Pampus-Blokkerhoek area contains 60 single-family houses with a height of 8 m and flat roofs. The sub-basins per gullypot range in area from 100 - 200 m² in Pampus-Blokkerhoek and from 100 - 260 m² on the Noorderwagenplein. The storm sewers in both basins consist of concrete pipes Ø 300 mm. The water in the sewer is dammed up by the V-notch weir used for the discharge measurements in such a way, that the bulk storage

in the pipes is filled up completely; only a small amount of dynamic storage is left. This allows the calculation of the sewer inflow from the measured discharge by

$$q_i = q_d + \frac{\partial s}{\partial h} \frac{dh}{dt}$$

q_i = inflow s = (dynamic) storage

q_s = discharge h = waterlevel near the weir

while $\partial s / \partial h$ is determined by experiments. The minimum measurable discharge is $0.005 \text{ m}^3 / \text{s/km}^2$ in Pampus-Blokkerhoek and $0.0066 \text{ m}^3 / \text{s/km}^2$ on the Noorderwagenplein.

Table 1 Features of the 3 basins for hydrological research in Lelystad

name	land use	area (ha)	slope (%)	% of area covered	% of covered area			water quality	recording since
					roof	asphalt + bridges	brick/ tile		
Pampus-									
Blokkerhoek	housing	2.0	0-2	41	30	32	38	no	1968
Noorderwa-									
genplein	parking	0.7	0-2	99.6	-	45	55	no	1969
Bastion	housing	4.5	0-2	66	43	6	51	yes	1982

In the Bastion area 270 single family houses and apartments are realised in 2 to 4 storey buildings. The roofs of these buildings have an angle of about 25° with the horizontal. The diameter of the sewer-pipes ranges from $\emptyset 300 - 500 \text{ mm}$. Because of the water quality measurements the water could not be dammed up, so the inflow cannot be calculated. The discharge measurements are done with an electromagnetic flowmeter. The minimum measurable discharge is $0.033 \text{ m}^3 / \text{s/km}^2$. The precipitation is measured with 2 ground level raingauges with an orifice of 0.4 m^2 .

The data logging system is controlled by a desk top computer. The system only registers significant changes in the values (event-sense registration). The shortest recording interval for each instrument is less than 20 s. Moreover, every 2 hours the values of all instruments are recorded, to have some recordings in times that a steady state appears.

URBAN STORMWATER RUNOFF POLLUTION

E.J.B. Uunk

ABSTRACT

In the Netherlands, until recently very little attention was paid to the quality of stormwater runoff.

In the paper, the results of two recent Dutch research projects concerning urban stormwater runoff pollution are presented.

Compared to the situation in other countries, the stormwater runoff in both Dutch basins contained very little suspended solids and oxygen demanding matter.

In one basin, the stormwater runoff was rather unpolluted for most constituents considered. This was partly caused by the mixing with drain-water prior to discharge. In the other basin, some pollution was found, in particular with respect to faecal bacteria, lead and zinc, mineral oil and certain polycyclic aromatic hydrocarbons. The nutrient concentrations in the stormwater runoff from both basins were fairly high.

The necessity to control stormwater runoff pollution depends on the degree to which receiving surface waters are affected. Control measures to be considered are listed.

1 INTRODUCTION

In the Netherlands, two systems are used to drain urban areas:

1. a subsurface drainage system to transport infiltrated precipitation and, in some areas, seepage;
2. a sewer system to transport stormwater runoff mainly from covered surfaces.

A sewer system is always needed, a subsurface drainage system only where groundwater control requires it.

When the stormwater sewer system is also used to transport the wastewater produced in the urban area, it is called a combined system; if the wastewater and the stormwater are transported by separate sewers, the system is called a separate system. In this system, the stormwater sewers discharge directly on receiving surface waters. In the combined system, most of the storm water runoff is treated prior to discharge (figure 1); the remainder is discharged directly following intensive precipitation, together with a part of the wastewater produced at that time and sediments, scoured from the sewers. Generally, the sewers discharge on small-sized urban canals or ponds.

In addition to these two main systems, a number of modifications of both systems exist, like the so called improved separate system. This paper focuses on the ordinary separate system.

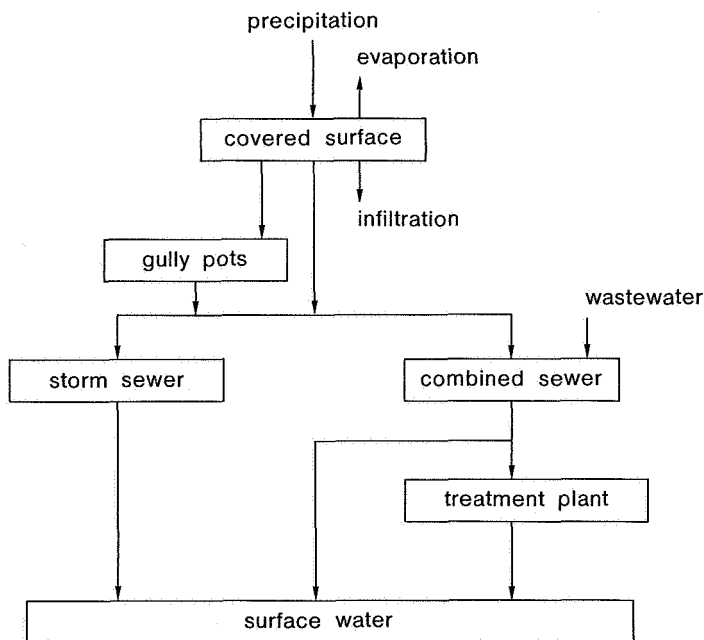


Figure 1 The transport of urban stormwater runoff by a separate and a combined sewer system

In the past, stormwater runoff was considered to be clean. Now that we have learned that stormwater (precipitation) is polluted in many respects, stormwater runoff obviously has fallen under suspicion as well.

The locations where the stormwater can become polluted after reaching the urban ground surface are indicated in figure 2.

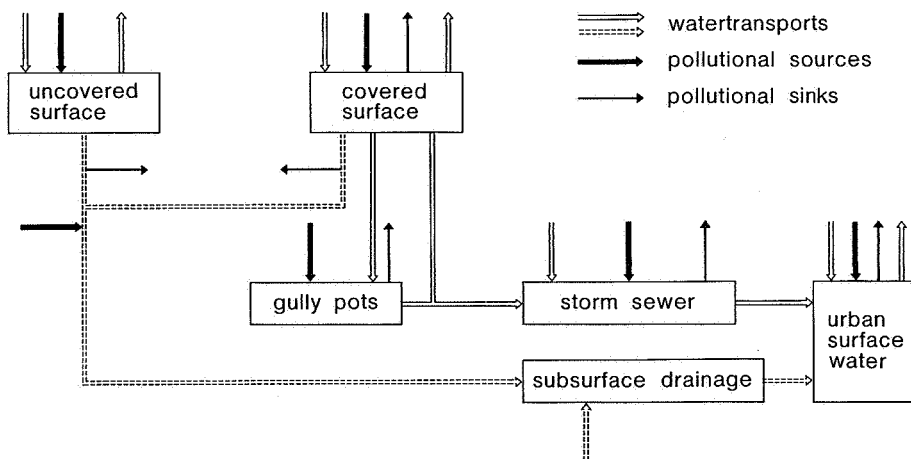


Figure 2 Stormwater runoff: pathways and pollutional sources and sinks

Pollutional sources of the (covered) surface are:

1. motorised traffic.

This source can be subdivided in:

- exhaust emissions from fuel combustion,
- fuel and lubricant spills,
- particle emissions from wear of brake linings, tires and clutch,
- and
- corrosion.

Main pollutants involved are some organic micropollutants and heavy metals;

2. atmospheric fall-out/precipitation.

Air pollution, originating from industrial and domestic sources and (again!) motorised traffic, appears as dust and soot particles, aerosols and in gaseous form. The pollutants, present in the air,

may reach the earth again in dry periods (dry deposition) and in wet periods (precipitation). Precipitation and dry deposition can be a significant source of pollutants in urban stormwater runoff. The precipitation effects a wash-out of pollutants from the atmosphere and, once turned into runoff, it may easily entrain the soluble substances and small particles, emanating from dry deposition. The main pollutants from this source in connection with stormwater runoff are some organic micropollutants and heavy metals;

3. litter (paper, plastics, etc.).

Because of its large average size, litter is likely to be only a minor constituent of runoff;

4. faecal deposition by animals (dogs, birds).

This can locally be a very significant source of, particularly, bacteriological pollution;

5. corrosion.

Corrosion of gutters, fences, lampposts and such can lead to a significant heavy metal contamination of the stormwater runoff;

6. vegetation and erosion.

The amount of material, entering stormwater runoff from these sources can be considerable. Qualitatively, the material is rather harmless;

7. industrial point sources.

The category is very diverse and only locally significant.

Further pollutional sources of infiltrated stormwater and stormwater runoff are:

- illegal dumping or discharge of polluted materials like used paint or motor oil into gully pots;
- dissolution of pollutants from the soil profile (generally unimportant);
- entrance of wastewater through "false connections" or through percolation from wastewater sewers.

In some older separate sewer systems, up to 5 to 10% of false connections are reported. In these cases, the entrance of wastewater is the prime source of organic and bacteriological pollution of the stormwater runoff.

On the other hand, there are also pollutional sinks, in particular the (already mentioned) retention by the soil profile, street sweeping, and

the cleansing of gully pots and sewers.

2 SIGNIFICANCE OF QUANTIFYING URBAN STORMWATER RUNOFF POLLUTION

Why do we want to know exactly how large the pollutional loads associated with urban stormwater runoff are?

Until recently, the only reason was to allow for the comparison of the pollution, from separate and combined sewer systems (the "emission trace"). Annual loads were established, typically only for BOD or COD, for the sake of the system choice for areas to be sewered. Discussions on the separate system focused on the number of false connections to be taken into account.

At this moment, the "emission trace" is only considered adequate for some toxics like mercury and DDT ("black list" substances). More generally, the emphasis is now on the assessment of the consequences of the discharges from sewers on the quality of receiving surface waters (the "immission trace").

Starting point are the functions of the affected surface waters. On the basis of these functions, water quality goals can be formulated for these waters.

Urban surface waters usually play a role in urban water management and town planning and serve as a habitat for aquatic life and as a medium for recreational activities. As a result, they have to comply with the so called "basic water quality" standards, with fishing water standards and, possibly, with swimming water standards. In the future, in addition ecological goals will be formulated. Comparing actual water quality with the relevant standards may result in a need for correctional measures. This requires quantitative insight into the factors that determine the actual water quality and into the way they influence water quality.

In urban surface waters receiving stormwater runoff, the discharges by the storm sewers are often the most important factor, determining water quality. Therefore, quantification of both urban stormwater runoff

pollution and of its consequences for the receiving waters is needed. Although this paper focuses on the former subject, some receiving water impacts will be indicated.

3 STORMWATER RUNOFF POLLUTION

Below, the first year's (1983) results of two recent Dutch research projects concerning stormwater runoff pollution are presented.

3.1 The experimental set-up

The two investigations concerned are:

1. The urban water research project Lelystad, carried out by the IJsselmeerpolders Development Authority. Location of the project is the experimental basin "Bastion" in the new town Lelystad. The project is carried out to contribute to the optimization of the planning and management of urban waters, in particular in the Lake IJsselmeerpolders. In this project, both the quantification of stormwater runoff pollution and the assessment of receiving water impacts are addressed (Van de Ven, 1982).
2. An investigation, carried out by Dwars, Hederik en Verhey Consulting Engineers in an experimental basin in the town Heerhugowaard. This investigation forms part of the STORA-38B project that aims at developing quantitative guidelines for the set-up and planning of sewer systems, utilizing information on emissions of pollutants. The Heerhugowaard project is restricted to quantification of stormwater runoff pollution. It is sponsored by the NWRW (Netherlands National Research Committee on Sewerage and Water Quality).

Table 1 contains some features of both basins. Both basins are residential areas; the Bastion basin has a higher population density and percentage covered surface than the Heerhugowaard basin. Both basins are completely flat.

In both basins, the stormwater runoff is finally discharged on the receiving surface water by one storm sewer outfall. In Heerhugowaard, this sewer also serves for the transport of drainwater, whereas in

Lelystad the drainwater is discharged separately.

Table 1 Features of the two experimental basins

		Bastion (Lelystad)	Heerhugowaard
	units		
Surface area	ha	4.5	12.5
Covered surface	ha (%)	3.0 (66)	6.8 (54)
Population density	inh/ha	160	90

In both basins, the stormwater runoff flow was measured continuously, while flow proportional sampling started above a certain (low) runoff flow (Heerhugowaard) or precipitation intensity (Bastion). Within a certain runoff event, generally several samples were taken. Not all runoff events could be sampled. When the flowmeter was not working, no samples were taken. This situation occurred in less than 10% of the time so that it hardly influences the reliability of the findings. Among the measured flows, the very small flows were not sampled. Fortunately it appears, both in the Bastion as in Heerhugowaard, that in general the water quality of the runoff does not depend on the runoff volume so that the presented results may be considered to be representative for the water quality of very small runoff volumes as well. All in all, in the Bastion 143 runoff events were sampled, in Heerhugowaard 39.

3.2 Quantitative aspects

The year 1983 was rather wet with extreme rainfall in May and November, while June and July were fairly dry months. In the Bastion, annual precipitation amounted to 928 mm. The flow measurements in the storm sewer related to 727 mm of precipitation and yielded on average runoff coefficient of 0.61. As a result of the drainwater impact on the storm sewer flow no runoff coefficient could be determined yet for the Heerhugowaard basin.

3.3 The quality of the stormwater runoff

Of the results of the water quality investigations, only (flowproportional) mean concentrations per runoff event are presented.

3.3.1 *Comparison with other stormwater quality data*

To give an idea of the quality of the stormwater runoff in both basins, in table 2 for a number of constituents mean concentrations over 1983 are presented and compared with results of similar investigations in other countries. The project from the USA concerns the "Nationwide Urban Runoff Programm" (NURP, 1983), in which the stormwater runoff of 81 experimental basins in 22 U.S.-cities was examined. Among the experimental basins were both residential and commercial areas, but no large industrial areas. The "various investigations" involve investigations in a number of experimental residential basins in Western Germany (Brunner, 1975), Finland (Melanen, 1981), Norway (Lindholm, 1978), Australia (Weeks, 1981), Switzerland (Roberts, 1979) and England (Mance, 1978).

One of the most remarkable features of the Dutch basins is, as table 2 shows, the relatively low suspended solids content of the stormwater runoff. Apparently, as a result of the flatness of the area and the fairly high contribution of paved surfaces, much less particles are picked up from the surface than elsewhere. In Heerhugowaard, the stormwater runoff contains more - mineral - suspended solids than in the Bastion basin, possibly as a result of the lower paving percentage.

The BOD, a measure for the presence of biodegradable organic matter, is in the Dutch basins also much lower than in the other basins, as is the COD, a measure for the total presence of oxygendemanding matter. Presumably this is, apart from the low solids content of the stormwater, mainly caused by the ascertained absence in both basins of false connections (which are presented in most other basins).

This absence of false connections enhances the suitability of both basins for evaluating the quality of stormwater runoff.

Table 2 Water quality of stormwater runoff:

arithmetic means of a number of (flowproportional) event mean concentrations

units: mg/l, except for faecal coliforms: counts/100 ml

Constituent	Bastion	Heerhugo- waard	USA*	Various investigations
Total suspended solids	33	-	180	80-900
Mineral suspended solids	21	36	-	20-850
BOD	3	2.5	12	7-14
COD	20	35	82	37-120
Total P	0.47	0.32	0.42	0.18-0.8
Total Kjeldahl-N	1.9	2.2	1.90	1.6-3.3
Nitrate-N	1.4	-	0.86	1.0-1.7
Total lead	0.05	0.01	0.182	0.05-0.44
Total zinc	0.43	0.09	0.202	0.10-0.8
Total copper	0.01	0.01	0.043	0.01-0.13
Faecal**	3,000	-	21,000	-

* for "median" experimental basin

** median for summer months (April-September)

In contrast with the BOD and COD, the nutrient content of the stormwater runoff in the Dutch basins is not significantly different from that in other areas.

As for heavy metals, the two Dutch basins differ considerably. The Heerhugowaard basin shows very low concentrations for all heavy metals examined, which may partly be caused by the relatively low traffic impact in the area. A more important factor appears to be the diluting effect of the drainwater discharged with the stormwater runoff. When compared to the other figures in table 2, the Bastion basin shows a low copper concentration, a fairly low lead concentration and a high zinc concentration.

The bacteriological contamination of the stormwater runoff in the Bastion is considerable but nevertheless low, compared to the U.S. basins where false connections often determine the bacteriological quality of the stormwater runoff.

3.3.2 *Suspended solids*

The substances discharged with the stormwater runoff can be classified in two categories:

- a. a fraction, present in or bound to suspended solids, and
- b. the dissolved fraction.

What is this suspended matter, present in the stormwater runoff, made up of?

First of all, of soil particles (clay and sand) and dust particles, present on, in between or close to the pavement, before being picked up and washed away by the runoff. This material is predominantly inorganic (mineral) by nature.

Organic by nature are leaves, grass cuttings and, more significant, animal faeces (in particular of dogs), which also contribute to the suspended solids in stormwater runoff.

In the Bastion basin, in 1983 about two-thirds of the discharged suspended solids were inorganic-mineral- and one-third organic by nature. Both fractions and especially the smaller particles in these fractions have a large binding capacity for substances that are in the first instance entrained by the runoff in dissolved form. As a result of this, the loading of the discharged solids with the pollutants can be much higher than that of the original particles. As a considerable part of the polluted solids will settle readily in the receiving surface water, water (and sediment) quality problems associated with the solids discharge are expected to be confined to the vicinity of the sewer outlets.

As was anticipated the suspended solids content of the stormwater runoff is strongly (cor)related to the maximum precipitation intensity occurring in the course of a runoff event. For, this intensity determines the force with which particles can be loosened from the surface

on which they were present before the event. Figure 3 exemplifies this relationship for the mineral suspended solids concentrations in both Dutch basins. The lines and correlation coefficients in figure 3 refer to a quadratic increase of solids concentration with precipitation intensity, which gave a better fit to the data set than a linear relationship.

As it turned out, the solids concentrations correlate even better with maximum runoff intensity than with maximum precipitation intensity. Apparently, the runoff intensity describes more closely the loosening of particles that takes place not only on streets, sidewalks and roofs, but also in gully pots and in sewer pipes.

As summer periods generally comprise higher precipitation and runoff intensities than winter periods, in summer higher solids concentrations are found. A large number of pollutants is, at least partly, bound to solid particles. Therefore, in summer stormwater runoff is more polluted than in winter for a number of constituents: this is illustrated in table 3. Of course, other factors than runoff intensities are also responsible for seasonal changes. The seasonal change in water temperature for instance will influence rates of biological processes and, as a result of that, variables like BOD and total Kjeldahl-N. Table 3 includes also chloride as a constituent which is not bound to the suspended solids. Chloride concentrations are relatively high in winter, due to the application of road salt for de-icing purposes (highest concentration determined: 740 mg/l).

	CORRELATION COEFF. (R)	NO OF MEASUREMENT
BASTION	.854	64
HEERHUGOW.	.771	38

MINERAL SUSP.
SOLIDS
(MG/L)

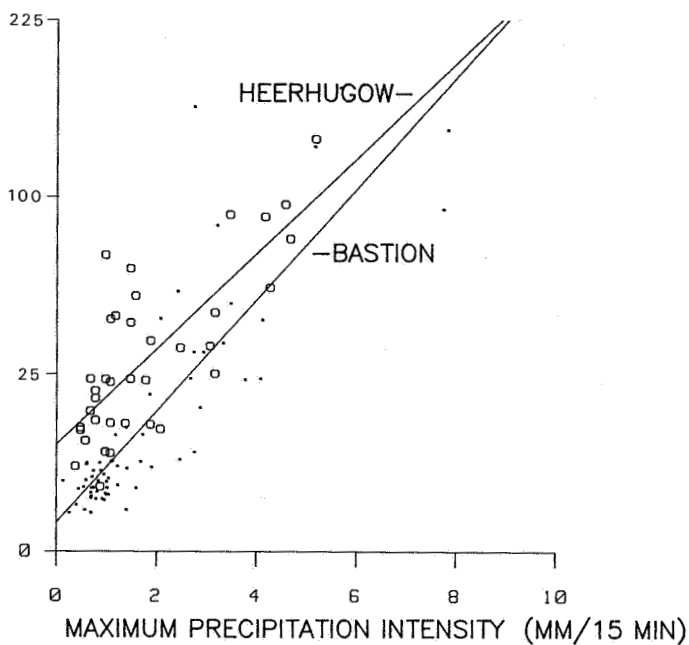


Figure 3 Mineral suspended solids in stormwater runoff and maximum precipitation intensity

Table 3 Water quality of stormwater runoff (Bastion): seasonal fluctuations and relation with suspended solids. All units: mg/l; except faecal coliforms: counts/100 ml

Constituent	r*	average concentration	
		summer	winter
Mineral suspended solids	0.998	31	9
BOD	0.79	3.7	2.3
COD	0.95	25	16
Total-P	0.50	0.54	0.41
Total Kjeldahl-N	0.73	2.2	1.6
Total lead	0.91	0.10	0.06
Total zinc	0.72	0.41	0.46
Total copper	0.82	0.011	0.009
Faecal coliforms	0.62	8,700	4,500
Chloride	-0.09	24	65

* correlation coefficient of linear regression with total suspended solids

3.3.3 *Oxygen demanding matter*

As table 3 indicates, the BOD of the stormwater runoff is fairly low in summer too. Therefore no direct problems with respect to the oxygen balance of receiving waters are to be expected, the more so as the stormwater runoff generally contains a sufficient amount of oxygen. A fairly good relation was established between BOD and COD (figure 4). A COD/BOD ratio of 5 follows from the regression equation, indicating a fairly low biodegradability of the organic matter in the runoff. Only a minor part of COD appears to be unrelated to BOD, this fraction possibly concerns oxidizable inorganic matter.

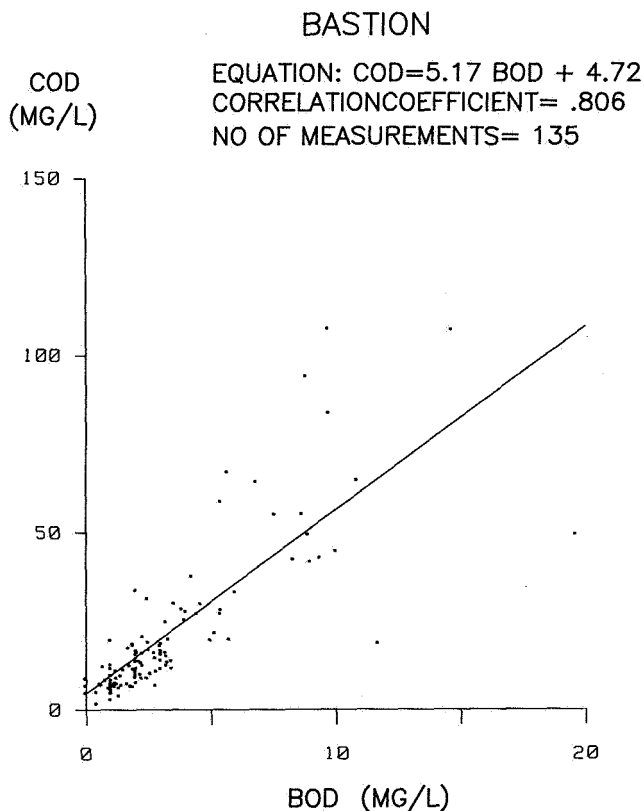


Figure 4 Relation between BOD and COD in the stormwater runoff
(Bastion basin)

3.3.4 *Nutrients*

The nutrient concentrations in the stormwater runoff are rather high. The phosphorus concentrations in particular do not meet Dutch standards, devised to control eutrophication in receiving water. Conceivable sources of nutrients are:

1. precipitation.

In particular for ammonium and nitrate nitrogen, the precipitation is one of the prime sources, as the concentrations in the precipitation (1.0 and 0.8 mg N/l, respectively) do not differ too much from the concentrations in the stormwater runoff (1.15 and 1.4 mg N/l,

respectively). The organic N and total-P concentrations in the precipitation however are that low (0.05 and 0.06 mg/l respectively) that their impact on stormwater runoff quality is negligible.

2. entrainment of particles by the runoff.

The entrainment of particles by the runoff results in an emission of nutrients in particulate form. In the Bastion basin, two-thirds of the phosphorus and one fourth of the nitrogen is discharged in particulate form. Particles contributing a lot of nutrients are organics, in particular animal faeces.

In table 4, the particulate P- and N-emissions in the runoff from the Bastion basin are compared with an estimate of the P- and N-runoff from fertilized agricultural soils in the Netherlands. This estimate is based on the assumptions that 1% of the manure produced and 0% of the fertilizers applied run off. It appears that the P- and N-runoff in urban areas like the Bastion is higher than in agricultural areas!

Table 4 Phosphorus and nitrogen runoff

	Runoff (kg/ha,yr)	
	P	N
Residential area (Bastion)	1.2	3.1
Estimate for agricultural areas	0.6	2.2

3. mineralization/dry deposition.

In contrast with the first two processes, mineralization and dry deposition take place in the periods in between runoff events.

They produce dissolved nutrients: orthophosphate, ammonium- and nitrate-nitrogen. As a result, the concentrations of those constituents tend to increase with increasing dry weather period. This is illustrated in figure 5 for ammonium nitrogen.

For orthophosphate, mineralization (in sewer pipes and, in particular gully pots) appears to be the most significant process, while for the nitrogen compounds dry deposition is equally important.

BASTION

N-NH₄
(MG/L)

CORRELATION COEFFICIENT (R) = .688
NO OF MEASUREMENTS = 108

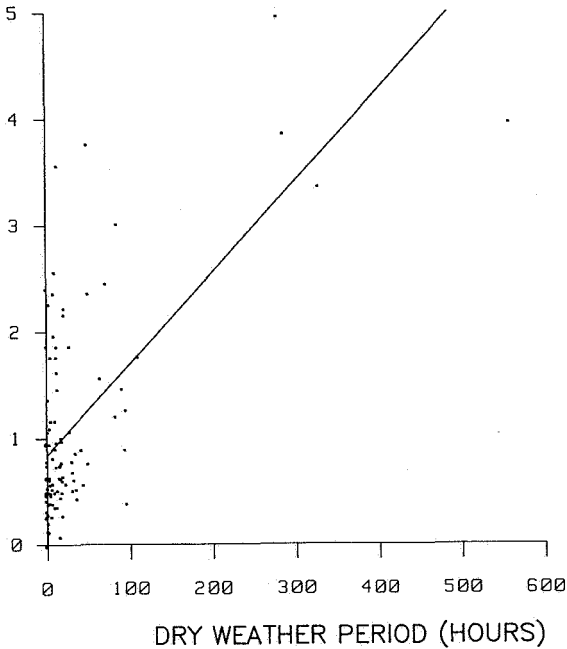


Figure 5 Ammonium in stormwater runoff as a function of dry weather period (Bastion basin)

It is to be expected that the dissolved nutrients, produced by these processes and in particular by the mineralization in gully pots, are discharged with the first part of the stormwater runoff following a dry period. This "first flush" phenomenon was indeed observed in the Bastion basin (Uunk, 1984).

3.3.5 Bacteriological contamination

That runoff of manure-like material plays a significant role is confirmed by the bacteriological contamination of the stormwater runoff. As

mentioned before, this cannot be ascribed to wastewater entrance into the storm sewers. For the Bastion basin, the absence of wastewater entries was a.o. ascertained in a storage experiment during which no change of water levels could be observed in the sewer system while no stormwater entered and discharge was prevented.

That the bacteriological contamination is of animal origin could be ascertained for the Bastion basin by the high density of faecal streptococci that was found in the stormwater runoff. It is reported (Olivieri, 1977) that faecal coliform (FC)/faecal streptococci (FS) ratios below 0.7 indicate an animal origin of bacteriological contamination. In the Bastion basin, the FC/FS ratio is generally below 0.6.

Incidentally, salmonella's could be detected in the stormwater runoff so that real health risks are conceivable in case of intensive contact with this water.

3.3.6 *Heavy metals*

In table 5, heavy metal concentrations in the runoff from both basins are compared with Dutch heavy metal standards for surface waters and heavy metal concentrations in precipitation (in the Bastion basin). As discussed before, the runoff from the Heerhugowaard basin shows low heavy metal concentrations. The runoff from the Bastion basin shows low copper and chromium concentrations and not too high cadmium concentrations. The lead concentrations, though low when compared to the situation abroad, are high when compared to the Dutch surface water standard for lead. The dry deposition of lead, originating from local exhaust emissions, appears to be the main source of lead in the Bastion area.

The zinc concentrations in the Bastion runoff are high. Corrosion of zinked fences that abound in the basin is suspected to be the main source of zinc. At the moment, experiments are carried out to quantify the emissions from the source.

The precipitation may be the main source of copper and cadmium in the Bastion basin and a significant source of lead and zinc.

Table 5 Heavy metal concentrations in stormwater runoff and precipitation and according to surface water standards (all concentrations in mg/m³)

Metal	Maximum allowable surface water concentration	Average concentrations		
		precipitation	stormwater	runoff
		Bastion	Heerhugowaard	
Lead	50	16	54	6
Zinc	200	116	430	85
Copper	50	8	10	9
Chromium	50	5	11	7
Cadmium	2.5	0.6	0.8	0.2
Nickel	50	-	-	4
Mercury	0.5	-	-	0.1

The zinc, originating from precipitation and from corrosion, is in a dissolved state, when transported by the stormwater runoff.

In comparison with zinc, lead is adsorbed readily onto particles. As a result, a relatively large part of the lead in the stormwater runoff is associated with the suspended solids and only a minor fraction is in a dissolved state. In figure 6, the presence of lead and zinc in the stormwater runoff from the Bastion basin is related to suspended solids concentrations. The slopes of the regression lines indicate the lead and zinc contents of the suspended matter. These are that high that for instance Dutch standards for the quality of sewage sludge to be applied in agriculture are violated.

BASTION

	CORRELATION COEF. (R)	NO OF MEASUREMENT
ZINC	.715	68
LEAD	.911	60

(MG/L)

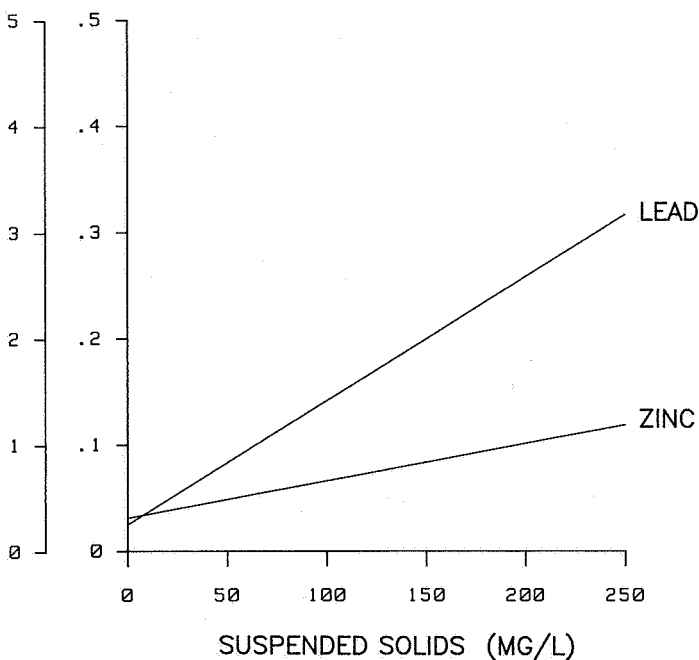


Figure 6 Lead and zinc in stormwater runoff in relation to suspended solids content (Bastion basin)

3.3.7 *Not (easily) biodegradable organics*

Finally some comments on the presence of non or not easily biodegradable organic compounds in the stormwater runoff.

The stormwater runoff contains a considerable amount of mineral oil. In the stormwater runoff from the Bastion basin, an average oil concentration of 1.4 mg/l was established. The oil content of the stormwater runoff may be somewhat underestimated. Sampling took either place in

plastic containers (automatic sampling), resulting in possible losses to the container walls, or in glass containers. Sampling in glass containers was performed by hand, generally near the end of the storm event, when the runoff was relatively clean. It appears that oil spills from parked cars and dumping waste motor oil in gully pots are the main sources of mineral oil.

Polycyclic aromatic hydrocarbons have also been detected in rather high concentrations in the runoff from the Bastion basin. The median total-concentration of the "Borneff six" amounted to 0.5 µg/l, i.e. five times the relevant Dutch standard for surface waters. The exhaust emissions by the motorised traffic appear to be responsible for this exceedance. Of the remaining micropollutants examined, only pentachloorphenol and lindane turned out to be significant. These two compounds appear to be omnipresent in the aquatic environment through, so that possibly non-specific sources are responsible for the concentrations observed. All sampling for organic micropollutants and for salmonellae took place by hand (in glass containers) near the end of the runoff events.

4 CONTROL OF STORMWATER RUNOFF POLLUTION

As it appears, stormwater runoff can be polluted to such an extent that measures control the emission of pollutants may be advisable. As indicated before, the consequences for the receiving surface water have to determine the extent to which measures are opportune. Below, some of the most important measures that can be considered are summarized. The following categories can be distinguished:

1. "non-specific" measures, like the abatement of air pollution, which may result in reduction of the emissions of heavy metals and of organic micropollutants;
2. local regulations, like regulations to oblige dog-owners to keep their pet's faeces from certain public grounds or regulations to penalize dumping of wastes in gully pots of storm sewers and/or in surface water;
3. measures during the (re)construction of a sewer system like
 - the prevention of false connection, a.o. by the application of

- different materials for stormwater and wastewater sewers,
- maximizing storage and infiltration capacity in the drainage area to reduce runoff (intensity),
 - locating discharge points in such a way that negative effects on receiving water are minimized,
 - using, if available, water of good quality to flush surface waters that receive stormwater runoff,
 - disconnecting polluted surfaces like car-parks, markets and main roads from storm sewers,
 - the application of end-of-pipe treatment facilities like retention and detention basins;
4. maintenance and control measures, like
- street sweeping,
 - cleansing gully pots,
 - other sewer maintenance and control activities,
 - controlling whether false connections have been illegally;
5. public information and education.

5 CONCLUSIONS

1. Stormwater runoff can be polluted to some degree, in particular with respect to faecal bacteria, lead and zinc, mineral oil and certain polycyclic aromatic hydrocarbons.
- The emission of nutrients, in particular of phosphorus, can add significantly to the eutrophication of receiving surface waters.
2. If, and if so to what extent, measures to control the emission of pollutants associated with the stormwater runoff are necessary, depends on the consequences of this pollution for receiving surface water. Investigations into these consequences are needed.
3. Significant measures to control stormwater runoff pollution are a.o.:
- the abatement of air pollution;
 - the prevention of false connections;
 - control of dumping of wastes;
 - reduction of runoff (quantity and intensity);

- disconnecting polluted surfaces from storm sewers;
- public information and education.

REFERENCES

- BRUNNER, P.G., 1975. Die Verschmutzung des Regenwasserabflusses im Trennverfahren, Diss., München
- LINDHOLM, O., P. BALMER, 1978. Pollution in storm runoff and combined sewer overflows, Proc. Int. Conf. on urban storm drainage, Southampton
- MANCE, G., M.M.J. HARMAN, 1978. The quality of urban stormwater runoff, Proc. Int. conf. on urban storm drainage, Southampton
- MELANEN, M., 1981. Quality of runoff water in urban areas, Publ. 42 of the Water Research Institute, Helsinki
- ROBERTS, P.V., L. DAUBER, B. NOVAK und J. ZOBRIST, 1979. Schmutzstoffe im Regenwasser einer Städtischen Trennkanalisation, in Stuttgarter Berichte zur Siedlungswasserwirtschaft, 64, p. 125-145
- U.S. Environmental Protection Agency, 1983. Results of the Nationwide Urban Runoff Program, volume I-Final Report, Washington
- UUNK, E.J.B., 1983. Pollution in urban runoff, Contribution to the MAB-5 Workshop on Land use impacts on aquatic systems, Budapest
- UUNK, E.J.B., 1984. De vuiluitworp van regenwaterriolen en hun invloed op het oppervlaktewater; de eerste resultaten van een onderzoek in Lelystad, H₂O (17), p. 57-64
- VEN, F.H.M. VAN DE, 1982. Monitoring water quantity and water quality in an urban basin. In: Advances in hydrometry (proceedings of the Exeter symposium), IAHS-publ. no. 134
- WEEKS, C.R., 1981. Pollution in urban stormwater runoff, Proc. 2nd Intern Conf. on urban storm drainage, Urbana

POLLUTION EMISSION FROM COMBINED SEWER SYSTEMS;
A FULL SCALE INVESTIGATION PROGRAMME

D. Vat

ABSTRACT

A full scale investigation and monitoring programme was started in 1981 to gain better insight into the processes that influence pollution emission from sewer systems.

In this paper, the first results are presented of a regression analysis of collected data in two catchment areas served by combined sewer systems.

The analysis shows that maximum rainfall intensity and seasonal variation in the run-off process greatly influence the amount of sewage sludge in the stormwater discharges.

The COD, P-tot and Pb concentrations in the stormwater discharges show a good correlation with the corresponding concentration of the dry residue of the non solved constituents.

Cost-effective rehabilitation measures and maintenance programmes to reduce pollution emission from combined sewer systems should be more directly focused on the reduction of sewage sludge discharges.

1 INTRODUCTION

Stormwater discharges from sewer systems are a major source of surface water pollution.

From a technical and economic point of view it is hardly possible to eliminate these diffuse pollution sources entirely.

One of the problems involved is the difficulty in demonstrating the cost-effectiveness of rehabilitation measures. Consequently, it is hard to formulate a well grounded policy aimed at a responsible reduction of such diffuse discharges. The situation sketched applies in particular to the emission from combined sewer systems.

In the 70's attempts were made to analyse the emission problems of sewer systems by means of mathematical simulation models (Wiggers, 1978; Vat 1978). However the lack of data from operational situations for verification restricted the value of these models.

In 1978, the Research Organization of the Water Authorities in the Netherlands (STORA) took the initiative to carry out a full scale investigation and monitoring programme on the emissions from combined and separate sewer systems. The full-scale monitoring itself was commenced in 1981. The programme is being implemented by DHV Consulting Engineers BV. The sampling programme is being carried out by members of the STORA. In 1983, a supplementary investigation programme was set up, in cooperation with the Ministry of Housing, Planning and the Environment, to study the effects of pollution emissions from sewer systems on the quality of the receiving surface water.

The first results of the current investigation programme with respect to the emission from combined sewer systems, were presented in Copenhagen in 1984 (Bakker, 1984).

The present contribution provides a further analysis of the collected data up to December 1984.

The monitoring programme will continue until the end of 1986.

It will be clear that the conclusions presented in this paper have a preliminary character.

2 AIM AND SET-UP OF THE PROGRAMME

2.1 Aim of the programme

The aim of the investigation and monitoring programme is to gain more insight into the relative importance of the major factors affecting pollution emission from sewer systems. This will allow better quantification of the cost-effectiveness of rehabilitation measures. Development of emission models based on monitoring of operational situations will permit the knowledge obtained to be used for water quality management.

2.2 Set-up of the investigation programme

With respect to the pollution load of stormwater discharges various processes and pollution sources can be distinguished. The contribution to the final pollution emission depends on factors which are typical for the process considered. These processes and the major pollution sources are shown schematically in Figure 1.

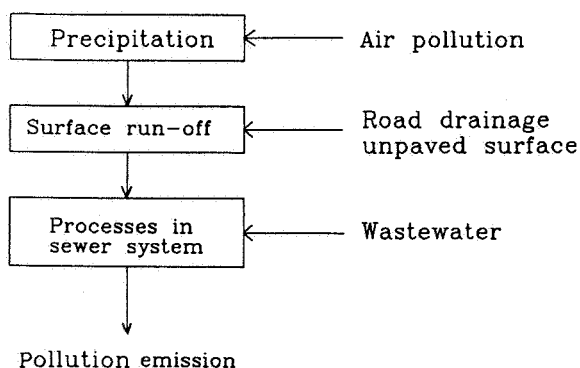


FIG. 1 PROCESSES AND POLLUTION SOURCES

The processes and pollution sources can also be described by parameters. The aim of the study is to examine the relative importance of the major parameters. From a point of view of pollution emission these parameters can be seen as "decisive factors". Some of these decisive factors are presented in Table 1.

A distinction can be made between precipitation data and area data. The area data are of particular importance for the transferability of the investigation results to other catchment areas.

Table 1 Decisive Factors

precipitation data	area data
- rainfall intensity	- size of the catchment
- rainfall depth	area
- rainfall duration	- paved surface
- dry weather period	- storage capacity of the sewer system
	- capacity of the sewage pumping station
	- dry weather flow
	- average terrain slopes
	- slopes of sewers

In the first stage of the project the most appropriate method to analyse the results of the monitoring programme was studied. The results of this study made it clear that all individual process components should not be investigated separately (STORA Workshop, 1980).

The monitoring programme implemented concentrates on the registration of the intensity and duration of precipitation (input) and the flow, duration and pollution of stormwater discharges (output). The various processes contributing to the pollution load are not monitored in detail but both input and output factors are studied by regression analyses.

2.3 Set-up of the monitoring programme

After a period of preparation in which a prototype of the recording and sampling apparatus was developed and tested, the monitoring programme was started in 1981. It will continue till the end of 1986 and covers six catchment areas in all.

An overview of the areas monitored, types of sewer systems, terrain slopes and periods of monitoring is given in Table 2.

Table 2 Overview of Catchments Areas Monitored

area	type of sewer system	terrain slopes	period of monitoring*
Loenen (gld.)	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 length of the period of monitoring depends on the type of sewer system and the frequency of stormwater discharges into open water.

3 RESULTS OF THE MONITORING IN LOENEN AND OOSTERHOUT

3.1 Area data

The major area data from the monitoring areas Loenen and Oosterhout are reproduced in Table 3.

A distinction is made between:

- data that characterize the area
- data that characterize the sewer system

Table 3 Data of the Areas Monitored

Decisive Factor	Loenen	Oosterhout
Size of the catchment area [ha]	56,5	22
Paved surface [ha]	15.8	11.6
Percentage paved area [%]	28	53
Population	2050	2270
Average terrain slope	moderate	flat
Storage capacity of the sewer system [mm]	5.7	5.3
Pump capacity*) [mm/h]	0.88	0.97
Theoretical overflow frequency [per year]	9	9

*) the dry weather flow not included

The theoretical average overflow frequency given in the table was calculated with the standardized "Kuiper model", using the pointplot rain data of De Bilt (Koot, 1967).

The fact that the theoretical overflow frequency for both systems is 9 is coincidental. It does however provide the possibility to examine whether assessment of the pollution from combined sewer systems on basis of the theoretical overflow frequency is reasonably reliable.

3.2 Overflow frequencies and run-off coefficients

Observed overflow frequencies and run-off coefficients were established for both monitored areas on the basis of the measured precipitation and overflow volumes. The overflow frequencies are given in Table 4.

Table 4 Theoretical and Observed Overflow Frequencies

area	theoretical overflow frequency [per year]	observed overflow frequency [per year]
Loenen	9	15.7
Oosterhout	9	12.4

It is noteworthy that in both cases, observed overflow frequencies are higher than the theoretical average overflow frequency.

No firm conclusions can be drawn on a basis of the observed overflow frequencies. Several aspects have to be taken into consideration such as:

- the monitoring period could have been relatively "wet"
- the rainfall pattern in Loenen and Oosterhout may differ from that in De Bilt
- the sewer system (including pumping stations) may not have functioned without breakdowns
- the performance of the "Kuiper model" for calculation of the theoretical overflow frequency

To study these aspects a separate project was started, primarily to study the reliability of the "Kuiper model". In this project special attention is given to the variations in the value of the run-off coefficient and the distribution of precipitation across the Netherlands.

Figure 2 shows the calculated run-off coefficients per storm event plotted against the month of occurrence.

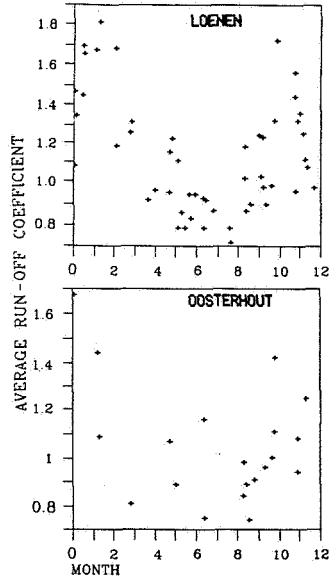


FIG. 2

These figures indicate a seasonal affect. The results have been used in the further analysis of the emission data.

3.3 Emission data

The following data have been obtained from each overflow event by means of registration, flow sampling and sample analysis:

- the discharge into surface water as a function of time
- the pollution of the stormwater discharge expressed by the parameters BOD, COD, dry residue of the non solved constituents, P-tot, N-Kj, Pb and Zn.

The data were filed in data-bases after careful scrutiny.

These primary data have been used to calculate some characteristic parameters of the pollution emission and to perform the regression analysis of precipitation data and emission data.

One of the characteristic parameters is the mean value of the average pollution concentration per overflow. These values are shown in Figure 3.

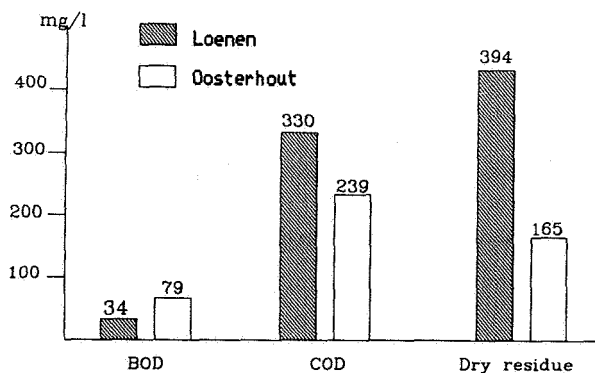


FIG. 3 MEAN VALUE OF THE AVERAGE CONCENTRATION

The mean values of the average concentrations of BOD, COD and the dry residue for the two areas taken into consideration differ quite markedly. In Loenen relatively high dry residues were found while in Oosterhout the BOD mean value is relatively high.

This indicates that different processes may dominate in the two catchment areas. The maximum values of the average concentrations per overflow for both areas are given in Figure 4, in which the mean values from Figure 3 are included for comparison.

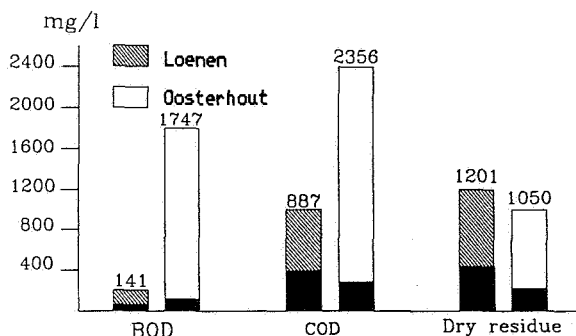


FIG. 4 MAXIMUM VALUE OF THE AVERAGE CONCENTRATION

It is remarkable that in Oosterhout the maximum values for BOD and COD deviate relatively greatly from the related mean values, i.e. 1:20 and 1:10 respectively instead of 1:3 to 1:5 for the other pollution parameters.

The differences noted above can probably be attributed to the following local circumstances.

Loenen

The relatively high value of the dry residue concentration of the sewage overflow is mainly caused by the characteristics of the area (see Table 3).

Loenen is located in a hilly area and the catchment area monitored has a relatively high percentage of unpaved surface.

During heavy rainfall, anorganic material will be washed down from the unpaved surfaces, resulting in high dry residue values.

Oosterhout

The high peak values of the BOD and COD concentrations are caused by sludge deposits in the sewer to the overflow device.

These deposits are the result of wastewater discharges from some ten houses that are directly connected to the overflow sewer main, in which velocities are very limited during dry weather flow. Further analysis showed that the high concentrations occurred during two showers with relatively high rainfall intensities.

In the monitoring programme extra attention is now being given to these sludge deposits.

4 REGRESSION ANALYSIS RESULTS

4.1 Methodology

The aim of the regression analysis is to determine to what extent the various decisive factors are related to the characteristic emission parameters.

In the phase of the study under consideration, the relations between various combinations of input and output parameters have been examined. Two groups amongst those already studied are considered further, i.e. relations between precipitation data and emission data and relations between the various emission data.

4.2 Precipitation data and emission data

On the basis of the available data, a large number of relevant relations were studied in general. It appeared from this study that there is a clear relation between the maximum rainfall intensity and the average concentration of some pollutants in the overflow water.

From the literature, it was to be expected that the run-off process on paved and possibly unpaved surfaces affects the pollution emission. As shown in 3.2, the seasonally dependent character of the run-off process can be represented by the variation in the value of the run-off coefficient.

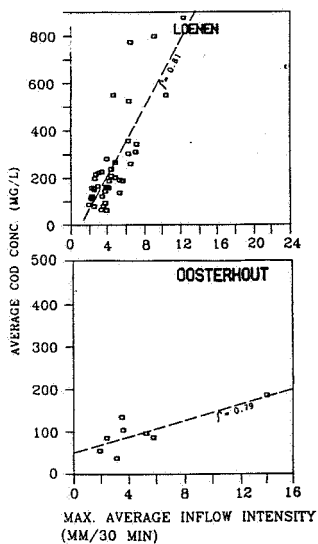


FIG. 5

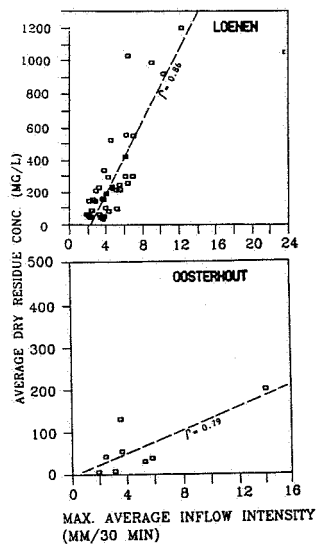


FIG. 6

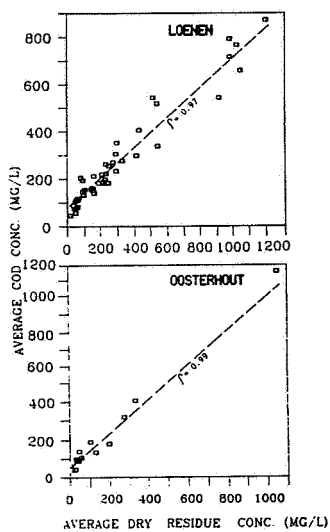


FIG. 7

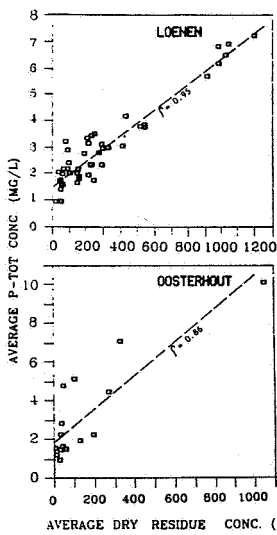


FIG. 8

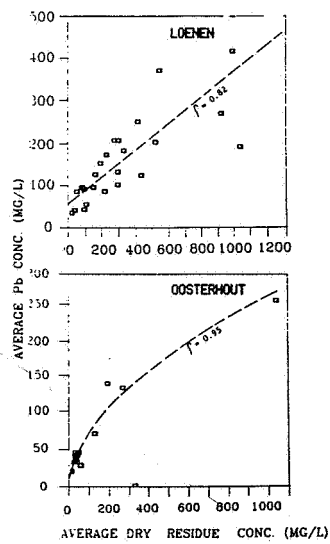


FIG. 9

To study the influence of both the maximum rainfall intensity and the seasonal variation in the run-off coefficient on a specific emission the concept of "the maximum value of the average inflow intensity" has been introduced.

Maximum value of the average inflow intensity (mm/30 min.) = maximum value of the average rainfall intensity during a period of 30 minutes x the average run-off coefficient during the storm event.

In Figures 5 and 6, the average concentrations of the COD and the dry residue in the monitoring areas Loenen and Oosterhout have been plotted against the corresponding maximum value of the average inflow intensity. It appears from the calculated correlation coefficients that there is a clear relation.

If it is assumed that the average dry residue concentration is a good representation of the amount of sludge that is stirred up and discharged, it is of major importance that this complicated phenomenon can evidently be described by the maximum value of the average inflow intensity.

4.3 Comparative emission data

To increase the insight into the major processes and the relative importance of the decisive factors, a search for relations between the average concentrations of the various pollution parameters was also undertaken. In connection with the anticipated effect of the stirring up of sewage sludge on the pollution emission, the relation between the average dry residue concentration and several other average concentrations were examined.

These relations for the average concentrations of COD, P-tot, Pb for both measuring zones are shown in Figures 7, 8, and 9.

It appears from these figures that the concentrations of COD, P-tot and Pb show a good correlation with the average dry residue concentration. This means that these pollution components are bound to a major extent to the sewage sludge that is discharged during an overflow.

The remaining pollution components, particularly N-Kj and Zn, do not have such a clear connection with the sewage sludge. These substances are evidently better dissolved in the sewage.

5 CONCLUSIONS

5.1 Consequences for model building

The importance that had already been attached to the stirring up of the sewage sludge was confirmed by the results of the monitoring and analyses. There proved to be a clear correlation between the amount of overflow sewage sludge expressed in the average dry residue concentration and the introduced concept of the "maximum value of the average inflow intensity". It also appeared that the sewage sludge is not of equal consequence for all pollution components. In preparing the emission models, the sludge bound pollution and the pollution from dissolved substances must be given separate attention.

5.2 Perspectives for design and maintenance

The results now available provide grounds for the conclusion that it is not only the general design of the sewer system (storage and pump capacity) that influences the pollution emission, but also the detailed design and the maintenance situation.

Both quantitative and qualitative aspects of the emissions appear to be incidentally strongly influenced by pump breakdowns, extreme sludge deposits, etc.

It may be said that current methods of evaluating sewer systems have serious limitations.

Major decisive factors, such as maximum rainfall intensity and seasonally varying values of run-off coefficients, are not taken into account.

Moreover, further distinctions will have to be made between the various pollution components, such as oxygen-binding matter (BOD, COD), eutrophicating matter (P-tot, N-Kj) and heavy metals (Pb and Zn).

It is of great importance that more attention is given to the sewage sludge. Besides the avoidance of sludge deposits by means of a good detailed design, efficient sewer flushing programmes based on sewer inspection can contribute. Allied measures, aimed at removing the settleable sludge from the sewage overflow, can also produce a notable improvement. Settling basins and swirl separators seem to be the most promising devices under Dutch circumstances.

The recent development of new technologies can act as an important stimulus in this field.
It is the task of policy makers to give these new technologies a chance.

REFERENCES

- BAKKER, K. and V. WIERINGEN, H., 1984. The influence of rainfall characteristics on the pollution emission. IAWPRC Wat. Sci. Techn. Vol. 16, Copenhagen, pp. 285-295.
- KOOT, A.C.J., 1967. The storage and the discharge capacities of sewerage systems and the operating frequency of storm overflows: Dutch methods of calculation.
Institution of Civil Engineers, Symposium on Storm Sewage Overflows, page 135-141.
William Clowes and Sons Limited London.
- VAT, D. and WIGGERS, J.B.M., 1978. Urban stormwater management: Dutch practice in an international perspective.
Université de Liège Colloque International.
Traitement combiné des eaux usées domestiques et industrielles.
- WIGGERS, J.B.M., 1978. In what way is water pollution influenced by sewerage systems CHO/TNO.
Urbanization and water management. Proceedings of technical meeting 34.
- EMISSION OF POLLUTION FROM SEWERAGE SYSTEMS.
Collected papers of the workshop held on 27 and 28 November 1980 and published by STORA Rijswijk.

COMBINED SEWER OVERFLOW POLLUTION ABATEMENT

A.S. Hogendoorn - Roozemon

ABSTRACT

The realization of extra in-line storage capacity in combined sewerage systems for pollution emission reduction purposes is discussed in relation to the application of special combined sewer overflow (CSO) devices. Apart from other advantages, it is estimated that application of CSO-devices leads to more cost-effective pollution abatement. The most promising CSO-devices for practical application in the Netherlands are briefly reviewed.

1 INTRODUCTION AND PROBLEM DEFINITION

In several countries in Western Europe, amongst which Belgium and the Netherlands, the theoretical, average overflow frequency f_o is used as one of the design criteria for combined sewer systems. The f_o is considered a function of storage volume in the network and interceptor capacity. Usually the interceptor capacity is limited on considerations of sewage treatment plant size. Consequently, extra in-line storage capacity has to be realized in order to meet the f_o criteria, i.e. only for reasons of surface water quality control.

It is doubtful, however, if extra in-line storage capacity is the most cost-effective pollution abatement. In this paper an alternative approach is proposed, based on the application of special CSO-devices to reduce the overflowing pollution load where necessary. Seven types of CSO-devices are listed, while the most promising devices for the Dutch conditions will be briefly discussed. To illustrate some features a hypothetical example is worked out, offering a rough estimate of the financial consequences of these CSO-devices versus extra in-line storage capacity.

2 IN-LINE STORAGE CAPACITY

The application of the f_0 as design criterion often leads to larger diameter piping to obtain extra in-line storage capacity.

This measure, meant to reduce pollutants emission is not only costly, but also has certain other disadvantages:

- . It increases the residence time of the wastewater in the system, thereby further degrading it's quality and increasing the possibility of odour nuisance at the pumping stations or treatment plant.
- . It will occur more often that the system is partly filled when a second rainfall event presents itself. The total storage volume is not always available.
- . During low flow conditions more sedimentation within the sewer lines will occur.

The last mentioned factor is of great importance since the lions' share of the pollution load caused by combined sewer overflows usually consists of resuspended sludge deposits. This is due to the fact that in flat area's the major part of the sewer network is situated below the overflow level, in order to maximize storage volume. This results in minimal sewer-line gradients and thus conditions favourable to sedimentation.

In Table 1 the minimal shear stress necessary to prevent sedimentation in sewers is compared with the near maximum values (sewer lines

completely filled) obtained for the indicated conditions, which are representative for the situation in the Netherlands. It is clearly indicated that in general self cleansing of these collection systems is quite doubtful.

Table 1 Shear stress in completely filled sewer lines, as compared with the values necessary for self cleansing.

pipe diameter (mm)	gradient	shear stress (N/m ²)
200	1:500-250	0.98-1.96
300	1:750-500	0.98-1.47
400	1:750	1.30
500	1:1000	1.23
600	1:1000	1.47
700	1:1000	1.72
Necessary for self cleansing in combined sewerage system		2-3.5

An increase in pipe diameters, with otherwise the same hydraulic conditions, increases the probability of sedimentation. This is markedly illustrated by Broeker (1984), who experimentally determined the critical flow velocity needed for self cleansing of combined sewers (Figure 1).

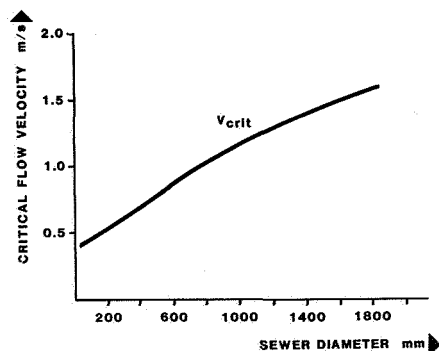


Figure 1

Critical flow velocity necessary for self cleansing operation of combined sewers, as a function of sewer-line diameter (Broeker, 1984)

From the above mentioned it follows that sedimentation processes within the sewers are favoured with increasing sewer diameters at equal hydraulic loading. Therefore Berlamont and Smits (1984) stipulate the possibility, that increased in-line storage capacity may even have a negative effect on pollution emission.

3 COMBINED SEWER OVERFLOW DEVICES

A preliminary study on emission reducing combined sewer overflow devices has recently been carried out within the framework of the NORW, the Dutch National Research Programme concerning sewer system design in relation to receiving water quality (VROM/NWRW, 1984; Coppes, 1985).

Different types of devices were evaluated on the basis of direct applicability under the typically Dutch conditions, as specified earlier.

The devices considered were:

- physico chemical and biological treatment units
- special filters and (micro) strainers (Drewing, 1979)
- the helical bend separator (Sullivan, 1975)
- tea cup separator (Wilson, 1978)
- sedimentation tanks
- improved overflow chambers
- swirl concentrator.

The physico-chemical and biological treatment units as well as the filters and strainers generally are not feasible, due to the low overflow frequencies.

The helical bend and tea cup separator are still in an initial phase of development, while their construction is rather complicated.

Although further research is needed on many aspects of these often newly developed devices, the swirl concentrator and the "improved overflow chambers" were considered to offer the best perspectives, besides the sedimentation tanks, which already are applied on a larger

scale. The improved overflow chambers have not yet outgrown their developmental stage either, but the simplicity and low cost of these devices make them attractive for practical application.

In the following the "improved overflow chambers" and the swirl concentrator will briefly be reviewed. For further detailed information the reader is referred to the publications by Coppes (1985) and VROM/NWRW (1984). Sedimentation tanks will be discussed in the paper presented by de Ruiter.

Four types of "improved overflow chambers" are distinguished, which mainly have been developed in the United Kingdom:

- the stilling pond (Figure 2) (Nicoll, 1978; Halliwell, 1982; Burrows, 1982)
- the high side weir, with longitudinal overflow weirs parallel to the inflow (Figure 3)
- the shaft overflow, with rectangular and circular design shapes (Figure 4) (Burrows et al., 1984)
- the vortex chamber with peripheral spill (Figure 5); the foul outlet pipe is set in the centre of the chamber floor and the overflow occurs over a weir formed over 50% of the peripheral wall. A scumboard retains the floatables.

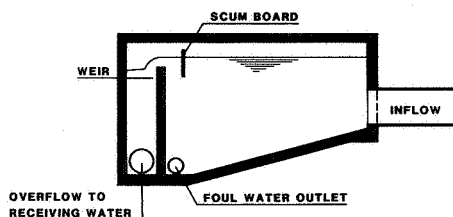


Figure 2 Stilling pond overflow

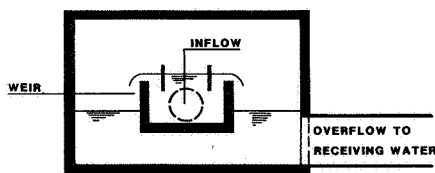


Figure 3 High side weir

These chambers all have in common that they are somewhat sensitive to hydraulic overloading, their design capacity ranging from 40 -80 m³/m².h.

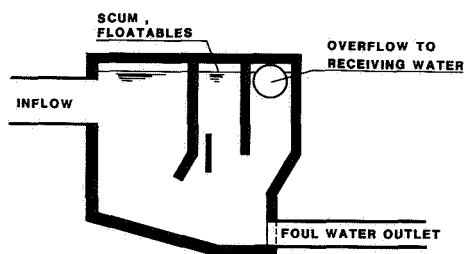


Figure 4 Shaft overflow

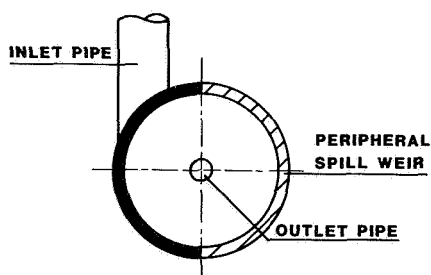


Figure 5 Plan of the vortex chamber overflow with peripheral spill

Although no full scale experimental data are available, laboratory experiments indicate removal efficiencies of 10 -35%, depending on the inflow velocity, particle form, size and weight.

The stilling pond is the earliest developed device and already finds practical application. It is, however, expected that the shaft overflow will render higher removal efficiencies. The stilling pond and the shaft type both have the disadvantage of a rather deep foul outlet pipe.

The vortex chamber with peripheral spill is the latest development, showing very promising test results, with a better performance than the stilling pond of comparable volume. (Balmforth et al., 1984).

The swirl regulator-concentrator, finally, consists of a circular shaped chamber with low tangential inflow and overflow over a central circular weir plate. The device operates on the basis of the gentle swirl action, which imparts liquid-solids separation (Figure 6) (Field et al., 1977). The dry weather flows or the low volume foul concentrate is diverted through a channel in the floor into a bottom orifice discharging to an interceptor sewer.

The swirl has successfully been applied and tested in the United States, Japan and Norway. In situ measurements during 11 overflow events rendered a removal efficiency of 51 - 82% of the BOD load and 33 - 82% of the suspended solids. (Field et al., 1977).

These data agree with Lygren's (1980) findings in Norway, showing removal efficiencies of 32 - 78% of the settleable solids at near design capacity.

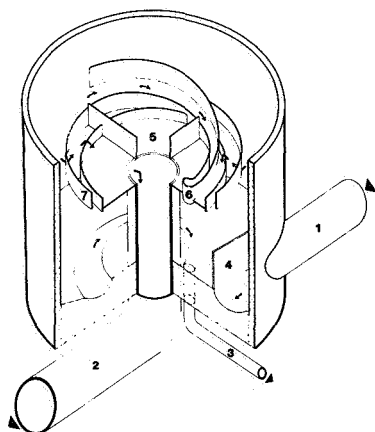


Figure 6

Isometric view of the swirl concentrator (Field et al., 1977)

- 1 = tangential inflow
- 2 = overflow
- 3 = foul sewer outlet
- 4 = flow deflector
- 5 = overflow weir and weir plate
- 6 = floatables trap

The "improved overflow chambers" as well as the swirl concentrator have modest space requirements, contain no moving parts, nor do they need any external energy supply. The necessary maintenance is therefore quite limited.

4 FEASIBILITY

The differences between traditional sewer design with extra in-line storage capacity and the alternative approach, where CSO-devices are applied, are illustrated by an example. The following features are considered: theoretical overflow frequencies, pollutional loads and cost estimation.

The arbitrary situation is a catchment of 50 ha impervious area with 5 overflow sites and 10.000 inhabitants.

An effective in-line storage capacity of 3500 m³ (7 mm) is assumed for the traditional design and 2500 m³ (5 mm) for the alternative, both with a pumping overcapacity of 350 m³/h (0.7 mm/h).

The calculations have been made by means of a sewerage pollution emission model (RAINY DAISY) using a sequence of 72 years of rainfall data (weather station De Bilt) as input.

A runoff coefficient of 1.0 is assumed, while the mixing, sedimentation and resuspension processes are simulated. Mineralization processes occurring within the sewer lines have not been taken into account, although they may be quite relevant. (Hogendoorn-Roozemon, 1984). The results emanating from the simulation model should therefore be used for comparison purposes only, and do not necessarily represent the actual pollution loads of such a catchment area, the more so as many factors are site specific.

First of all some features are considered of the 7 mm storage system versus the 5 mm system without CSO-devices.

Figure 7 shows the reduction in the overflow frequency by the 2 mm extra storage capacity. In both cases, however, a marked dispersion is discernible. This means that the f_0 value relevant to this situation can regularly be exceeded.

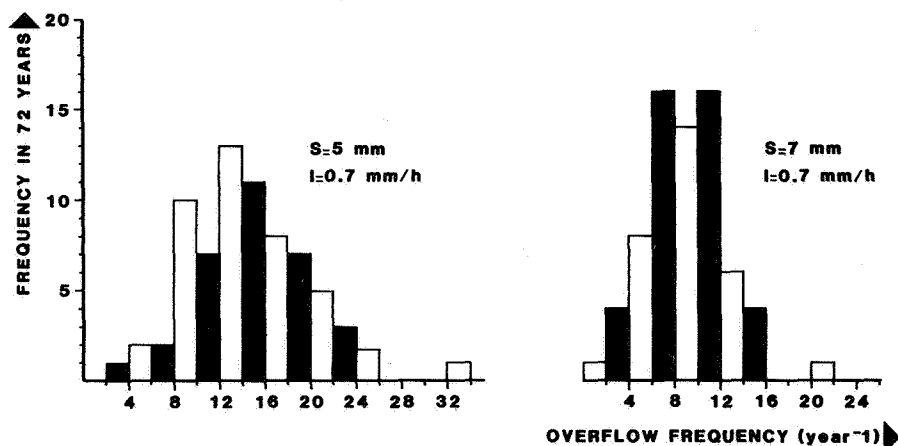


Figure 7 Frequency distribution of the annual overflow frequency for the indicated catchment area, with a storage volume (S) of 5 and 7 mm respectively. I = Interceptor capacity

In Figure 8 the individual overflow events are classified according to their pollutant load categories. It shows, that most of the extra overflow events occur in the lower pollution-load region. At the overflows with the higher pollution loads, the extra storage capacity has little effect, while in reality these overflows have the most pronounced environmental impacts.

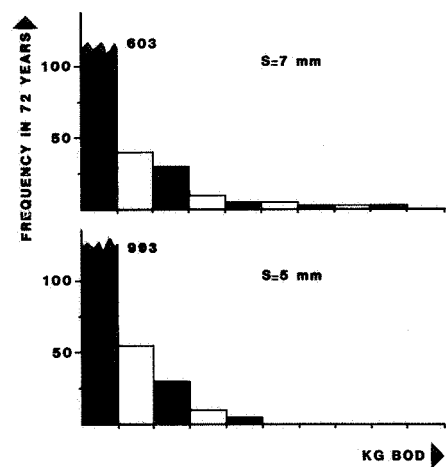


Figure 8
Frequency distribution of individual overflow BOD loads for the indicated catchment area, with a storage volume (S) of 7 and 5 mm respectively.

When a gradual increase in sedimentation with increasing sewer line diameter is assumed, the average annual pollutant load as a function of storage volume is presented in Figure 9.

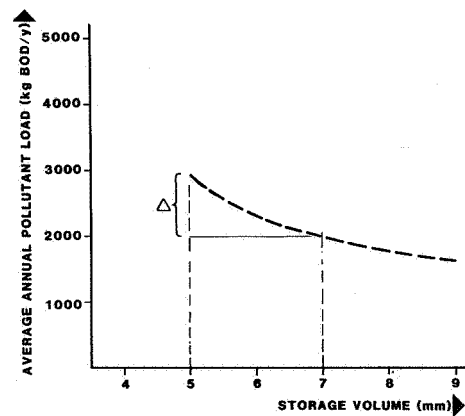


Figure 9
Average annual pollutant load as a function of storage volume

The difference in average annual BOD load between the 5 and 7 mm storage alternatives amounts to a mere 20%. When this "sedimentation" effect is omitted the difference in annual load amounts to a maximum of 40%. This value, analogous to the findings of Wiggers (1977), is to be considered as the fictitious maximum.

The overall effect of CSO-devices will depend on the design rainfall intensity and the design hydraulic loading of the device.

Assuming the values summarized in Table 2, rough estimates of the costs pertaining to the two systems have been made and are related to overall removal efficiencies.

Table 2 Assumptions on removal efficiencies applied in feasibility calculations

device	loading (m ³ /m ² .h)	removal efficiency (%) at design rainfall intensity	
		40 l/s.ha	60 l/s.ha
improved overflow chambers	40	25	35
	80	20	30
swirl concentrator	60	45	60
	150	30	45

Based on rough cost estimates the financial consequences of the traditional (extra 2 mm in-line storage capacity) and alternative (pollution reduction by means of CSO-devices) approach are compared in Figure 10a and b. Figure 10a refers to the "improved overflow chambers", while Figure 10b illustrates the situation for application of the swirl concentrator.

All the possible combinations in CSO-device application are to be found within the shaded area's: i.e. high and low hydraulic loading, high and low design rainfall intensity, number of overflow devices applied ranging from 1 - 5.

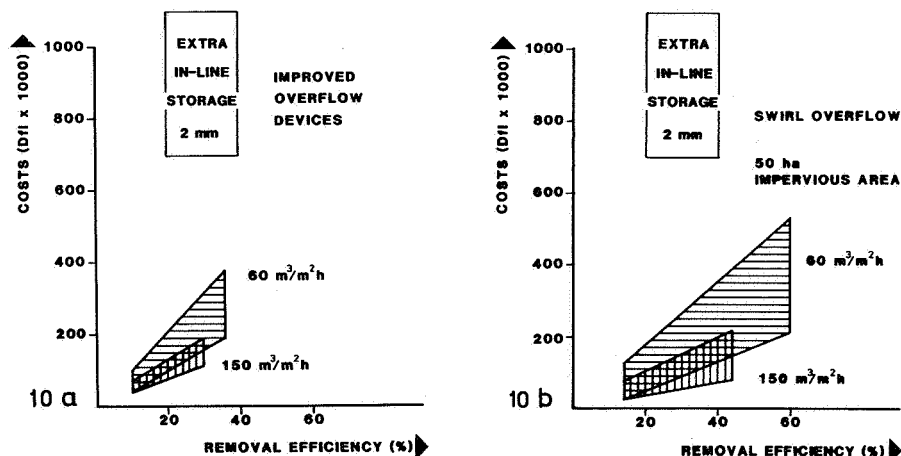


Figure 10 Cost estimation of indicated pollution abatement measures as a function of overall removal efficiency

Although Figure 10 is based on many assumptions and is only partly derived from actual measurements in the field, it distinctly illustrates the promising perspectives CSO-devices offer in the optimization of combined sewerage systems.

It is for this reason that an extensive research programme in the Netherlands has started on this subject. Two swirl overflow devices have already been built and a 3 year automated monitoring programme will soon commence at the location "Goes". In addition, a thorough study of several types of the "improved overflow chambers" is presently being carried out (VROM/NWRW,1985). Field studies are included for the near future.

5 CONCLUSIONS

Besides sedimentation tanks, the improved overflow chambers and the swirl concentrator promise to be handy tools in the optimization of combined sewerage system design in the Netherlands. They therefore have been selected for further research and field monitoring.

Application of these CSO-devices is estimated to be more cost effective than the realization of extra in-line storage capacity.

Additional major advantages are:

- limited residence time of waste water in the sewerage system
- effective use of the available storage volume
- reduced sedimentation within the sewer lines
- greater design flexibility and the possibility of selective source control.

It is believed that application of these CSO-devices enables a more differentiated approach in sewerage system design, in which the receiving surface water, with all its different functions and capacities, can play a more central role.

REFERENCES

- BALMFORTH, D.J., LEA, S.J. and SARGINSON, F.J., 1984. Development of a vortex storm sewage overflow with peripheral spill. In: Proceedings of 3rd International Conference on Urban Drainage. Ed: Balmer, P. et al., Göteborg.
- BERLAMONT, J. and SMITS, M., 1984. The overflow frequency as a criterion for the design of combined sewer systems. In: Proceedings of 3rd International Conference on Urban Drainage. Ed. Balmer, P. et al., Göteborg.
- BURROWS, R., ALI, K.H.M. and PATEMAN, D.R., 1984. A circular shaft overflow chamber for large interceptor schemes. In: Proceedings of 3rd International Conference on Urban Drainage. Ed: Balmer, P. et al., Göteborg.
- BROEKER, H.W., 1984. Impact of depositions on sewer operation. In: Proceedings of 3rd International Conference on Urban Drainage. Ed: Balmer, P. et al., Göteborg.
- COPPES, J.G.A., 1985. Vuiluitworpreducerende voorzieningen en afvoerregulerende middelen in rioolstelsels (in Dutch). H₂O No. 8.
- DREHWING, F.J., et al., 1979. Disinfection/treatment of combined sewer overflows. Cincinnati, U.S. Environmental Protection Agency, 1979. Report No. EPA-600/2-79-134.

FIELD, R.I., MOFFA, P.E. and MAC ARTHUR, D.A., 1977.

Treatability determinations for a prototype swirl combined sewer overflow regulator/solid-seperator.

Progress in Water Technology No. 6, p. 81.

FIELD, R., 1975.

The dual functioning swirl combined sewer overflow regulator and concentrator. Water Research (1975) No. 5/6, p. 507.

HALLIWELL, A.R. and SAUL, A.J., 1980. The use of hydraulic models to examine the performance of storm sewage overflows. Proceedings Institution Civil Engineers (1980) Part 2. June, p. 245.

HOGENDOORN-ROOZEMOND, A.S., 1984. Sedimentation, mineralization and resuspension of sludge in a sewerage system. In: Proceedings IAWPRC Conference Amsterdam, September 1984, Pergamon Press, New York.

LYGREN, E., 1980. Hvirveloverløp Avskilling av sedimenterbart materiale og flytestoffer i overløpsvann (in Norwegian). Norsk institut for vannforskning, Oslo.

NICOLL, E.H. and MC.GILLIVRAY, R., 1978. Report of the working party on Storm sewage (Scotland): A review. Water Pollution Control (1978) No. 2, p. 157.

SULLIVAN, R.H. et al., 1975. The helical bend combined sewer overflow regulator. Cincinnati, U.S. Environmental Protection Agency, 1975. Report No. EPA-600/2-75-062.

VROM/NWRW, 1985. Literature review on improved overflow devices (In Dutch). BKH consulting engineers, The Hague.

VROM/NWRW, 1984. Report on review and selection of emission reducing devices for sewer systems (in Dutch) Witteveen+Bos, consulting engineers.

WIGGERS, J.B.M., 1977. In what way is water pollution influenced by sewerage systems? In: Urbanization and water management. CHO/TNO Proceedings and informations no. 24.

WILSON, G.E. et al., 1978. Hydraulic design of the stormwater Teacup. Sacramento, Eutek systems, 1978.

THE EFFECT OF STORAGE SETTLING TANKS ON POLLUTION EMISSION FROM COMBINED SEWER SYSTEMS

D. ten Hove, M.A. de Ruiter and D. Vat

ABSTRACT

One of the possibilities for reducing pollution of surface waters by overflows from combined sewer systems is the use of storage settling tanks.

These tanks detain sewage sludge that is stirred up by heavy rainfall. In addition part of the sewage is retained in the tanks, pumped back to the sewer system with the settled sewage sludge when rainfall has stopped and discharged to the sewage treatment plant.

In The Netherlands, storage settling tanks have been built at major overflow points at several locations.

However, there has always been some doubt about the extent to which composition of overflows was influenced. Therefore two such tanks were monitored in the period 1983-1985. One was near the city of Kerkrade in a hilly area in southern Netherland, the other near Amersfoort, a town in a fairly flat area in the centre of the country.

From the measurements made, it appeared that the tanks gave a reduction of about 50% of the chemical oxygen demand (COD) of the overflow, particularly due to settlement, while the combined effect of storage and settlement gave an average reduction of the total pollution outfall, expressed in kg COD per overflow occurrence of 60-80%. Although it proved that the detention effect of the tanks decreased proportionately as the amount of precipitation increased, it was remarkable that the overflows with the largest water discharges were not identical with the

overflows which carried the greatest pollution load (kg COD).

1 INTRODUCTION

In The Netherlands more than 90% of the sewer systems are combined systems, i.e. wastewater and rainwater are discharged to the sewage treatment plant by the same sewers. The capacity of these plants is generally about four times the dry weather flow, equivalent to approx. 1 mm/h on a paved surface.

As The Netherlands is rather flat (comprising the deltas of the Rhine and the Maas) the sewer systems have a rather large storage capacity (approx. 5 to 8 mm). When there is more precipitation than can be stored or discharged to the sewage treatment plant, the excess water is discharged by overflow to surface waters.

The surface waters often consist of rather small ornamental ponds in urban areas.

In the past it was assumed that, because of dilution of the wastewater with rainwater, these overflows would have little or no deleterious effect on the quality of the receiving surface waters. However, for several decades it has been clear that there is a noticeable negative effect. In these small urban waters, overflows often cause fish mortality, nearly always due to lack of oxygen. Various investigations have made it clear that overflows are far more polluted than the dilution with rainwater lead to expect (ref. 1, 2).

The chief explanation of this phenomenon is the presence of a relatively large amount of sludge in the overflow, stirred up and discharged from the sewers by the sudden rising rate of flow. For some years, research on this phenomenon has been carried out in The Netherlands at various places (ref. 3). One of the possibilities to restrict the effect of overflow is to construct storage settling tanks near the overflow locations. The overflow water is passed through these tanks so that the sewage sludge is retained by settlement. In addition, the extra storage provided by the tanks reduces the frequency of overflow.

Following examples from abroad (ref. 2), more than a dozen storage settling tanks have been built at major overflow locations in The

Netherlands. However, there was some regarding the usefull effect on the pollution load of overflows. Therefore investigations were carried out from 1983 - 1985 at two of these tanks to determine the composition of the overflow water. One tank is in Kerkrade, a hilly area in the south of The Netherlands, and the other is situated near Amersfoort, in fairly flat land in the centre of the country.

2 MONITORING AT THE STORAGE SETTLING TANKS IN KERKRADE

2.1 Description of the Kerkrade storage settling tank.

Four storage settling tanks were built in the surroundings of Kerkrade in 1970, to protect a small reservoir in the Anselder stream, with a recreational character, from pollution by overflows from combined sewer systems in the locality. The area is rather hilly.

The investigation took place at the storage settling tank on the Vloedgraaf stream. The tanks were built to receive the overflow from a residential area with a population of approx. 14,000. There is practically no industry. The following table presents the main data on the area and the settling tank.

Table 1 Basic data, Kerkrade storage settling tank

paved surface residential area	69	ha
storage in sewer system	1	mm
storage in storage settling tank	3	mm
dry weather flow	0.3	mm/h
over-capacity	0.3	mm/h
total discharge to sewage treatment plant	0.6	mm/h

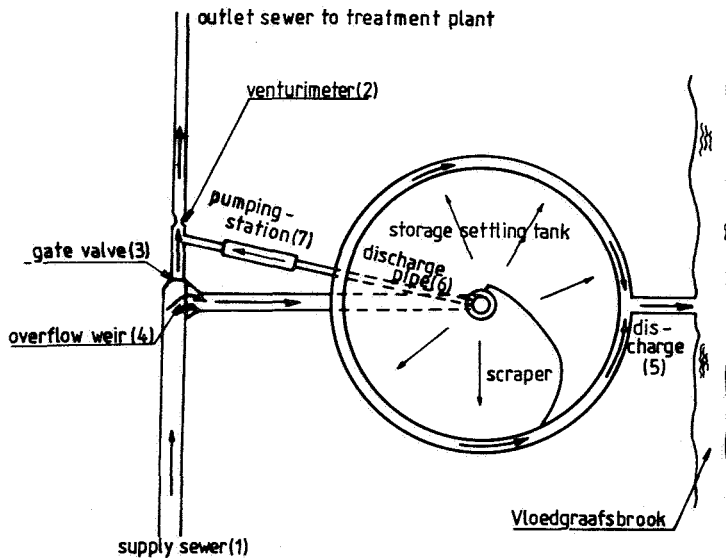


Figure 1 Storage settling tank Kerkrade

The tanks, which are all round open tanks of reinforced concrete, work as follows (see Figure 1):

Sewage brought in by the supply sewer (1) is continuously measured by a venturi meter (2). If this sewage flow is more than twice the dry weather flow (d.w.f.), which is the maximum capacity of the sewage treatment plant, the discharge is stopped at this value by an automatic electric gate valve (3). The sewage fills the supply sewer, flows over the crest of the overflow weir (4) and passes through the central supply pipe to the tank.

When the tank is filled up, the water also flows over the crest of an overflow weir at the outer wall of the tank into a drain (5). The drain (5) discharges the water to the Vloedgraaf brook.

Immediately the tank fills up, scrapers begin to operate, scraping the settled sludge to the centre where it is transported through a special pipe (6) and discharged to the outlet sewer (8) by a pumping station (7). The pumping station (7) pumps out the tank after rain has ceased. The sludge scrapers are fitted in a slowly

revolving bridge. This also houses a flushing installation which automatically comes into operation after the tank has been pumped out and cleans the tank floor with groundwater for about ten minutes to remove the remaining sewage sludge.

Thanks to this system of automatic sludge discharge and flushing with clean water, the maintenance of the tanks has proved to be minimal. Moreover, in spite of the fact that they are open tanks, situated only 100 m from a built-up area, there has not been a single complaint about odour in the 15 years that they have been in operation.

2.2 Description of the monitoring of the Kerkrade storage settling tank.

For about a year, the water flowing in at (4) and flowing out at (5) during the operation of the tank were sampled proportionate to the flow.

The sampling was done by a automatic sampling apparatus installed at (4) and (5). This apparatus was put into operation and adjusted by discharge monitoring with the aid of pressure gauges. The monitoring and sampling apparatus was designed by DHV Consulting Engineers, this company also carried out the investigation. As soon as the sampling apparatus began to operate, this was automatically signalled to the sewage treatment plant.

The samples, which were cooled at 4° C, were fetched within 8 hours, and transported to the Limburg Water Board at Roermond where they were analysed for chemical oxygen demand, etc.

2.3 Results of the monitoring of the Kerkrade storage settling tank

In the following table, the main results of the investigation are summarized.

Table 2 Results of the monitoring at the Kerkrade storage settling tank.

	calculated (1967)	measured (1984)
number of overflows from tank to surface waters (1/yr)	34	28
volume of overflow to surface waters (mm/yr)	120 mm/y	260 mm/y
average COD reduction due to settling (%)	50	51
average COD reduction due to storage and settling (%)	71	62
average COD-concentration (mg/l)	400	160

The far larger amount of water that overflowed in the investigated year in comparison to the calculated amount was remarkable (260 mm/y instead of 120 mm/y). This can probably be explained by the runoff from unpaved surfaces in this hilly area.

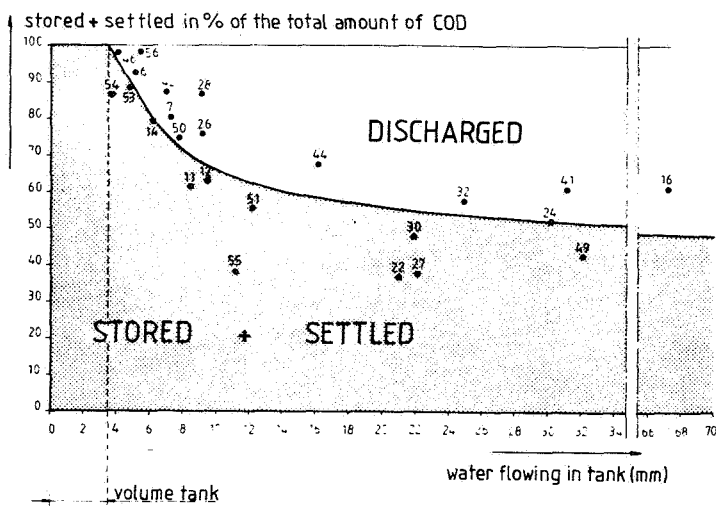


Figure 2 Effect of storage settling tank Kerkrade

When rainfall was heavy, the amount of settlement was generally - as was to be expected - lower than with a smaller water inlet. This can be seen from Figure 2, which shows per overflow occurrence (numbered from 1 to 35) what percentage of the total amount of COD brought into the tank with the incoming water remains there due to storage and settlement.

However, further study of the overflow data shows that the largest overflows were not identical in quantity to the overflows which brought the largest amounts of COD to the tank.

Probably a high precipitation intensity, which stirs up the sewage sludge, is more critical than the total amount of precipitation discharged.

The overflow water proved to be relatively less polluted than previously assumed on the basis of data from international literature; the average COD concentration of the water flowing into the tanks was 160 mg/l instead of the expected 400 mg/l. This can be explained by the dilution

because of the larger overflow volume.

3 MONITORING AT THE STORAGE SETTLING TANK IN AMERSFOORT

3.1 Description of the Amersfoort settling tank

In 1976, a storage settling tank was built for the overflow from a residential quarter with some 7,000 inhabitants in Amersfoort, a city in the centre of The Netherlands. There is no industry in the quarter, only a swimming hall.

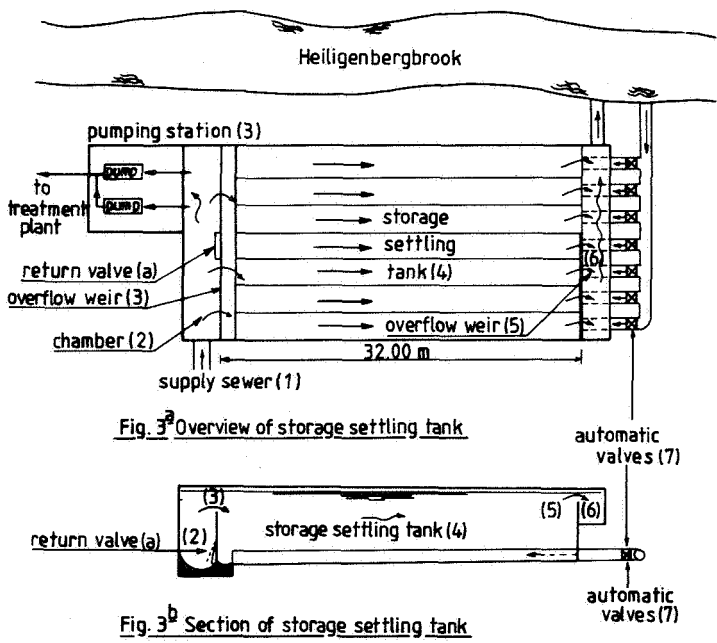


Figure 3 Storage settling tank Amersfoort

The following table presents the main data on the quarter and the settling tank.

Table 3 Basic data, Amersfoort storage settling tank

paved surface residential quarter	26 ha
storage in sewer system	7.3 mm
storage in storage settling tank	2.5 mm
dry weather flow	0.2 mm/h
over-capacity	0.8 mm/h
total discharge to sewage treatment plant	1 mm/h

The quarter is in a slightly sloping area. The major differences to the Kerkrade tank are the far greater storage in the sewer system (7.3 mm instead of 1 mm) and the greater capacity of the discharge to the sewage treatment plant (1.0 mm/h instead of 0.6 mm/h). The storage settling tank is a rectangular underground tank (see Figure 3). During dry weather, the wastewater passes the tank at (1) via the chamber (2) and is pumped up by the pumping station (3) and discharged to the sewage treatment plant. When the supply is greater than 1 mm/h, the chamber (2) fills and discharges the water over the crest of the overflow weir at (3) into the empty tank (4). When the tank is filled-up, the water also flows over the crest of the overflow weir (5) and is discharged through the chamber (6) to the surface water in the Heiligenberg brook. At the end of the rain shower, when the water supply is less than 1.0 mm/h, the water level in the chamber (2) drops and the water in the tank presses open a return valve at (a) (see Figure 3b). The tank is pumped out by the pumping station (3), after which automatic cylindric valves at (7) (see Figures 3a and 3b) are opened and surface water from the brook flows into the tank, flushing the settled sewage sludge to the pumping station (3), from which it is pumped to the sewage treatment plant.

The automatic flushing system has functioned completely satisfactorily in the nine years since the tank was built. Except for the pumping station (3), practically no maintenance has been needed.

3.2 Description of the monitoring of the Amersfoort storage settling tank.

From January 1983 to the end of 1984 the water flowing into the tank and the water discharged from the tank to the stream was monitored.

An automatic discharge-, registration and sampling apparatus was set up in front of the weir crest at (3) to monitor and sample the water flowing into the tank. A second apparatus was placed near the weir crest at (5) for the water flowing out. The discharge was automatically calculated from the registration of levels measured by pressure gauges. The sampling apparatus, designed by DHV Consulting Engineers who also carried out the investigation, took samples proportionate to the water volume passing the crest of the overflow weir. The samples were cooled. The samples were transported to the laboratory and analysed for COD etc. within six hours.

3.3 Results of monitoring the Amersfoort storage settling tank.

The results are summarized in the following table.

Table 4 Results of monitoring Amersfoort storage settling tank.

	calculated	measured
number of overflows from tank		
to surface waters (1/yr)	5.5	8.8
volume of overflow discharged		
into surface waters (mm/yr)	24	22.6
average COD decrease by settling (%)		47
average COD decrease by storage		
and settling (%)		81

In Amersfoort too, when rainfall was heavy the amount of settlement was lower than with smaller rainfalls. This appears from Figure 4, in which the percentage is given for the measured overflow-occurrences (numbered from 1 to 15) of the total amount of the COD (entering the tank with the water flow, that remains in the tank due to storage or settling.

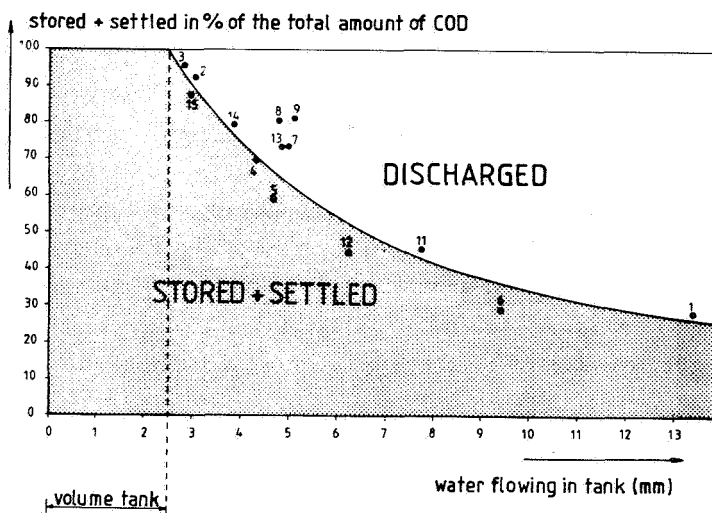


Figure 4 Effect of storage settling tank Amersfoort

A further analysis showed, more clearly than in Kerkrade, that the largest overflows are not identical in regard to the volume with the overflows that contain the largest amount of pollution, measured as COD. This is illustrated in Table 5 below, in which the five largest overflows in regard to water quantity and the five largest overflows in regard to COD flowing to the tank are presented next to each other. The overflow number is also given for each overflow occurrence. It appears from the table that the largest overflow in regard to water amounts, is not the largest in regard to pollution load.

Table 5 Largest amounts of water and largest pollution load per overflow.

sequence number	volume of water	overflow number	COD (kg)	overflow number
1	3,477	1 (C)	674	9 (C)
2	2,452	6 (C)	554	3 (W)
3	2,001	11 (C)	441	12 (W)
4	1,620	12 (W)	332	14 (W)
5	1,338	9 (C)	285	1 (C)

Moreover, the table above indicates whether the overflow took place during the cold season (October to April) or the warm season (April to October). It then appears that the overflows with the largest water quantities particularly occur in the cold season (C) four out of the five, whereas those with the largest pollution load per occurrence occur relatively more frequently in the warm season (W), three out of the five.

4 CONCLUSIONS

An investigation was carried out at two storage settling tanks in two different places in The Netherlands, built and operated under different circumstances. It appeared that due to the application of these tanks, the quantity of organic matter measured as COD that is discharged to surface waters is reduced by an average of approx. 50% by settling and of 60-80% by the combination of storage and settling.

The maintenance of these tanks proved to be very practically, because the settled sewage sludge was automatically flushed back to the sewer after rainfall had creased.

REFERENCES

1. GAMESON, A.L.H., and DAVIDSON, R.H., 1963, Storm-water Investigations at Northampton, Journal and Proceedings of the Institute of Sewage Purification
2 pp 105-135
2. DUNBAR, D.D. and HENRY, J.G.F. 1966, Pollution control measures for stormwaters and combined sewer overflows
Journal W.P.C.F. 38 : no 1 pp 9-26
3. VAT, D. 1985, Pollution emission from combined sewer systems; a full scale investigation programme, Proceedings technical meeting CHO/TNO, April 1985

WATER QUALITY EFFECTS IN SURFACE WATERS RECEIVING STORM WATER DISCHARGES

R.H. Aalderink and L. Lijklema

ABSTRACT

A general analysis of water quality impacts of storm water discharges is presented on the basis of the rates of the concomitant processes and the spatial scales of the effects. The practical use of this analysis is illustrated for the research of the water quality effects of storm water discharges from a combined sewer system in a retention pond in Loenen. Results are presented for the oxygen dynamics, the bacterial die-off and accumulation of heavy metals.

1 INTRODUCTION

The contribution of storm water discharges to the pollution of surface waters varies within wide ranges. The volume of discharges, the associated load of pollutants, the frequency of events and the characteristics of the composition of the storm water are obviously important factors. Sometimes storm waters from separate or combined sewerage systems are the main source of pollution, but often also effluents from waste water treatment plants or diffuse sources contribute to the loading of receiving waters. The proportions vary for the different compounds having environmental impacts. The intensity of the effects is furthermore a function of the properties of the receiving water such as volume, flow, hydraulic detention time, water depth and other morphological factors, and of the chemical composition.

At last also the environmental conditions (temperature, wind, irradiation) interact with the inputs and affect the transformations within the water system, causing a sometimes wide variation of impacts over the seasons. The numerous possible combinations of factors contributing to the water quality preclude the development of a simple recipe for the analyses of water quality impacts of storm water discharges. Yet a certain generalization can be made on the basis of temporal and spatial scales for the assessment of the effects, both in terms of measuring strategy and in terms of description and/or modelling

1.1 Spatial and temporal scales

Water quality impacts may cover a wide range of time scales which are controlled by the rates of the concomitant processes in the surface water. High rate processes such as acute fish kills by toxic matter or chemical oxidation-reduction reactions are directly related to the dynamics of the discharge: intensity and duration of the discharged load, the local mixing conditions and the process rates control the extent of the effects in space and time. The event will be highly dynamic; an assessment requires a detailed and frequent sampling or continuous recording taking in account strong local gradients. On the other hand the high rates will lead to a speedy decline of the effect and especially in stagnant receiving waters only a restricted zone will be affected. Water quality effects related to processes with moderate rates, for instance BOD-oxygen consumption or bacterial die-off will affect a wider area but for the assessment and description a less detailed analysis in time and space is required. For slow changes such as accumulation of nutrients or heavy metals in sediments the immediate effect of a storm water discharge is of minor importance; the total (average) annual loading may be sufficient to judge the consequences of a storm water outlet. In such situations the evaluation may be limited to an assessment of the percentage of the contribution from storm water discharges to the overall load. However, for substances involved in processes of a higher rate, the immediate effect may be the more serious.

For instance one oxygen depletion due to a storm water event in a period of one or two years may upset a receiving water more than the continuous discharge of an effluent, although the latter contributes over 98% of the total annual BOD-load.

Figure 1 presents a qualitative picture of the relation between the rates of the processes indicated and the spatial scale of the effects.

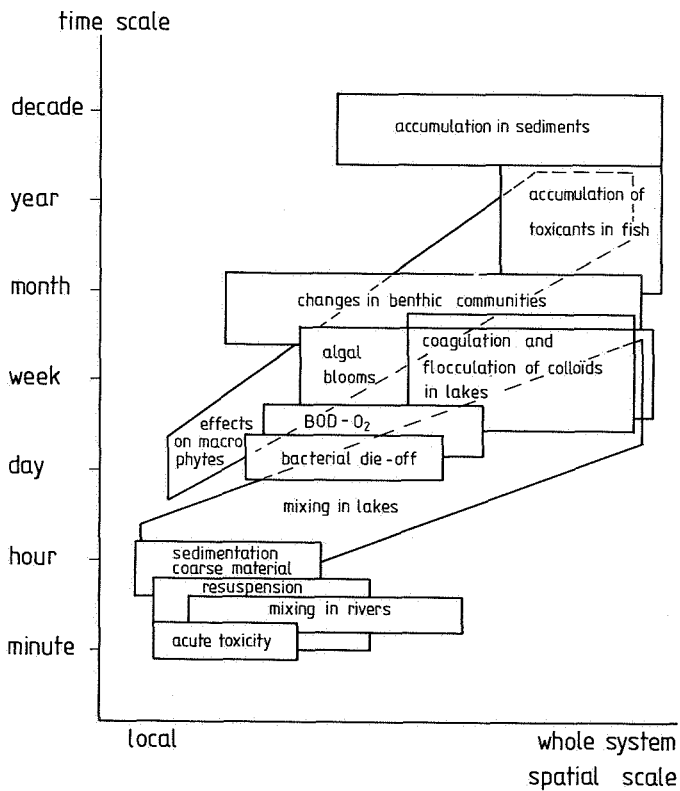


Figure 1 Relation between the rates of processes and the spatial scale of the effects

Due to its nature the effects in running waters will tend to afflict the water quality over longer distances as the slug of pollutants runs down the stream, but -when considering a fixed point in space- the duration will be shorter in comparison with stagnant water.

For processes with intermediate rate constants the analysis may take into account the probability of overlapping events. Especially in a flat country with a high storage capacity in the sewer systems a fairly long period (about 10 hrs) is required to pump the system until it is empty. Hence storms anticipated by a short dry period have a higher probability to cause an overflow and the probability of interference of events is enhanced in comparison with complete independency.

The practical conclusion for our research of the water quality effects of storm water discharges from a combined sewer system, including the measuring strategy, will be discussed in a subsequent section.

1.2 Deterministic or statistical approach?

The methodologies applied in the assessment of water quality effects of storm water discharged into receiving waters fall into two broad categories.

Statistical methods are based on the principle that the properties of the storm events (distribution of intensity, duration, volume and dry periods, including correlations between the properties) are somehow preserved during the transformation from pluviogram through the flow in the sewerage system and finally in the overflowing quantity. These methods have the advantage that when once the shape of the distribution of the discharged load has been assessed, the frequency of exceeding a water quality standard can be obtained readily from predicted water quality distributions round an average effect. The main problems with this methodology are the correct treatment of the correlations, the availability of reliable and sufficient data on water quality of storm water and a correct prediction of the transformation from precipitation into flow and overflow.

Deterministic methods use time series of rainfall and model the sequence of events in the sewer system, at the discharge point and in the receiving water. Calibration can be based on measured quantities during one or more events. Calculations based on records of rainfall will allow a statistical treatment of the resulting water quality data and yield frequency distributions of the violation of standards. These methods give insight in the interrelations between the processes and can handle more detail. Nevertheless the calculations often suffer from a poor representation of quality, especially regarding the sewage and the discharged overflow. Also the procedures are laborious and consume much computer time.

2 THE PRESENT RESEARCH

The overall aim of the study is to compare both methodologies, particularly with respect to the relation between discharge and water quality in the receiving water. Only deterministic aspects will be discussed here with emphasis on the sampling strategy as related to spatial and temporal scales, and some results and their interpretation in simple models. The research is a part of a large study by the NWRW (National Working Group on Sewerage and Water Quality). In the framework of this study and in cooperation with STORA (Foundation for Applied Research in Wastewater Treatment) a large data set on discharges and rainfall has been collected by DHV-Consultants at several locations in the Netherlands. One of the locations is the retention pond in Loenen, where we measured the water quality effects and studied individual processes in 1984. In 3.5 years 55 discharges have been sampled, of which 17 occurred in 1984. Of these events 11 were sampled and recorded more or less completely.

The village of Loenen has a combined sewerage system, draining about 16 ha of impermeable surface in a slightly sloping area. The average frequency of discharges as observed is 15 per year. Figure 2 shows the dimensions of the pond and the location of the inflow of storm water and outflow over a weir and the sampling stations. The pond receives percolating groundwater at a rate of about 10 cm/day; the average depth is 1m.

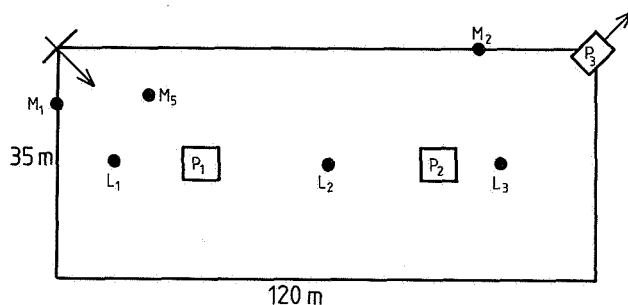


Figure 2 Dimensions of the retention pond and location of the sampling locations

2.1 Sampling strategy

The input of the overflowing mixture of sewage and storm water was measured by an automatic sampling and registration system which results in a continuous flow measurement and individual samples for the analysis of water quality variables with a frequency proportional with the flow. Although this sampling strategy had been established long before the water quality in the receiving water became the subject of study, the data proved to be adequate for this study also. The total load could be calculated, but for detailed studies, e.g. of the mixing process in the receiving water, the variations of flow and concentrations within the event could be assessed as well. The variables measured included COD, BOD, Kjeldahl-N, P-total and suspended solids in the samples as such and after settling. Total heavy metals were also measured. The quality as measured within the receiving water and the frequency of sampling is summarized in table 1. This shows clearly that variables related to fast processes are measured either continuously or with a high frequency.

Table 1 Water quality sampling programme in the pond

Sampling locations: see figure 1

Variable	Sampling Frequency	Location	Process/Effect studied
Temperature	Continuous	P1	Background information
Conductivity	Continuous	P1 P2 P3	Mixing
O ₂	Continuous	P1 P2 P3	O ₂ dynamics, especially after events
BOD, COD, NH ₄ -N, Kjeldahl-N,	0.5-4 hours after event	P1 P2 P3	
ortho-P, total-P	weekly	P3	mass balance
indicator micro-organisms	daily after events	P1 P2 P3	survival after event
suspended solids	0.5-4 hours after event	P1 P2 P3	sedimentation
<hr/>			
N, P, heavy metals ash free dry weight and in sediments + depth of sediment layer	monthly	L1 L2 L3	Accumulation of sediment and of nutrients and heavy metals
<hr/>			
macrophytes	twice a year	. . .	Effects upon
macrofauna	monthly	M1 M2 M3	ecosystem
Phyto- and Zoo-plankton	day after event; biweekly	L1 L2 L3	
Epiphytic diatoms	three month	- - -	

The conductivity proved to be a useful indicator for mixing because there was a difference in conductivity of the seepage water and the overflow. Occasional detailed studies were made of processes such as sedimentation with sediment traps, reaeration with a floating cap purged with nitrogen, seepage and sediment oxygen demand. Results will be presented for the oxygen dynamics, the bacterial die-off and accumulation of heavy metals.

The interpretation of the observed oxygen concentration after an overflow has been based on the basis of ideal mixing in the pond. This is certainly not justified for the initial stages of the event. Three periods can be discerned:

- the period during the overflow, generally in the order of hours and occasionally up to half a day. In this stage the level in the pond will rise, resulting in an increased outflow at the outlet weir. The flow pattern in the pond is controlled mainly by the impulse of the discharge and to a lesser extent by the flow regime induced by the outlet.
- after the end of the overflow, the level in the pond will gradually return to its equilibrium in which the infiltration rate is equal to the discharge over the weir. In this period the flow pattern will be characterised by the flow at the outlet and by wind. The length of this period is in the range of a few hours up to half a day.
- the third stage is the equilibrium condition in which wind controls the mixing conditions and possibly density differences between the infiltrating cold groundwater and the water in the pond.

During the two initial stages the assumption of ideal mixing is not correct because net advective transport plays a dominant role. However, the length of this period -less than one day- is much shorter than the time scale of the processes controlling the oxygen concentration; see figure 3c. During the third period no major differences in concentrations of variables at the measuring points were observed, so that at least for these sites ideal mixing could be assumed. The sensors were located at mid-depth in the pond.

The equations for the BOD and O_2 concentration in this period can be written as:

$$\frac{dL}{dt} = -K_d L - \frac{1}{\tau} L + \frac{1}{\tau} L_{in} \quad (1)$$

$$\frac{dO_2}{dt} = -K_d L - \frac{1}{\tau} O_2 - \frac{SOD}{D} + \frac{K_L}{D} (O_{2s} - O_2) \quad (2)$$

in which

L	the ultimate BOD concentration	g/m ³
L _{in}	the ultimate BOD in the infiltrating water	g/m ³
O ₂	the actual oxygen concentration	g/m ³
O _{2s}	the saturation concentration of oxygen	g/m ³
K _d	the deoxygenation rate constant	g/m ³
τ	the hydraulic detention time	d
K _L	the mass transfer coefficient for oxygen at the air-water interface	m/d
SOD	the sediment oxygen demand	g/m ² .d
D	depth of the water	m

The processes considered are deoxygenation by decay of organic matter (BOD), outflow of BOD and oxygen, input of BOD by the seepage, consumption of oxygen by the sediments and reaeration. BOD removal due to settling is considered to be minimal in the third period, the infiltration water is anoxic and primary production can be neglected for the period under consideration. The solution of equations 1 and 2 requires assessment of the correct parameters and initial conditions.

The hydraulic residence has been estimated from the observed conductivity at location P1, see figure 3a. The infiltrating water has a higher conductivity than the mixture of storm water and pond water after an event. The best-fitting value for τ obtained in this way is 2.8 day, which is much shorter than the value obtained from the average infiltration rate (τ ≈ 9 days). Two reasons can at least partly explain the discrepancy:

- the infiltration rate varies, the observed range being 3-16 cm/day and during heavy rainfall periods the higher ground water table will enhance the infiltration rate
- the well-mixed assumption is not valid; this may be due to the presence of dead zones, e.g. in the corners of the reservoir or in areas with growth of Potamogeton natans. Also small temperature differences between the (cold) infiltration water and the pond water may cause inhomogeneity during calm weather conditions.

The BOD decay rate constant has been assessed from the data of figure 3b by application of equation 1. The value obtained in this way is K_d = 0.22 day⁻¹ is normal for the conditions.

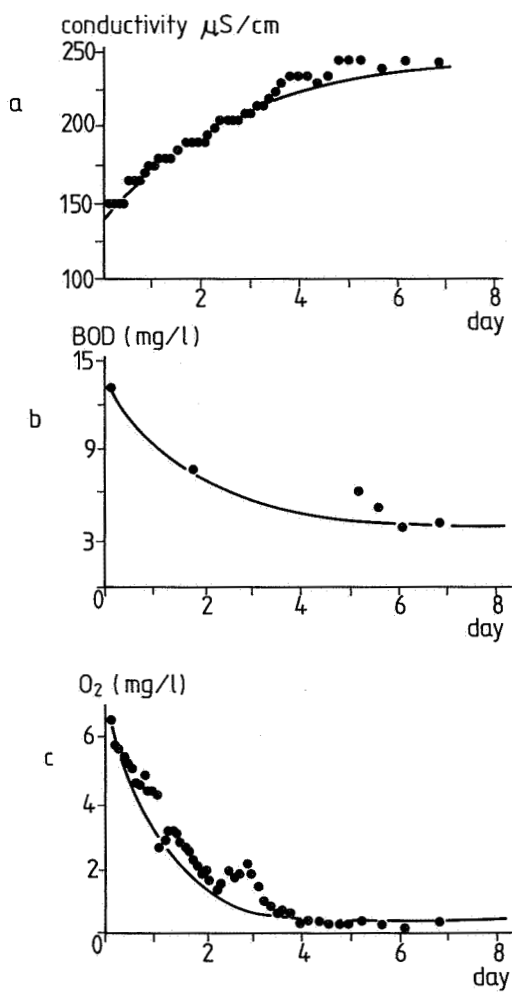


Figure 3 Measured and simulated conductivity (a), BOD concentration (b) and oxygen concentration (c) at location P1

Mass transfer coefficient measurements for oxygen across the air-water interface yielded an average value of 0.185 m/day.

Although direct estimates of the sediment oxygen demand were available from column experiments in the laboratory, these values cannot be considered as representative due to the absence of infiltration in these columns. Hence the SOD was estimated by application of equation 2 for stationary conditions, $\frac{dO_2}{dt} = 0$, in a period without primary production (no daily variation in O_2 concentration), or:

$$-\frac{1}{\tau} O_2 - \frac{SOD}{D} - K_d L + \frac{K_L}{D} (O_{2s} - O_2) = 0$$

With $\tau = 2.8$ day, $D = 1$, $K_d = 0.22 \text{ d}^{-1}$, $L = 4 \text{ g/m}^3$, $K_L = 0.185 \text{ m/d}$, $O_{2s} = 11 \text{ g/m}^3$ and $O_2 = 1 \text{ g/m}^3$ this results in a $SOD = 0.6 \text{ g/m}^2 \cdot \text{d}$. This value is the result of balancing terms which are subject to experimental error and therefore contains all model and parameter uncertainties.

Figure 3c compares measured and simulated O_2 concentrations for an event in october 1984 in which 4200 m^3 stormwater was discharged. After 4-5 days a minimum O_2 concentration is reached, the low reaeration rate than causes a slow increase to the 1 g/m^3 equilibrium value. The initial high value of about 6 g/m^3 is due to the oxygen content of the discharged storm water.

In the spring and summer the primary production prevents the occurrence of prolonged anoxic periods.

4 BACTERIOLOGICAL QUALITY

The objective of this part of the investigations was to assess the magnitude and duration of bacterial pollution after an overflow. The normal sampling frequency was biweekly, but after an event sampling was intensified to a maximum of one sample per day. The water was analysed for

- Fecal streptococci
- E. coli
- Total coli
- Fecal coli

Figure 4 shows data on *E. coli*, other indicator organisms showed a similar behaviour.

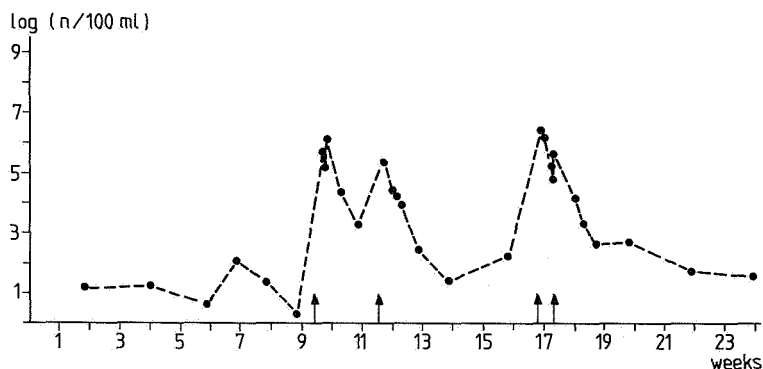


Figure 4 Observed number of *E. coli* at location P1

Overflows cause a sharp increase in numbers, followed by a gradual decrease caused by

- die-off and possibly grazing
- sedimentation
- dilution by the infiltrating water

The overall removal rate as gathered from the observed concentrations is about 1 day^{-1} , which means that it takes about 2 weeks before the concentration is reduced from 10^7 micro organisms per 100 ml initially to the background concentration of 10-100 micro organisms per 100 ml.

The contribution of dilution to the overall removal rate is about 0.35 day^{-1} . The numbers in sediment traps were occasionally also estimated and compared with the concentrations in the overlying water. From this an estimate of the sedimentation rate can be obtained. This value is biased by growth and/or die-off in the sediment trap and subject to considerable experimental error. Most results however indicate that after the two first phases of an event which are controlled by the hydraulic phenomena, settling is of minor importance and generally less than 0.1 day^{-1} . Hence the die-off rate is around 0.6 day^{-1} at the environmental conditions (Temp $\approx 14^\circ\text{C}$, pH = 6.5-7.0). This is similar to literature data (Gordon et al., 1978).

It is well known that the survival of pathogens and indicator organisms in sediments is better than in aerobic water. Therefore we also measured the numbers in the sediment. The average numbers in 100 ml sediment are:

Fecal coli 10^6
 Total coli $3 \cdot 10^5$
 Fecal streptococci $1 \cdot 10^4$

These numbers are considerably higher on a volumetric basis than for the water phase. Figure 5 shows the change in numbers per gram of sediment (top 10 cm of sediment sampled) in a period with two successive events.

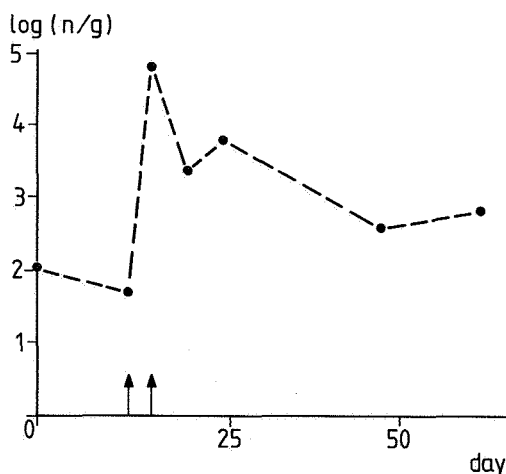


Figure 5 Observed number of *E. coli* in sediment at location L₁ (top 10 cm of sediment samples)

Sedimentation apparently induces a marked increase in numbers; the subsequent decrease apparently is much lower than in the water phase. As the scatter in data will be high due to local variations in numbers, poor reproducibility of the sediment sampling procedures in the field and experimental errors in the separation of bacteria and sediment particles, the survival has been measured also in the laboratory. This would reduce the first two errors mentioned.

Figure 6 shows the results; the die-off rate is similar as in figure 5 and of the order of 0.1 day^{-1} . Hence an enhanced initial concentration of $10^5/\text{gram}$ will be reduced to the average observed background concentration of $10^2/\text{gram}$ in about 2 months. As the recurrence of overflow events is of the order of 1-2 months this explains the continuously enhanced background concentration in the sediment.

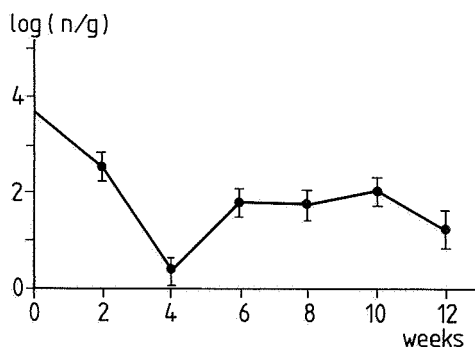


Figure 6 Survival of *E. coli* in sediment measured in the laboratory

A general conclusion is that sediments around storm water sewer outfalls will be contaminated more or less permanently with pathogens and indicator organisms and so are a potential source of infection.

5 ACCUMULATION OF HEAVY METALS

This is a subject where the total loading of the system is of more interest than initial mixing and dispersion. The analysis of sediment around the outfall will give an indication of the accumulation of heavy metals and of the distribution of the contaminated sediment in space. For this purpose in 1984 the thickness of the sludge overlying the original sandy sediment has been measured monthly and the content of three representative heavy metals was estimated. Figure 7 shows the results.

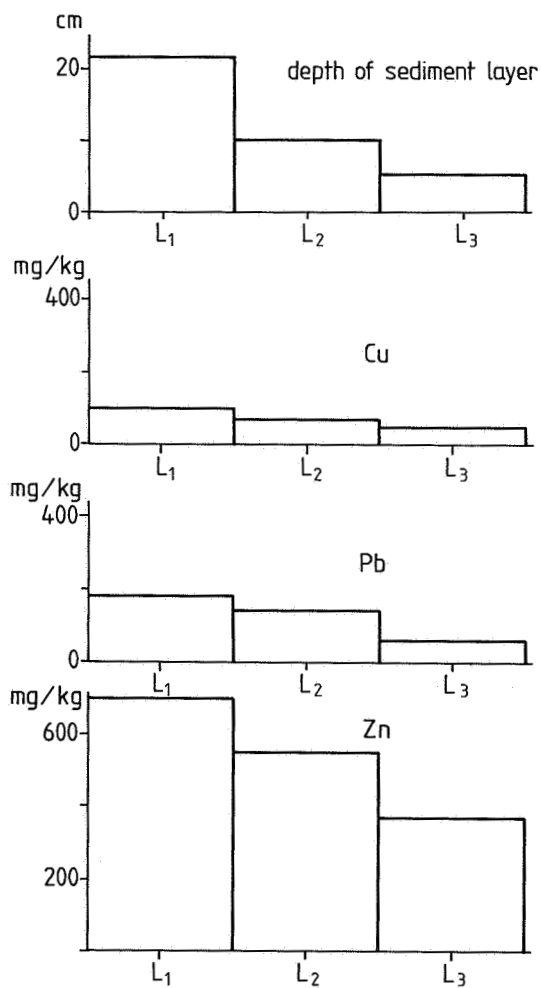


Figure 7 Depth of sediment layer and content of Cu, Pb and Zn at locations L1, L2 and L3

These results indicate clearly gradients in thickness and concentrations. The stagnant character of the receiving water is responsible for this behaviour but details of the mechanisms resulting in the existing condition cannot be elucidated due to lack of detailed information (resuspension, mobilisation of heavy metals, concentration related to particle size distribution and settling rates as related to size etc.).

The rough estimate of the total quantity of heavy metals from the available data (figure 6) has been compared with the analytical data of the 55 overflows sampled during three years. Combined with the age of the pond, 15 years, extrapolation of the total loading of the pond is possible. Assuming that the heavy metals are mainly associated with the suspended solids and taking into account that about 60% of the particles are retained in the pond, the expected accumulation can be calculated; see table II.

Table II Accumulation and input of heavy metals

Metal	Total load (kg)	Loading of sediment (60%), kg	Observed (kg)	%
Cu	35	21	8	38
Pb	52	31	15	48
Zn	107	64	60	94

The table suggests that the estimate for Zn is fairly good, but that Cu and Pb either are mobilised or that the input based on the more recent data has been overestimated due to a gradual increase in loading. Mobilization is not a probable explanation because Zn is generally much more mobile than Cu and Pb. A gradual increase of Pb contamination in the watershed due to traffic is certainly a reasonable argument but especially for Cu the trend in the percentages cannot be explained.

The level of contamination of the sludge is well below the standards in the Netherlands for disposal of chemical residues or for application of sewage sludge on farm land.

REFERENCES

- GORDON, R.W. and FLIERMANS, C.B., 1978. Survival and viability of *Escherichia coli* in a thermal altered reservoir. *Water Res.* 12: 343-352.

WATER MANAGEMENT PLANNING IN URBAN AREAS

J.J. de Graeff

1 INTRODUCTION

The first subject that will be dealt with is water management planning in urban areas. It should be briefly recalled what form water management planning will take in the future (this was the subject of the CHO-conference in Lunteren in spring 1984). Secondly, the role of towns and cities in this respect will be considered and the part played by the municipal authorities in connection with planning will be briefly described. Thirdly and finally, in slightly more detail attention will be paid to the respective tasks of the municipal authorities and the water-boards with regard to local water management and not just planning. All of these are subjects which will be treated in the other papers, so here only an introduction will be given.

2 WATER MANAGEMENT PLANNING IN URBAN AREAS

First of all, the future water management planning will be considered. The report of the last CHO-meeting (Proceedings and Information No 32) outlined how planning will be organized in future. In a nutshell, it may be said that the various existing planning arrangements will be integrated with each other. On the one hand this is desirable as a means of streamlining and simplifying regulations as part of the government's policy of deregulation, while on the other hand it is

based on the consideration that water quality and water quantity are interrelated matters, as are the management of surface waters and groundwater. The existing Policy Document on Water Management and Indicative Multi-year Programme for Water will be amalgamated into a single Policy Document on Water Management, indicating the main lines of the policy to be pursued by central government on water management in the Netherlands.

At the provincial level, the water quality plans, the groundwater plans and the envisaged planning in relation to water quantity are similarly to be incorporated into a single provincial water management plan, containing the main outlines of provincial water management policy. The main instrument of provincial planning is the designation of waters to perform particular functions. In this way it is intended that the broad outlines of policy on water management, physical planning and the environment will be harmonized.

At the management level it is envisaged that management plans should be drawn up which indicate in concrete terms what the manager intends to do in order to perform his duties and how his plans fit in with the other areas of policy mentioned before. Provision has not been made for complete integration of management plans, because different authorities may be responsible for administering qualitative and quantitative aspects. Of course, it is desirable that management plans should be drawn up in close consultation between the authorities concerned, and de facto integration of management plans would be a natural and desirable goal to aim at in cases where a single authority is vested with the various responsibilities. Integration of management plans has therefore to become a statutory obligation in the case of waters administered by the central government.

The planning structure described, is to be laid down in the Bill on the Water Management currently before the Lower-House of Parliament; in March this year the Minister of Transport and Public Works tabled a draft amendment for this purpose during the

advisory stage of the Bill. This will be followed by the memorandum in reply, accompanied by the finalized amendment, after which the Parliamentary procedure will run its course. Assuming that the amendment becomes law, the existing planning regulations will be deleted from the Pollution of Surface Waters Act, the Groundwater Act and the current Water Management Bill.

In the description so far no mention was made of the relationship between water management plans at central, provincial and management level or between water management plans and environmental policy plans. It may be sufficient to observe on the one hand that a certain hierarchy exists among the water management plans of the various authorities; planners at the more local level must always bear in mind the constraints which spring from the policy laid down by the more central authority or authorities. On the other hand it is also important that each authority should remain in a position to perform its own responsibilities. The fact that a hierarchy exists does not mean that planners at a higher level of government are free to usurp the powers and responsibilities of those at lower levels. This is an area of potential conflict concerning on which strong views are held by some of my colleagues. For the moment this point will not be taken: it is hoped that the Minister will make rapid progress in the further processing of the Bill. In the country at large, the various authorities whom it concerns are already implementing the plans for the new planning structure. On the one hand, this process could be promoted by providing a formal basis for it, while on the other hand unnecessary and undesirable discrepancies in form, content and procedures could thus be avoided.

Municipal authorities.

The first point to be made about the role of the municipal authorities in the new planning structure is that the municipalities have a lot of work to do in connection with water management, particularly in urban areas. The other articles of the present Report will, no doubt, provide a clear account of the matter. Here only a few examples will be given. When expansion plans are implemented, land must be drained so as to achieve and maintain an appropriate

groundwater level. Among the infrastructural components which may be required are pumps, weirs, water courses and reservoirs to drain water into. It is essential to have an adequate drainage system in order to manage water quality properly, while another precondition is flow and adequate renewal of urban waters. The functions of such waters are intimately linked with the planning of urban land-use.

The last example while on this subject is a recent publication by the Civil Engineering Planning Department of the Delft University of Technology concerning the problem of high water levels in an urban area. It is entitled: "When it rains we hold our breath" - a striking illustration of the horizontal links which, as I have indicated, exist in the work of public administration. This being so, it is essential that the municipal authority should be closely involved in the planning of water control at management level. The question is how this involvement should be expressed. In this context firstly reference can be made to section 12 of the Water Management Bill as it would read if the Minister's draft amendment is adopted.

Subsection 1 lays down that whoever is responsible for administering surface waters may draw up a management plan for them. For example, municipal authorities may do so, particularly in cases where part of an urban area is withdrawn from the authority of a water-board because it is separated from a polder and incorporated into the territory of a municipality for the purpose of water management. If another governmental body is responsible for administering quantity or quality, the power to draw up management plans for the area concerned is vested in the body concerned. In virtually every case this is the water-board; at least, this is true if we disregard Groningen, Friesland and Utrecht, where the provincial authorities are responsible for water quality.

Thus in many places it is the water-board which is responsible for "planning", while in some towns both the water-board and the municipal authorities will draw up management plans. Moreover, there is a trend for more environmental powers (and in some cases powers in the sphere of water management) to be vested in the municipal authorities in the four largest cities, so that it is not

inconceivable that in some cases the municipal authorities may be the only ones to draw up management plans (on an integrated basis or otherwise). In view of this situation, two points will be raised, which will form the third and final section of this paper.

3 TASKS OF MUNICIPALITIES AND WATER-BOARDS

Firstly, regardless of who is responsible for drawing up water management plans, close cooperation between the municipal authorities and the water-board will be essential in urban areas as a means of enabling water management and physical planning to be properly coordinated. This is very important, since on the one hand water management in urban areas is closely bound up with areas of policy for which the municipal authorities are responsible and on the other hand it is evident that urban water management does not stop at the boundaries of the municipality but is indissolubly linked to water management throughout the area for which the water-board is responsible.

In this connection it is gratifying that the Act on Physical Planning is being reviewed, and that it will be stipulated in the Act that the burgomaster and aldermen are obliged to consult the appropriate water-boards when preparing a structure plan or development plan. Conversely, water-boards are required to consult the appropriate municipal authorities when preparing management plans. The model management plans drawn up by the Association of Water-Boards likewise recommend this. The Bill on the Water Management does not lay down any such obligation, but leaves it to Provincial Councils to regulate the preparation of management plans. It is to be hoped that Provincial Councils will bear this point in mind when drawing up their regulations; however, the question arises why the Bill does not provide for the matter to be regulated by Order in Council. If this would be done, the arrangements for water management would be similar to those for physical planning, and unnecessary discrepancies between procedures would be avoided.

As a last point on the subject of cooperation it is worthwhile to recall the Guidelines on the Administration of Water Defences and Water Quantity Management in Relation to the Construction of Buildings by Municipalities, known for short as "the Guidelines with the long name", which were published recently by the Association of Water-Boards. These Guidelines, which Mr Dragt will deal with in his paper contain a number of useful rules for relations between municipal authorities and water-boards where there are urban expansion plans.

The subject of the Guidelines, which are based on the assumption that the urban area is within a water-board area, conducts to the very last subject. The point is whether it is really desirable that water management planning in urban areas should be partly the responsibility of the municipal authorities and partly that of the water-board. There are objections to this on practical grounds, to name but one aspect. It means that both must acquire expertise and employ manpower in the same field. This is a point which has also been taken into account in the deregulation operation to which was referred earlier. Such fragmentation of responsibility is contrary to the policy of streamlining and clarifying structures. But the issue goes deeper than this. The essential point is how water management should be organized in urban areas; responsibility for planning is merely a consequence of this. This has been a topical issue for some time in connection with discussions on the pros and cons of withdrawing responsibility for parts of polders from water-boards and vesting it in municipal authorities. The recent plans of North Holland Provincial Council to make the municipality of Amsterdam entirely responsible for water quality management in Amsterdam, and more generally the current ideas about transferring powers to the municipal authorities in the four largest cities in the Netherlands (Amsterdam, Rotterdam, The Hague and Utrecht) contribute towards this topicality. The subject is a delicate one, nonetheless a few words must be said about it.

In the author's view the question of what authority should be responsible for water management as a matter of principle is

fundamental. The government gave a clear answer to it in its 1977 policy document "Towards a new management of water-boards, in which it noted that responsibility for water management and water defences should be determined in accordance with natural geographical boundaries on grounds of efficiency and cohesion of policy. It further noted that certain groups are particularly concerned with water management. It was concluded that under these circumstances the water-board was best suited to take responsibility for local and regional water management. By making water-boards responsible for water management one ensures that the authority and the subject for which it is responsible are suited to each other and that those parties who are most concerned, and who themselves bear the financial costs, participate in administration.

However, the policy document went on to say that it was important to provide safeguards for the desired interaction between the special-purpose body and the general administrative body and to adapt the structure of the water-board itself, inter alia to take account of the fact that a wider range of interests need to be considered under the heading of water management nowadays than used to be the case. The general tenor of this thesis and conclusion was accepted by the Lower-House of Parliament in 1978. The present government continued previous policy in the 1984 Policy Document on Water Management which was mentioned.

The ideas quoted remain as valid as ever, and apply in their entirety to relations between municipal authorities and water-boards. The main decision is whether administrative districts for the management of water should follow municipal boundaries or the geographical boundaries suggested by the routes of water courses and the boundaries of other surface waters. The latter is preferable. The problems caused by ignoring municipal boundaries, which owe their existence to administrative considerations, can more readily be solved by cooperation than can the problems which would be caused by ignoring natural geographical boundaries. To cut across geographical boundaries would also be contrary to the aim of integrating water management, which the current Bill on the

Water Management is intended to achieve. It must be realized that any violation of the physical geographical unit which the territory of a water-board forms can only be to the detriment of water management.

It may be worth mentioning that a working party comprising representatives of central and provincial government and water-boards was set up a few years ago to draft a Water-board Bill, and that it presented the results of its labours last year. In doing so, it created the prospect that it may be possible to establish safeguards for a modern system of water-boards, inter alia by means of the planned administrative amplification of the water-board.

In the author's opinion there are therefore good reasons, and perhaps stronger reasons than previously, to make water-boards responsible for water management - and hence planning - within urban areas. In order for water management to be as effective as possible, this should at any rate be the basic principle, although not an absolute decree: if there are strong reasons or very special circumstances militating against adherence to the principle, it should be possible to depart from it. This indeed is the line that has so far been followed, at least as far as responsibility for water quality is concerned. The reason why it has been followed is, apart from the already mentioned administrative and practical grounds, in order to bring greater financial resources to bear on the problem than a municipal authority would be able to.

In the sphere of water quantity management there exists a certain reluctance to transfer authority over any part of the urban territory from water-boards to municipal authorities. In some cases the process is now even being reserved. The main reason for this trend is that it is felt that a water-boards which is structured, organized and equipped along modern lines should be capable of performing its duties properly in urban areas just as elsewhere. In this connection the Minister of Transport and Public Works can be quoted in answering to questions from the Upper-House of Parliament during recent considerations of the 1985 budget:

"It follows from the views expressed in the Government Policy

"Document 'Towards a new water management of water-boards'
"that water-boards should also be responsible for water management
"in urban areas. Nonetheless, circumstances may make it
"impossible to exclude the possibility of withdrawing responsibility
"for water management from a water-boards in a specific case if
"this is virtually the only way of providing a just and
"effective solution to a conflict of interests. My guiding
"principle here is that the authority which wishes to assume
"responsibility for water management instead of a water-board
"should demonstrate the problems to which a continuation of the
"normal situation would give rise."

It would accordingly be a retrograde step to depart from this line,
for example in the context of policy on the four large cities.
To avoid any misunderstanding it must be emphasized that the
author is not at all in favour of polarization between water-boards
and municipal authorities. On the contrary, although he believes
in the dictum 'Every man to his trade' at the same time he should
like to advocate an effective cooperation between those of different
callings.

Water management in urban areas should in principle be organized along
uniform lines, in the form which unambiguous government statements
have repeatedly referred to. There exists an interlocking nature of
the responsibilities of municipal authorities and water-boards and
an effective cooperation is required in order to ensure that these
responsibilities are carried out properly.

MANAGEMENT OF WATER IN THE URBAN ECOSYSTEM

H.W. Kroes

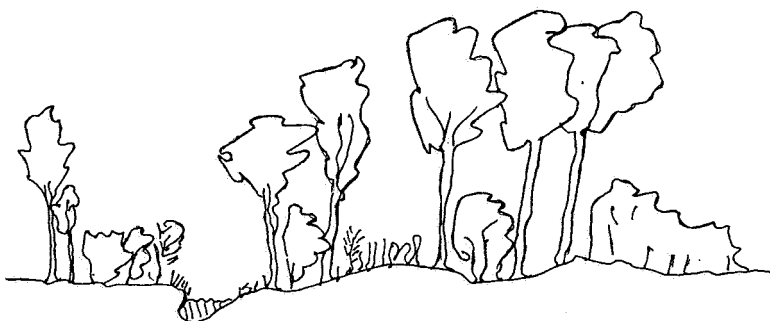
ABSTRACT

In the Netherlands, competent authorities for water management are the provinces or specific water authorities: municipal authorities play no role, except for the management of sewer systems. I would like to propose that, at least in the case of cities with more than 100.000 inhabitants, municipal authorities should become competent in the integrated management of the urban water system (surface water, ground water, sewer system). The main argument is the necessity of managing the city as an urban ecosystem, i.e. managing a system of interconnected living and non-living components and flows of energy and matter.

1 CHARACTERISTICS OF ECOSYSTEMS

I would like to explain my position by first examining what an ecosystem is and afterwards describing how a city can be thought of as an ecosystem, an urban ecosystem.

An ecosystem is an interacting whole of living organisms and a non-living environment. This is a rather abstract definition and we may get a better grasp of the ecosystem concept by having a look at the various ways an ecosystem may be



A. NATURAL ECOSYSTEM

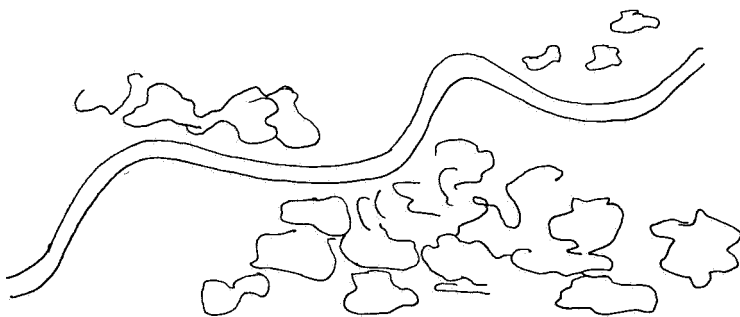


B. AGRO-ECOSYSTEM

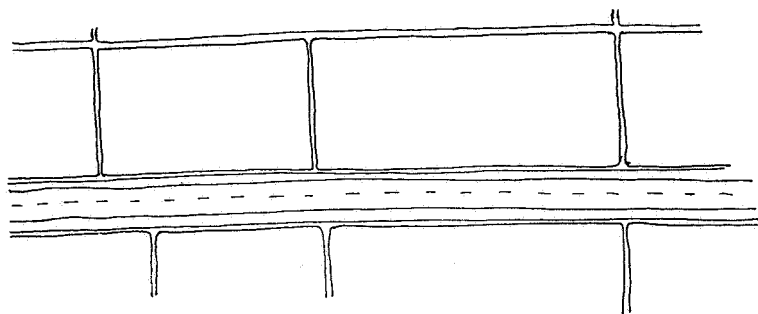


C. URBAN ECOSYSTEM

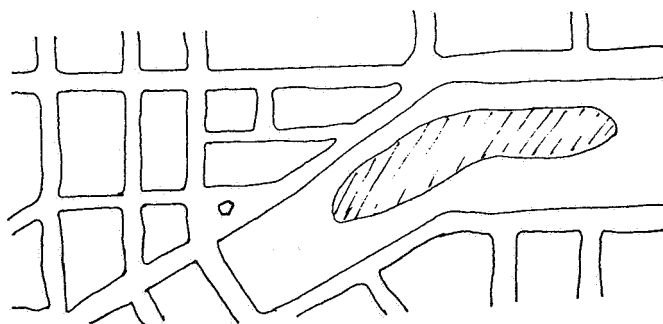
FIGURE 1 "SIDE VIEW" REPRESENTATION OF ECOSYSTEMS



A. NATURAL ECOSYSTEM



B. AGRO-ECOSYSTEM



C. URBAN ECOSYSTEM

FIGURE 2 "MAP" REPRESENTATION OF ECOSYSTEMS

pictured (i.e. represented on paper). There are several, rather different ways of representing one ecosystem. I will give an impression of the following: (1) side view, (2) map (= top view), (3) diagrams of components and flow of energy and matter, (4) lists of words and numbers, i.e. descriptions of species composition, values of physico-chemical parameters etc. The order is one of increasing abstractness.

(1) The "side view" representation

A drawing or photograph of the side view of an ecosystem is the most realistic representation in the sense that it resembles most what we see with our own eyes when we look at a real ecosystem: we see organisms and abiotic structures, e.g. trees, hills (Figure 1A). In the case of aquatic ecosystems the view is an imaginary one - as it were through the walls of an aquarium or through the eyes of a diver.

One should realise that, although this representation is the least abstract of the various ways of picturing the ecosystem, it is already very abstract in comparison with the real ecosystem. For example, it does not show the complete diversity of organisms, but only part of it (mostly the macroscopically visible species and those of high density). Neither does it show the many motions, the flows in and out of the ecosystem, and the processes of growth and decomposition.

(2) The "map" representation

The map is a drawing or an aerial photograph giving a top view of an area. In most cases the scale is such that no individual organism is visible (Figure 2A). In that sense the map is more abstract than the side view. The map, however, accentuates an important characteristic of the ecosystem concept: the spatial boundaries that can be thought to enclose the system, especially in the horizontal plane.

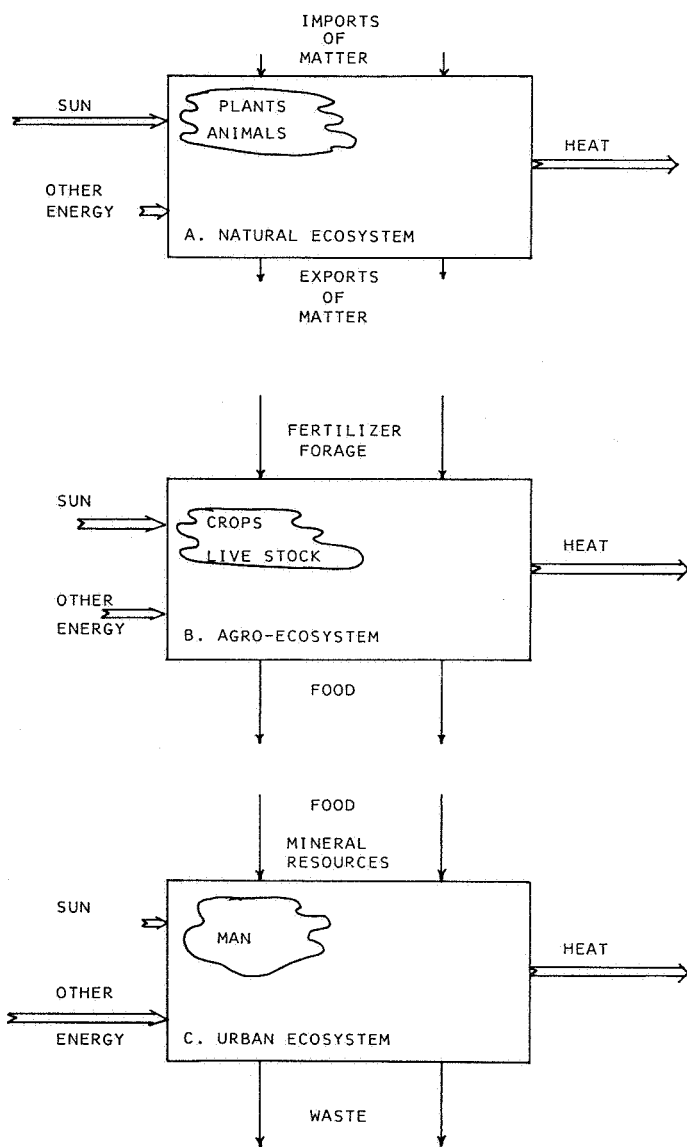


FIGURE 3 "DIAGRAM" REPRESENTATION OF ECOSYSTEMS

Another important characteristic is that one can have ecosystems on different scales, in other words that the area of an ecosystem can vary from a few square meters to the size of a continent or even to the size of the surface of the earth (the biosphere). It depends on the choice one makes for studying ecological processes; it is convention in any case to choose "natural" boundaries, e.g. shorelines, vegetation limits, etc.

(3) The "diagram" representation

The diagram representation is useful to distinguish the main components of an ecosystem (the "primary niches": see Kroes, 1977), i.e. the living organisms: producers, consumers and decomposers, and the abiotic substrates: inorganic matter and organic matter.

Diagrams are further used to show the flows of energy and/or matter that enter or leave an area and the flows of energy and/or matter between the components of the ecosystem. They can also be used to show other relations between the components, for example competition between species. Traditional energy-matter flow diagrams (e.g. in Anderson, 1981) illustrate the important fact that an ecosystem almost always depends on energy import from the outside (in most cases in the form of sunlight) and that transport of matter is much more an internal affair. Ecosystems can in principle maintain a closed circuit of the flow of matter, although there is a great variation in the ratio of import and export versus internal flow of matter between real ecosystems.

Figure 3A is an example of a diagram with main energy and matter flows for natural ecosystems.

(4) The "words_and_numbers" representation

Lists of species (or other taxa) of organisms are a common

A. NATURAL ECOSYSTEM

Potamogeton pectinatus	dissolved oxygen
Nuphar lutea	(yearly minimum) : 5 mg/liter
Butomus umbellatus	
Limnaea stagnalis	temperature
Cloeon dipterum	(yearly maximum) : 23 °C
Rana esculenta	
Gasterosteus aculeatus	

B. AGRO-ECOSYSTEM

Secale cereale	soil pH	: 6.2
Claviceps purpurea		
Centaurea cyanus	phosphate (P) in	
Hordeum murinum	solution in soil	
Micromys minutus	moisture	: 0.4 mg/liter
Passer domesticus		

C. URBAN ECOSYSTEM

Homo sapiens	carbon monoxyde	: 45 mg/m ³
Canis familiaris		
Felis domestica	sulphur dioxyde	: 0.8 mg/m ³
Sturnus vulgaris		
Musca domestica		
Poa annua		
Calendula officinalis		

FIGURE 4 "WORDS AND NUMBERS" REPRESENTATION
OF ECOSYSTEMS

way of describing the living part of the ecosystem. So are lists of values of physico-chemical parameters characterizing abiotic aspects of a specific ecosystem. They are, of all four ways of description, the most abstract because several levels of translation are necessary to bridge the gap between reality and representation. They are, by virtue of that abstraction, also the most exact of the four, an aspect that is dangerous because it may convey the impression of reality, while, as I said above, this way of representation is very abstract. Figure 4A gives an example.

At this point we could try to reformulate the definition of an ecosystem given in the beginning of this chapter (see Kroes, 1977, for details):

"An ecosystem is a part of the surface of the earth that is represented in such a way that the following aspects are prominent:

- populations of animals, plants, and lower organisms are components of the system ("niches"),
- there are three distinct substrates: soil, water and air,
- the system has horizontal boundaries: it is limited in space,
- important processes in the system are the flows of energy and matter, both internal and cross-boundary,
- these processes are kept going by the populations of living organisms with sunlight as an ultimate source of energy; populations can be grouped in producers, consumers and decomposers,
- species composition is very sensitive to external influences such as changes in energy supply, climate and

the flow of matter into or out of the system; these external influences themselves, however, are less sensitive to changes in species composition of the ecosystem,

- there is no central steering mechanism."

It should be noted that the ecosystem concept is very flexible, especially in an spatial sense. A small pond or a large ocean may both be described in ecosystem terms. The principle of description is the same in both cases. This is also one of the reasons that the ecosystem concept is very convenient for application in the case of man-made systems.

2 ECOSYSTEMS AND HUMAN SOCIETY

So far I have described characteristics of natural ecosystems, in which man plays no role, or at most a minor one. In the past 100.000 years, however, large parts of the world have been radically altered by human activities. At present, large areas are being used for agricultural purposes (i.e. for the production of human food, clothing, etc.) and smaller, but still extensive, areas for housing, industry and transport.

In spite of the fact that areas used for human activities differ markedly from natural areas, they can be described as ecosystems. The advantages of doing this are (1) a facilitation of comparison between natural and "man-made" areas and (2) the inherent possibilities that the ecosystem model provides for incorporating components and processes (organisms, non-living substrates, flows of matter and energy, etc.) into a set of representations as given above.

Therefore, it is convenient to distinguish three broad categories of ecosystems in the present world:

- natural ecosystems
- agro-ecosystems
- urban ecosystems.

Figs. 1-4 give a comparative general impression of the characteristics of the three types of ecosystem.

In the side view representation (Figure 1) the monocultural aspect of the agro-ecosystem and the rectilinear aspect of the urban ecosystem versus the natural ecosystem are evident. Important to note is the addition in the urban ecosystem of two substrates: buildings and roads ("roads" is to be taken in a broad sense: not only conventional roads of land and water, but also all kinds of pipelines, cables, etc. for the transport of matter, energy and information). So the urban ecosystem has five substrates in stead of three: water, air, soil, buildings and roads.

The map representation (Figure 2) shows the rectilinear features of both the agro- and the urban ecosystem, as compared to the natural ecosystem. The drawings also show the importance of roads and buildings as components of the urban ecosystem.

The diagrams of matter and energy flow (Figure 3) illustrate several characteristics of urban and agro-ecosystems versus natural ecosystems.

First, the sun as an all-important direct source of energy is for the urban ecosystem replaced by fossil fuel, nuclear energy, and, to a lesser extent, wind and water energy. Although many agro-ecosystems depend on light energy from the sun, mechanization has been made possible through the use of fossil fuel and specialization has led to the creation of agro-ecosystems that use light energy only in an indirect way (live stock breeding on the basis of imported forage).

Second, urban and agro-ecosystems are more open to imports and exports of matter. The interdependence of both types of ecosystems as regards food production and consumption is evident, but it is only gradually different from flows of matter between natural ecosystems (e.g. a forest and a river system). New in comparison with natural ecosystems is the large flow of mineral resources into the urban (and to a certain extent also the agro-) ecosystem. In modern society, this flow of minerals sustains the production of housing, utility building, industrial plants, roads, transportation and communication means, clothing, and all kinds of consumer goods. In a qualitative and quantitative sense it is completely different from the flow of matter between natural ecosystems. One of it's, still unsolved, consequences is the production of large amounts of solid and liquid waste; society tries to get rid of it by dumping it into natural ecosystems (e.g. rivers), thereby uncontrollably degrading those systems.

Third, species composition is different from that in natural ecosystems. Agro-ecosystems are managed intentionally to produce one or a few kinds of product (= species). Urban ecosystems are built intentionally to house people (= Homo sapiens). This is not to say that those types of ecosystems do not harbour many other species. In some agro-ecosystems a great diversity of "wild" species is found and urban ecosystems often harbour a large variety of species (e.g. birds, trees, shrubs, exotics: Müller, 1980). There is, however, a continuing trend of diminishing diversity in those systems, especially in the agro-ecosystems, as a consequence of the use of pesticides, erosion, upscaling, etc. See also the "words and numbers" representation (Figure 4).

3 MANAGEMENT OF TERRESTRIAL AND AQUATIC ECOSYSTEMS

Now I would like to return for a moment to the category of

natural ecosystems. They can be divided into two types: aquatic and terrestrial ecosystems. The substrate combination water - soil - air (aquatic system) produces a set of abiotic conditions quite different from that of combination soil - air (terrestrial system). As a consequence, not only species composition, but also flows of energy and matter are clearly different. In practice, therefore, terrestrial ecology and aquatic ecology (the last-named including limnology and marine ecology) are separate disciplines. Of course, there are many interactions between land and water, and integrated approaches, in science as well as in environmental policy, are often necessary. Nevertheless, it is reasonably possible in environmental management to manage water (aquatic ecosystems) and land (terrestrial ecosystems) as separate entities.

This goes for natural ecosystems. In the Netherlands, about 65 per cent of the area is used for agriculture, 25 per cent is forest, nature reserves, recreational area, and water, and 10 per cent is urban area (Centraal Bureau voor de Statistiek, 1980). Therefore, management of aquatic ecosystems is only partly a matter of managing natural ecosystems.

A large part of water management takes place in the context of agro-ecosystems and urban ecosystems. In agro-ecosystems, surface water (ditches, brooks, canals, polder lakes, etc.) may - in the same way as in natural areas - be managed as separate systems, because water systems are sufficiently independent from the land systems (managed by individual farmers). In the urban ecosystem, however, the simple distinction between land and water is replaced by a much more complicated combination of land, water, air, buildings and roads, interwoven on a small scale. Therefore, it is impractical to split up management of the urban ecosystem into management of the separate components.

Management of surface waters in the Netherlands presently is

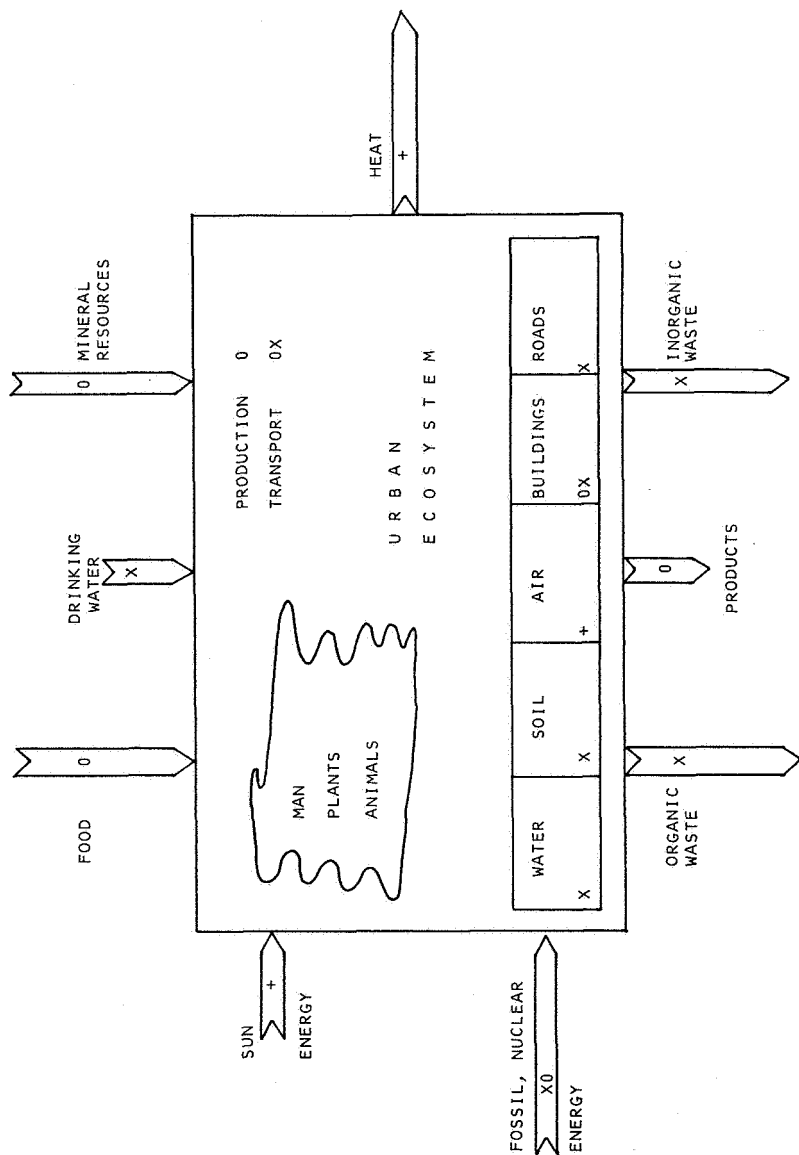


FIGURE 5 DIAGRAM OF THE URBAN ECOSYSTEM
 EXPLANATION OF SYMBOLS: X = MANAGEMENT BY PUBLIC AUTHORITIES,
 0 = MANAGEMENT BY PRIVATE PERSONS OR ORGANIZATIONS
 + = NO MANAGEMENT

the competence of the national government for the large waters of national importance, and of provinces or water authorities (mostly one to five per province) for all other waters. Municipal authorities have no legal competence in water management. This means that urban waters in a city are managed by a province or a water authority, while all the other components of the urban ecosystem are managed by the municipality. As I announced in the abstract, I propose that urban water, at least in the large cities, be managed by municipal authorities. The reason is that, in the urban ecosystem, a partition of water and land ecosystems (readily possible in natural and in agro-ecosystems) is not efficient in urban ecosystems. In the following, I will try to explain more fully why this is so.

4 MANAGEMENT OF URBAN ECOSYSTEMS

Figure 5 shows a diagram like that in Figure 3, but in more detail. It contains the main substrates, living components and the flows of energy and matter. Moreover, the elements of the diagram are marked, indicating whether (in the Netherlands) processes or components are managed by public authorities or by private persons or organizations, or not managed at all.

It is seen that, in the urban ecosystem, public authorities manage the substrates water, soil, roads, buildings (partly), the transport of gas, drinking water, electricity, information (partly), as well as a large part of the disposal of solid and liquid waste (sewerage, waste water treatment). Private persons or organizations manage the imports and distribution of food and mineral resources, the part of the energy flow that consists of oil and coal, and one of the substrates (partly): buildings. Some flows and ecosystem components are not managed in the conventional sense: inflow of sunlight, outflow of heat, the substrate air.

It is clear from this inventory that management of surface water in the city is part of the management of the substrates in the urban ecosystem. It is the only component, however, that is not managed by the municipal authority, but by a province or a water authority. There are such close links of urban surface waters with other components of the urban ecosystem that they should be managed as part of that system, i.e. by the municipality. There are more reasons to do so:

- urban waters are mostly man-made: the aquatic system is of a generalized kind, requiring no specific ecological expertise or protection of specific species or communities,
- urban waters are close to the places where we live and work: daily contact of most people with water is urban water, therefore it should be managed by a public authority as near as possible to the public,
- urban waters have many links with the sewerage system (direct discharges, overflows): the authority managing the sewer system (the municipality) should manage the urban waters too,
- in practice, the legally competent authorities (provinces, water authorities) have, in the past 15 years, showed little interest in management of these waters, perhaps because other tasks took priority.

I believe that most cities of 100.000 inhabitants or more, consisting of typical urban ecosystems, are sufficiently equipped to manage urban surface waters. Specifically, this would imply the following tasks and powers:

- planning of integrated management of sewerage systems and surface waters (possibly in the still wider frame-

- work of environmental planning for the city),
- maintenance and cleaning of sewers and surface waters,
 - water quality surveying, monitoring, and reporting,
 - the power (in the framework of the Water pollution control act) to grant permits for discharges of waste water into sewers or urban surface waters,
 - operating a closed financial system for abating the costs of management of sewerage systems and surface waters on the basis of levies in the framework of the Water pollution control act, replacing current levies by provinces and water authorities on the one hand, and municipal sewerage tax on the other,
 - possibly - in some cases - operating sewage treatment plants.

REFERENCES

- ANDERSON, J.M., 1981. Ecology for environmental sciences: biosphere, ecosystems and man. Edward Arnold (Publishers), London, 175 pp. ISBN 0 7131 2814 3.
- CENTRAAL BUREAU VOOR DE STATISTIEK, 1980. Algemene milieu-statistiek 1977-1978. Staatsuitgeverij, 's-Gravenhage, 276 pp.
- KROES, H.W., 1977. The niche structure of ecosystems. J. theor. biol. 65: 317-326.
- MÜLLER, P., 1980, Biogeographie. Verlag Eugen Ulmer, Stuttgart, 414 pp. ISBN 3 8001 2459 9.

ODUM, E.P., 1971. Fundamentals of ecology. W.B. Saunders
Company, Philadelphia, 574 pp. ISBN 0 7216 6941 7.

MUNICIPAL INVOLVEMENT IN WATER IN AN URBAN ENVIRONMENT

L.J. Pieterse

1 INTRODUCTION

Municipalities, in this context seen as centres of population of some considerable size with a reasonably developed level of activity, had for many centuries an important relationship with the surface water within their boundaries. This relationship was originally focussed on the qualitative aspect and was governed mainly by economic considerations. Most industries in these centres were dependent on water as a raw material, both for the food industry (breweries) and for the textile industry, to name just a couple. It is hardly surprising that this situation frequently led to conflicts between the towns and the water-boards in which they were situated. A very interesting piece of history with just such a background was the conflict between the Delfland water-board and the town of Delft, that lasted for about as long as the Eighty Years War. You can read a summary of this in the catalogue of "Het Prinsenhof" museum in Delft which accompanies the exhibition on 'The town of Delft, 1572-1667'. The conflict was, it seems, finally settled by the 'States of Holland' in 1632, after five court sessions proceedings between the two disputing factions. So conflicts in water management are not a recent feature, and neither are shortages of good quality water confined to the present day. This in spite of the fact that the interests and criteria are now quite different and that the systems are of a different structure.

Two developments have changed the situation :

Firstly: the necessity to remove the surface water from the centres of population which grew rapidly in the nineteenth century, and where heavy pollution occurred. That could be done by two methods: either by flushing it away or by catching it in separate systems: the sewerage systems. Both methods were employed, separately and together.

Secondly: the need of towns to expand, even in areas which, on account of their low-lying altitude, seemed less suitable. This led to raising the ground or to lowering the level of the surface water. Here too, both alternatives were employed, either separately or combined.

Anyone examining the relationship of the municipalities with water in an urban environment cannot, therefore, restrict himself to surface water, but must also devote attention to the sewerage system and should study the effects of urban expansion.

2 THE FUNCTION OF SEWERAGE

The primary role, in my opinion, of municipalities in water quality management has rather receded into the background. The same thing has happened as with many other aspects of public health. Once a threat has been apparently beaten, it is treated as if it has been eliminated for all time, even though constant vigilance is called for.

What is this primary role then?

In short: The potentially dangerous waste water produced in centres of population should either be rendered non-hazardous or should be removed from these centres. The latter method is the customary one.

The sewerage system, as it has developed during the last hundred years, has proved itself the ideal means of doing this. However, in serving

this function, the rule observed until recently was that waste water should be got rid of as quickly as possible, once it has been collected. No hold-ups were allowed. This freed the surface water in the centres of population from a significant source of pollution and delivered the population from a threat. This was not, incidentally done without large investments. But the problem was in this way transferred to the watercourses outside the population centres. The problems that arose there have been dominating water management in recent decades.

Finally, post-war industrialization has added numerous potential hazards to waste water, which are not so much related to hygiene. Here as well, the same criterion applies: the rapid removal of waste water from centres of population is, in my opinion, of the highest priority.

3 THE CONSTRUCTION OF NEW URBAN AREAS AND THE REDEVELOPMENT OF OLD AREAS

While originally only those areas were built upon which were sufficiently high, the centres of population expanded during the last century into low-lying waterlogged areas which in their original state were considered as being unsuitable. In the process, water levels were altered and canals were excavated to control the water regime. In general, this expansion resulted in a loss of surface water area. The elimination of this water area was accompanied by the loss of their disposal function, while in addition the nature of the surface also changed. In order to solve this problem the discharge of rainwater was combined with the discharge of waste water. These were, however, minor effects. By far the greatest effect of this sort of expansion was the change in ground-water level.

In addition to urban expansion, urban redevelopment was also undertaken. A large number of water courses (whose primary urban function had meanwhile been lost) were quite simply filled in,

usually for the purpose of constructing arterial roads. The area of surface water within municipal boundaries has decreased sharply in the course of time, especially as a result of it losing its traditional functions and due to the threatening character it possessed.

Only in recent years has an appreciation of its value developed and has an aesthetic and recreational function been assigned to water in towns.

4 PRIORITIES

This has been a very short summary of the relationship which in the course of time has developed between municipalities and water management.

What are the priorities of municipalities?

As regards the sewerage function, the removal as rapidly - and safely as possible of waste water from the population centres. In addition, the most effective management of the sewerage system, by which nuisance from water for the population is avoided. Lastly the maintenance, aimed at preventing and removing stoppages in the waste water courses.

As to the urban development, a high priority was assigned to surface water satisfying the requirements of hygiene and aesthetics and that also lending itself to modest forms of recreation. In addition, a good management of surface water and groundwater levels.

These priorities are, of course, set out from a rather one-sided point of view. It depends to a large degree on the organization of water management within the municipal boundaries, both qualitatively and quantitatively as to whether the municipality should itself weigh up its priorities against other priorities or whether consultation with other management bodies is necessary

in order to reach agreement on the order of priorities. However, no municipality is able to establish its own priorities, whether or not modified. So that gives rise to problems. I have tried to inventorize these as generally as possible. The relative gravity of the problems will vary from place to place. I can also not rule out the possibility that I may have missed specific problems.

5 PROBLEM AREAS

5.1 Present situation

The requirement that waste water be removed as rapidly as possible from population centres has been greatly influenced by principles of modern quality control. The means of discharge from the system has been greatly changed, both qualitatively and quantitatively.

The first problem that arises is that for some mysterious reason the criteria laid down by the bodies responsible for quality control differ greatly. A town such as Rotterdam, which has to deal with four quality management bodies is confronted with four different sorts of limiting conditions. This forms an obstacle to a uniform management of the sewerage system. But that is not all: it also creates a form of inequality that manifests itself primarily in the cost sphere.

As regards the criteria themselves, the sometimes longer residence time of the water in the system can lead to processes which could possibly accelerate corrosion. But not enough is yet known about corrosion of concrete pipes filled - usually partly - with waste water for anything further to be said on the subject.

It is nevertheless worth noting that the sedimentation in the sewerage system is certainly not going to diminish, if anything it is more likely to increase. This sewage sludge has recently started to become a problem. In a number of cases, it does not satisfy the criteria which would permit it to be dumped. This will give rise to problems in the future, particularly in areas with small-scale industry. It is to be hoped that this problem will become a thing

of the past when these smaller industries are also made subject to licensing and control.

Another criterion, that of the permissible overflow frequency, forms a second problem area. The aesthetic and hygienic characteristics of urban surface water mean that even a single overflow is in fact undesirable. Even the discharge of rain water from paved areas often produces a distasteful picture.

Avoiding such an occurrence entirely is, of course, impracticable. In Rotterdam, a fairly stringent criterion is, however, observed in this respect, based on the old pumped situation. With the then very high overcapacities, an overflow frequency of not more than 3 to 4 times per year was observed. Compensation for this extremely strict criterion has been found not by higher investment in disposal means or overcapacity, but in a pumped overflow into the river.

Finally, the rather rigid application of the current computation methods, in which the most unfavourable situations for all connected systems are simply all added together gives rise to at least a suspicion of overdimensioning. Compensatory effects which can arise through differences in the situation for large-scale areas were not made use of and could not be utilized, since the capacities derived from the calculation were fixed.

The qualitative aspect of the discharge of municipal waste water is also rather strange. Both the connection permit and the overflow permit presented to the municipalities as model impose pretty stringent requirements - at least in the province of South Holland - as regards the quality of the waste water for disposal. This is strange, since the organization of tasks is in general formulated by the quality-control bodies such that the municipalities apply a Discharge Order solely to protect the sewerage system. The model Discharge Order does not rule out far-reaching requirements (see Article 10), but quite a commotion is stirred up if a municipality sticks too closely to it.

Besides the fact that municipalities are hindered in fulfilling the conditions in the above-mentioned permits, this also means that municipality policy is dependent on the priorities set by others. But the quality control body will not be very receptive to anyone with a batch of sludge whose composition and pollution falls outside the criteria set by the Law on Chemical Waste Disposal.

Finally, the surface water itself. The problem here is directly related with the as yet undefined function of urban surface water and the requirements arising from this. I have previously said something about this and Mr Kroes has already dealt with these problems in greater detail.

5.2 The near future

Is the problem fully described?

By no means. Up to now we have just made observations on the current situation.

In the meantime, other problems are looming into sight. Some of them have already been mentioned, but their extent is still not entirely evident.

Some will be mentioned.

The problem of diffuse discharges, of which there are two categories. The first we think we can do something about. They fall directly under the Surface Water Pollution Act. It is only the checking that is physically impossible. It mainly concerns discharges of residues of consumption goods, such as oil, chemicals and the like. What is quite clear here is that the collection of many of these materials is subject to a high barrier. Collection campaigns only succeed if this barrier can be lowered or if there is something to be had for the trouble.

The second category is more problematical. It includes materials which are released diffusely from road surfaces or from building sites, or merely from the not so clear sky. In both cases the municipality will become involved in combating them.

Another problem is the maintenance of sewerage systems. There are some very far-fetched stories about this subject. But one of the problems in this area is that up to now there is no method which can predict maintenance needs to a reasonable degree of accuracy. So for the present the age criterion is used. It has been found in Rotterdam that this is not a reliable criterion. We are currently searching for a relationship between age on the one hand and corrosive attack as a function of the settling of the system on the other hand. This is not simple. There is a lot of research to be done.

Thirdly, there is the application of the currently employed mathematical models for complex systems. I have already mentioned that systems are being designed by simple superposition of the least favourable situations. The concept 'central control' has meanwhile entered the scene. But it should be realized that the advantage to be gained with fixed pumping capacities is only limited. Flexible capacities are possible and even - contrary to what is generally assumed - at competitive costs relative to the old and familiar fixed capacity systems.

Fourthly, there are the problems of groundwater.

The situation is far more complicated here, as there is a gap in the legislation. On the one hand there may be a drainage problem, as a result of which the ground becomes marshy and cellars flood at high groundwater levels. From a technical point of view, there is but one solution for this: improved drainage. What is not clear is who bears the obligation to provide this. Mr H.H.A. Teeuwen has written an interesting article on this subject in 'De Ingenieur' of October, 1984. No 10. page 43. The other side of the coin is less easy to regulate from a technical point of view. It is the problem of excessive drainage causing the groundwater level to fall and endanger (especially wooden) foundations. This is, of course, a problem which occurs in specific areas with equally specific soil condition and composition. The causes may also be specific and local in nature, but may also be quite general. A characteristic is, however, that in the areas concerned, substantial settling is

quite common.

A couple of reasons for this can be identified:

- The streets in the areas concerned are laid on layers of sand. After settling has occurred, these are raised. In this way trenches are formed with a draining effect on the groundwater level, especially if connections are thereby formed with areas whose water level is maintained at a lower elevation.
- As the street subsides, so do the sewer pipes. The house connections break off and then take groundwater from the surrounding soil.
- Uneven settling can cause the sewer pipes to break or develop cracks and in this way remove water from the sand cunette. This has a comparable effect as the above-mentioned link with areas pumped to a lower water level.

I have restricted myself to causes not arising from anyone's fault. There are, of course also cases where someone else is to blame. It is these effects in particular which stand in the way of the efficient approach for which Mr Teeuwen makes a case in his article.

Finally there is the problem that is being extensively studied at the present time: the relationship between sewerage and water quality. This subject has also been broached previously, here and elsewhere.

A question that has not been voiced up to now, however, is the following:

If the results of the investigations lead to a different policy, and if this policy in turn requires new facilities - like associated facilities for the sewerage system - to which category would they then belong? To sewerage or purification works? It is clear to me that what we are here faced with is a financial problem. This, incidentally, is the case with all problems. The financial aspects of the problem areas, are beyond the scope of the subject of this lecture.

Neither the problems associated with preparing sites for the building of urban expansion projects will be dealt with. In the next paper it is proposed to examine this subject, referring to a recent publication of the Union of Water-boards. This 'guideline' gives an excellent exposé of the problems in this field and even proposes solutions for them.

6 SOLUTIONS

Please do not expect now a list of ready-made answers to the problems that have been raised. Many of them will have to be solved by a process of consultation, in which this exposé will hopefully contribute to the discussion, if only by creating an understanding of the innumerable problems facing municipalities.

There is one important factor missing in this consultation process, namely, a sort of forum in which municipalities could get to gripe with their technical problems. This lack results in the water-boards which are approached by the municipalities in respect of technical problems are faced with as many different manners of approach as there are municipalities. There are differences between groups of municipalities as regards their size, but some degree of order would seem desirable.

Moreover, the lack of such a forum is also the reason why in many cases each municipality has to rediscover solutions to problems which have already been solved elsewhere. This in turn means that even after problems have been identified, it takes a long time before they are eliminated, which is hardly beneficial to good water management.

Solutions will probably require great effort and consultation and will certainly also have financial consequences. For some of the problem areas mentioned, one already hears rumours of vast sums which are said to be required for their solution.

Clarity can only be obtained by thorough study. The municipalities

have not done much in this direction. However, the Ministry of Housing, Physical Planning and Environmental Protection (and before that the Ministry of Public Health and Environmental Hygiene) and also the IJsselmeerpolders Development Authority have made great efforts to fill this gap and still do so.

It might therefore be argued that the task to a certain extent is fulfilled. However, it is done in a manner whereby the priorities and the limiting conditions for the studies are determined by others instead of by the municipalities,

In the whole process of reorganization, the rules and solutions that sometimes emerge may raise the question if this is really what we have been waiting for.

But until the municipalities themselves take the initiative, there will be no change in the situation and they will just follow meekly.

7 SUMMARY

The municipalities have long been closely involved in water management within their boundaries. Their requirements are specific and directed particularly to the urban population and their welfare. There are three aspects by which this involvement manifests itself in modern water management:

- the sewerage function
- drainage of areas earmarked for expansion
- the surface water itself.

There still remain a large number of problems to be solved in the present situation. These relate particularly to the relationship between municipalities and the bodies responsible for quality control. In some cases the problems are specific to individual municipalities.

New problems are presenting themselves, or have already made themselves known. They include many specifically municipality-related aspects, half a dozen of which have been elucidated in the foregoing.

If the municipalities, possibly in co-operation with others, are to find a solution to these problems, the first and foremost precondition will be that they combine their forces and themselves define a policy for studies into finding solutions.

THE WATER-BOARD, ALSO IN URBAN AREAS

J.S.J. Dragt

ABSTRACT

In his lecture Mr De Graeff explained the responsibility and position of the water-boards with regard to the water management of The Netherlands. These bodies are designated by various names, depending on their specific tasks such as: polder-board, joint college of water-boards, polder district, sewerage purification authority etc. In the following the general designation "water-board" will be used also because both in the explanatory memorandum to the draft bill on the water-boards and in the revised Constitution of the Netherlands it is taken that the concept of "water-board" as such needs no definition, "no more than in concepts of municipality and province do".

1 QUANTITATIVE WATER MANAGEMENT IN URBAN AREAS

The main task of a water-board consists of the conservancy of defence works of waterways and appurtenant structures in the interest of water control and navigation. The jurisdiction may comprise the territories of several municipalities. As a result the municipalities and the water-boards often "run across" each other in performing their tasks. In this encounter, "running across" means a mutual attuning of the interests to be looked after by the two agencies in organizing and managing the water element in an urban area. This is precisely the subject of this symposium. In parenthesis it should be mentioned that the municipality and the

water-board also run across each other outside the urban area; i.e. in the rural areas where both agencies are dealing with physical planning. By means of a number of practical examples it will be shown how a good co-operation between a municipality and a water-board resulted in well-integrated solutions of problems of urban extension, but also where this co-operation jogged along.

Below it will be outlined in which way the water-boards can, will and must be active within urban areas.

Where different governmental bodies sit at the table to consult how in a certain situation the water management should be effected the starting point of the individual parties to the discussion is often to keep the costs for themselves as low as possible.

In politics, the concept of segregation has been introduced to indicate this way of operating only within one's own field of interest. Technicians also are guilty of this practice.

It needs little argument that this is not a very efficient way of starting the solution of the problems. Let alone that an efficient solution will result from it. The starting point must be co-operation and not only consultation. Only then an optimum solution can be attained in the field of policy as well as in the technical and financial fields. If the starting point is co-operation, a reasonable solution for the distribution of the costs will always be found.

A second remark concerns the point of time at which the co-operation should start. Well then, that cannot be too soon. Development of urbanization projects requires the collection of all kinds of information. Consequently, for a fertile co-operation between water-board and municipality to be possible, the start of this co-operation should be as early as possible. Otherwise, there is a great risk of positions having been taken up and segregation having settled - we people are just made that way. For that matter, early co-operation also applies to other bodies involved in the development of projects.

1.1 Guideline

The problems signalized above have given rise for the Union of Water-Boards to consider how the water-boards have to carry out their tasks

when building over of an area is realized under the responsibility of a municipality. Recently for that purpose the publication "Guideline on the defence against the water and the water quantity management by water-boards in relation to the realization of the building over of an area by municipalities" has been issued.

It would carry too far to discuss this guideline in detail, but some main items should be mentioned.

The problem is how the task of a water-board as laid down in the regulations with its managerial, technical and financial responsibilities can be executed when a municipality wants to build over an area.

The guideline distinguishes site development from the water management. The former is a municipal task, the latter the task of the water-board. It is clear, however, that site development and management are two matters that cannot be considered independently from each other. The guideline assumes that site development is exclusively a municipal task. Based on this assumption all kinds of physical planning decisions will have to be taken that will be affected by the management of water or that themselves affect the management of water. In the former case the influence can be such that finally even the lay-out changes. In the latter case adaptation of the water management system may be necessary. Once the new lay-out (building over of an area) has been realized, the water-board has to adjust its management system to the new situation.

As examples that must be classified under site development may be mentioned: filling in, drainage, sewerage, excavation of canals and moats and the like and their draining by means of pumps.

Water management after site development includes the evacuation of excess water from rainfall which is draining off rapidly.

Nationally applicable definitions of the responsibilities cannot be given because the tasks of the water-board, in accordance with the regulations, have been commissioned to differ too much in the Netherlands. Consequently, it is risky to give examples of solutions as they soon are going to lead a life of their own. Some practical situations may clarify how in specific cases these tasks are interpreted.

1.2 An example of the water-board De Aa

Within the scope of the necessary consultation pursuant to Article 8 of

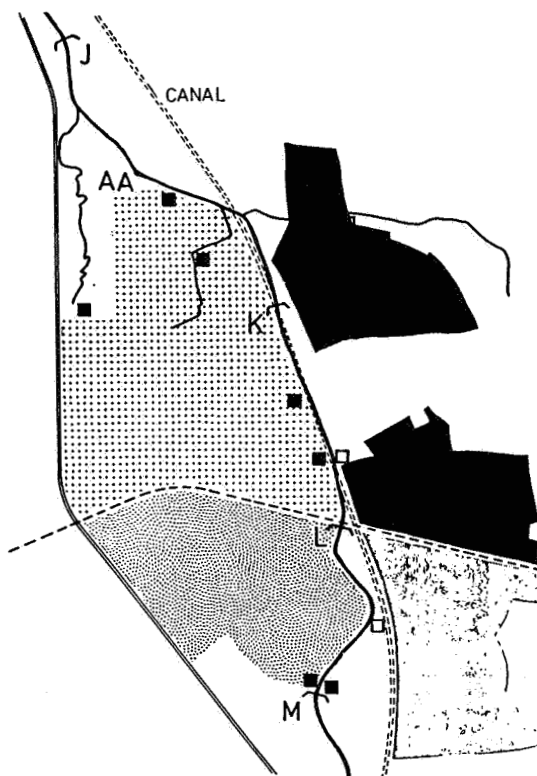


Figure 1 The situation of the present and future overflows

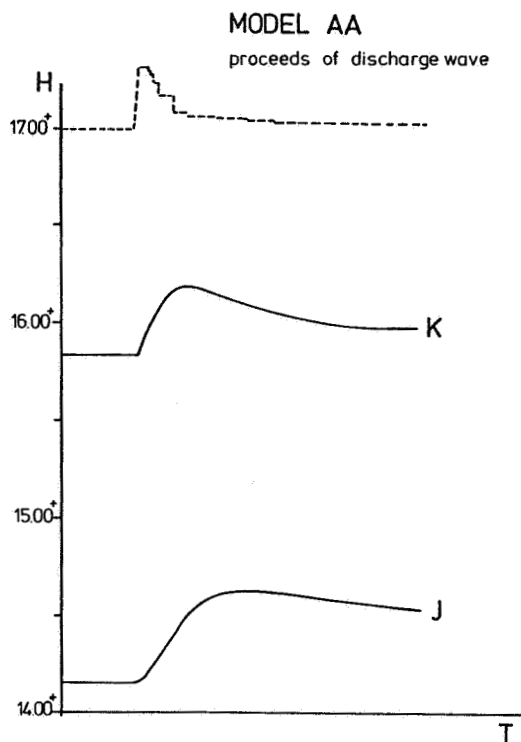


Figure 2 The outflow hydrogram of one of the overflows is indicated by a dotted line

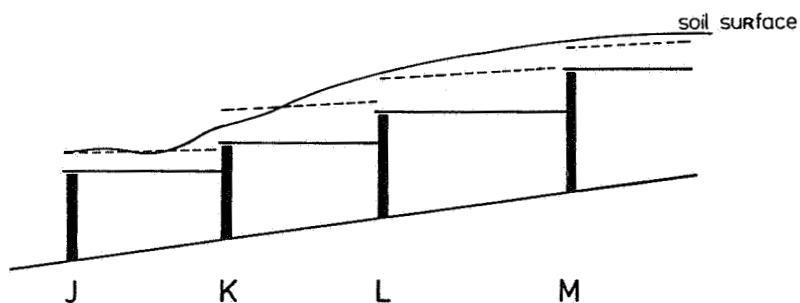


Figure 3 Levels as a consequence of overflows

the Decree Physical Planning, municipality X brings to the notice of the water-board (see Figure 1) the draft of an relatively large zoning plan. The plan forms part of three large town plans and together they form a structural intervention in the water management of the region. In an earlier stage the water-board had already argued for the establishment of a working group ad hoc to consider in which way adaption of the water management could be made. The above zoning plan in the meantime had struck out a line for itself, while the investigation of the working group that had indeed been established was still going on. The answer of the water-board to the Article 8 procedure can easily be guessed:

Objections!

It is more interesting, however, to know what were the results of the working group. First and foremost, however, we must mention the standpoint of our water-board as to the discharge of urban water.

In general, it means that upon draining to water courses - mostly via overflows - a capacity of 50% of the design discharge for the water course must be provided, so that the 50% of the design discharge is available for the discharge of the urban rainwater that will drain off rapidly. The remainder must be stored. Also, the water-board always pleads to establish the necessary storage according to the Physical Planning rules.

The above zoning plan provided for the discharge of the overflow water to the Zuid-Willems-canal, which will be diverted in the future. No attention was paid to the necessity of discharging for many years the additional overflow water - having a maximum intensity of about $5\text{m}^3/\text{sec.}$ - via the Aa.

Neither was it clear whether this magnitude of discharges would produce an unacceptable quality of the water of the river Aa.

All in all the water-board foresaw great problems, the more so as the above-mentioned general practice on the capacity of 50% of the design discharge could not be applied here; for this purpose about 10 weirs in the Aa would have to be automated. The results of the investigation is summarized in Figure 1, 2 and 3.

Figure 1 gives the situation of the present and future overflows.

In Figure 2 the outflow hydrogram of one of the overflows is indicated by a dotted line.

For all overflows such an overflow diagram has been made and introduced into the calculation.

The other curves are hydrographs at various nodal points in the Aa. The computer calculation gave as result maximally occurring levels as a consequence of overflows, partly shown in Figure 3. From this it could be concluded that, apart from storage section J-K, the levels occurring in the Aa would nowhere give rise to inundations. For storage level, which is justified, because the agricultural significance of the area concerned as minimized by urban development. As the water-board at first feared that storage ponds had to be included in the expansion plans, this did not appear to be necessary after the investigation provided the water-board would be prepared to accept exceedances of short duration of the target levels. Consequently, it was not necessary to strictly stick to the above basic standard. As these matters did not form any problem, an enormous saving in the zoning plan could be obtained.

1.3 An example of the water-board De Dommel

A typical example of a water-board having taken over the responsibility is provided by the case of the water-board De Dommel. For many years already there had been the discussion whether or not De Dommel between Eindhoven and Boxtel had to be improved.

Calculations of the desirable high water line had been carried out in the nineteen sixties. Sewerage plans and design levels of new zoning plans were tuned to the lowered high water line to be expected. When the water-board finally decided not to improve the Dommel between Eindhoven and Boxtel, it was also decided that this might not be at the expense of a worsening of the drainage and dewatering situation of the adjacent urban areas. The result of this decision was that in particular St. Oedenrode was provided with a system of pumped overflow which guaranteed sufficient drainage at high Dommel levels.

The guidelines of the Union, as stated above, give clear recommendations for the responsibility of the water-board if urban developments from part of the assignment of the water-board. If, however, it is a

depoldered area*) which depends on the discharge of its overflow on the adjacent district of the water-board, an entire different situation arises. Then the water-board does not have a statutory responsibility in such an area. In most cases the water-board will impose entirely unilaterally per licence the conditions for discharge to the water management system under its control.

That is a bad thing, because also in this case by means of an examination of the possibilities there may be an alternative solution involving lower costs for the society. Here, too, a condition is that the water-board will be involved in the plans at an early stage. Naturally, all financial consequences are in these cases for account of the municipality, the costs of official assistance being included. These are the disadvantages of depoldering!

1.4 Summary

The above examples make it clear, and the guideline points to that too, that early and structured consultation is necessary in preparing the zoning plans. Also in 1977, in a lecture on a similar subject to that of this symposium, the writer of this report pleaded already for the inclusion of a chapter "wet-structure plan" in the zoning plans. The experience has been that not yet every town-planning expert is aware of this necessity.

In an appeal procedure even a provincial government was of the opinion that such a "wet structure plan" - from which "destinations" result - needs not form part of the zoning plan.

2 WATER QUALITY MANAGEMENT IN URBAN AREAS

In the Netherlands the water quality management for the water courses which are not non-Governmental waters has been entrusted to the Provinces, who in most cases have in turn delegated this management to water-boards

*) A "depoldered area" is an area which has been withdrawn from the jurisdiction of a water-board because of physical reshaping of the area and administrative reasons.

specialized in it, or sewerage purification authorities especially brought into existence for it.

These bodies have to take care of the realization and maintenance of a good quality of the surface water. The basis for this assignment has been the fact that water quantity and water quality and their management are intimately related. Since a judge pronounced a sentence on it, it is also an established fact what has to be understood by surface water;

"each water that at any moment of the year is visible and communicates with other water ought to be considered surface water in the legal sense".

Consequently, it is clear that the water quality management of the water-board also extends to the urban waters, both the canals of Leiden or Delft, and those at Almelo or Baarle Nassau, which dry up temporarily. Now in which way manifests the water-board itself as to the water quality management of the surface water in an urban area? As stated earlier these waters form part of a unit of water management: they run off to the water outside the town or the waters from outside the town flow through them. By means of the system of water quality plans functions are conferred to the surface water. It is clear that these functions, apart from depending on the possibilities, also depend on the destination a certain surface water has. This demonstrates the coherence of the quality management inside and outside the town.

It is the task of the water-board to pursue as much as possible the functions that once have been established, and that cannot be done separately within the boundaries of the built-up area. Therefore, Mr Kroes' suggestion to set the municipality an independent task as to the management of the urban waters, in the sense of the Pollution of Surface Water Act should be rejected for this very reason.

In which way then should the water quality manager see his task in urban areas?

Even more than in the rural area, there should be a better tuning in the urban area between the zoning plan within the scope of the Physical Planning Act and the functional assignment of the surface water within the scope of the Pollution of Surface Water Act. There are no legal co-ordinating committees. The Report on Water Management, which is now under discussion in the Lower House, also establishes this shortcoming. Only an early co-operation between the quality manager and the municipality, similar to the one on water quantity management, can provide a

clear tuning, the more so as the urban waters clearly affect the surface waters in the rural area. Kroes and Pieterse have outlined the functions of the urban waters. In the draft report by working group V - subgroup 1 - of the CUWVO (Implementation Committee Pollution of Surface Water Act) "Ecological quality objectives for the Netherlands surface waters" some extensive particulars can be read on the functions of urban waters. It is clear, and everyone working in practice will know, that considering the very functions of urban waters, one should not nourish great expectation of the quality of those waters. In this connection it should also be observed, that in large parts of the Netherlands the water management in the built-up areas takes place around and in a system of canals, ditches and lakes which in some cases may be dry for a part of the year. Then water quality management assumes a particular significance!

Now in which way does the water-board try to integrate the water quality management in the urban areas into the overall water quality management in its jurisdiction?

Considering the large differences in areal size of the water-boards and also considering the differences within a given area, only a general approach can be formulated. Furthermore, only the influence of combined sewer systems on the quality of the urban waters is considered. The most important key of water-board in managing the urban water quality is the draining licence based on the Pollution of Surface Water Act. Flushing and rinsing are among the essential tools of quality management. The draining licence is known to everyone. In particular it is required where a combined sewer system has been installed and overflows are unavoidable. Therefore, the water-boards care intensively involved in the municipal sewerage plans. Indeed, the effects of the unavoidable overflow should be restricted as much as possible and not every outfall on the surface water is acceptable. Nevertheless the negative effects of an overflow cannot always be prevented.

Then sometimes little remains of the water quality desired for urban waters as described in the CUWVO (Implementation Committee Pollution of Surface Water Act): the city pond stinks and even if there were fish in it, it would die; the overflow ditch does not exactly present an esthetic appearance.

Experience has taught that a great number of cases the poor quality of

the urban waters could have been prevented if a zoning plan had been set up in which both physical planning and water management had been integrated.

Current practice is that in many cases after a rainy period the quality of urban waters is bad. In parts of the Netherlands this problem can be alleviated by means of flushing of the water courses.

In many cases, however, only a decrease in the emission of pollutants of an overflow may lead to improvement of the water quality, in town and also in rural areas, but one should not expect wonders from it.

In this respect Mrs Hoogendoorn indicated some possibilities in her lecture. This is one of the most important fields of research of the Netherlands Research Committee on Sewerage and Water Quality.

2.1 Water quality management

Now what is the position of the water quality manager in managing the urban waters? It is the task of the water-board to see to it that the object for which the surface water is intended, with the pertinent standards, is realized also in town. In this respect the water-board may conduct either a passive or an active management.

In case of a passive management one restricts oneself either to making conditions or to supplementary conditions to the draining licence. This means in case of new urban expansions that location and extent of the overflow must be weighed against the receiving capacity of the surface water. Then first and foremost there must be a well-designed sewer system, in general having a 7 mm standard storage capacity and 0.7 mm/h pump capacity. As these units are reasonably well controllable and I prefer these to the criterion of the overflow frequency which can never be well controlled.

Demands that go beyond the above might be set to new sewer systems - for instance specific overflow structures - if the water-board can unambiguously show that this is necessary from the viewpoint of proper water quality management. It can be justified that the licensee then also sees to the expenses.

It becomes more problematic if two different applicants for a licence will make use of the same surface water, in which situation it can be justified that one direct overflow discharge is acceptable, provided

that at the other overflow provisions are made. Who then has to carry the can? Probably both! For shortly the overflow provision becomes standard condition in the licence, "for safety's sake".

In existing situations the adaptation of the conditions for the licence is a legal possibility. In most cases introduction of supplementary provisions proved to be unfeasible because of financial constraints. Therefore, it is recommendable, that the water-boards play a more active role in combating negative effects of overflow on surface water. Know-how and financial opportunities have also been the reasons to the implementation of technical purification works to the water-boards. In that case the water quality management plan will serve as a foundation.

In this way the practical implementation of the management will run parallel to the above-mentioned water quantity management of the water-board in town. For the water quality managers to be capable of developing any action in respect of the above, the central government will have to extend the helping hand to them. The highness of the current levies, which already necessitates a temporization here and there in realizing the primary purification works, does not allow also additional sewerage provisions to be carried out shortly. This is even less possible in the municipal sphere.

Consequently, Mr Kroes' suggestion to assign to the municipalities an independent task in the quality management - in the sense of the Pollution of Surface Water Act - of the urban waters system, is not considered to make much sense.

If the Ministry who is represented by Mr Kroes would give a financial injection to the water quality managers, as has been done to the municipalities in connection with the arrangement for expensive sewerage works, one may be convinced that particularly good results will be obtained in improving the aquatic urban environment. Actually, such a financial arrangement would have to be a logical sequel to the enormous financial contribution from the Ministry of Housing, Physical Planning and Environmental Protection to the research programme of the Netherlands National Research Committee on Sewerage and Water Quality.

As to this view the issue is not who on formal Pollution of Surface Water Act grounds will have "to carry the can" of the said overflow, but in which way an efficient approach can be achieved of the water pollution by an overflow; for that matter this overflow belongs to a well-

dimensioned sewer system.

In the sense of the guideline for the quantity management a guideline might also be drawn up for the management quality. Then in the diffuse border area between sewerage and surface water quality the segregation, or in other words, the wasting of money, could be prevented.

Finally, an example of the influence of zoning plans on the water quality will be given.

2.2 An example of the water-board De Aa

The qualitative aspect was also investigated in the above-mentioned zoning plans in which drainage to the Aa was foreseen.

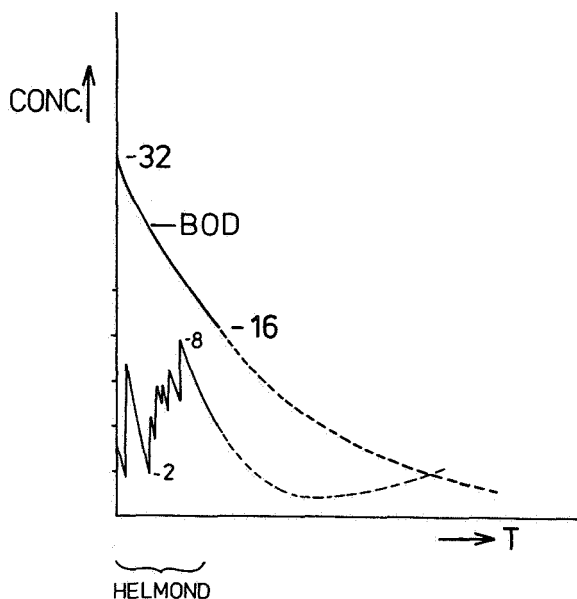


Figure 4 Proceeds of BOD and O_2 after an emission of pollutants

In this investigation only the parameters of Biochemical Oxygen Demand and Oxygen were involved. In Figure 4 both parameters are given in relation to either time or downstream distance. It concerns the resulting pollution bung produced at the last overflow. It can be derived from the Figure how the pollution bung, while travelling downstream, decreases in concentration.

There is something peculiar with the oxygen content. At the moment that this oxygen content reaches a critical value, the aerating effect of the weirs present again provides a substantial oxygen supply. Consequently, the oxygen content nowhere falls below 2 mg/l during the passing of the pollution bung, which was considered acceptable for short periods. Therefore, on account of these calculations it is considered justified to leave out supplementary measures for the reduction of the emission of pollutions.

The more so as the situation in respect of the diversion of the Zuid-Willemsvaart will be entirely different after 10 years. Based on the above, the water-board could after all agree with the zoning plans. A favourable thing was that in this case the problem of cost allocation did not arise.

3 RECOMMENDATION

In so far this has not been done yet, water boards and municipalities ought to make approaches towards each other in an early stage upon realization of the plans. Only in this way plans can be achieved that are acceptable for society at acceptable costs. In doing so, the task of the water-board has to begin where the water reaches the surface from the built-up area. Financial support for such a policy from the public means is necessary.

By means of a number of tests the effectivity could be tested of an urban water quality programme to be implemented by the water-boards.

GROUNDWATER MANAGEMENT: GOAL, REQUIREMENTS AND METHODS

A.J. Hebbink

ABSTRACT

In the Netherlands the new residential areas mostly are concentrated in areas where the groundwater levels are too high. But not only in the new housing estates, also in the old residential quarters inconvenience may be experienced from shallow groundwater. To prevent disadvantages of high groundwater levels, a good groundwater management is necessary. In this article a description is given of the urban hydrological system. Secondly the drainage criteria are mentioned which are necessary to control the groundwater level. A brief impression is given of some measures which can be taken to lower the groundwater level. More detailed attention is paid to one of the possible measures, namely subsurface drainage. Different types of subsurface drainage systems are applied. Each system is described with its advantages and disadvantages. The experiences with subsurface drainage, applied in Lelystad and Almere, are indicated.

1 INTRODUCTION

There are statutory regulations stating who is to control the open water. But there are no regulations stating who is to control the groundwater. In the case of groundwater the well-known saying seems to hold: "What the eye does not see the hart does not grieve over". Yet the groundwater management is very important. To prevent inconvenience of high groundwater levels, a good groundwater management is

necessary. During building activities a high groundwater level can cause some problems. For example by a bad load-bearing capacity of the building site. Besides inconvenience can be found in installing cables and conduit pipes. After the building activities a high groundwater level can be disadvantageous to buildings and houses. Just imagine damage to walls, carpets, parquet floors and gas-pipes. Also the road-construction can be damaged by a high groundwater level. Finally the height of the groundwater level influences the growing possibilities of plants and trees.

It is also possible that there are some problems with groundwater levels, which are too low. For example, this can result in withering and dying of plants and trees or in damaging of the foundation (rotten wooden piles). However, no attention is paid to problems with low groundwater levels.

In this article first an impression will be given of the urban hydrological system. Of great importance are the requirements which the groundwater management in urban areas has to meet. In brief attention is paid to some measures which can be taken to lower the groundwater level. More detailed an impression is given of one of the possible measures, namely subsurface drainage. A description will be given of drainage materials, subsurface drainage systems, design criteria, installation and maintenance of subsurface drainage. Finally some problems are mentioned which can occur at the application of subsurface drainage.

2 URBAN HYDROLOGICAL SYSTEM

In general the precipitation which falls on an unpaved area, infiltrates into the ground. In dry circumstances the infiltration capacity of a sandy soil varies from 40-80 mm/hour and of a clay-soil from 30-50 mm/hour. In wet circumstances the infiltration capacity varies respectively from 20-50 mm/hour and from 0-10 mm/hour. It appears that under wet circumstances the infiltration capacity of a clay-soil can be zero. Partially the infiltrated water can be stored in the ground. But the

greatest part has to be discharged to open water.

An important part of the precipitation which falls on a paved area, flows over the paved area to sewers. However, from several researches it appears that there can be an important infiltration through some paved areas, such as tiles and bricks. Infiltration capacities are found varying from 5-15 mm/hour.

The amount of water which has to be discharged, is fixed by the surplus of precipitation. To design a drainage system especially the surplus of precipitation during the winter period (October to March) is important. From measurements of the station Lelystad-Haven it appeared that once in the 10 years, the surplus of precipitation during the winter period can be more than 400 mm. This means that on an average about 2 to 2.5 mm/day have to be discharged.

Established in 1968 an ongoing urban hydrological research project is being conducted in Lelystad at several catchments. One of these catchments has a surface of 2 ha and 44% of the surface is paved. From this research project it appeared that about 40-50% of the annual precipitation is discharged by subsurface drains and only 20-25% in storm water sewers.

3 SOME ELEMENTS OF MAKING THE AREA SUITABLE FOR BUILDING

Every year many areas are withdrawn from their original destination in behalf of house-building. Till the year 2000 still about 1 million houses will be built. When we assume a density of 30-40 houses per ha at least 25.000 ha will be zoned for building houses.

Before building operations, first the area has to be made suitable for it. This can enclose the following elements: choice of the street level, raising the building site with sand, prognosis of the subsidence, and the installation of drainage and sewerage systems. In the next pages attention is paid to the drainage systems. First the drainage criteria are mentioned which a properly drained area has to meet.

4 DRAINAGE CRITERIA

The drainage criterion is defined by the distance between the soil surface and the highest admissible groundwater level.

On a building site it must be possible to use the appropriate transport vehicles and to store building materials, all this without excessive inconvenience from groundwater. In general the excavations have a depth of 0.5 m below the soil surface. It is taken as criterion that the groundwater level has to be lower than 0.2 m below the bottom of the excavation. So the drainage criterion for the building of houses is 0.7 m below the soil surface.

The cables and conduit pipes have to be installed or repaired without inconvenience from groundwater. In table 1 the minimum depth of several cables and conduit pipes is given. From this table it appears that the drainage criterion is about 0.6 to 1.0 m below the soil surface.

Table 1. Minimum depths of several cables and conduit pipes in m (NEN 1739)

low tension cable (< 10 kV)	0.60
high tension cable (> 10 kV)	0.90
telephone cable	0.50-0.60
gas pipe	0.60-1.00
conduit pipe	0.80
sewer	1.00
town-heating	0.50-0.90

Also the construction of roads may not experience inconvenience from groundwater. For arterial roads the drainage criterion is 1.0 m below the soil surface. When the rush of traffic is less or it concerns small roads, a drainage criterion of 0.6 m below the soil surface is sufficient. In the case of (public)gardens it is not possible to give prevalent criteria because just variation in the drained situation is one of the possibilities to get a varying natural environment. In general the groundwater level may not rise higher than 0.4 to 0.5 m below the soil surface.

5 METHODS TO CONTROL THE GROUNDWATER LEVEL

To meet the drainage criteria mentioned before, in urban areas the following methods are possible:

- lowering the groundwater level by the installation of a drainage system (canals, ditches, subsurface drainage);
- raising the area with a layer of sand;
- lowering the groundwater level by lowering the open water level;
- a combination of these methods.

The mentioned methods will have different effects on the groundwater level. Where the area is raised with sand, there may well be settlement owing to the groundwater being pressed out of the semi-permeable layers. A lowering of the groundwater level in areas with seepage, may well result in an increase of seepage. In areas with infiltration, the infiltration will decrease and may well change to seepage.

Owing to the vulnerable geohydrological situation, a lowering of the groundwater level will lead to a fall in the groundwater levels in the surrounding area. Since in these areas, changes in the geohydrological system can have far-reaching consequences, especially for the vegetation, falls in groundwater levels are as a rule unacceptable.

In the next pages more detailed attention is paid to the subsurface drainage.

6 SUBSURFACE DRAINAGE

At present subsurface drainage systems mainly consist of corrugated plastic drain pipes. To prevent inflow of soil particles and to improve the entrance resistance envelope materials are used. In the case of homogeneous clay and heavy loamy soils there is no risk of soil particles being washed into the drain pipes. For this reason pipes can be laid without covering or envelope material. In the case of light loamy and sandy soils, however, there is a risk of silting up and the pipes are covered by or enveloped with a filter material. In order to avoid disappointing results first of all the materials are tested in the

laboratory or/and in the field. The research is primarily concerned with the entrance resistance of the materials, as well as with filtering operation and durability. In recent years coconut fibre and polypropylene fibre envelopes are mainly used. A good deal of silting up with sand, when using coconut fibre envelopes, tended to occur in fine sandy soils. Besides coconut fibre can decay fairly quickly in certain circumstances. When using polypropylene fibre envelopes there is no silting up in fine sand and the material is unlikely to decay. On the other hand, polypropylene fibre is more expensive than coconut fibre.

In special cases gravel is used as cover material.

The main functions of the subsurface drainage systems are:

- acceleration of soil subsidence;
- control of the groundwater level during building activities;
- improvement of the load-bearing capacity of the building site;
- control of the groundwater level after the building activities have been completed.

6.1 Subsurface drainage systems

First of all a distinction can be made in horizontal and vertical drainage systems. At vertical drainage the groundwater is discharged by way of sand or gravel piles to deeper lying well permeable sand layers. At horizontal drainage we can distinguish the following systems: single drainage system, composite drainage system, crosswise drainage system, block drainage system and road body drainage system.

6.1.1 *The single drainage system*

The single drainage system consists of drains laid parallel to each other, at distances which are dependent on the permeability of the ground. The drains discharge their water directly into ditches or canals. The single drainage system is temporarily used before and during building activities. This system is also used for sport fields and recreational areas. In figure 1 the principle of the single drainage

system is given.

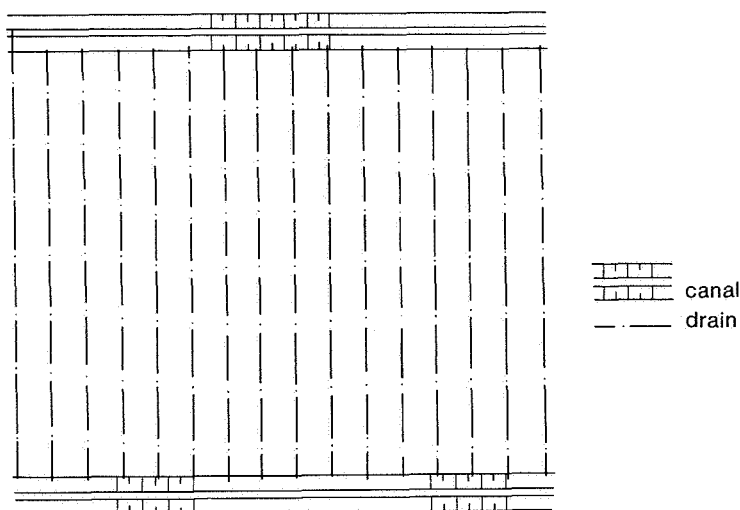


Figure 1 The single drainage system

The installation and maintenance of the single drainage system is very cheap. The disadvantage is that this system is easy to disturb by sewers, conduit pipes and pile driving.

6.1.2 *The composite drainage system*

In the urban areas there are mostly too few ditches and canals or too many obstacles to suffice to lay only a single drainage system. In that case a composite drainage system can be applied, by which the drains are connected to main-drains. The drains discharge their water into sewers or canals by way of the main-drains. The drains can be maintained from canals, cleaning entrances or junction boxes. The theoretical set up of the composite drainage system is shown in figure 2.

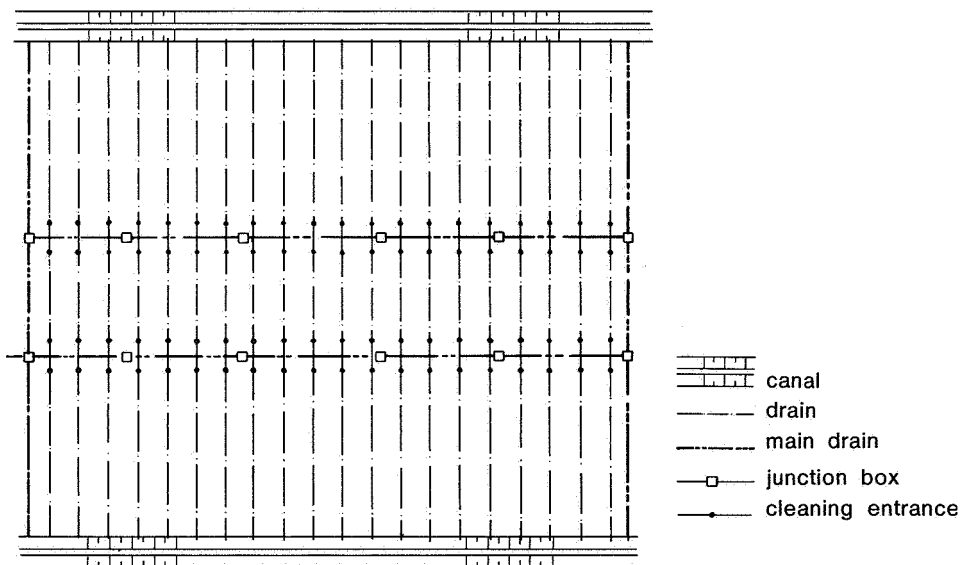


Figure 2 The composite drainage system

The installation of the composite drainage system is relatively cheap. A disadvantage of this system is, that by damage of a maindrain the system is easily disturbed and consequently the drainage of a relative great area functions not sufficiently. Besides, when the distance between the canals becomes greater, the composite drainage system becomes more complicated and more vulnerable.

6.1.3 *The crosswise drainage system*

To avoid disruptions of drains as much as possible a crosswise drainage system can be applied. Two single drainage systems are laid perpendicularly to each other. The upper system is laid about 0.15 m above the lower system with a view to avoid disruption of the lower system by the installation of the upper system. As it is very important that water can flow from one system to the other, the drains have to be covered with proper cover material, for example gravel. In principle the drains only can be maintained from ditches or canals. The dimensions of the crosswise

drainage system, as installed in Almere-Stad, are shown in figure 3.

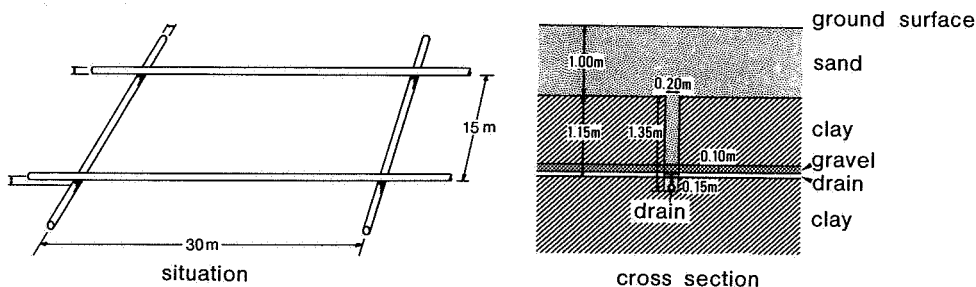


Figure 3 The crosswise drainage system

Should disruptions occur the crosswise drainage system keeps a possibility to discharge the water by way of the intersections. A disadvantage of this system is that the installation is relatively expensive, as a consequence of the usage of gravel as cover material.

6.1.4 The block drainage system

In view of the fact that the building pits have to be dry, drains can be laid around buildings or houses. The drains mostly are installed previous to the building activities. The drains are laid almost directly against the front. In behalf of the proper functioning of the block drainage system, it is very important that there is at least 0.3 m well-permeable sand under the building or the house and that there is a connection between the sand in the building pit and the sand in the drain trench. The drains discharge their water in the sewers or main-drains and sometimes directly into the open water. The drains are mostly maintained from cleaning entrances. The principle of block drainage is shown in figure 4.

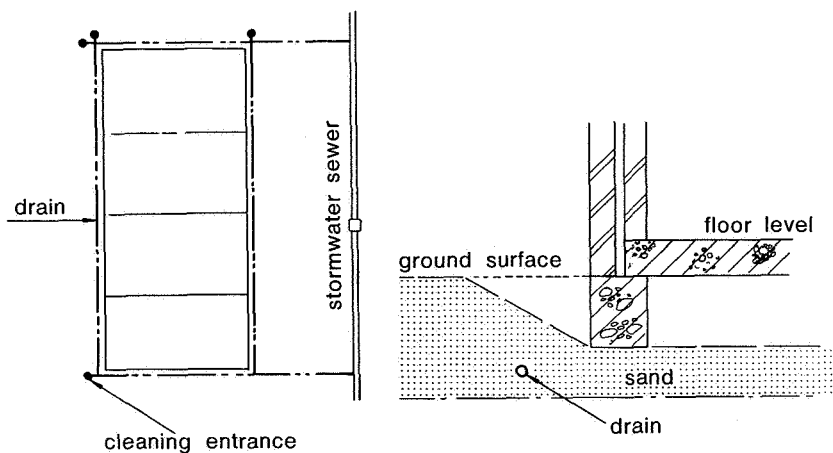


Figure 4 The block drainage system

Instead of laying drains around buildings or houses, it is also possible to lay the drains under the building or the house. In this case the drains are installed after the excavation and pile-driving. The drains are connected to the storm discharge. The block drainage is a more or less reliable drainage system. The drains can hardly be disturbed. On the other hand, the installation of block drainage is expensive.

6.1.5 *The road body drainage system*

When the soils have not a sufficient load-bearing capacity, the road can be constructed on a sandfill of 1 to 1.5 m dependent on the soil type and the admissible traffic load. If the bottom of the road body exists of badly permeable ground a subsurface drainage system is necessary to control the groundwater level and to improve the load-bearing capacity. The drains mostly are installed in a little trench at both sides of the road. At mutual distances of about 200 m uncorrugated drains are laid perpendicular out of the road body drains to ditches or canals. The drains are maintained from cleaning entrances or directly from the open water. The principle of the road body drainage system is given in figure 5.

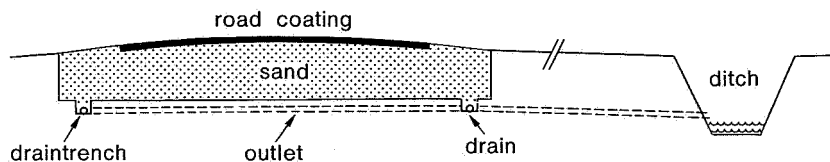


Figure 5 The road body drainage system

6.2 Design of a subsurface drainage system

To meet the earlier mentioned drainage criteria, some design criteria have been developed on account of the subsurface drainage systems. The design criterion is a relationship between the groundwater level and the drain discharge. It means that with a certain amount of drain discharge, the groundwater level may not rise above a certain level. The design criteria are based on a frequency once a year. In table 2 the design criteria are given for several destinations in an urban area.

Table 2 Design criteria

Destination	Groundwater level in m below soil surface	Drain discharge in mm/day
Building site	0.60	10
Residential areas	0.70	5
Sport fields etc.	0.50	15
Recreational and green areas	0.50	7
Cemeteries	0.35*	10

*(in m below the bottom of the coffin)

To meet these criteria the drains must have a certain distance between each other. To calculate the spacing of drains several formulas have been developed. In practise mostly the Hooghoudt's formula is used:

$$Q = \frac{8.k2.d.h + 4.k1.h^2}{L^2}$$

Q = drain discharge in m^3/sec

L = spacing in m

k_1 = hydraulic conductivity under drain level in m/sec

k_2 = hydraulic conductivity above drain level in m/sec

d = thickness of equivalent layer in m

h = difference between drain level and highest admissible groundwater level in m

6.3 Depth of the subsurface drains

In practice there is a great variety in drain depths. A mostly made distinction is: shallow subsurface drainage (till 1.5 m below soil surface) and deep subsurface drainage (deeper than 1.5 m).

Owing to the building activities (excavations, pile driving, installation of sewers and conduit pipes) a previously laid subsurface drainage system will be disturbed more or less. As the drains are laying deeper, there is less change that the drains are disturbed. On the other hand the repair of these drains is more expensive.

Practically mostly the drainage criteria and the maximum depth of the drain are given, for instance the wish to discharge the drains above the open water level. Inside this range several drain depths are possible. Which drain depth you have to choose is hard to say. It depends on various factors:

- Drainage machines. At present the drainage machines are capable to install the drains at great depths. In general this need not be a restrictive factor.
- Sewerage. When the drains discharge their water into the sewerage system, the depth of the sewers is determinant for the depth of the drains. On the other hand, if the drains may not be disturbed by the sewerage system the drains have to be laid at great depths; in general more than 2 m below soil surface.
- Upper limit. In connection with the frost-limit a ground cover of at least 0.6 m is necessary.
- Cables and conduit pipes. To avoid disruptions of cables and conduit-pipes the drains have to be installed deeper than 1.2 to 1.4 m below

soil surface.

6.4 The installation of the subsurface drains

When there are no obstacles the drains generally are installed with a drainage machine. At present these machines can install the drains to depths of 2 to 4 m below soil surface. The drain depth is usually regulated by a laser plane. When the drains are laid under wet circumstances, it could happen that the draitrenches are filled with badly permeable material or that there is a great variation in the drain depth. Both are disadvantageous for a well-functioning of the drainage system. In urban areas much additional drainage has to be laid (collector-drains, main-drains, block drainage). Since at the moment that the additional drainage is laid other activities are often taking place, the drains have to be laid with a hydraulic crane. It is self-evident that this way of installation is much more expensive than having the drains laid with a drainage machine.

6.5 Control and maintenance of the subsurface drains

To find defects in the drainage system it is necessary to control the drains directly after installation. An indication of the functioning of the drainage system can be obtained by measuring the groundwater level at several places. When the groundwater level is high during long periods the drainage system does not function very well. The drains have to be cleaned. At present drains can be cleaned up to 200 to 300 m from the outlet. When the drains are longer, cleaning entrances have to be installed (figure 6). In main-drains of collector-drains junction boxes are placed to have the possibility to clean these drains. From a cleaning entrance or a junction box the drains can be cleaned over a distance of 100 to 150 m.

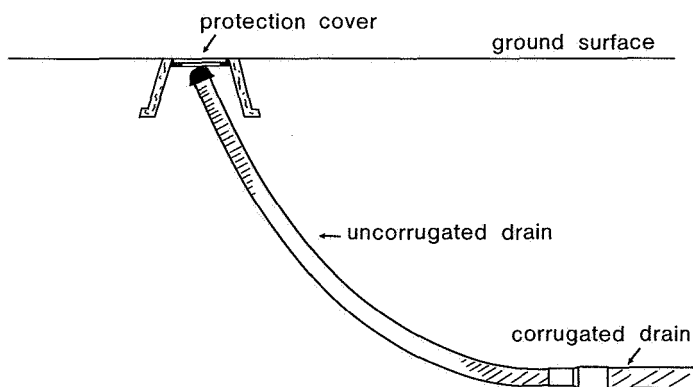


Figure 7 A cleaning entrance

Maintenance of the drains is mostly carried out with special "flushing machines". Water is pumped into the drain through a reinforced hose with a special nozzle, which then passes the entire drain. The operation can be done under high, middle or low pressure. Experience has shown that low pressure flushing (< 25 atmosphere) and middle pressure flushing (± 35 atmosphere) are usually preferred to high pressure flushing (> 80 atmosphere).

The frequency at which maintenance of the drains has to be done is dependent on the measure of pollution, but also on the use of the drained area. The deposition of iron in and directly around the drains results in a poor functioning of the drainage system. So regular maintenance is necessary. In areas with little or even no iron at all a maintenance of the drains once in the 5 to 10 years is sufficient. The drains of sport fields and cemeteries generally are cleaned every year, because the repair of these drains is very expensive and sometimes not possible at all.

6.6 Problems with subsurface drainage

The installation of a subsurface drainage system is a more or less expensive affair. So a drainage system is carefully prepared and installed.

And we may expect that the system functions well. Nevertheless there could be already made some mistakes at the installation. Besides some disruptions could occur sooner or later after installation. Defects which can occur during the installation of the drainage system are:

- The drains are not connected to each other. It is possible that wrong connection-pieces have been used.
- Broken drains. The drains can already be broken at the moment that they are laid in the ground. It is also possible that the drainage machine shoves away and disturbs the drain.
- The drains are not laid level. When the drains are not installed level or straight, the reinforced hose cannot enter the drains sufficiently resulting in only a part of the drainage system being maintained.

Defects which can occur sooner or later after installation are:

- Silting up with soil particles. The danger of silting up with soil particles is the greatest directly after installation of the drains, when the surrounding of the drain has an instable structure. A good deal of silting up with sand occurs in fine-sandy soils.
- Clogging by iron. The deposition of iron takes place in the drain-trenches, the envelope or cover materials or the drains, especially at the outlets. This results in an increase of the entrance resistance and so the drains do not function sufficiently.
- Growth of roots in the drains. The materials in the drain trench often has a good structure for the growing of roots. When the roots reach the drain they will clog the drain and the drain will not function sufficiently any more and sooner or later there will be no discharge at all.
- Broken drains at the outlet. This problem arises when mowing the slope of canals and ditches. The drains are disturbed and the drain discharge decreases or even ends.
- Disturbance by building activities. In many cases the drains are disturbed by pile driving, the construction of sewers or the installation of cables and conduit pipes.

7 CONCLUSIONS

- When the drains are lying deeper the costs of installation and repair are increasing.
- To prevent disruption by cables and conduit pipes the drains have to be laid deeper than 1.2 to 1.4 m below the soil surface.
- To get an optimum functioning of the drains, the draitrenches have to be filled with well-permeable material, for instance sand.
- When the canals or ditches are not widely spaced and consequently the sewers are not lying at great depths it is possible to lay only a crosswise drainage system. Practically this system can only be disturbed by pile driving. In view of the greater possibilities of discharge by way of intersections, the crosswise drainage system has a good chance to survive. It is necessary that the space between the upper and the lower system is filled up with gravel.
- To keep a well-functioning drainage system regular maintenance of the drains is necessary. In other words the drainage system must have capability for being maintained.
- Until now there is made no allowance for juridical aspects with regard to the subsurface drainage. A possibility to make allowance for these aspects is to drain public areas as far as possible independent on private areas.

REFERENCES

- DAM, C.H. VAN AND J. SCHOTKAMP. Infiltratie in bestratingen. R.IJ.P.-report 1983-Abw; IJsselmeerpolders Development Authority, Lelystad.
- DIJK, W. VAN, H. HENGVELD AND A. OVERWATER. Onderzoek naar de wijze van bouwrijp maken in twaalf plaatsen in Nederland. Flevobericht no 135; IJsselmeerpolders Development Authority, Lelystad 1978.
- HEBBINK, A.J., a.o. Evaluatie drainage in Lelystad en Almere. Flevobericht no 240; IJsselmeerpolders Development Authority, Lelystad.
- HEBBINK, A.J. AND H. DE ROOS. Drainage van stedelijke en recreatieve gebieden in de Flevopolders. R.IJ.P.-report 1984-34abw; IJsselmeerpolders Development Authority, Lelystad.

- KREMER, R.H.J. and E. SCHULTZ. 1978. Urban groundwater: Usage impediments and administration. Proceedings and information no 24. Committee for Hydrological Research TNO, The Hague.
- SCHOLTEN, J. and G.A. VEN. Drainage materials survey in the IJssel-meerpolders. R.IJ.P.-report, 1984-28abw; IJsselmeerpolders Development Authority, Lelystad.
- SEGEREN, W.A. AND H. HENGVELD. Bouwrijp maken van terreinen. Stichting Bouwresearch 99; The Hague 1984.
- VOORTMAN, B.R. De maandelijkse waterbalans van een woonwijk en een parkeerterrein in Lelystad. R.IJ.P.-report 1984-35abw; IJsselmeerpolders Development Authority, Lelystad.
- WERKGROEP BOUWRIJP MAKEN. Bouwrijp maken in de Flevopolders. R.IJ.P.-report, 1981-1abw/ad; IJsselmeerpolders Development Authority, Lelystad.

MANAGEMENT OF GROUNDWATER; A DAILY JOB IN OUR OLDER CITIES

R.H.J. Kremer

ABSTRACT

The lowering of the groundwater table caused by the leakage of sewer mains may cause damage to existing buildings. In Amsterdam a groundwater monitoring system has been installed to avert this damage. The information acquired from this system is also very valuable pertaining to problems caused by excessive groundwater. In recent years Amsterdam has been frequently confronted with these problems and a number of cases have been investigated. The individual solution that has been applied in each circumstance is dependent upon the particular source of the problem. From the cases described in this paper, the conclusion may be drawn that groundwater problems may arise in old city centres as well as in new districts. Eliminating the crawl space would mean a better guarantee for dry houses.

1 INTRODUCTION

The management of groundwater is poorly regulated, this in contrast to the management of surface water. A series of laws, regulations and by-laws is available, which has been developed by successive generations. Particularly, the recent Pollution of Surface Waters Act has to an important degree contributed to the solution of the problems concerning groundwater in the deeper aquifers in Amsterdam. In the sixties local industries withdrew about 25 million m³ groundwater from deep wells at a depth varying from 60 to 100 m.

The lowering of the piezometric level of the third sandlayer (fig.1) seemed likely to cause damage to building in course of time. In 1983, even before the enactment of the Groundwater Bill this amount was reduced to about 7 million m³, which as a result has greatly reduced the probability of damage.

This paper will discuss the subject of phreatic groundwater. The Groundwater Act which has been in force for more than one year now, does not deal with this subject. Drainage and dewatering of land has been excluded from this law, which implies an exclusion of the management of phreatic groundwater as well. This stems from a lack of insight in the behaviour of this groundwater as well as a lack of experience for its management with respect to technology, costs and quality. However, during the last few years Amsterdam has been so frequently confronted with problems caused by groundwater, that the need to investigate the causes and to find solutions became very pressing.

A study of some of these cases may possibly contribute to a better understanding of the behaviour of groundwater and eventually even to the prevention of groundwater problems in the future.

2 THE CHARACTER OF THE SUBSOIL AND THE GROUNDWATER

The Amsterdam subsoil shows a greatly stratified profile in which water bearing sandlayers and impermeable clay- and peatlayers alternate. The first, second and third sandlayers are separated from each other by clayey sand (Alleröd) and clay (Eemien) and were all deposited during the Pleistocene. The height of the piezometric level that is found in these confined aquifers is some meters below the phreatic groundwater table. This is the result of the drainage works of several low lying polders in the neighbourhood of the city. However, a rise has been noticed following a considerable decrease of deep water extraction in the last few years.

Above the sandlayers, lying at a depth of about NAP - 12 m (40 feet), holocene clay- and peatlayers with extensive sandy marine deposits at places are encountered. From the founding of this city these layers have been gradually covered with a fill in order to elevate the building sites to a higher level than that needed for agricultural work and to an elevation greater than the flood level of the former Zuiderzee.

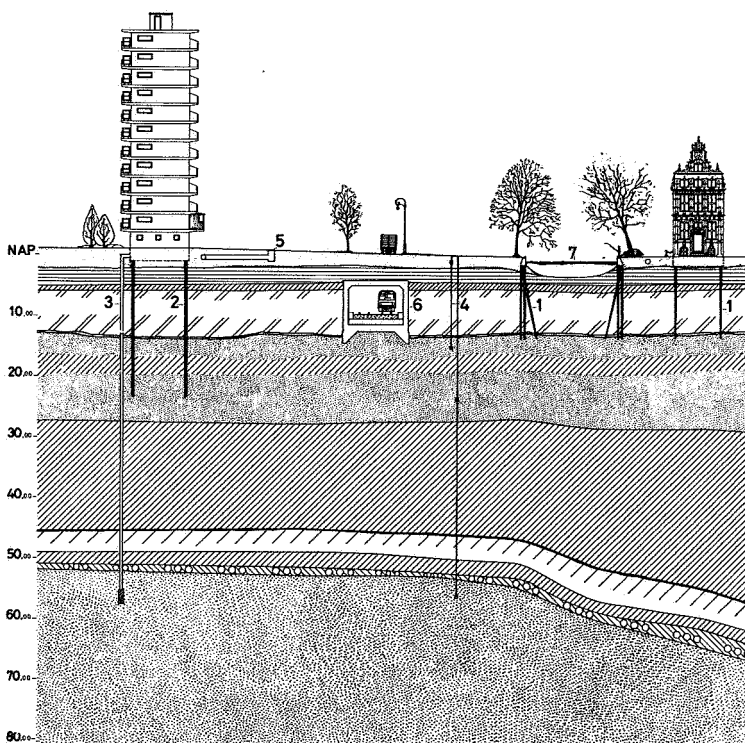


Figure 1

1. Timber pile foundation in so-called first sandlayer
Average foundation level NAP - 13 m
Load capacity between 50 and 80 kN
2. Concrete pile foundation in second sandlayer
Foundation level between NAP - 17 m and - 24 m
Load capacity 200 to 1000 kN
3. Well points to extract groundwater from third sandlayer
(e.g. for cooling)
Depths up to 120 m
4. Stand pipe piezometer for regular observation of ground-
water levels
5. Sewage system
6. Underground railway founded directly on first sandlayer
7. Average city water table NAP - 0,4 m

The phreatic groundwater regime includes both the holocene layers and the sand fill. On account of their small permeability the holocene layers are of little importance to the drainage of groundwater. Both the composition and the thickness of the sand fill show great variations.

In a number of districts man has built on 4 à 5 m well draining sand brought in from the moorlands, but in many cases peat and clay originating from canals and covered with a thin sandlayer were used. Especially in the last case the groundwater flow is influenced by the type and extent of the buildings, because the foundation level extends into the "mud"layers. If such is the case where a number of apartment blocks enclose a central area, a kind of "bathtub" effect occurs in the inner court area. In a number of areas only the street levels have been raised with fill while the gardens have been maintained at the polder level. The water table in these areas is maintained by means of separate polder drainage sewers, which have replaced the original ditches.

With the preparation of building sites the street elevation was chosen at a level of about 1,10 m above the surface water level, in both the old inner city centre and the expansion areas. These average water levels were considered to be the standard groundwater elevations in connection with the construction of timber foundations.

The Amsterdam Building By-Law fixes the pile butt level at 0,20 m below the standard groundwater level. However, foundation inspections show that foundation wood is found often at a higher level, even above the surface water table.

An investigation into the variations of the groundwater level that occur in some of the districts has shown that relatively large seasonal fluctuations of 1,0 à 1,2 m are not uncommon. These fluctuations are barely influenced by the surface water of canals. Safety margins in the butt levels of timber foundations (top of pile) are therefore small or nonexistent so that the possibility of wetting and drying of the pile head due to the fluctuation in the level of the groundwater is very real.

To drain building sites in Amsterdam no drains have been applied. Drains are usually placed only in sport fields and the play grounds of parks. Since 1970, horizontal prefab drains at 2 m depth have been

used instead of the ditches and trenches usually applied for the reclamation of sites. This type of drainage is only meant for use during the construction phase and is not maintained after the construction. It is assumed that after the construction of sewers and roads a sufficiently low groundwater level will be established. In connection with construction of the suburbs of south and west Amsterdam this method performed as expected. There were but seldom complaints of problems due to excessive groundwater. In § 7 the fact will be discussed that under certain circumstances problems do arise. Whenever in the older districts main and secondary sewers are renewed, nowadays a drain is often placed adjacent to the new sewer in the excavation trench in order to replace the drainage function of the old and often leaky sewer.

3 GROUNDWATER MANAGEMENT - A START?

Two elements associated with the city infrastructure can bring about an acute disturbance in the normal groundwater level, namely water conduit pipes and sewers. I will go into the matter of water pipes in § 6. Sewers will cause drainage of the surrounding areas in the cases where pipes and connections are not constructed completely water-proof. The leakage of deep lying sewers, usually the trunk and main sewers, can lower the groundwater table substantially and as a result may cause damage to timber foundations (fig.2).

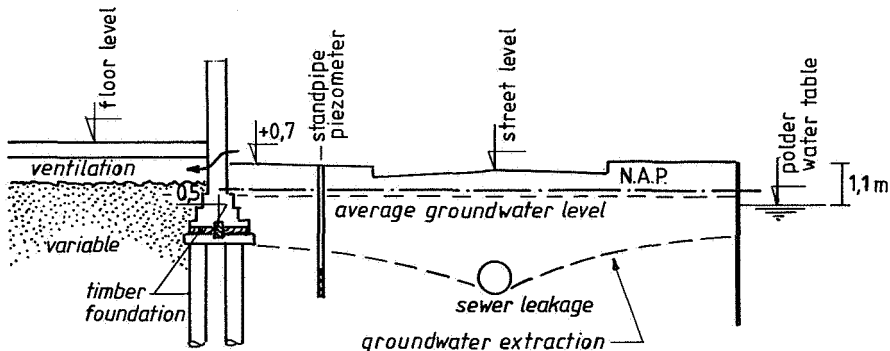


Figure 2 Groundwater extraction as a result of sewer leakage

In Amsterdam a groundwater monitoring system has been installed along side the sewer system. In about 2000 stand pipe piezometers along the

trunk and main sewers the piezometric head is registered. The measurements are compared with a warning level and excessively low groundwater levels are detected. The major lowerings are selected and they make the basis for a program of inspection. Regularly the measurements are so low that the inspection indicates that repair of the sewer is necessary. It has been proved that the effects of repair can be checked by means of the piezometers very well. The primary function of the monitoring system is to prevent damage since detection and repair of the leakage can be carried out very quickly. At the same time the efficiency of the sewage system is improved since the transportation and the purification of a considerable amount of groundwater is prevented. Moreover the understanding of the groundwater regime is improved by the monitoring system. Much data is acquired concerning the different groundwater levels in various areas as well as the seasonal fluctuations in these areas. This gives the opportunity to predict changes in the groundwater regime. Although this monitoring system contributes to groundwater management, it has not been installed for this reason, it has been primarily installed to prevent damage that may be caused by the malfunctioning of a municipal construction. This distinction becomes quite clear from the fact that a proposal for a general groundwater observation system distributed over the city has failed to get the approval of the municipal board.

4 GROUNDWATER CONTROL AND TOWN RENOVATION

In the Amsterdam district "Indische buurt" which was built around 1925, many houses are being renovated while other houses are being replaced. A quality survey on structural aspects shows that in the larger building blocks settlements have occurred in the order of 0,2 m to 0,6 m during these last 60 years. Nevertheless the decision was made to renovate these houses as no large differential settlements are expected to occur within the next 25 years. The renewal of the district also comprises the reconstruction of services and roads, including the raising of the latter to the original design level. The data from the sewer monitoring system showed lowered groundwater levels in several streets varying from 0,75 m to 1,50 m. A large number of leaks was found during the inspection of the sewers making

a prompt and extensive repair and renewal project necessary.

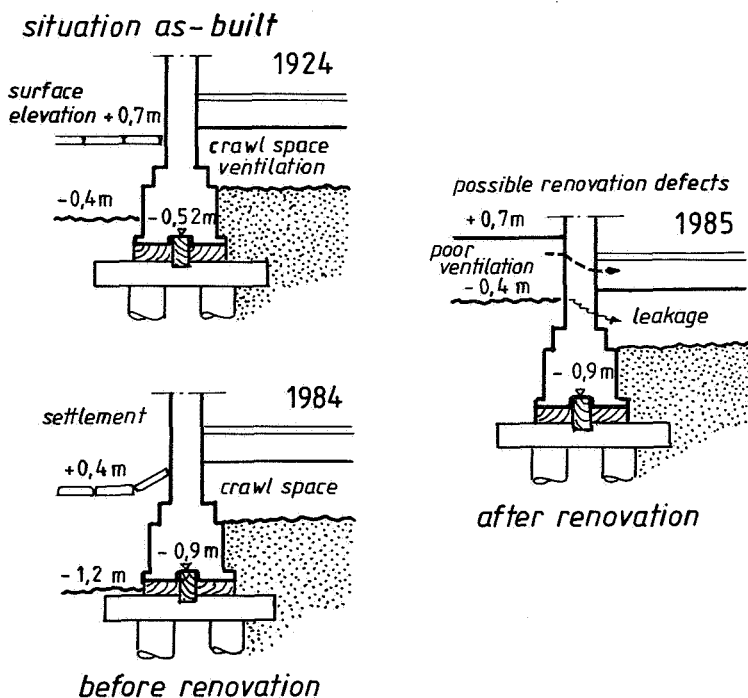


Figure 3 Foundation as-built, before and after renovation

From the above mentioned facts the following may be inferred (fig.3):

- the ground floor level is now far below street design level;
- the crawl space has settled under the "normal" groundwater table in some places;
- the pile head level of the timber foundation is only 0,1 m under "normal" groundwater in some places.

As a consequence of the renewal of the sewers the average groundwater table will rise to an average level of NAP - 0,4 m. For that reason the groundwater will inundate a number of crawl spaces. If these spaces would be kept dry by means of horizontal drains, the result would be the drying out and rotting of the timber foundations. On similar occasions a solution has been found by making connections in the crawl spaces and the installation of a waterpump in a collector basin at the lowest part of the crawl space.

The raising of the street level to the original design level is not

desirable in these circumstances. If the streets were raised, the ground floors would be below street level and furthermore the ventilation shaft of the crawl spaces would disappear under the pavement. The blockage of the ventilation opening would lead to an extreme relative humidity which will cause a process of wood rot in floors and dampness in the walls.

It is essential for the process of town renovations that a detailed survey of existing and future groundwater tables is available as well as data concerning foundation and crawl space levels. On this basis the necessary provisions can be made and the new design levels for the pavement determined.

5 SPAARNDAMMER DISTRICT, INCONVENIENCES CAUSED BY GROUNDWATER

It was October 1979 when the first complaints of serious problems in connection with groundwater were registered. Specifically a number of apartment buildings in Zaanstreet and Spaarndammersquare were involved. In recently renovated houses, plaster work was damaged by excessive moisture and dampness, possibly as a consequence of groundwater that had been absorbed from the foundation. Measurements of the groundwater level showed a remarkable raise to about 0,5 m below surface.

The Spaarndammer district is situated north-west of the city centre (fig.4). The railway Amsterdam-Zaandam, the Spaarndammerdike and Spaarndammerroad form the boundaries of this district, the latter being part of the high water dike along the river IJ. In this area, which was built around 1910, in most cases town renovation implies renovation of the old buildings.

Drainage

There are no canals or other surface waters in the district itself. As a result the area drains to the river IJ and to some city canals outside the district at rather large distances. The phreatic groundwater flow is hampered by the high water dike. The average water table of the canals is NAP - 0,4 m. South of the railway a polder area still exists where the water table is maintained at NAP - 2,15 m. The sandfill of about 2 m thick, placed during the reclamation period, determines the flow directions of the phreatic groundwater.

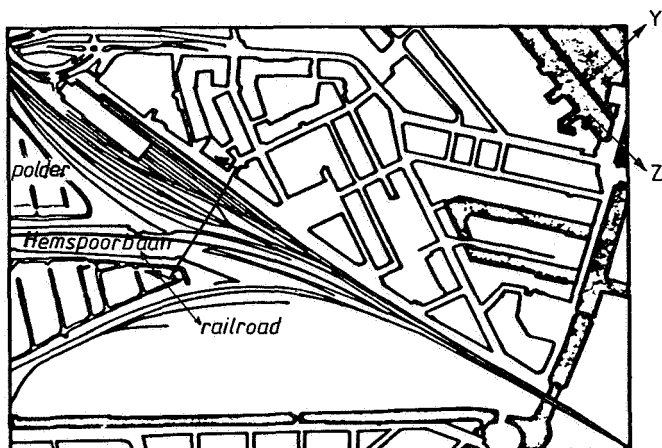
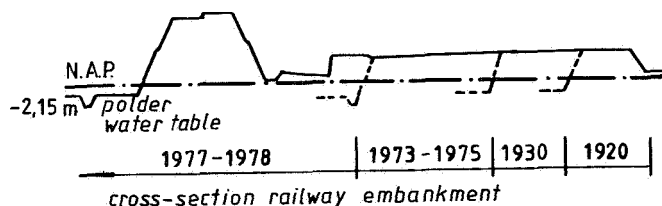


Figure 4 Plan Spaarndammer district

As shown in fig.5 remarkable raises in piezometric head have occurred in the years 1974/1975 and 1977/1978.

Possible origins of the problems

Firstly for the investigation of the origins of the problems, associated with excessive groundwater, the data on precipitation and evaporation have been collected. The period of measurement showed notable differences. The poor correspondence between the net infiltration and the groundwater level could not provide an explanation. Moreover these climatic effects had not caused any significant changes in groundwater levels in other quarters of the city.

During the period 1969 to 1980 a great number of sewers was renewed. In Zaanstreet and the surrounding area the most important renewals took place in 1969 and 1972. The graph of changes in the piezometric head with time (fig.5) shows no effects of the sewer construction

work as the piezometric head remains reasonably stable until the winter of 1973/1974.

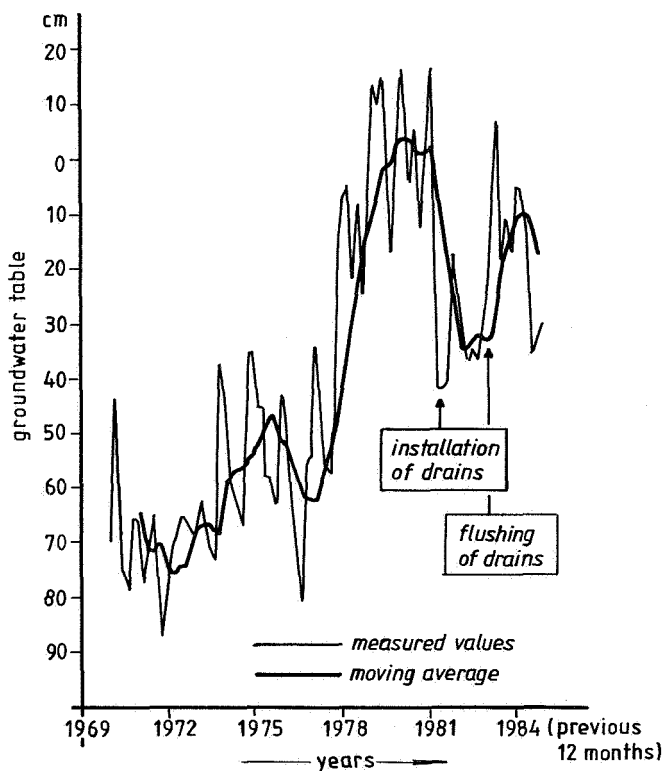


Figure 5 Changes in piezometric head with time

Finally the possible influences of the railway embankment were investigated. Changes in the flow pattern of groundwater became evident. The maximum rise had been effected in the western part of the area near the embankment and in addition the direction of flow was completely reversed.

Up until 1977 the groundwater flow was directed to the railroad, but after the strong rise of the water table during the winter of 1977/1978 the groundwater drained from the embankment into the housing area. In 1977 the railway yard was extended and during the period 1977-1980 the "Hemrailroad" was constructed. As a result the width of the embankment was enlarged from 80 m to 260 m.

Another consequence was the increase in distance between the housing

area and the deeper lying polder south of the railroad. The combination of enlargement of the embankment and the increase in drainage distance had important hydrological consequences for the area. In the new situation large quantities of groundwater drained into the housing area.

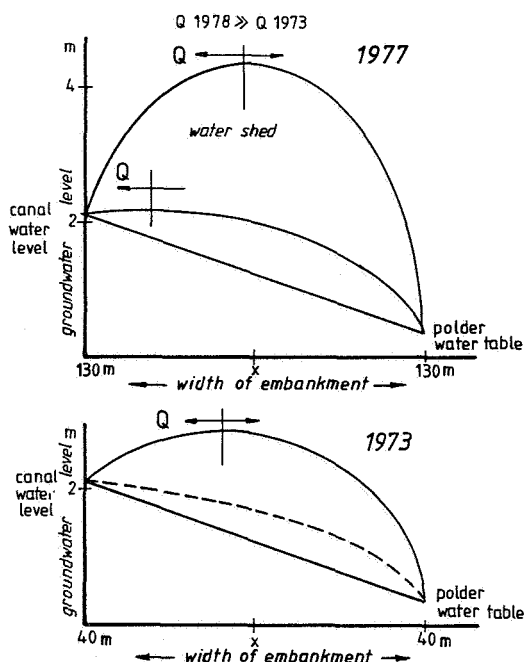


Figure 6 Groundwater flow railway embankment 1977 compared to 1973

Remedial measures

As soon as the hydrological analysis indicated that the groundwater problems originated in the enlargement of the railway embankment, it was decided to install drainage facilities along the railroad. The first section was installed in January 1981. It soon proved to be effective and consequently additional sections have been installed. The groundwater table was lowered to the normal level of approximately NAP - 0,4 m (fig.5). Clogging of the drain reduced the discharge capacity relatively quickly and effected a new rise of the groundwater. Flushing by a high pressure water jet was carried out in November 1983 (fig.5) and proved to be effective.

Although the groundwater level was lowered, the amount of moisture and dampness in the basement and ground floor walls remained excessive.

New consultations and inspections brought to light that the ventilation of the crawl spaces was so poor that the relative humidity was about 95%. A reduction to approximately 60% was obtained by the construction of extra ventilation apertures. In addition a non-renovated house proved to be dry while in the neighbouring one the renewed plaster was wet and mouldy. Specific measurements in the plaster and the brick work showed the plaster itself to be the origin of the trouble. The plaster was hygroscopic. The solution was found by removal of the damp plaster work on the ground floor. This meant a great relief for the residents and for the housing association as well. If the problem had originated in the capillary absorption of groundwater by the brick work, it would have been necessary to cut the walls and put an isolating layer in the cuts. Such an operation would have been very costly and would have been a real nuisance.

6 COMPLAINTS ASSOCIATED WITH GROUNDWATER PROBLEMS

Nowadays complaints associated with groundwater problems can be noted in the newspapers more and more frequently. In most of the cases the authorities are blamed for lack of attention to the situation. In some cases the municipality has been sued for damage by residents. The city of Amsterdam also receives complaints more frequently nowadays.

A summary of the complaints in recent years is given below, including their causes and the solutions selected. Most of the complaints concern problems in houses ranging from damp and mouldy walls, stuffy rooms, foul smell and some water in the crawl space, to basements with water pouring in as if all taps were open.

The complaints are received by the following municipal services:

- The Municipal Water Board which has a special crew for leakage investigation and repair
- Public Works, Department for Sewerage and Water Management that has a specialized staff for cleaning and repair of sewers, or the Department for Soil Mechanics and Hydrology that has a large number of piezometers for groundwater monitoring and that can assist in

the investigations by making chemical analysis

- Building and Housing Department that assists tenants with all kinds of problems with respect to their houses

To prevent citizens from being treated like a rubber ball bouncing from one office to the other is not easy. Close cooperation of the municipal departments involved is necessary in order to resolve the problem. Often the investigation takes a lot of time and effort since frequently the source of the trouble cannot be found in the house of the person complaining.

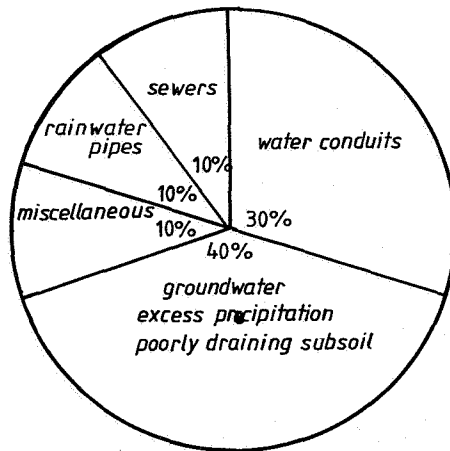


Figure 7 Complaints associated with excess groundwater

The 1982 review on groundwater problems shows that 30% of the cases were caused by leaks in the water distribution system (fig.7).

The leak investigation crew of the Water Board detects most of the leaks, but sometimes this method fails as illustrated below.

In May 1981 a number of basements were inundated and gardens were flooded in the Van Breestreet. Initially, the source of the problem was expected to be found in the special drainage sewer for this district. The area is still part of a former polder with a separate pumping station. As no piezometers were available in this area the sewer itself had to be checked. In the meantime water analyses were made, the result of which indicated leakage of tap water.

Since a leak detection investigation had already been carried out with no result in the houses of the tenants who initially complained

as well as in the neighbouring homes, the Water Board had to be pressed for a second inspection. This time the crew successfully located the leakage source. It became apparent that the whole neighbourhood had been flooded by the leakage of one single house connection. During the same inspection tour another problem was solved. In the adjacent building groundwater had been a problem all winter. Foul smelling water had permeated the gardens and in November 1980 the leak detection crew had jumped to the conclusion that the problem originated in blocked sewers. However, after repair of a small private sewer pipe and subsequently the repair of the rainwater pipes of the adjoining church, the water table was still very high. In connection with the second inspection in the other building, this block was investigated again more thoroughly. Also in this case the origin was a leak in one single service pipe.

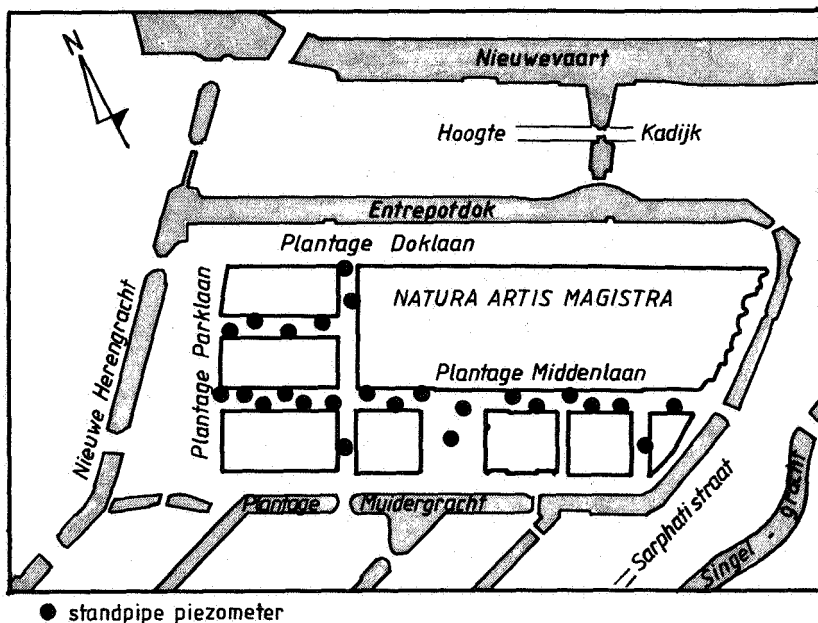


Figure 8 Plan Plantage district

During the same period a large number of complaints were received from the Plantagequarter where the main road, Plantage Middenlaan, had recently been renovated (fig.8). The road had been resurfaced, all of the cables and service pipes renewed and a large masonry sewer

replaced. Many inspections and measurements were performed by the Sewerage Department as well as by the Water Board, but without any result. The piezometers indicated a high groundwater table in some areas, but the water analyses gave no information on the possible origin. In many cases the chlorine content gives a good indication of the source, but this time values ranging from 10 to 1300 mg/Cl⁻ per litre were measured and furthermore during a period of several months large variations in different piezometers were noted. Gradually the investigation revealed that the groundwater situation had been a real concern during the road reconstruction, but also that it had not been much better for years. The residents had finally stopped complaining because the source of the trouble could not be found. Several tenants had already installed a small pump in the basement of their houses.

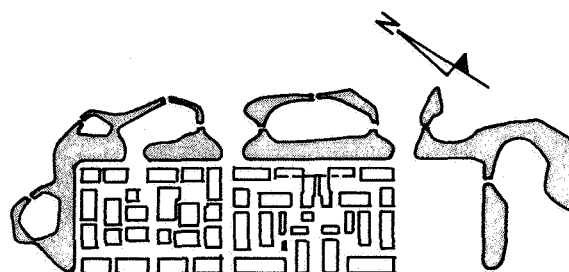
In this case the trouble was basically caused by the applied system of land reclamation. About 1682 the area was developed as pleasure ground and for that reason peat and clay were deemed suitable as fill materials. When house building was permitted in 1858, the area was filled with approximately 0,5 m sand. Even though the distances to the canals are relatively small, this thin sandlayer does not permit adequate groundwater drainage. Moreover the former pleasure ground surface at approximately NAP level is almost impervious and prevents the groundwater from draining below this level.

To obtain practical information on the effectiveness of drains an in situ test was performed. At three locations a drainage pit filled with gravel was constructed in the road and connected with the sewer. In the pits the water table was lowered to the mean level of NAP - 0,4 m, but the radius of influence was only 1,5 m to 2,5 m. From this it was quite clear that installation of drains along the road would not result in drainage at the houses and gardens. The only way to keep the houses dry is by waterproofing or pumping them individually.

7 GROUNDWATER INCONVENIENCES IN MODERN DISTRICTS

As mentioned in § 2, no drains are installed in new housing areas and in the modern outlying districts of Amsterdam hardly any groundwater inconveniences are encountered.

In the Geerdinkhof district of the Bijlmermeer some problems have occurred. The municipality is in somewhat of a dilemma in this case as the high groundwater level was not caused by any municipal activities (railway construction, sewer renovation etcetera) and the situation does not give cause for the Building and Housing Department to step in.



Plan-view of Geerdinkhof

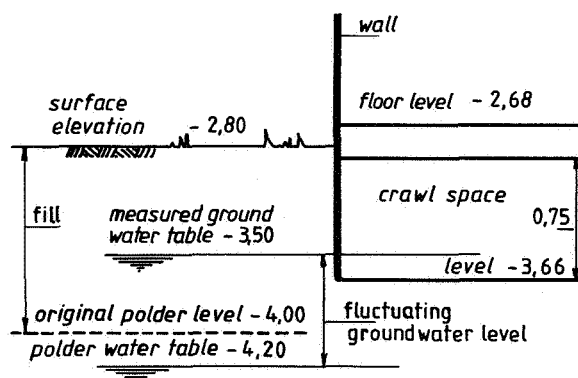


Figure 9 Plan-view and cross-section at ground level Geerdinkhof

Geerdinkhof is a small district with family houses situated on the outskirts of the Bijlmermeer (fig.9). This deep polder was reclaimed by a hydraulic fill with a sandlayer 2 m in thickness varying to a minimum of 1,70 m at the periphery of the polder. The water table has been maintained at the original level of NAP - 4,20 m and the roads were constructed at NAP - 2,80 m. The difference in height of 1,40 m is a little more than normal. Geerdinkhof is surrounded by newly dug canals on three sides, so that drainage should not be a problem.

In 1977 however, shortly after completion of the housing project, the residents complained of problems due to high groundwater particularly in the crawl spaces. They asserted that the high groundwater level was caused by some fault in the process of landfill. That is why they demand an improvement of the drainage facilities.

The municipality maintains that the high water table will be lowered in due time as a consequence of the construction of roads and sewers. Moreover the municipality has stated that basically the origin of the problem is to be found in the construction of the houses. In other words the work has been carried out without taking into account the natural groundwater situation. From an investigation into the groundwater level in the Bijlmermeer it has been found that in many places the water table is 0,50 m to 0,75 m higher than the adjacent canal level. A possible cause for the higher water table could be due to seepage from the first sandlayer as the water table in the polder is approximately 2 m below the piezometric head in this layer. From the technical point of view no design errors have been made in the land reclamation work or in the construction of roads and sewers. If this is the case with the construction of the houses remains to be seen. The construction has been carried out in accordance with the building regulations as the ground floor is considerably more above the normal groundwater table than the required 0,45 m. In case of a timber floor the construction of the crawl spaces would have to meet several specifications on leakage and ventilation, but with concrete floors the specification does not apply. Even the usual concrete slab to cover the subsoil was abandoned for economic reasons. Nevertheless this omission is undesirable as the corrosion of service pipes may be accelerated and the water will attract vermin to the crawl space. Generally speaking drainage is the appropriate measure for lowering groundwater. When applied in this case the drains would have to be installed at NAP - 3,90 m to result in dry basements, taking into account 0,20 m capillary rise in the sand fill (lit.1). It is evident that the drains have to be situated adjacent to the houses. If the drains would be placed in the roads, the groundwater will not be lowered enough at the houses which is unacceptable. One may question the application of a traditional crawl space in this case. The use of a crawl space originates with the need for ventilation of

timber floors. The secondary use of the crawl space was for the installation of service pipes and cables. As timber floors are used infrequently in modern housing, the application of crawl spaces should be reconsidered both from the technical and the economical point of view.

In the Geerdinkhof case the crawl spaces are constructed in such a way that as a consequence the drainage facilities should meet very high standards. For that purpose large investments are necessary for the installation and maintainance of the drains.

In consultations between the Building and Housing Department an the Public Works Department some new projects have been reviewed to determine what the consequences will be with respect to the installation of pipes and cables in case of building without crawl spaces. It has been concluded that proper solutions without extra costs are available.

In addition it should be mentioned that in the report "Crawl spaces, yes or no?" edited by the Ministry of Building (lit.2) it is concluded that abandoning crawl spaces leads to cost saving solutions as the thickness of the insulation layer on the ground floor can be reduced.

According to this case history of a housing district on newly reclaimed ground and according to the recent report "When it rains, what will happen?" (lit.3) Geerdinkhof is not an isolated case, so that the conclusion may be drawn:

Building without a crawl space a better guarantee for dry feet!

REFERENCES

- 1 SEGEREN, W.A. and HENGVELD, H., 1984. Preparation of building sites. Report nr. 99 Building Research Foundation, Kluwer Technische Boeken B.V., Deventer.
- 2 "Kruipruimte, wel of niet?", 1983. Report Ministry for Building.
- 3 BORREMAN, N. and GEERTSE, N., 1984. Als het regent houden wij ons hart vast. Report nr. 7 Series on civil town planning, Technical University, Delft.

GROUNDWATER CONTROL: LEGAL ASPECTS

F.J.G. Gerritsen

ABSTRACT

The array of legal instruments regarding groundwater management has been split up. The existing regulations allow the municipality sufficient freedom to draw up regulations as to phreatic groundwater level control.

The municipality is free to include active groundwater management into its "household". There is no corporate duty, however, for the municipality as to manage and control of the groundwater level as such.

If the municipality will proceed to an active groundwater level control, private persons suffering damage as a result of that active management may successfully appeal to a court of civil judicature so as to obtain indemnification.

1 INTRODUCTION

In the last few years, also by an increasing number of complaints from the citizens, the attention of the municipalities for the problems concerning the groundwater levels has highly increased. As a consequence, pressure to achieve some kind of municipal groundwater management has increased, too.

The array of legal instruments as to groundwater management has been split up. In the following it will be investigated in a nutshell how much room the existing legal regulations leave for a municipal groundwater management. Successively will be dealt with:

- the Groundwater Act;
- the Provincial Groundwater By-Law;
- the Bill on the Water Management;
- the Building By-Law;
- the Local Government Act.

Furthermore, also by analyzing some jurisprudence, the legal consequences will be further entered into of an active municipal groundwater management in the form of phreatic groundwater level control by means of horizontal draining systems.

2 LEGAL SCOPE

2.1 The Groundwater Act

The Groundwater Act (hereinafter: GWA) from 22nd May, 1981, is meant as a substitute for the Groundwater Act Waterworks. On 1st June, 1982 and 1st November 1983 a restricted number of articles of the GWA came into force. The GWA is operative as a whole from 1st September, 1984.

According to the preamble, the purpose of the GWA is to promote a proper management of the groundwater. It is considered necessary "to make rules as to the groundwater withdrawal as well as to the groundwater recharge for that purpose".

Already in Article 1 of this Act we read that the Act does not apply to the extraction of groundwater as a consequence of de-watering or draining of grounds. Extraction of groundwater is defined in Article 1 as "extraction of groundwater by means of a device".

Article 1, Paragraph 3, opening lines and sub a GWA

This Act does not apply to the extraction of groundwater: by drainage. An exception is made here for the extraction of phreatic groundwater.

In the Explanatory Memorandum (EM) this exception is further entered into:

"Actions (...) that do not have the withdrawal of groundwater for their purpose but do affect also the groundwater level, such as the pumping for level control will be outside the scope of this Act. This does not exclude that based on other considerations derived from local interests there may be a need for regulations as to these actions. The regulating competence of the lower authority continues to be maintained as to this point." (EM, the Lower House 1975-1976, 13705, No. 3, page 33).

So, the autonomous freedom of the local authority to draw up regulations as to groundwater level control is left intact.

A recording and licence duty applies to the non-excluded groundwater withdrawal or recharge.

Article 14, Paragraph 1 GWA

"It is forbidden to withdraw groundwater or to infiltrate water, unless the County Aldermen granted a license for this purpose."

So the County Aldermen are the authorizing body. In pursuance of Article 15 GWA Provincial States can designate by by-law exceptions to the provision in Article 14 GWA.

Article 15, Paragraph 1 GWA

"The prohibition described in Article 14, first paragraph, does not apply to the withdrawal of groundwater in the cases in By-Law designated under Our approval by the Provinciale States."

In pursuance of Article 19 GWA, for the municipality (Mayor and Aldermen) only an administrative and advising task has been reserved.

2.2 The Provincial Groundwater By-Law

A Provincial Groundwater By-Law clearly bears the character of a supplement to the GWA. Because a by-law shall not adopt articles that have already been incorporated in the law, it should be read in connection with the GWA.

Within this scope the contents of the various provincial (draft) groundwater regulations will not further be entered into.

As anticipated the provincial regulations will not provide any further nuances of the exceptive clause mentioned in Article 3 of the GWA regarding the withdrawal of groundwater for draining.

This leaves open the question which technical measures affecting the groundwater levels, such as draining, pumping and infiltration systems (active groundwater management) still are within the concepts of draining. In many cases the GWA will not apply to a horizontal drainage device, in contrast with a vertical standpipe drainage. Within this category, too, a distinction could be made between:

- drainage by gravity;
- drainage via pumped surface water;
- drainage via a pumped sewage system of a pumped well;
- drainage with direct pump suction.

It is not clear whether the provinces will use such a distinction in drainage concepts.

For their interpretation at any rate the objective of the GWA, the exceptive clause in Article 1, paragraph 3 of the GWA and the relation between the GWA and the Housing Act Building Regulation have to be taken into account.

2.3 The Bill on the Water Management

On 29th March, 1982 in the Lower House a Bill on the Water Management has been introduced (hereinafter to be called Bill-WM). The Bill-WM has a twofold objective. On the one hand the Bill provides instruments for a coherent and efficient policy and management in the field of the water management as a whole. On the other hand, it provides rules for the quantity management of surface waters. So, the Bill-WM is both a general Bill to coordinate the water management and a specific Bill for the quantity management of the surface water.

The Bill introduces, in combination with plans introduced by the Pollution of Surface Water Act (PSWA) and GWA, a planning system for the entire field of the water management. At Governmental level a Policy Document on Water Management will, in addition to the Indicative Multi-year Programme for water quality and soil protection, function as a testing frame for the provincial plans that have to be implemented in phases, and for managing instruments.

The Province should draw up three different plans:

- the provincial water management plan, in which has also have to be included the main lines of the quantity management to be conducted in the plan area (with the exception of the National Waters);
- the provincial groundwater management plan;
- the provincial water quality plan.

In addition, the quality manager draws up a quality management plan and the quantity manager - voluntarily or assigned by the County Aldermen - a quantity management plan.

There is an indirect relation between the quantity management plan and the provincial groundwater management plan, as both plans should be effected within the rough scopes of the provincial water management plan. If a municipality is the quantity manager, as e.g. Amsterdam, it will be able to or will have to draw up a quantity management plan in observance of the water management plan and - indirectly - of the groundwater management plan.

For practising quantity management, the Bill-WM provides the array in the form of a system of licences, the "water agreement" and a duty to register. A duty to license only applies in cases assigned for the purpose of drainage of extraction of water on or at the water surface, respectively. The assignment for National waters will be given by General Administrative Order and for other waters by a provincial by-law that has to be approved of by the Crown. To what extent the system of licences of the Bill-WM will affect groundwater level control by means of quantity management of the surface water, is still unclear. Maybe the assignment will clarify this.

2.4 The Building By-Law

Most municipalities based their Building By-Law on the model-Building By-Law of the Vereniging van Nederlandse Gemeenten (= Association of Netherlands Municipal Corporations). A number of paragraphs of this model By-Law concerns the groundwater level. In most cases they contain provisions to be made (e.g. soil enclosure) or requirements that for buildings, yards and sites the groundwater level has to be taken into account.

From Article 383 of the model Building By-Law "Safety measures in building, maintaining and demolishing of buildings" it can be derived that the municipality is involved in the maintaining of the groundwater level: "When building trenches are pumped, water must not be extracted from the ground in such a way that the groundwater level in the surroundings is lowered, so that building foundations might be damaged."

This article stresses that a certain way of groundwater extraction is forbidden; a fine is imposed on offences. In this respect the Amsterdam Building By-Law has been formulated even more stringently. The clause: "When building trenches is lowered, so that", is there replaced by "Water must not be extracted from the ground in such a way, that,".

On account of articles in the Building By-Law the municipality can only act as a guardian of groundwater levels by making writs on the provisions of Article 25 of the Housing Act, to the owners of the buildings or sites concerned to take measures against infringements of these rules.

This way of approaching may, however, not be expedient since costs involved in enforcing administratively what could be considered as a private offence, could come to the municipality's expense. As appears from the jurisprudence, integral redress of costs is undesirable, when the writ is not only due to the condition of the dwelling or the way of habitation but concerns also a matter of public interest (Department of Jurisdiction "Council of State", 23-08-1983, Gemeentestem (a Trade Journal), No. 6763, page 77). This would apply when there is ground-

water nuisance over a larger surface area (e.g. a residential district). Here lies a question of public interest, making other measures probably preferable above those on the strength of the Building By-Law.

2.5 The Local Government Act

In the Local Government Act groundwater is not mentioned. Yet within this scope a number of articles are of importance.

On the strength of Article 168 of the Local Government Act the town council is authorized to make rules: "The town council makes the By-Laws that are required in the interest of public order, morality and health, and others relating to the municipal housekeeping."

Basically, a municipal groundwater by-law (passive groundwater management) would be among the possibilities on the strength of Article 168, on the understanding that this has already partially been provided for by law and higher By-Law (GWA, provincial By-Law). Article 193, paragraph 1 of the Local Government Act ordains that local By-Laws must not step into matters that are of general governmental or provincial interest.

In Article 3 the GWA declares not to apply to "other types of withdrawal for drainage". There would be no conflict with Article 193, paragraph 1 of the Local Government Act, if the council would make a by-law that would lay down rules for the said groundwater extraction (e.g. drainages). Taken for granted, of course, that this by-law would meet the criteria of Article 168 of the Local Government Act.

The above does not yet solve the question whether the groundwater management falls under the municipal housekeeping, i.e. to what extent the municipality is bound to undertake the active or passive management of the groundwater.

The Local Government Act does not answer this question. The customary conception is that as long as a higher authority has not assumed a certain matter, the municipality is free to include it into its housekeeping. So, there is no obligation to conduct a groundwater management.

If the municipality should include in its housekeeping an active groundwater management, e.g. in the form of controlling the groundwater level by means of drainage systems, this would be based on Article 175 of the Local Government Act: "It (the town council) commands the construction or improvement of municipal roads, water conduits, streets, squares, canals, buildings, works and plants."

2.6 Summary

Municipal groundwater management will have to remain within the legal scopes. In the following a summary will be given of the regulations that have to be taken into account, if active groundwater level management will be practised by means of horizontal drainage systems.

The Local Governmental Act leaves sufficient room for active management of municipal groundwaters. The municipality is free to include the active groundwater management into its housekeeping. Neither will the GWA and the provincial by-laws present many problems as, in general, horizontal drainage systems will not come under the régime of these by-laws. The Building By-Law seems to be an improper instrument for a total approach of preventing high groundwater levels, because it leaves basically the solution of this question to private persons.

A question that has not been mentioned previously, is the discharge of drainage water. If drainage water is discharged on the municipal sewerage, either a licence or an exemption can be required on the strength of a drainage by-law on sewerage. Because groundwater is either a carrier of polluting substances, or it has to be considered as a waste water because of an oxygen deficiency or too high temperature, a license may be required on the basis of the Pollution of Surface Water Act for discharge of drainage to surface water.

In case of discharge to surface waters, whose quantity management is not in municipal hands, requirements can be made by the surface water quantity manager, on the strength of the future Water Management Act (WMA).

3 LEGAL CONSEQUENCES

From the above it may be concluded that the municipality does not have any corporate duty as to the management and the control of the groundwater level as such. Consequently, the municipality cannot be reproached for negligence in respect of damage as a result of fluctuations of the natural groundwater level. Moreover, it seems that in practice the groundwater level cannot be effectively controlled. This vision is supported by a sentence of the President of the Rotterdam Court, given on 15th August, 1969, NJ 1972 No. 265, in which it has been judged that a subjective right to a certain groundwater level cannot be granted and public authorities have no duty to regulate the groundwater level.

Also the sentence of the President of The Hague Court in a summary jurisdiction between a number of inhabitants of dwellings in a location of newly build houses and the municipality of Alphen aan den Rijn (municipality in the Province of South-Holland) is interesting in this respect.

In November and December 1979 a number of newly built dwellings in the building location "Ridderveld II" in Alphen aan den Rijn have been troubled by a too high groundwater level. Only part of the houses in Ridderveld II was at that time inhabited. In eight of the dwellings that had already been occupied, the groundwater rose in the crawl spaces as far as the groundlevel floor. In one house the so-called "leefkuil" (recess in the livingroom) filled, so that the flooring was damaged. In another dwelling the hot-air system that recessed in the floor was damaged. The inhabitants required from the municipality to lower the groundwater level, because the ground bought by them did not meet the definition of "building ground", at that time. Finally, proceedings for a conditional injunction were taken. The arguments of the municipality were among others that horizontal draining pipes could only be effective after a certain time during which the ground of the site could settle. When during this period draining appears to be not effective, the builder should be kept responsible for keeping dry his building trench. Water nuisance, if any, had been of a temporary nature, because the future water management system did not yet function fully; also not all roof-gutters had as yet been connected to the sewer system.

The President of the Court met the arguments of the municipality. Accordingly the municipality was not bound to take measures to lower the groundwater level (in building ground during the building phase) such that the crawl space under the groundlevel floor of the plaintiffs' dwellings would become and/or remain (practically) dry. For the time being the President did not see either that such measures would fall under the corporate duty of the defendant.

In this case the method according to which the site was prepared for building was really tested for its lawfulness. The court decided that the municipality had to demonstrate that it "prepared the building area normally, i.e. in accordance with the standards considered technically justified by experts".

Can the municipality be held liable for damage as a result of an active groundwater level control by means of horizontal draining systems?

This question cannot easily be answered. Here a distinction should be made between damage resulting from an illegal (local) governmental action and that resulting from a legitimate (local) governmental action. In general it can be put that the municipal liability for the carrying out of works, among which draining systems, essentially does not differ from private persons' liabilities.

The general requirements for liability on the strength of Article 1401 of the civil code are: illegitimacy, fault and causality. The illegitimacy of a certain activity or of its ommittance is in practice mostly determined by the question whether the damage could have been foreseen and prevented. This is being judged by following criteria:

- the nature of the activity;
- the extent of the probability of the damage;
- the seriousness of the consequences;
- the extent of the difficulty of taking precautionary measures.

In a situation as mentioned this means that before the municipality starts installing the horizontal draining systems it has the duty to:

- make an investigation into the possible consequences;
- to warn those concerned as to the consequences to be expected;
- to take precautionary measures.

Also when the draining is being executed by a building contractor, the municipality remains, as the principal, liable to third parties as to the duty to warn them and take precautionary measures. This may be illustrated by means of a verdict of the Amsterdam Court, dated 15th April, 1977 (BR, 1978, page 81). The draining of a building trench for the installation of a culvert caused the subsidence of a campshed and a garden partition. Both the implementation and the management were commissioned to third parties by the Principal, a Union of Inland Polders. Nevertheless, also the Union of Inland Polders was held liable and sentenced to indemnify.

Before liability is assumed, there should also exist a causal connection between the damage sustained and the illegitimate activity; therefore it is necessary to show that the damage would not have been caused, if the activity should not have taken place and that it is proved satisfactorily that the damage is the result that may be reasonably expected from the activity.

But what will happen when an active control of the groundwater level can stand the lawfulness test and nevertheless there is damage as a result of this legitimate (local) governmental action?

This comes under the so-called domain of the "administrative compensation". The government takes action in the public interest. These action can adversely affect some citizens. Must the government make good this incidental damage? The judge can only sentence the government to indemnification, if the damage results from an illegitimate action. Already in 1944 the Supreme Court found a solution to this problem in the so-called Dune drinking-water judgement (SC 18-02-1944, NJ 1944, Nr. 226): For the drinking-water supply the municipality of The Hague extracted water from a dune area. As a result a number of market-gardeners sustained damage, because the soil dried up. They successfully conducted an indemnification action on the strength of Article 1401 of the Civil Code. The Supreme Court judged the water catchment in itself admissible, but the non-simultaneous presenting of indemnification illegitimate.

If the municipality proceeds to an active groundwater level control, private persons who may sustain damage as a result of that active management may successfully appeal to a court of civil judicature to obtain indenmnification, regardless whether this damage results from either legitimate of illegitimate governmental activities.

REFERENCES

- BORREMAN, N., GEERTSE, N. 1984. When it is raining we hold our breath, the issues of a high groundwater level in urban areas, a stock-taking (in Dutch) Publicatiereeks Civiele Planologie No.7, Department Civil Physical Planning, Delft, University of Technology.
- GEMEENTE ZAANSTAD, 1981. Groundwater levels part 2 - managerial aspects and liability (in Dutch), Zaanstad.
- SCHIPPER, G.M. de, 1984. Administrative Compensation (in Dutch). Tijdschrift voor Overheidsadministratie, 12.

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