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METHODS FOR CALCULATION OF ANNUAL AND EXTREME OVERFLOW EVENTS FROM COMBINED SEWER SYSTEMS

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ABSTRACT

Overflow from combined systems constitute an increasing source of pollution of receiving waters, as compared to daily wastewater discharges which undergo treatment to a still higher extent. The receiving water problems from overflows are significant both in a long term scale (mean annual load) and in a short term scale (extreme event load). A method for computation of both annual and extreme load is presented. It is based on historical rain series and the use of a time-area model and simple pollutant mixing model in runoff calculation. Statistical calculations for both mean annual load and extreme events have been applied to the computed overflow series. Based on the computerized method simple manual calculations methods have been developed, resulting in graphs and tables for annual load and extreme load.

KEYWORDS

Historical rainseries; urban runoff; combined system; overflow; basin; pollution; time-area method.

INTRODUCTION

Ultimately the use of historical rain series in overflow calculations calls for the most complete runoff models. In practice the expenses in using these advanced models will not be justified due to the uncertainties in the description of the catchment area, and knowledge of receiving water reactions. In order to achieve operational methods simplifications must be introduced consistent with the nature of the problem. A staged approach can be used (Harremoës, Jensen and Johansen, 1983). A basic model in the staged approach for overflow computations, is described in this paper.

OUTLINE OF THE PROBLEM

In most combined sewer systems overflow structures form an integrated part of the system. This is a result of system expansions from outlet of untreated wastewater to the nearest receiving waters to centralized treatment plants and construction of interceptors. To avoid excessive costs in interceptor construction and to protect the water treatment plant from shockloading during rain events overflow structures are necessary parts of the system. Recent investigations, Lager and others (1974); Hogland and Niemczynowicz (1979) have shown that the preliminary assumption of negligible pollutant load from these structures cannot be justified. This applies to both the mean annual load, causing eutrophication and the extreme shockloads, causing oxygen depletion (Field, 1981; Harremoës, 1981).

The main objective of water quality planning projects is to quantify and predict the impact on the receiving waters from present and future outlets. In this scope the exact figures for the discharge are necessary but intermediate remedies. Because different types of receiving waters react differently to the same discharge of pollution, the computation of water and pollutants discharge have to be linked to computation of the reactions of the receiving water in order to make it possible to make statements regarding the effect of the discharge (Harremoës, 1982; Hvidtved-Jacobsen and Harremoës, 1982).

The demands on the computation method include both mean annual load and extreme shockloads because the aim is to reflect the statistical nature of the events. In many cases this has been done by use of synthetic rain events, Arnell and others (1983). As there is no unique relationship between statistics on the rain series and statistics on the runoff series the use of synthetic rain events will give rise to uncertainties in addition to significant uncertainties involved in modelling the runoff transformation.

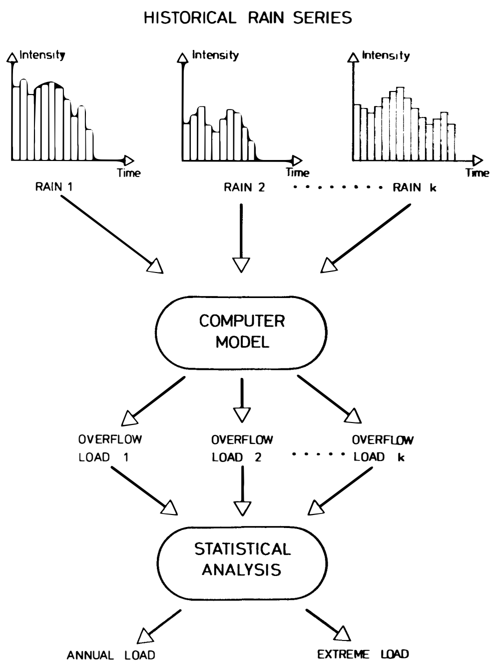


Fig. 1. Principle for calculation of discharge to receiving water from overflow structures. The rainseries should be continuous and the runoff model should reflect all phenomena in the runoff process.

Our main approach is to use historical rain series, transform every rain event to an overflow event by means of a runoff model, and make the relevant statistics on the overflow series. This principle is illustrated in Fig. 1. The computer costs of runoff modelling and input preparation are high for these ideal calculations. Therefore it is necessary to introduce such simplifications in the modelling, that the basic processes can be produced for a minimum of expense without loosing the ability to make reliable computations.

Rain Series

In Denmark historical rain series are available from the Water Pollution Control Committee. In the 1930's the Association of Municipal and Harbour Engineers founded a number of rain measuring stations equipped with gauges of the pluviograph type. Data from two of these stations, Gentofte and Odense, have been digitized, Jensen and others (1983). Together with data from a network of 43 stations of tipping bucket type and telemetric registration, Harremoës and Henze (1981), they form the input to runoff modelling. For the present project data from the Odense station has been used. The total registration period amounts to 47 years. During this period the station has been operated both summer and winter for 33 years. In digitizing the data was used an event definition of 1 hour minimum inter-event time, and a minimum rain depth of 0.6 mm. Table 1 presents the basic characteristics from the above mentioned 33 years series. For the overflow calculations rain events with a depth of less than 3 mm were excluded from the series.

TABLE 1 Characteristic figures for the Odense series. (33 years).

	Rain >0.6 mm	Rain ≥3 mm
Number	142 y^{-1}	48 y^{-1}
Mean annual rain depth	464 mm	331 mm

Hydrology

The runoff volume from the surfaces is computed from equation 1:

$$V = 10\phi_p \alpha_p F (R - S_p) \tag{1}$$

where

V : runoff volume (m^3)

F : total area (ha)

R : rain depth (mm)

S_p : initial loss on paved area (mm)

ϕ_p : runoff coefficient for the paved area

α_p : fraction of the total area that is paved.

From measurements on 3 catchments the initial loss is estimated to be in the range 0.4 - 1.0 mm and the runoff coefficient for the paved area in the range 0.7 - 0.9, Johansen and others (1983). These figures are consistant with the results from other investigations (Arnell, 1982).

Hydraulics

A modified version of the time-area method is used for the runoff computation of overland flow and pipe flow in the catchment area. For the flow in the interceptor a constant travel time is used, either optional or computed as full flow travel time. The basic time-area approach is illustrated in Fig. 2. For a given velocity isochrones are computed and the accumulated contributing area is plotted against the runoff time to give the time-area curve. Traditionally the time-area curve is computed manually and used in manual design methods. An alternative is to use a runoff model to compute the time-area curve.

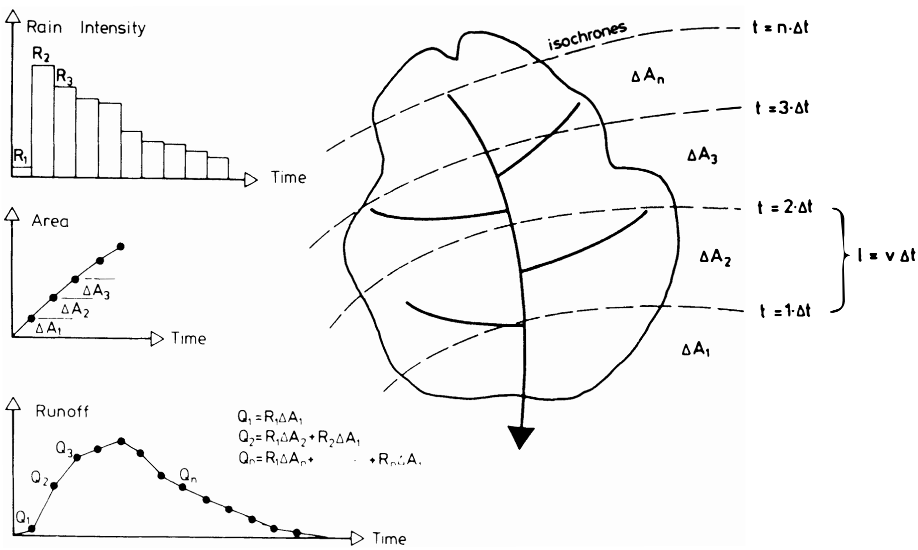


Fig. 2. Computation of time-area curve and transformation of rain event to runoff by the time-area method.

The basic assumption in this method is that the runoff velocity is constant, which implies that the method becomes linear. This is opposed to the kinematic and dynamic wave approaches which involve non-linear relationships between water depth and velocity. By means of the kinematic wave assumptions it is possible to transform this relationship into a relationship between the actual travel time and the full flow travel time as a function of the ratio between the actual mean rain intensity and the design rain intensity, Jensen (1981). Equation 2 expresses this relationship:

$$\frac{t_c}{t_f} = 2.58-10\left(\frac{i_m}{i_f}\right) + 26.8\left(\frac{i_m}{i_f}\right)^2 - 36.5\left(\frac{i_m}{i_f}\right)^3 + 23.7\left(\frac{i_m}{i_f}\right)^4 - 5.6\left(\frac{i_m}{i_f}\right)^5 \qquad (2)$$

i_m : Mean intensity of actual rain event

i_f : Design intensity for the inflow pipe to the overflow structure

t_f : Full flow travel time for the catchment area

t_c : Travel time according to i_m.

Figure 3 illustrates the use of the formula. From the basic time-area curve (in this case derived by ILLUDAS and a constant rain intensity of 1.75 μm/s) it is possible to construct time-area curves for an arbitrary mean intensity of a rain event. Once the time-area curve for a specific rain event has been constructed, the runoff is computed, taking into account the time variation of intensity during the event. This modification of the time-area concept is an approximation to the kinematic wave methods accounting for the difference in mean flow velocity between the events.

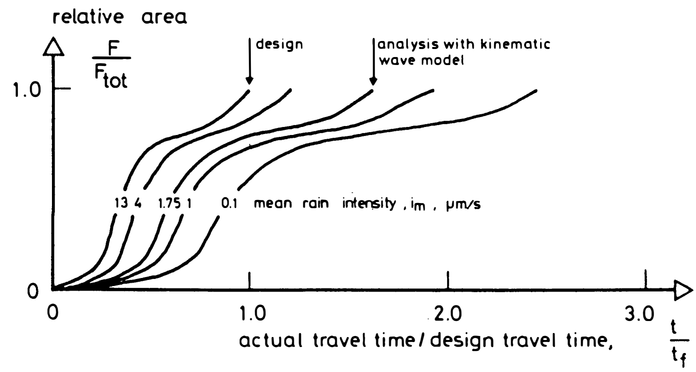


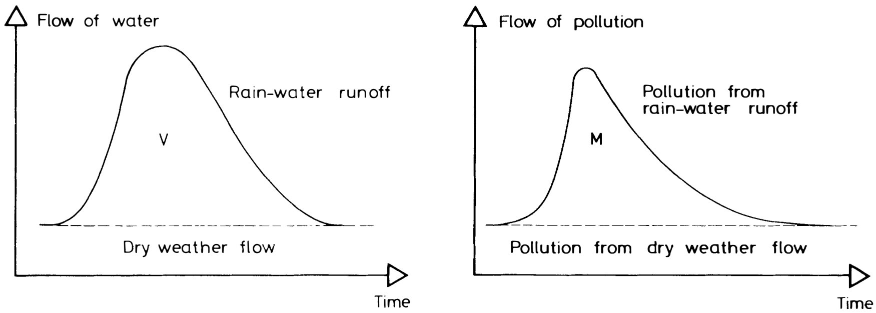
Fig. 3. The modified time-area method makes use of a time-area curve which is computed by a kinematic wave model. The runoff time is adjusted as function of the mean intensity of the specific rain event. From the new runoff time it is possible to create a specific time area curve to the actual rain event from the basic time-area curve, derived by a kinematic wave model.

The division of flow in an overflow structure is modelled by assuming constant outflow to the interceptor. When the inflow exceeds the outflow an eventual basin starts to fill up. When the available volume of the structure is full, the overflow starts to operate. These computations account for the decrease in basin volume from water remaining in the basin from previous events.

Pollution

The pollution of the runoff water is simulated by a simple mixing model. Figure 4 illustrates the concept of rainwater runoff and dry weather runoff. From the basis of field measurements of pollution in the total runoff and the dry weather flow, it is possible to compute the pollution of the rainwater runoff. The reverse procedure is used in the model, from known values of pollution in rainwater runoff and dry weather runoff, is computed the pollution load of the total runoff. In Table 2 values for pollutant concentrations are shown.The resulting pollutograph is routed through the system by use of the time-area method. The settling of pollutants in the basins is modelled as a function of the residence time of the basin.

The "first flush effect" is ignored by this method. Actual measurements (Johansen and others, 1983), have demonstrated, that when 50% of the water have passed the point of measurement approximately 60% of the pollution have passed. This indicates that "first flush effects" are insignificant for this type of computation.



$C_r = \frac{M}{V}$ mean rain-water runoff concentration

Fig. 4. The concept of rain water runoff and dry weather runoff. The total pollution can be separated into the components of rain-water runoff and dry weather runoff.

TABLE 2 Values for Pollution in Rain-Water Runoff and Dry Weather Flow. The Values are Flow Weighted Mean Values on an Annual Basis. (Johansen and others, 1983).

	Rain-water runoff g/m^3	Dry weather flow g/m^3
SS	179	130
Total-P	2.1	9.9
Dissolved-P	1.0	7.5
Total-N	8.6	32.0
Dissolved-N	4.1	27.8
COD	120	253
Dissolved-COD	31	125

Output

The result of the simulation is a series of overflow volumes and loads, one series for each overflow structure. From these series, it is possible by appropriate statistical techniques to generate the discharge of water and pollution in a time scale suitable to the type and problems of the receiving waters. It is most common to characterize the discharge in two ways.

- o Mean annual discharge
- o Extreme discharge corresponding to preselected return period.

In the next section is described results from model simulations for selected catchments values. They are used to generate simple diagrams and tables for the annual and extreme discharges of water and pollutants from a single overflow structure, Johansen (1983). For a system of overflow structures where there is interaction between the structures, these diagrams and tables do not apply, but the total model has to be used.

ANNUAL LOAD

For 120 different combinations of interceptor capacity, a, and full flow travel time, t_f, where a is in the range of 0.05 µm/s to 3.2 µm/s and t_f is in the range 5 to 240 min, 120 overflow series are computed. From each series values are computed for:

- o Overflow volume
- o Frequency of overflow
- o Duration of overflow
- o Mixing factors between rain water runoff and dry weather flow.

The interceptor capacity, a, is defined as:

$$a = 100 \frac{Q_i - Q_d}{F} \tag{3}$$

Q_i: Constant flow to interceptor (m^3/s)

Q_d: Dry weather flow (m^3/s)

F: Impervious area (ha)

a: Interceptor capacity (µm/s)

The figures 5 to 8 show the resulting mean values plotted as function of interceptor capacity and travel time. An example will clarify the use. For an interceptor capacity of 0.4 µm/s and a travel time of 45 min Fig. 5 gives a mean annual overflow of 90 mm. From Fig. 6 appears that the frequency will be 38 y^{-1} and from Fig. 7 that the overflow is operating 67 hours/year. For computation of the pollutional load Fig. 8 is used. This diagram shows m_y/m_s, where m_y and m_s are given by equations 4 and 5

$$m_s = \frac{Q_i - Q_d}{Q_d} \tag{4}$$

$$m_y = \frac{V_o - V_{od}}{V_{od}} \tag{5}$$

where

m_y : Mixing factor between discharged rainwater runoff and dry weather runoff on an annual basis

m_s : Mixing factor between rainwater runoff and dry weather runoff when the overflow starts to function

Q_i : Flow to the interceptor, assumed constant during the event (m^3/s)

Q_d : Dry weather flow, assumed constant during the event (m^3/s)

V_o : Total volume of storm water and dry weather flow discharged to the recipient on an annual basis (m^3/y).

V_{od}: Total volume of dry weather flow discharged to the recipient on an annual basis (m^3/y).

The values for Q_i and Q_d are structure and catchment dependent, so m_s is an independent parameter in relation to overflow values. Once m_s is known m_y can be determined from figure 8. The pollution load is then calculated from Equation 6:

$$P = \frac{V_o}{1 + m_y} (C_d + m_y C_r) \tag{6}$$

where

P : Discharged pollution of each substance to the recipient (kg/y).

V_o: Discharged water to the recipient (m^3/y)

C_d: Pollution concentration in dry weather flow (y/m^3)

C_r: Pollution in rain water runoff (g/m^3).

The previous exemplified catchment data can be supplemented by m_s=5, C_d= 500 g/m^3 and C_r=100 g/m^3, where the concentrations exemplifies an arbitrary chosen substance. From Fig 8 is found that m_y/m_s=2, which then gives m_y=10. Equation 6 then gives P=136 kg /year.

In case that there is a basin connected to the overflow, the results have to be corrected for the water remaining in the basin during the event. This is done by means of tables like Table 3. In these are set up the ranked overflow values, for a given combination of interceptor capacity and runoff time. Assuming that the basin volume equals 10 mm, one reads that V_o is bigger than 10 mm for ranks less than equal to 53. The overflow frequency is then 53/33 = 1.6 y^{-1}. In the column for V_o is read that in these 53 events a total of 846 mm is discharged. From that number has to be subtracted the sum of water remaining in the basin in the 53 events, so the net overflow volume is (846-530)/33 = 9,6 mm/y or for an impervious area of one ha.: 96 m^3/y. From the previous computation without basin the mean concentration in the overflowing water can be determined to 151 g/m^3. Assuming there is no sedimentation in the basin, the overflowing pollution load is then 96x151 g/year=15 kg/year. In this calculation no attention is paid to the effect of the decrease in basin volume from water remaining from previous events. When it is necessary to include this effect (large basin volumes and small capacities of the interceptor) the graphs and tables have to be updated based on the total computer model, according to the staged approach outlined in Harremoës, Jensen and Johansen (1983).

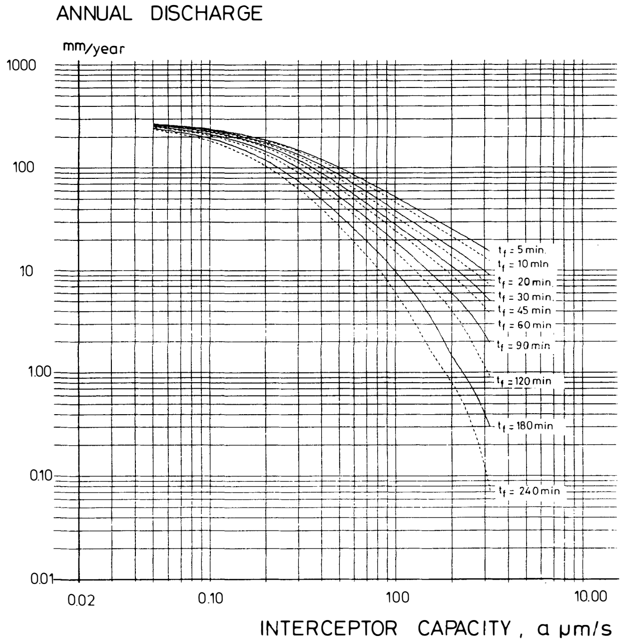


Fig. 5. Annual discharge of water as function of the interceptor capacity, a, and the full flow travel time for the catchment, t_f.

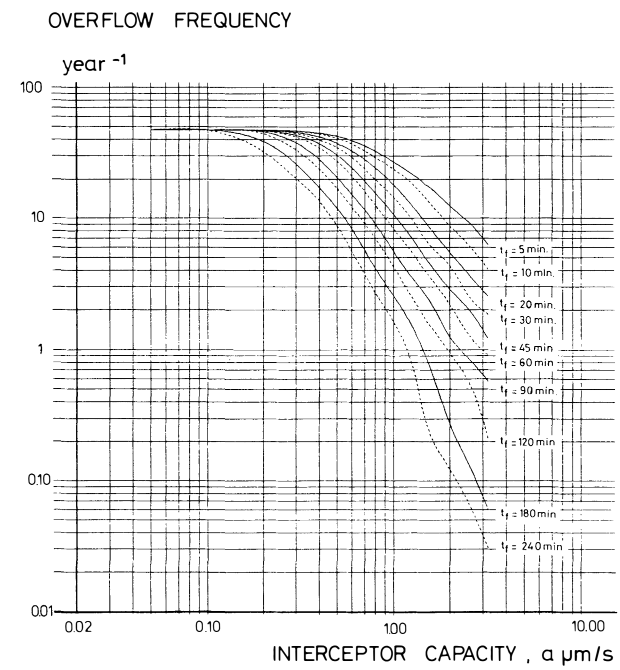


Fig. 6. Frequency of overflow as function of interceptor capacity, a, and full flow travel time, t_f.

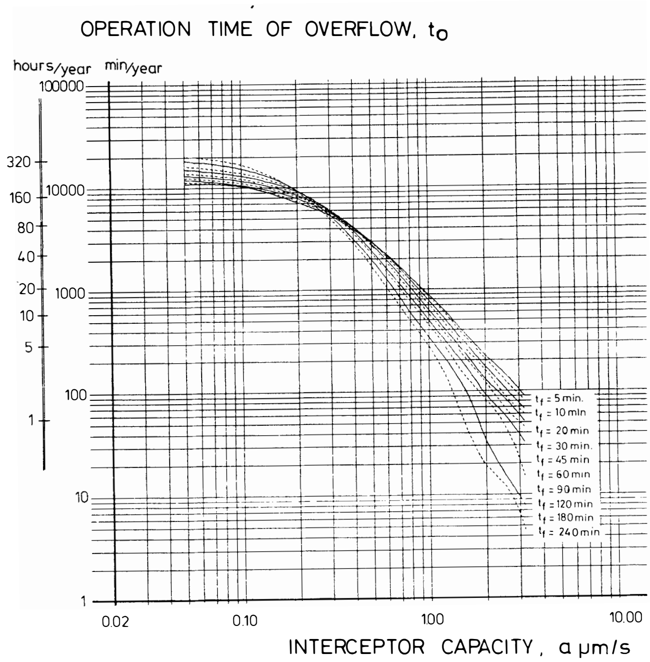


Fig. 7. Operation time of overflow as function of interceptor capacity, a, and full flow travel time, t_f.

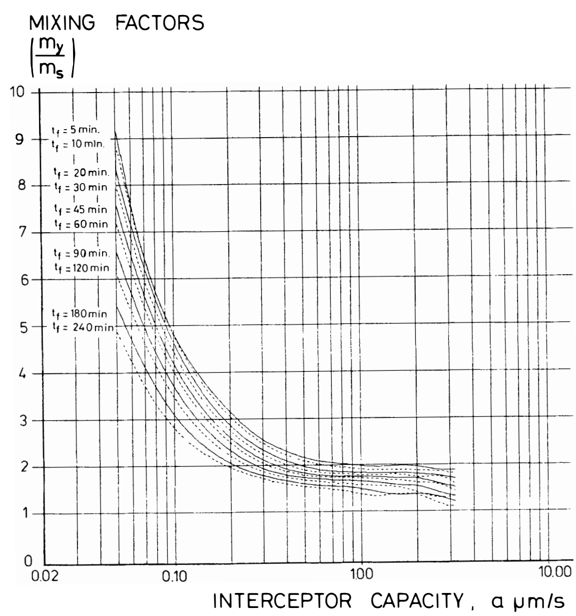


Fig. 8. Relative mixing between rain-water runoff and dry weather flow on an annual basis, as function of mixing when the overflow starts to operate, m_s, the interceptor capacity, a, and the full flow travel time, t_f

EXTREME OVERFLOW EVENT

For each of the above mentioned combinations of interceptor capacity, a, and travel time, t_f, ranking has been performed of the events according to the discharged water volume, V_o. These ranked values are listed in tables together with the duration of overflow, t_o, and the summed overflow of water ΣV_o (used for basin overflow computation as previously described), see Table 3. For a preset rank, M, it is possible to pick out the value of discharged water for a given combination of interceptor capacity and travel time. The pollution load, P, can be computed from Equation 7:

$$P = C_d V_d + C_r V_r \tag{7}$$

On the assumption that rainwater runoff and dry weather flow is totally mixed while the overflow is operating Equation 7 can be rearranged:

$$P = V_o \left(C_r + \frac{t_o Q_d}{V_o + t_o Q_i} (C_d - C_r)\right) \tag{8}$$

where

- P : Pollution load (kg)
- V_o: Discharged water to the recipient (m^3)
- V_d: Discharged dry weather flow to the recipient (m^3)
- V_r: Discharged rainwater runoff to the recipient (m^3)
- C_r: Pollution concentration in rainwater runoff (g/m^3)
- C_d: Pollution concentration in dry weather runoff (g/m^3)
- t_o: Duration of overflow (s)
- Q_i: Flow to interceptor (m^3/s)
- Q_d: Dry weather flow (m^3/s)

Table 4 is an example of such a ranked list of overflow events. Due to the inherent large variation in t_o, it is not certain that the rank of the pollutant load is the same as the rank of the discharged water. It is recommended that the pollution load is computed for a number of events, with the rank in question as the median. Ranking then can be performed of the pollution load, and the appropriate event selected. Included in Table 4 is an example of pollution computation. The example apply to a value of Q_d=0.002 m^3/s, Q_i=0.006 m^3/s, C_d=500 g/m^3, C_r=100 g/m^3 and the paved area 1 ha. The pollutant discharge, P is computed from Equation 8, and listed in Table 4, together with the rank of P, M_p. It is obvious that there are major differences in the ranks M_v and M_p. The rank of water M_v=17 (return period 1.9 years) produces a pollutant disharge with a rank, M_p=6 (return period 5.5 years).

The computational results have been compared to results of measurements for a number of overflow structures on an annual basis. In Fig. 9 is plotted the computed mean annual overflow of water against the measured overflow volume for a specific year. The 95% confidence limits are based on the standard deviation of the annual overflow series. Assuming normal distribution of the annual overflow series the 95% confidence limits are computed from mean value ± 2 x standard deviation. From the comparison appears that the model results on a 95% confidence basis are in accordance with the measured values.

TABLE 3 Rank Ordered List of Overflow Values for the 55 Largest Overflow Events for an Interceptor Capacity of 0.4 μm/s and a Runoff Time of 45 minutes. The List Expresses the Rank, M_V, of the Overflow Volume, V_O, and the Rain Identification (yymmdd hhmm for the Start of the Rain) together with the Operating Time, t_O, and the Summed Volume. By that List it is possible to Compute the Overflow from Basins, and the Overflow of Pollutants (see Table 4).

M_V	RAIN IDENTIFICATION		V_O mm	t_O min	ΣV_O mm
1	720729	1756	49.6	345	49.6
2	610905	1217	34.7	130	84.3
3	610613	1538	28.2	140	113
4	640607	1523	23.1	210	136
5	580525	2111	23.0	180	159
6	650505	1605	21.0	100	180
7	710924	234	20.7	160	200
8	740811	1353	19.4	70	220
9	650721	1541	18.8	85	239
10	720724	2348	18.7	105	257
11	590815	1321	18.7	165	276
12	670918	2218	18.5	95	295
13	740712	716	18.1	85	313
14	730921	1307	17.2	400	330
15	390830	1755	16.9	105	347
16	730928	1449	16.9	120	364
17	710529	221	15.9	570	380
18	720811	250	15.8	415	395
19	680627	1538	15.8	175	411
20	670810	809	15.2	115	426
21	410723	804	15.1	210	441
22	670802	1809	15.1	130	457
23	570604	1428	14.9	115	471
24	761130	1812	14.7	360	486
25	590815	2017	14.3	480	500
26	710815	731	14.1	110	514
27	620721	1350	14.1	110	529
28	550507	147	13.9	240	542
29	560703	47	13.8	285	556
30	740801	736	13.6	115	570
31	530516	1810	13.5	65	583
32	670918	28	13.4	100	597
33	570720	939	13.3	305	610
34	790825	1258	13.3	130	623
35	740809	1723	13.2	70	637
36	670922	2	13.2	575	650
37	700916	1807	13.1	350	663
38	710627	814	13.1	320	676
39	670529	2050	12.9	355	689
40	671016	2219	12.2	345	701
41	560818	1418	12.2	275	713
42	410805	604	11.7	195	725
43	360522	126	11.7	240	737
44	360719	207	11.6	355	748
45	630610	1422	11.6	115	760
46	640908	704	11.5	125	771
47	740805	411	11.1	255	783
48	530618	2245	11.0	865	793
49	590815	936	10.9	90	804
50	570727	1609	10.6	65	815
51	530610	41	10.5	245	825
52	730707	1914	10.5	65	836
53	630618	1743	10.2	140	846
54	630815	639	10.0	390	856
55	410611	513	10.0	300	866

Table 4 List of Pollution Discharge, P, Computed on the Basis of Table 3 and Equation 8. The Ranks, M_p, of the Pollution Differs from the Rank of the Discharged water volume, due to the Large Variation in the Operating Time of the Overflow, thus increasing the Contribution from the Dry weather Flow.

M_v	RAIN IDENTIFICATION	V_o mm	t_o min	P kg	M_p
1	720729 1756	49.6	345	62.8	1
2	610905 1217	34.7	130	40.2	2
3	610613 1538	28.2	140	33.9	3
4	640607 1523	23.1	210	30.7	4
5	580525 2111	23.0	180	29.7	5
6	650505 1605	21.0	100	25.1	10
7	710924 234	20.7	160	26.7	8
8	740811 1353	19.4	70	22.4	15
9	650721 1541	18.8	85	22.3	16
10	720724 2348	18.7	105	22.4	14
11	590815 1321	18.7	165	24.7	11
12	670918 2218	18.5	95	22.3	17
13	740712 716	18.1	85	21.6	20
14	730921 1307	17.2	400	27.7	7
15	390830 1755	16.9	105	21.0	22
16	730928 1449	16.9	120	21.5	21
17	710529 221	15.9	570	27.8	6
18	720811 250	15.8	415	26.0	9
19	680627 1538	15.8	175	21.8	19
20	670810 809	15.2	115	19.5	24
21	410723 804	15.1	210	21.8	18
22	670802 1809	15.1	130	19.9	23
23	570604 1428	14.9	115	19.2	25
24	761130 1812	14.7	360	23.9	13
25	590815 2017	14.3	480	24.7	12

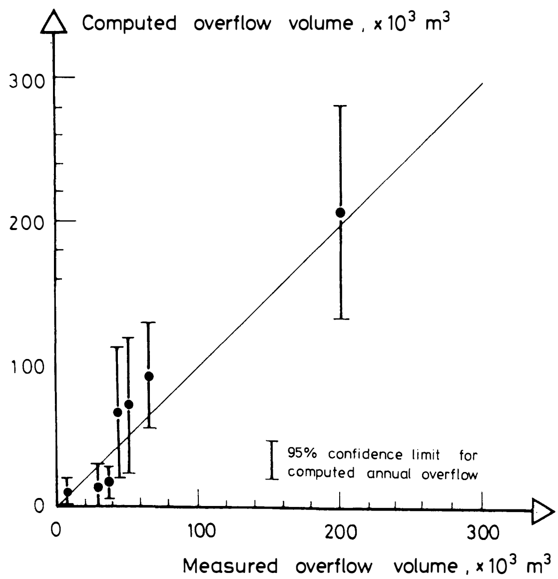


Fig. 9. Comparison between measured and computed overflow volumes for a number of overflow structures. (Johansen and others, 1983).

CONCLUSION

On the basis of a historical rain series it has been possible to develop a simple runoff model of a modified time-area type, for computation of overflow series for each overflow structure in a sewer network. Statistics on these series can generate the load on receiving waters in a time scale appropriate to the type of problems of the receiving water.

Based on the computer method simple graphs and tables have been developed for manual computation of both annual and extreme discharge from single overflow structures where there are no such structures upstream.
Comparison of models results with measured values of annual water discharge from a number of overflow structures shows that the model gives results which on a 95% confidence basis are in accordance with the measured values.

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