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Pollution Loads Discharged from Combined Sewer Overflows: Theoretical Approach and Long Term Numerical Assessment

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The paper describes a theoretical analysis and a numerical assessment of pollutant loads discharged from Combined Sewer Overflows (CSOs) – with or without stormwater tanks – into the environment.

The theoretical approach was based on certain simple assumptions, reasonably valid if the time scale of the problem involved is long enough (month/year), in that single-event simulation is not interesting at all. Two main parameters related to the rainfall regime were found to be significant: the total volume of water discharged from the structure and the effective mixing factor between sanitary sewage and storm runoff.

A numerical assessment of these two parameters was then made, on an annual basis, by means of a long-term rainfall series recorded in Milan, Italy.

Both the "simple" CSO structure and the CSO coupled with stormwater tanks (on-line or off-line) were considered. The resulting graphs make it possible to evaluate the total annual load discharged from CSOs into the environment and the potential reduction obtained by adding a storage capacity to the overflow.

This estimation could be of interest for persistent pollutants (phosphorus, heavy metals) discharged into low-recirculation bodies (lakes, estuaries, lagoons, closed seas).

Introduction

Combined sewer overflows are the main hydraulic structures – and frequently the only ones – built into the drainage network to control the quantity and quality of waters discharged into surface water bodies. CSO structures must be designed observing two specific criteria:

- to preserve, for all expected discharges, good hydraulic efficiency, so as to reduce interceptor capacities and treatment-plant dimensions and costs;
- to ensure that pollution loads transferred from the drainage network into the environment are small enough to avoid acute effects (shocks) or long-term damage (accumulation) in the water body.

This paper deals with the second of these criteria. The aim was to define a simple procedure that can be applied for evaluating, on a long-term basis, the total load released from CSO structures into the environment. The pollutional impact of CSOs has been found to be very significant in most European countries. Ellis (1989) pointed out that, in the UK, between 35-40% of CSOs structures are responsible for serious pollution, either acute or chronic, in surface water bodies. Research carried out in other countries (Torno *et al.* Eds. 1986; Ellis Ed. 1989) substantially confirms the magnitude of the problem.

Several experimental studies have been conducted in recent years with a view to developing new types of CSO structures capable of reducing pollution in the overflow discharge while maintaining good hydraulic efficiency. Structures based on vortex motion and peripheral spill (Field 1984) or coupled with small stilling ponds (Balmforth *et al.* 1982) can concentrate certain types of pollutants (mainly particulate materials) in the interceptor outflow towards the treatment plant.

Nevertheless, in most of the new proposals (Balmér *et al.* Eds. 1984; Gujer and Krejci Eds. 1987; Iwasa and Sueishi Eds. 1990) and realisations (Stotz and Krauth 1986; Krejci *et al.* 1986), on-line and off-line storages are considered as the main means of reducing the impact of CSO structures.

In order to evaluate their efficiency, the theoretical and numerical analysis was extended to cover CSOs coupled with stormwater tanks.

CSO Impact on Receiving Waters

The impact of CSO pollution on the environment mainly depends on the characteristics of the receiving water body and on the pollutant in question. From an operational point of view, the effects of pollution caused from runoff can be separated into acute effects and accumulative effects. As Harremoës (1986) has pointed out, these two categories are only extreme examples of the same problem governed by different time-scales and the working approach can be synthesised in the following concept: "the evaluation of pollution discharged with runoff has to be based on extreme statistics for the quantity of the pollution discharged over a period characteristic of the time of turn-over for the pollutant in the receiving water".

The greater the time-scale extension, the less important is a detailed description of any single event. Interest must be focused on a reliable statistical evaluation of the total mass of pollutant discharged over all the events falling within the time-scale typical of the pollutant and of the receiving water body.

The importance of the scour and release of sediments in the drainage network, which can cause an increase in concentrations of pollutants in the first runoff waters (first foul flush) is a matter of controversy. Indeed, even if all researchers acknowledge its importance with regard to single events and to specific watersheds, its effect on the total load discharged in a longer time-scale is, according to some authors (Harremoës 1986; Johansen 1985) statistically insignificant.

If the aim is to assess the pollutant load discharged in a sufficiently long time-scale (month/year), such as the case of persistent pollutants in low-recirculation water bodies, is seems therefore reasonable to approach the problem adopting the following assumptions:

a) the concentrations of a given pollutant in storm waters and in dry waters at the inlets of the drainage network are constants;
b) the mixing of storm and dry waters is instant and total; consequently, sedimentation and scouring phenomena in the drainage network are negligible;
c) physical, chemical and biological transformation phenomena are negligible.

These simplifications are also useful to achieve some kind of generalization; on the other hand a) does not allow to apply the procedure to situations of multiple CSOs in series. Simulation of large networks with many overflow structures requires therefore the application of a specific computational model, capable to take into account the main characteristics of the network and of the interceptors (Jacobsen 1990).

Theoretical Analysis

CSO without Stormwater Tank

Let us consider a combined sewer system with a CSO structure at the outlet section. Let q_d be the dry-weather discharge and let q_i be the threshold discharge at which the overflow starts to operate. Let us assume that the dry-weather discharge q_d is constant and that the threshold discharge q_i is equal to the interceptor capacity at full stage.

Fig. 1 shows the situation during a rainfall event sufficiently heavy to involve the CSO structure. The stormwater discharge, generated by the rainfall, is added to the dry-weather flow, causing the overflow of a runoff water volume and given pollutant load. Let dV_a be the elementary water volume overflowed in an infinitesimal duration dt.

The above-mentioned hypothesis allows the following expression

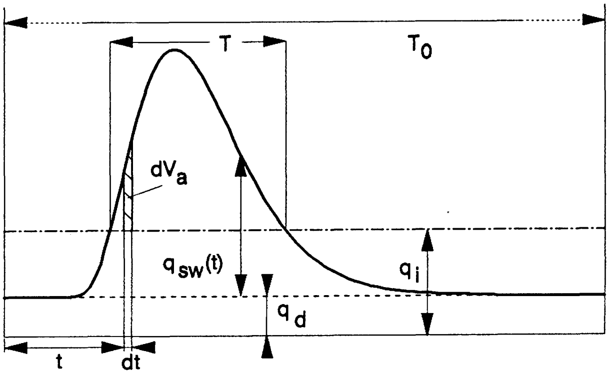

Fig. 1. Definition sketch of the overflow phenomenon and relative symbols.

$$c(t) = \frac{q_d\, c_d + q_{sw}(t)\, c_{sw}}{q_d + q_{sw}(t)} \tag{1}$$

where $c(t)$ is the time-varying concentration of a generic pollutant contained in the dry-weather flow q_d with the constant concentration c_d and in the storm discharge $q_{sw}(t)$ with the constant concentration c_{sw}.

The elementary mass discharged is, therefore

$$dP \equiv dV_a\, c(t) \equiv [q_{sw}(t) + q_d - q_i]\, \frac{q_d\, c_d + q_{sw}(t)\, c_{sw}}{q_d + q_{sw}(t)}\, dt \tag{2}$$

During the time-scale T_0 in question, the total mass of pollutant overflowing can be expressed as follows

$$P = c_d \int_0^T [q_{sw}(t) + q_d - q_i]\, \frac{1 + \alpha\, M(t)}{1 + M(t)}\, dt \tag{3}$$

where T is the total duration of CSO operation in the period of time T_0 and which contains the following two dimensionless variables

$$M(t) \equiv \frac{q_{sw}(t)}{q_d} \tag{4}$$

time-varying mixing ratio between storm and dry-weather flows, and

$$\alpha = \frac{c_{sw}}{c_d} \tag{5}$$

constant ratio of the concentrations of pollutant between the storm and dry flows.

Eq. (3) shows that, with the above-mentioned hypothesis, the functional relation between the total load discharged and the independent variables can be written as follows

$$P = f\{q_d, q_i, c_d, c_{sw}, q_{sw}(t)\} \tag{6}$$

For Eq. (4), the mixing factor $M(t)$ is a highly variable function, depending on the runoff regime $q_{sw}(t)$; in the time-scale T_0, however, only the estimation of an *effective* mean mixing factor, from which it is possible to evaluate the total pollutant mass discharged P, is of interest. Assuming that the fraction between overflowing volume and total volume running to the overflow structure is the same for all the events, Johansen (1985) obtained the following expression for evaluating the annual mean mixing factor $\tilde{M}$

$$\frac{\overline{M}}{m_s} = \frac{V_a}{(q_i - q_d)\ T_a} \tag{7}$$

where $m_s \equiv (q_i\text{-}q_d)/q_d$ is the mixing factor at the beginning of the overflow function and V_a, T_a are the annual water volume discharged and the related annual duration of overflow operation.

Using Eq. (7) to calculate $\tilde{M}$ approximately, the load of pollution can be computed from the expression, derived from Eq. (3)

$$P \equiv c_d\ V_a \frac{1+\alpha\ \overline{M}}{1+\overline{M}} \tag{8}$$

The author computed mean annual mixing factors $\tilde{M}$, based on the rainfall regime of Odense (Denmark) and for several values of interceptor capacities and times of travel in the sewer network.

Starting from Johansen's formulation, it is however possible to define an *effective* mixing factor $\tilde{M}$ so that the Eq. (3) would become rigorously valid

$$\begin{aligned} P &\equiv c_d \frac{1+\alpha\ \tilde{M}}{1+\tilde{M}} \int_0^T [q_{sw}(t) - (q_i - q_d)] \\ &\equiv c_d \int_0^T [q_{sw}(t) - (q_i - q_d)]\ \frac{1+\alpha\ M(t)}{1+M(t)}\ dt \end{aligned} \tag{9}$$

In Eq. (9), the left-hand integral is still the total volume V_a overflowed in the operational time T (related to the time scale T_0). It can be easily shown that $\tilde{M}$ is related only to quantitative characteristics of dry and wet-weather discharges, being independent of the pollutant concentrations c_d and c_{sw}. In fact, after some manipulation, Eq. (9) becomes

$$\tilde{M} = \frac{\int_0^T [q_{sw}(t) - (q_i - q_d)]\ \dfrac{q_{sw}(t)}{q_d + q_{sw}(t)}\ dt}{\int_0^T [q_{sw}(t) - (q_i - q_d)]\ \dfrac{q_d}{q_d + q_{sw}(t)}\ dt} \tag{10}$$

Eq. (10) shows that $\tilde{M}$ depends on the runoff regime $q_{sw}(t)$, on the difference $(q_i\text{-}q_d)$ and also on dry-weather discharge q_d, but it is independent of the (assumed constant) concentrations c_d and c_{sw}. The upper integration limit T, being the operational time of overflow in the timescale T_0, is only a function, for a given runoff regime $q_{sw}(t)$, of the exceedance discharge $(q_i\text{-}q_d)$.

By introducing the (known) mixing factor m_s when the overflow starts to operate

$$m_s = \frac{q_i - q_d}{q_d} \tag{11}$$

Eq. (10) can be expressed as ($m_s \neq 0$)

$$\tilde{M} \equiv \frac{\int_0^T [q_{sw}(t) - (q_i - q_d)] \dfrac{m_s\, q_{sw}(t)}{(q_i - q_d) + m_s\, q_{sw}(t)}\, dt}{\int_0^T [q_{sw}(t) - (q_i - q_d)] \dfrac{q_i - q_d}{(q_i - q_d) + m_s\, q_{sw}(t)}\, dt} \tag{12}$$

Eq. (12), linearized with respect to m_s in the range of practical interest ($0.4 \leq m_s \leq 5$), can easily be approximated with the functional relation

$$\frac{\tilde{M} - 1}{m_s} = f\{(q_i - q_d),\, q_{sw}(t)\} \tag{13}$$

With the hypoteses formulated, the total pollutional load can then be expressed in the synthetic form

$$P \equiv c_d V_a \frac{1 + \alpha \tilde{M}}{1 + \tilde{M}} \tag{14}$$

which is again Eq. (8) introduced by Johansen, but which is now rigorously valid in relation to the new definition of the effective mixing factor $\tilde{M}$.

Therefore, by introducing $\tilde{M}$, the functional relation Eq. (6) leads back to Eq. (14), which offers the advantage of separating the parameters of the quality (c_d, α) from those of the quantity (V_a, $\tilde{M}$) which, for a given runoff regime $q_{sw}(t)$, depend solely on the difference $(q_i\text{-}q_d)$ between the threshold discharge of the CSO structure and the dry-weather discharge.

The function V_a can, by integration, be deduced from the discharge-duration curve at the CSO location, being

$$\frac{dV_a}{d(q_i - q_d)} \equiv - T\{(q_i - q_d),\, q_{sw}(t)\} \tag{15}$$

This requires the direct knowledge of the runoff regime at the CSO structure or its determination on the basis of a historical time-series of rainfall through a rainfall-runoff model. If the time of concentration of the catchment can be taken as negligible and the runoff coefficients as constant, it becomes possible to determine the

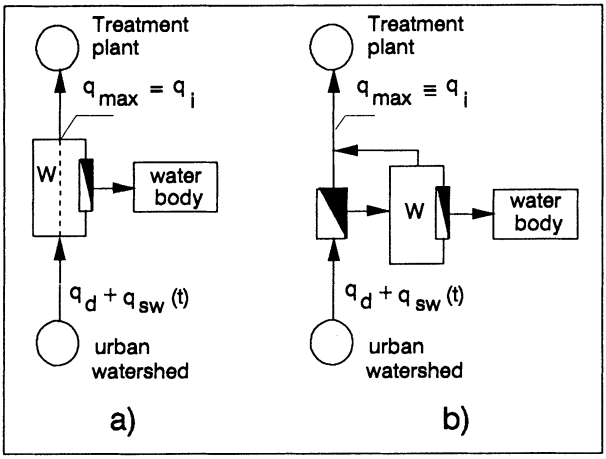

Fig. 2. Operational schemes of on-line (a) and off-line (b) storages.

function V_a directly from the knowledge of the rain intensity-duration curve (Sifalda 1977; Mignosa *et al.* 1990).

Assessing the function $(\tilde{M}\text{-}1)/m_s$, on the other hand, is more complicated, because it requires the mathematical modelling – albeit with the simple assumptions adopted – of the rainfall-runoff phenomena, and dry and stormwater mixing and discharging.

CSO Coupled with Stormwater Tank

Let us consider the situations outlined in Fig. 2. A stormwater tank is placed between the drainage network and the receiving body so that discharge towards the receiving body can take place only after the volume W has been completely occupied.

In the first scheme (on-line tank, Fig. 2a), the entire discharge q_d+$q_{sw}(t)$ flows to the tank, while the outflow from the tank towards the interceptor is limited to q_i. In the second scheme (off-line tank, Fig. 2b), only discharges in excess of the threshold value of the overflow q_i reach the tank. The tank is emptied at an assumed variable discharge q_u so as to saturate the residual conveying capacity of the interceptor, which is also assumed equal to the threshold value q_i. With this type of layout, tank volume is optimised in any case, and it becomes simple to verify that the two functional diagrams, conditions remaining equal, give the same volume discharged (Fig. 3). The opposite obtains with the total pollutant load discharged towards the receiving body, because of the different concentrations $c(t)$ in the on-line and in the off-line tanks.

The total load also depends on the magnitude of the phenomena of sedimentation or disappearance in the tank, and they in turn are linked with the various hydrodynamic situations that occur during filling and emptying of the storage. It is, therefore, rather difficult to generalise about what might be happening. Neverthe-

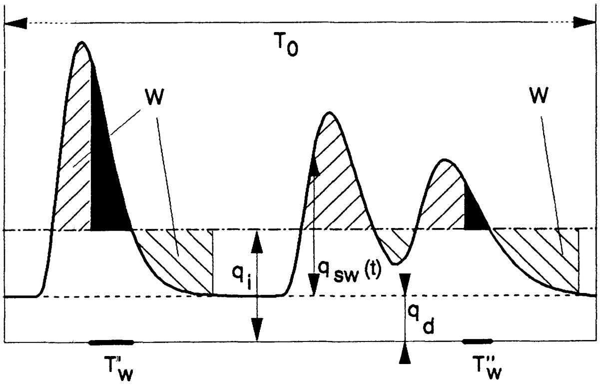

Fig. 3. Definition sketch of filling, emptying and overflow phenomena and relative symbols.

less, a schematic outline based on the assumption of the total absence of sedimentation in the tank can be of interest, considering that:

- it is interesting to evaluate the long-term accumulation of certain pollutants in the receiving water body (phosphorus, for example) and these are to be found in dry and wet-weather waters in a form with a low sedimentation capacity;
- an estimate of the pollutant loads discharged by the sewer overflows of the tanks is, therefore, precautionary.

If, therefore, also with the storage capacity, we maintain assumptions a), b) and c) page 29, Eq. (6) can be generalised as

$$P = f\{q_d, q_i, c_d, c_{sw}, q_{sw}(t), W, \mathrm{ON/OFF}\} \tag{16}$$

where, in addition to the variables already introduced, we have the effective volume of the tank W and its type (on-line, off-line). Referring back to Fig. 3, which is valid for both assumed types, and taking as negligible the routing effects above the tank overflow level, the total pollutant load is

$$P = \int_0^{T_w} [q_{sw}(t) - (q_i - q_d)]\, c_u(t)\, dt \tag{17}$$

where c_u is the concentration of pollutant in question in the discharged waters, corresponding, for the purposes of our assumptions, to the concentration present at the same moment in the entire tank, and $T_w = f\{(q_i - q_d), W, q_{sw}(t)\}$ expresses the operation of the sewer overflow in a given time-scale T_0, depending, here too, on the volume that can be stored in the tank.

From the equations for pollutant-mass balance and continuity in the tank

$$q_e(t)\, c_e(t) - q_u(t)\, c_u(t) = W(t)\, \frac{dc_u(t)}{dt} + c_u(t)\, \frac{dW(t)}{dt} \tag{18}$$

$$q_e(t)-q_u(t) = \frac{dW(t)}{dt} \tag{19}$$

where $q_e(t)$ and $c_e(t)$ are the pollutant input flow and concentration, and $q_u(t)$ and $c_u(t)$ are the related tank outflow (and internal) values, it is possible to show that concentration $c_u(t)$ can always be expressed as a linear combination of pollutant in the dry and wet-weather waters

$$c_u(t) = \lambda(t)\, c_d + \mu(t)\, c_{sw} \tag{20}$$

with $\lambda(t)$ and μ(t) depending only on quantitative parameters, so that

$$\lambda(t) + \mu(t) = 1 \tag{21}$$

Pollutant load P defined by Eq. (16) can, therefore, again be expressed in the form of Eq. (14). Indeed, since Eqs. (20) and (1) are formally identical, we need only state

$$V_a = f\{q_{sw}(t), (q_i-q_d), W\} = \int_0^{T_w} [q_{sw}(t)-(q_i-q_d)]\, dt \tag{22}$$

and

$$\tilde{M} = f\{q_{sw}(t), (q_i-q_d), m_s, W, \text{ON/OFF}\} = \frac{\int_0^{T_w} [q_{sw}(t)-(q_i-q_d)]\, \mu(t)\, dt}{\int_0^{T_w} [q_{sw}(t)-(q_i-q_d)]\, \lambda(t)\, dt} \tag{23}$$

to verify the assumption.

In Eqs. (22) and (23), T_w is the duration of operation of the CSO in the given time-scale T_0, depending, for a given runoff regime $q_{sw}(t)$, on the difference $(q_i$-$q_d)$ between the threshold discharge of the sewer overflow and the dry-weather discharge and on the effective tank volume W, while $\lambda(t)$ and $\mu(t)$ also depend on the type of tank (on-line, off-line) in question.

Linearising Eq. (23) with respect to m_s, we again have, within the field of practical interest ($0.4 \leqslant m_s \leqslant 5$), and with excellent approximation, the functional expression

$$\frac{\tilde{M}-1}{m_s} \equiv g\{q_{sw}(t), (q_i-q_d), W, \text{ON/OFF}\} \tag{24}$$

With a stormwater tank and with the assumptions adopted, therefore, the discharged pollutant load P can again be calculated with the expression Eq. (14) for any value of the concentrations c_d and c_{sw}. In this case, however, assessments of the two functions V_a and $(\tilde{M}$-1)/m_s depend, for a given runoff regime $q_{sw}(t)$, on the difference (q_i-q_d) between the threshold discharge of the overflow and the dry-weather discharge, on the volume of tank W and, as regards the second function only, on its type (on-line, off-line).

Numerical Analysis

According to usual engineering practice in Italy, the design of combined sewer overflows is based on a threshold discharge q_i equal to a multiple (usually 3-5) of the average dry-weather discharge q_d. It is well known that, with dimensions of this type, CSOs come into operation very frequently.

In order to determine the average quantitative characteristics of CSOs operation, it is therefore necessary to evaluate the entire runoff regime that reaches it, including moderate discharges, which are also the most frequent. Since direct information about runoff is very rarely available, it is normally derived from rainfall data, through rainfall-runoff models. It follows that, in the last analysis, we need a sufficiently detailed knowledge of the entire rainfall regime from which it becomes possible to deduce the sequence of discharges at the CSO section.

Should a stormwater tank be present, the frequency of discharges towards the water body is lower, the greater the volume of the storage. In spite of that, we still cannot neglect small and medium events: even though they do not directly cause spilling from the tank, they are responsible for the partial pre-filling of the storage (Geiger 1987).

The Rainfall Regime of Milan, Italy

The rain database used for simulation consists of the long term series of 17 consecutive years of rainfall recorded by the City Council of Milan at the tipping-bucket gauge station of Via Monviso. The entries were converted on to magnetic media (Brown *et al.* 1990) by digitising the time of each tip (0.20 mm). The running speed of the original strip, the best available in Italy, allows a time resolution of about 1-2 minutes, quite precise enough for requirements.

Due to the discrete type of information provided by the tipping-bucket raingauge, it is necessary to define a time of absence of tipping (equivalent to a threshold-intensity value) in order to subdivide the consecutive sequence into single events. Normally, this inter-event-time, or IET, is evaluated in such a way as to guarantee the statistical independence of rainfall events (Restrepo Posada and Eagleson 1982) or of discharges (Arnell 1982).

An analysis of the first type was carried out on the series in question by Piga *et al.* (1990), resulting in the definition of a 27-hour inter-event-time.

In the present study, on the other hand, we have used a more operative approach, assuming an inter-event-time of 1 hour. We did this so as to avoid considering excessive periods of rainfall with very weak intensities ($\leqslant$0.2 mm/h). In any case, since each event is identified by its beginning (year, month, day, hour, minute), the sum of events thus defined is virtually still a consecutive sequence and, as such, was used in further processing operations, updating the state of the drainage network and stormwater tank, if present, to the beginning of each event.

Table 1 sums up the main characteristics of the series obtained.

Table 1 – Main characteristics of the Milan-Via Monviso series (1971-87)

Years of observation	17
Average annual depth of rainfall [mm]	1,002.0
Average annual number of events with IET = 1 hr	142.0
Avarage annual rainstorm duration [hr]	560.8
Avarage annual rainfall intensity [mm/hr]	1.787

Rainfall-Runoff Modelling

The need to simulate the entire sequence of runoffs arriving at the CSO structure and the objective to reach some kind of generalization of the results, does not allow to work with highly sophisticated, physically based models of rainfall-rainfall excess-runoff transformation.

The contribution made to the runoff by permeable areas of the urban watershed – which for heavy storms can have a great effect on the volumes and flows into the drainage network – gradually loses importance until it becomes negligible in cases where interest is focussed on mean values.

Therefore, the mathematical modelling of hydrological losses can gradually be simplified until the contribution to the runoff of only the impermeable areas of the catchment is considered.

In a previous work (Mignosa *et al.* 1990) a depression storage loss on the impervious areas was considered, exponentially filled during rain and emptied during dry periods on the basis of a potential evaporation curve. The total specific capacity of depressions was assumed equal to 1 mm, as a ponded average between the small and frequent rains which did not cause any runoff (0.4-0.6 mm) and the high but rare rains in which the losses were higher (2-3 mm); this value is well in accordance with many experimental calibrations (Jensen 1990). The results showed a slight incidence on mean annual volumes discharged by CSOs structures. This moderate influence, together with the difficulty involved in defining the parameters of filling and recovery of depressions, and the effect of event subdivision criterion on the determination of dry periods – the only ones in which evaporation can take place – do not seem to justify such an approach.

It was therefore held sufficient to consider only the contribution of impervious areas, without any hydrological losses. This approach also makes it easier to generalise the results, which from now on will be expressed with exclusive reference to the impervious portion of the drained watershed.

As far as rainfall excess-runoff transformation is concerned, since there is no point in reproducing single events exactly, the intention being, rather, to generalise results, a simple linear model was adopted. The sequence of historical storms was then converted into runoff by means of a linear rainfall-runoff model (time-area method), with times of concentration ranging between 5 and 60 minutes, representative of most urban catchments with medium-small extensions (0-200 ha).

Results

CSO without Stormwater Tank

A summary of the results obtained from the simulation of the series of 17 years of rainfall events is given in Figs. 4 and 5.

Fig. 4 shows the mean annual volumes V_a discharged from the CSO structure as a function of the specific discharge $(q_i\text{-}q_d)/A_{\text{imp}}$ for the two extreme times of concentration considered (t_c=0-60 min).

Fig. 5 shows the corresponding values of the parameter $(\tilde{M}\text{-}1)/m_s$, from which, being m_s known, it is possible to deduce immediately the mean annual mixing factors $\tilde{M}$. Having determined the values of V_a and $\tilde{M}$ from the graphs, and knowing – or assuming – the concentration values of the pollutant in question in storm and dry-weather waters, it becomes easy to apply Eq. (14) and calculate the mean annual load of pollutant discharged toward the water body.

The trends of the curves shown in Figs. 4 and 5 deserve certain general considerations. As has already been said, CSOs are usually designed in Italy in order to start to operate at a discharge q_i equal to a multiple (usually 3-5) of the average dry-weather flow q_d. With normal housing densities and *per capita* sewage, this corresponds to specific discharge values $(q_i\text{-}q_d)\ A_{\text{imp}}$ of the order of 0.5-1.5 $\text{l}/(\text{s}\times\text{ha}_{\text{imp}})$. From Fig. 4, it can be instantly seen that, with these values, the average annual volume discharged towards the water body is still very high. In order to reduce the volume to any significant extent, it would be necessary to design the sewer overflows and all of the downstream works in terms of much higher specific discharges. Besides the higher cost of collecting waters held in the network in this way, similar flows are not compatible with efficient operation of the treatment plants we have today. For these reasons, the only solution that makes it possible to keep the dimensions of interceptors and treatment plants unaltered while reducing the impact on the receiving water bodies is the insertion of stormwater tanks between the drainage network and the overflow.

The graph in Fig. 5, which gives values of the parameter $(\tilde{M}\text{-}1)/m_s$, as a function of the specific discharge $(q_i\text{-}q_d)/A_{\text{imp}}$, shows a fast decrease as the latter increases. In spite of this, in the entire field under study, the values of the mean annual mixing factor $\tilde{M}$ remain much higher than the corresponding value m_s of the beginning of overflow operation. This last parameter, therefore, is not representative of the real behaviour of the sewer overflow, which contradicts what is usually held to be the case.

Figs. 4 and 5 also show that the influence of the time of concentration of the drained catchment on volumes discharged and on mean effective mixing factors is relatively moderate in the field of usual interceptor capacity values and for the times of concentration considered. Response time does have a greater influence on the mean annual number of overflows shown in Fig. 6. As much as these last values may be partly dependent on the IET used to separate the events, the trend of the

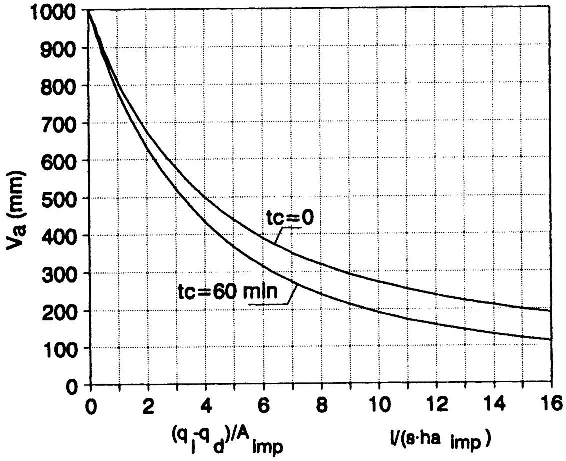

Fig. 4.
Mean annual overflow volumes V_a versus specific discharge $(q_i\text{-}q_d)/A_{\text{imp}}$ and time of concentration t_c.

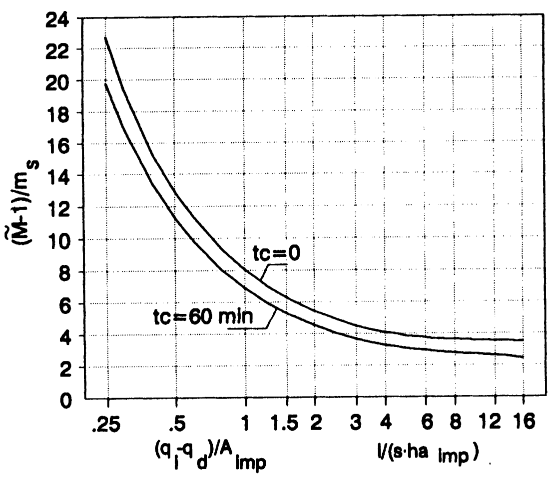

Fig. 5.
Effective mixing factors $\tilde{M}$ versus specific discharge $(q_i\text{-}q_d)/A_{\text{imp}}$ and time of concentration t_c.

curves shows that, even with the higher specific discharges, the mean annual number of overflow events hovers around a couple of dozen.

Certain regulations actually impose limits on overflow frequency rather than on the concentration or on the load discharged towards the water body, with threshold values always around a few events a year. This type of regulation is based on the assumption – somewhat simple and not applicable to all pollutants and all water bodies – that any uncontrolled flows help to degrade the receiving waters. To observe such regulations in the design of CSOs, one can understand that it is necessary to apply other measures than simply increasing the interceptor capacities. The solutions normally adopted range (Schilling 1990) from inserting stormwater tanks to a better exploitation of in-line storage capacity – already existing inside the drainage network – by means of real-time controls (Schilling

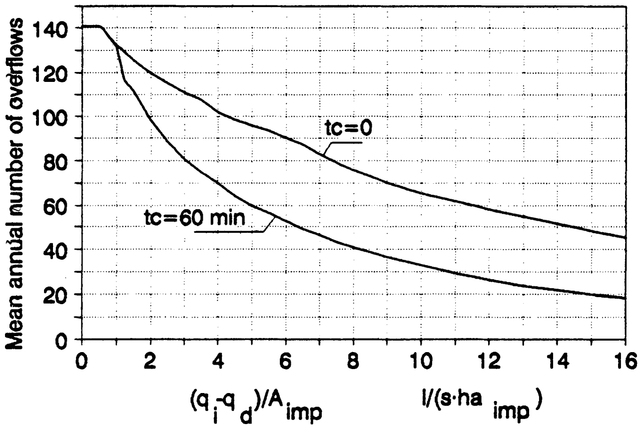

Fig. 6. Mean annual overflow frequency as a function of specific discharge and of the time of concentration (historical Milan-Via Monviso series, 1971-87).

1986, 1990), runoff reductions by means of controlled infiltration and/or overland storages, and so on.

While these latter measures require a specific analysis of the drainage network in question, the effect of inserting stormwater tanks can be analysed using the same procedures as are adopted for the CSO without storage capacity.

CSO Coupled with Stormwater Tank

The fact that discharged water volumes V_a and mean annual effective mixing factors $\tilde{M}$ depend little on the response time of the drained catchments has already been demonstrated in the case of a CSO without storage capacity in Figs. 4 and 5. With the insertion of a tank between the drainage network and the sewer overflow, the influence of this parameter becomes even less significant, because the storage capacity balances out the incoming flows. It has therefore been deemed legitimate to neglect the effect of rainfall excess-runoff transformation when determining the values of V_a and $\tilde{M}$ in the event of a stormwater tank.

The results obtained from the simulation of the series of 17 years of rainfall events (without transformation) are summed up in Figs. 7 and 8. Fig. 7 shows the mean annual volumes V_a discharged towards the water body as a function of the specific volume of the tank W (referring only to the impervious surface of the drained catchment) for various values of the specific interceptor capacity $(q_i\text{-}q_d)/A_{imp}$. With the assumed filling and emptying schemes, the diagram is seen to be valid for both types (on-line, off-line) of tank considered.

Fig. 8 shows the corresponding trends of the parameter $(\tilde{M}\text{-}1)/m_s$. In this case, the diagram is different for the two types of tank considered. In the on-line type, on account of the continual inflow of water and pollutants into the tank, even during

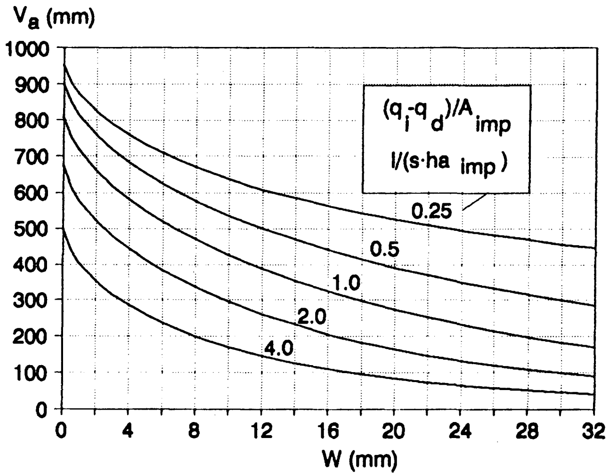

Fig. 7. Mean annual overflow volumes V_a as a function of the specific volume of the tank W and of the specific interceptor capacity $(q_i\text{-}q_d)/A_{\text{imp}}$ (historical Milan-Via Monviso series, 1971-87).

non-rainy periods, the mean effective mixing factors, other conditions remaining equal, are lower compared with those relative to the off-line tank. In practice, this means that the on-line type gives a higher mean annual load overflowed towards the water body. This trend becomes even higher, other conditions remaining equal, as the specific tank volume increases, since there is more likelihood that the storage is already partly full at the onset of an event that has caused an overflow.

Trends in the discharged – volume diagram, Fig. 7, show that the quantitative

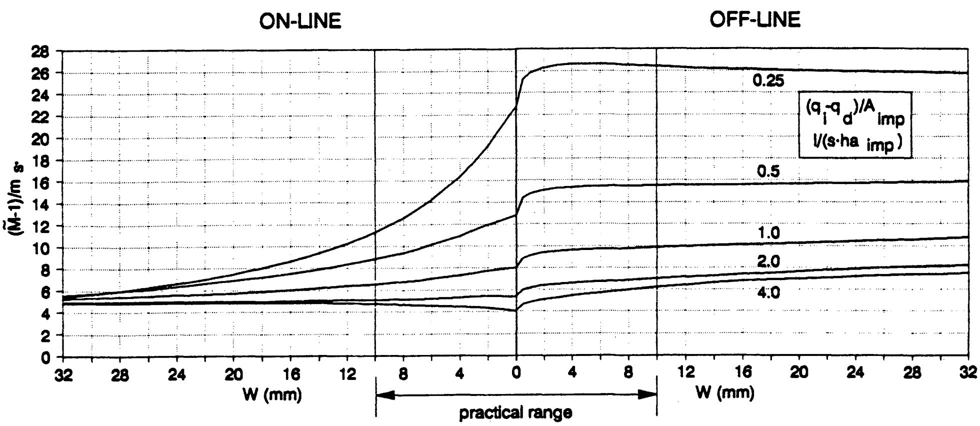

Fig. 8. Mean annual effective mixing factors $\tilde{M}$ as a function of the specific volume of tank W and of the specific interceptor capacity $(q_i\text{-}q_d)/A_{\text{imp}}$ for the two types of tank considered (historical Milan-Via Monviso series, 1971-87).

efficiency of the tank, defined as the ratio between volume retained in the network (V_i-V_a) – and hence conveyed to the treatment plant – and the volume flowing into the tank V_i (Krejci *et al.* 1986)

$$\eta = \frac{V_i - V_a}{V_i} \tag{25}$$

decreases as its specific volume increases.

Indeed, if there is not an adequate increase in outflow as the volume of the storage increases, tank-emptying times increase until they become comparable with or greater than the average inter-event-time between storms, and the tank is already partly pre-filled at the onset of each new event, as Geiger (1987) has pointed out.

For example, a specific storage of 10 mm (100 m^3/ha_{imp}) with a specific interceptor capacity (and emptying discharge) of 0.5 $l/(s \times ha_{imp})$ reduces the volume overflowed towards the water body from 900 mm/year (in absence of the tank) to 540 mm/year. The same storage capacity with emptying specific discharges of 1 and 2 $l/(s \times ha_{imp})$ gives an overflow 430 out of 810 mm and 300 out of 680 mm. The efficiency increases from a value $\eta=0.40$ in the first case to the values $\eta=0.47$ and $\eta=0.56$ in the second and third cases. Therefore, wishing to maintain flows in the network downstream the CSO location at values compatible with current interceptor and treatment-plant dimensions, it does not seem reasonable to plan storage capacities very much larger than the limits mentioned above (10 mm), independently of any other economic considerations.

A comparison between Fig. 7, for a CSO coupled with a stormwater tank, and Fig. 4, for a CSO without storage capacity, also makes it possible to draw certain conclusions about the use of the on-line storage already available inside the drainage network.

Let us take an example: a CSO without storage capacity with a specific discharge of 1 $l/(s \times ha_{imp})$ located at the outlet of an urban watershed with a time of concentration of 60 minutes. The mean annual volume overflowed corresponds to 780 mm, while without the damping due to overland flow and pipe routing (t_c=0), the volume overflowed would be a little higher, equal to 810 mm (Fig. 4). The same reduction effected by the network could be obtained using a stormwater tank of only 0.25 mm (Fig. 7). It becomes immediately clear that only a minimal part of the on-line storage of the pipe network, which is normally a few millimetres, is effectively exploited in order to reduce the entity of discharges at the CSO location. This can be understood easily if one considers that:

1) in the vast majority of events that cause an overflow, the drainage network remains practically empty for the whole duration of the runoff, since it is designed to convey much greater stormwater discharges (2-10 years return period);

2) the on-line storage of the drainage network is never used in syncronous mode, due to routing phenomena, unlike what happens in a stormwater tank.

Even if the second consideration leads us to consider the network, especially if it is on a steep gradient, as a syncronous storage with a much lower volume, all measures taken to exploit it better when small or medium events take place can be useful means of reducing the frequency and the entity of volumes discharged towards the water body.

Numerical Examples

As an example of the application of the procedure proposed, let us consider an urban centre with the following characteristics:

- 3,250 inhabitants;
- *per capita* sanitary sewage: 200 l/(cap.×day);
- directly connected impervious surface: 30 ha,

and we wish to assess the average annual load of phosphorus discharged by a CSO capacity with a threshold discharge $q_i=3q_d$ without storage capacity (a) and with an off-line storage capacity of 8 mm (b).

Phosphorus has been taken as the pollutant because it responds fairly well to the assumption of negligible sedimentation (it is contained to an amount of about 10% in the particulate material) and, usually, it is interesting to evaluate its cumulative effects on low-recirculation receiving waters (lakes, lagoons, closed seas) in the long term (season/year).

Assuming a production of 0.75 kg of phosphorus per inhabitant per year and an average concentration in storm-weather waters, on the basis of experimental data in the rainwater canalisation systems of separate drainage networks (Ellis 1989) of

$$c_{sw} = 0.5 \ g/m^3$$

by means of the graphs of Figs. 4-5 (case a) and Figs. 7-8 (case b) and by application of Eq. (14) we obtain the results summarised in Table 2.

Table 2 – Main parameters and results of the procedure application

	W [mm]	q_d $[\frac{l}{s \times ha}]$	q_i $[\frac{l}{s \times ha}]$	m_s [–]	c_d $[\frac{g}{m^3}]$	c_{sw} $[\frac{g}{m^3}]$	V_a $[\frac{mm}{yr}]$	$\frac{\dot{M}-1}{m_s}$ [–]	P $[\frac{kg}{yr}]$	P_{tot} $[\frac{kg}{yr}]$	$\frac{P}{P_{tot}}$ [%]
a)	0	7.5	22.5	2	10.3	0.5	900	25	237	2,600	9.1
b)	8	7.5	22.5	2	10.3	0.5	575	32	137	2,600	5.3
c)	0	7.5	22.5	2	450	200	900	25	56,400	167,250	34
d)	8	7.5	22.5	2	450	200	575	32	35,800	167,250	21

With the same population and stormwater-concentration data, a perfectly separate sewer system (without any wrong connection of sewage in the stormwater network) would have discharged a quantity of phosphorus equivalent to 150 kg/yr that is, approximatly the same as the case of the combined sewer with an overflow coupled with a stormwater 8-mm tank.

If we then assume a percentage of *wrong connections* amounting to 3-4% (in many respects considerable as physiological and difficult to eliminate), we would have to add to the previous value an estimated quantity of 75-100 kg/yr.

In all, we would then have an estimated average annual discharged phosphorus load of

$$150 \text{ kg/yr (stormwater flow)} + \begin{cases} 75 \\ 100 \end{cases} \text{kg/yr (wrong connections)} = \begin{cases} 225 \\ 250 \end{cases} \text{kg/yr}$$

equivalent to the load overflowing from the combined sewer system with no stormwater tank we examined earlier. This result is confirmed by Ellis (1989) who calculates the percentage of wrong connections at 2-5%, sufficient to cancel out the advantage – in terms of water quality in the receiving water body – generated by separating the networks.

We note that, in the case of phosphorus, the quantity conveyed by the storm waters ($\simeq$ 150 kg/year) is one order of magnitude lower than the quantity contained in the sewerage ($\simeq$ 2,450 kg/year). Other pollutants can be present in comparable quantities in the two waters (suspended solids, for example) or even in higher proportions in storm-weather waters (Pb, for example) (Ellis 1989). Consequently the entity of the annual load discharged, as against that produced, changes considerably, but the procedure can be applied as well (until the hypoteses can be assumed reasonably valid).

Let us, for instance, endeavour to estimate, using the same situations as previously, the average annual load of suspended solids discharged into the receiving water body (cases c and d). Even if possible build-up and wash-off phenomena in the drainage system (and in the stormwater tank, if present) can, in this case, make calculations based on the proposed methodology more doubtful, the results obtained do give useful, approximate indications.

Assuming a production of 33 kg of SS per inhabitant per year and an estimated average concentration in storm waters flowing into the drainage network of (Ellis 1989)

$$c_{sw} = 200 \text{ g/m}^3$$

we obtain the results reported in Table 2.

The high percentages discharges are in very good agreement with that reported in recent literature (Ellis 1986) for pollutant (such as SS or heavy metals) present in large quantities also in storm waters due to the wash-off of the drained surfaces.

Conclusions

The paper describes a theoretical approach and a numerical analysis aimed at determining the water volumes and pollutant loads discharged, on annual basis, by CSO structures with and without stormwater tanks.

Based on certain simple assumptions reasonably valid if the time-scale of the problem involved is long enough (month/year), the theoretical analysis made it possible to define the two quantitative parameters governing the phenomenon: the water volume discharged by the overflow and the effective mean mixing factor between storm and dry-weather waters.

A numerical analysis of these two parameters was carried out by means of a long-term historical rainfall series of 17 consecutive years recorded in Milan, Italy.

The resulting graphs, valid for the rainfall regime of Milan, showed that, for interceptor capacities commonly adopted in Italy for CSOs (and treatment plant) design, the water volumes and pollutant loads discharged form a significant proportion, respectively, of rainfall volumes and total loads produced on the watershed. Frequency of overflow function is also very high, corresponding to several dozen rainfall events per year.

It must be stressed that the adopted approach do not take account of the effect of the first-flush phenomenon which, if it is often jointly responsible for the shock effects on water bodies, can in certain cases also have a not negligible effect on the long-term impacts.

A definitive answer about the importance of these neglected phenomena could be assessed comparing the results of the proposed procedure with experimental data, on a long-term basis. Even if an extensive program of CSOs monitoring has been programmed for the next years, till now continuous records of quality parameters long enough to be useful for this purpose are not available for the Milano rainfall regime.

The results also show that, in order to significantly reduce the impact on receiving waters, it is necessary to explore solutions other than a simple increase in the interceptor capacities and CSOs threshold discharges: if such a solution is not very efficient in theory, it is actually impractical from an engineering point of view.

There can be other solutions, more suitable for reducing the impact of discharges on to the water body while still maintaining the customary dimensions of collection and treatment plants. Examples are the insertion of stormwater tanks, better exploitation of on-line storage capacity already existing inside the drainage network – by means of real-time controls (Schilling 1986, 1990), runoff reductions by means of controlled infiltration and/or overland storages, etc.

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