An experimental approach to the efficiency of a first-flush tank

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Abstract

First flush tanks are supposed to be useful to control the impact of the pollutants contained in combined sewer overflows over the receiving waters. Their design is based upon the idea that the first water in a storm event contains most of pollution caused by the cleaning of the catchment and the conduits (Gupta, 1996).

The design of these structures is often controlled by dilution rules which give retention volumes. These rules do not take into account the hydraulic behaviour of the structure itself, which is not well known so far, as these tanks include some hydraulic phenomena which are difficult to evaluate and even to measure (ATV-128, 1992; Stahre, 1990).

An experimental facility has been developed in the CITEEC Hydraulics Laboratory in A Coruña University in order to measure how the hyetographs and the pollutographs vary at the inlet and at the outlets of the structure. Discharge, accumulated volume and pollutant taxes have been measured in unsteady conditions for different specific volumes (m3/Ha) in the structure. The removal of the pollutant mass has been evaluated, as well as the mixing phenomena inside the tank.

The experimental facility consists of a scale model of a first-flush tank similar to those used by the Northern Water Authority in Spain. The hydrographs and the pollutographs have been obtained by direct field measurements in a combined catchment in Santiago de Compostela (Spain). A colorimetric tracer has been used to simulate pollution (neither settling nor reaction has been measured) and different volumes of the structure have been simulated by varying the discharge scales, according with a Froudian model.

The result consists of a series of graphs which show the amount of pollutant removed as a function of the tank volume, for some first-flush hydrograph-pollutograph peak distance, and a discussion of the actual efficiency in the removal of pollution in the structures installed in Spain and world-wide.

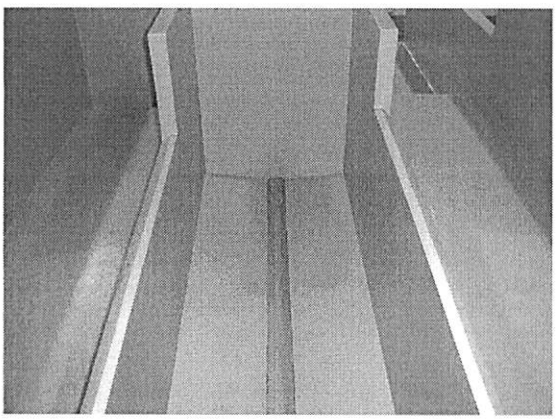
Description of the structure

The first-flush tank studied corresponds to the typical design of those constructed by the Water Authority of Northern Spain (Confederación Hidrográfica del Norte de España). They are based on German standards (ATV-128) and on British studies (BS Sewage 1987 – Liverpool formula).

These structures are not designed as settling tanks but only to prevent first-flush waters from the spill to the rivers (Temprano et al., 1996). They are small structures, with specific volumes of about 4-8 m3/drained Ha and consist of a main channel with a certain accumulation capacity, the first- flush tank itself and a spillway to the receiving media. The structure of these tanks can be seen in figure 1.

Experimental set-up

The experimental structure is a scale model of the original prototype, including an inlet, a main chamber, a first-flush tank and a spillway, as it can be seen in figure 1. The way it works can be seen in figure 3.



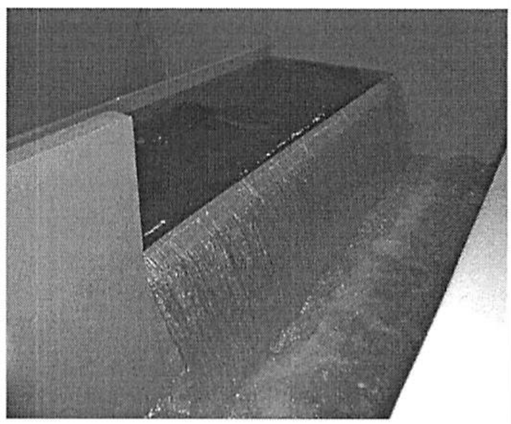


Fig 1.- Scale model of the first flush tank

An automatic sluice gate has been used to generate the adequate hydrographs, and a peristaltic pump provides a rhodamine pollutograph, according with the specifications of each of the experiences. Two more little pumps have been located at the WWTP outlet and at the spillway outlet so as to drive a small continuous amount of water to the Turner fluorometers which will define the output pollutographs in both exits by measuring the rhodamine concentrations.

The water depth in the two chambers have been measured with 12 DHI conductivity-based water level sensors, which are shown in figure 2. Water velocity has been measured in the points shown in Figure 2 by using Sontek ADV-3D velocity measurement devices. These velocity sensors have been placed near the center of the tank in order to have a first idea about the recirculation and settling velocities in the tanks.

A calibration process has been defined before starting the tests. The linearity of the fluorometers and level sensors has been checked, and the curves of supply from the automatic valve and from the peristaltic pump have been defined.

Once this first step has been done, the hydroghaphs and pollutographs will be generated by the computer by opening the valve or changing the velocity of the pump. All the parameters and operations are controlled by a VISUAL BASIC control file.

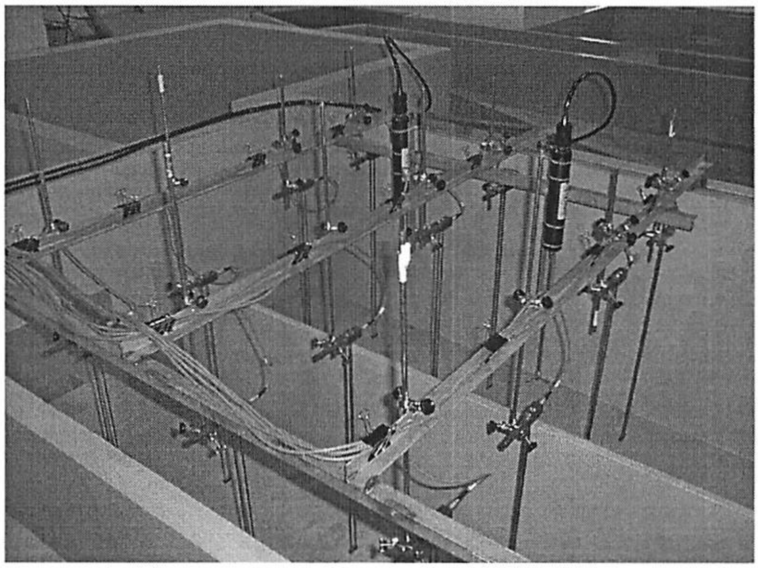


Fig. 2.- Level sensors and ADV velocity sensors

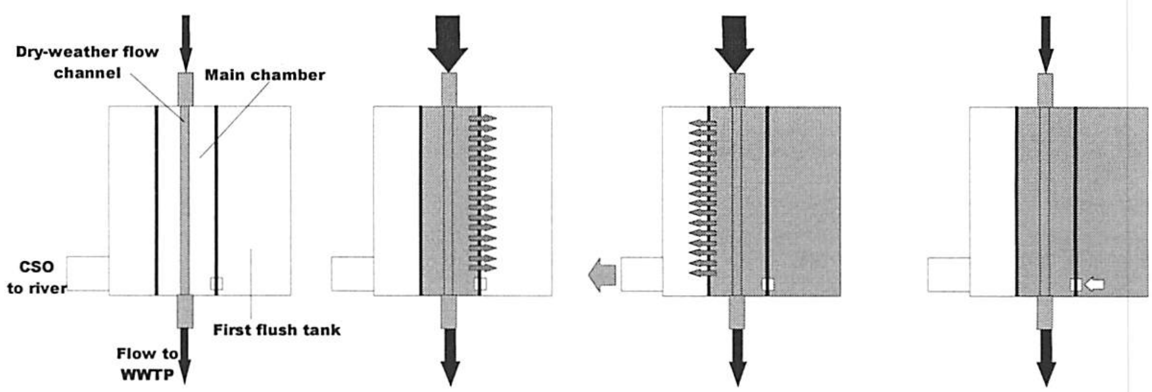


Fig 3.- Sketch of the structure studied

Experimental process and results

The scale of the model is considered as a variable, by means of changing the hydrograph and pollutograph scales. If we consider the actual scale between the prototype tank and the scale model, the geometric ratio is 5. If we place this structure at the outlet of the Ensanche catchment, the specific volume of the tank is about 8.7 m^3 per drained Ha. By applying a Froudian scaling law, we can calculate the scale hydrographs and pollutographs by reducing the times (scale $5^{1/2}$) and discharges (scale $5^{5/2}$). However, we can consider the tank to be associated to other specific volumes, such as

17.4 m^3/Ha, for instance, and assume that discharges, times, velocities, etc..., will vary according with the Froudian law. By doing so, we can use a single scale structure to model a full range of storage capacities, with a previously given hydrograph and pollutograph.

Froudian scaling law has been considered as representative of the filling and emptying processes, as gravitational effects seem to be the main ones. If we consider the possibility of a washing-up of the tank, in quasi-steady conditions, the gravitational effects would not be so important as shear stresses in the upper zone of the tank. These effects must be regarded using a Reynolds scaling law or directly by measuring them in the prototype, as setting up a Reynolds facility is not easy at all. The discharges and velocities involved would be much higher if we used a Reynolds law instead of a Froude law, so, if some washing up is noticed in the Froudian model, it will be reasonable to think that this phenomenon will be much more important in the prototype.

If we consider a time to peak of 30 minutes, a peak discharge of 1500 l/s, a base flow of 300 l/s, a recession time of 80 minutes and the linear trends in the hydrograph, the values to be used in the model can be extracted from table 1, which considers the specific volume of the tank as a fundamental parameter.

Table 1.- Scales and dimensions definition

	Scale				Peak Q	Time to peak	Recession time	Base flow
m3/Ha	**Geom**	**Volume**	**Disch.**	**Time**				
8.7	5.000	125.000	55.902	2.236	26.833	11.180	35.777	5.367
4.35	3.969	62.500	31.374	1.992	47.811	12.550	40.158	9.562
17.4	6.300	250.000	99.606	2.510	15.059	9.961	31.874	3.012
26.1	7.211	375.000	139.645	2.685	10.742	9.310	29.791	2.148

The values of the peak discharge, the time to peak, etc... have been obtained from real data measured in the Ensanche catchment (Cagiao et al, 1998), and they have been obtained as qualitative mean values over those parameters obtained from the top five rainfall events registered during a year, according with the peak discharge criteria. For the definition of the time to peak, the first raising of the curves have been considered.

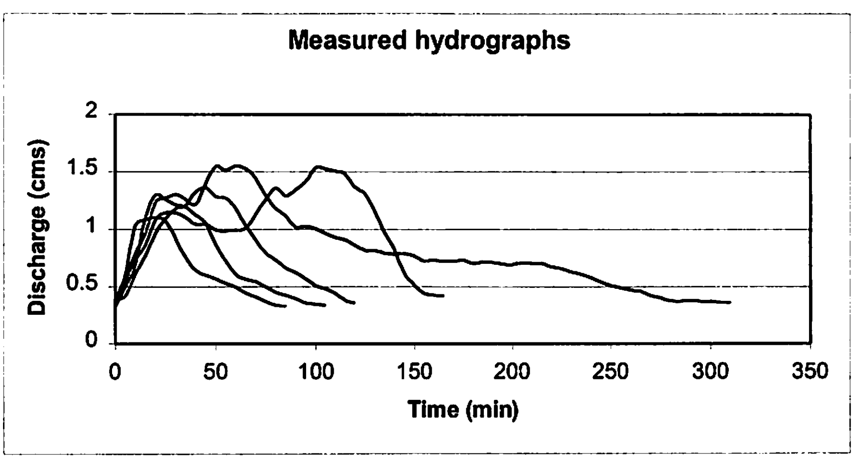


Fig. 4 .- Measured hydrographs

First flush has been modelled by adding a pollutograph to the hydrograph. The distance between the pollutograph and the hydrograph peaks has been also measured in the Ensanche catchment, using the suspended solids content as a paramenter. The delays are in the range 10-20 min, for a time to peak of about 30 min. Raising and decay curves for the pollutographs are about the same slope, so we have defined three pollutographs for each hydrographs, delayed 0 (no first flush), 10 and 20 minutes (prototype time), as it can be seen in figure 5

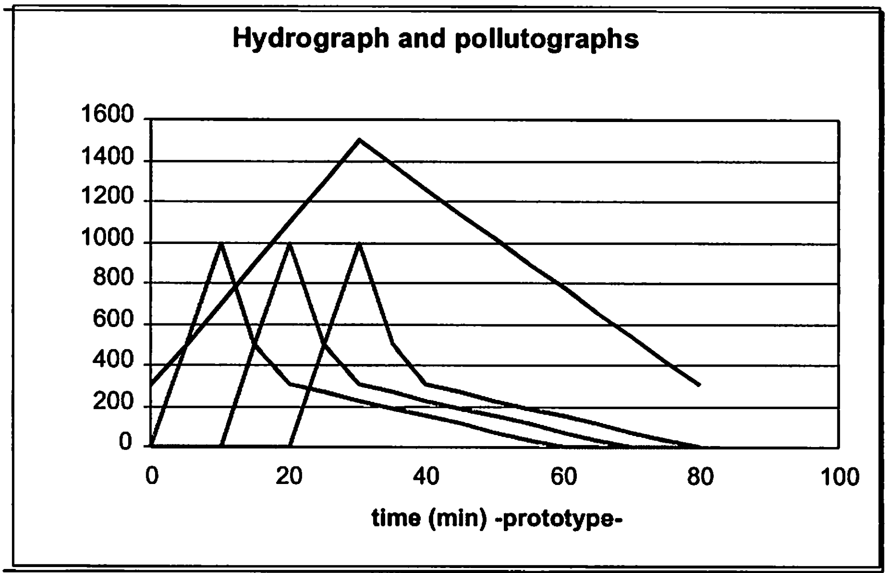


Fig. 5.- Experimental hydrograph and pollutographs

The full experimental work consists of a series of tests, covering a wide range of hydrographs and pollutographs, which is described in the table below:

Table 2: Test references

	Specific volume (m^3/Ha)			
Delay	**4.35**	**8.7**	**17.4**	**26.1**
0	*4-0*	*8-0*	*17-0*	*26-0*
10	*4-10*	*8-10*	*17-0*	*26-10*
20	*4-20*	*8-20*	*17-20*	*26-20*

Results

The first results extracted from the information provided by the tests are those explaining the qualitative behaviour of the tank. In figure 6, an evolution of the water depths and discharges is shown, in order to characterise this behaviour, and also to check some of the parameters involved.

According to this figure, some remarks can be done:

- The filling of the main tank and the first flush tank begins when their level sensors start gauging. That fact permits to check the volumes of both structures as the difference between the two discharge curves
- The main tank depth recordings show a small horizontal region while the first flush tank is being filled. A measure of the discharge driven to the first flush tank can then be done on a volumetric basis by considering the evolution of the levels in the first flush tank. The verification of the continuity equation can be checked if we compare the former discharge with the inflow and the outflow driven through the WWTP.
- The volume of the first flush tank can be also obtained as the difference between the inflow and the discharges driven to the WWTP in the tail of the figure, when first flush tank starts to be emptied.

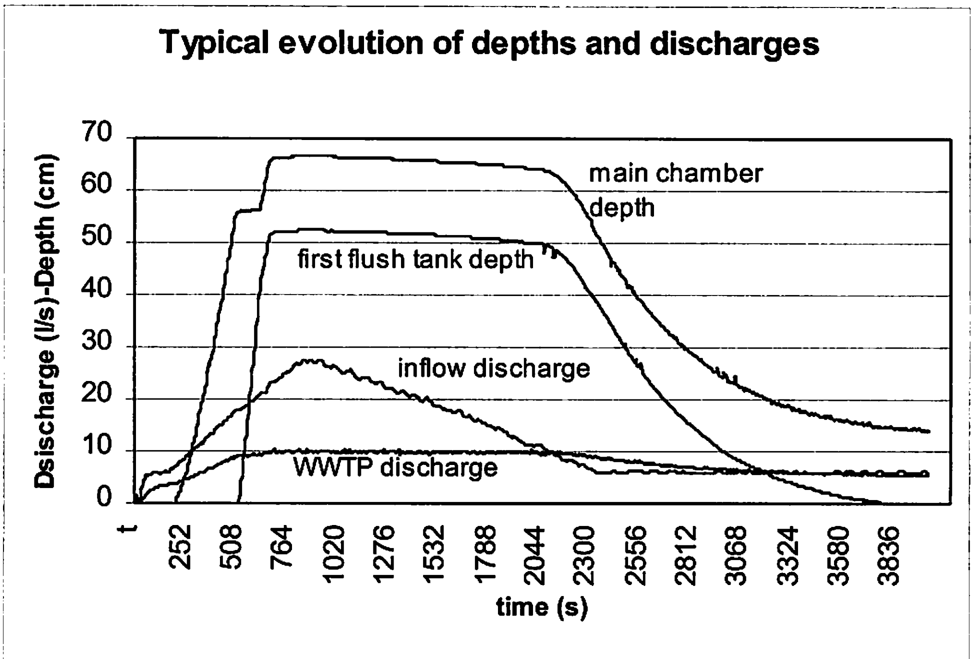


Fig. 6.- Typical behaviour of water depths and discharge distribution in the tanks (main chamber and f.f. tank floor levels are not the same)

All this checking criteria have been applied to the tests in order to warrantee a certain level of accuracy in the results. No errors over 8% have been noticed in any case. Further analysis will be carried out with all these data, with the aim of obtaining a detailed description of the fluxes in the tanks, the behaviour of the spillways, etc.

As a main result of this first approach to the hydraulic behaviour of the tank, we have focussed in the measurement of the efficiency of the structure, in terms of reduction in the release of dissolved pollutants (mass of rhodamine driven to the WWTP vs. mass of pollutant being released to the river).

In order to have a first approach to these data, a series of tests have been designed as explained before. A typical graph for these results is shown in figure 7, where the pollutant mass is expressed in terms of ml of a known solution (input) of rhodamine.

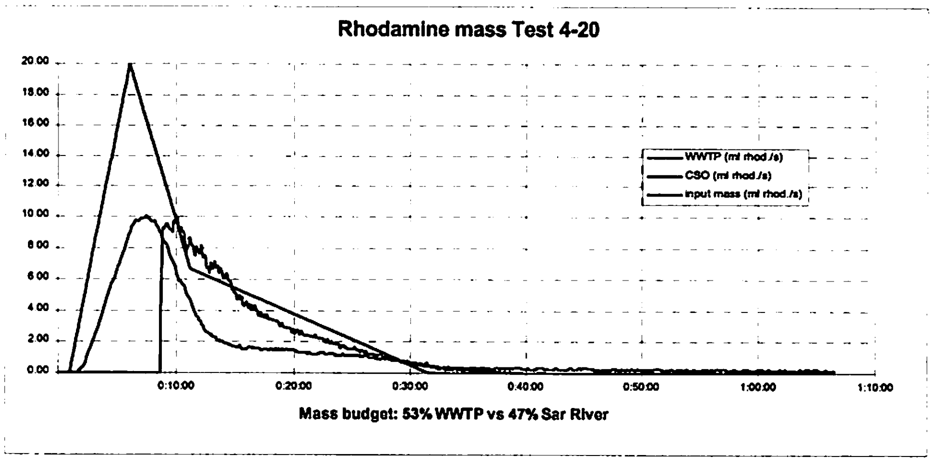


Fig. 7.- Typical test result

As it was expected, the concentrations of rhodamine in the WWTP outlet and in the spillway are equal, as can be observed in figure 8, which also shows that both fluorometers were measuring properly.

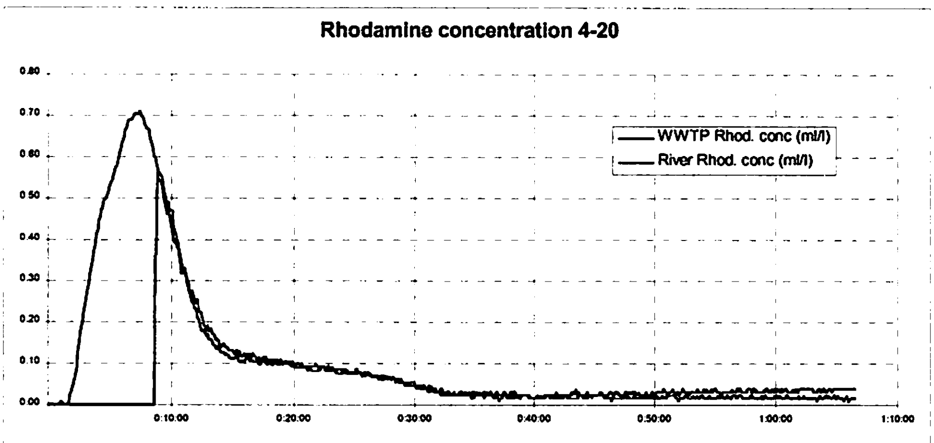


Fig 8.- Fluorometer reliability checking

If we consider a given tank volume, say for instance 4.35 m3/drained Ha, a clear difference in the efficiency can be observed depending of the existence or not of a first flush. In the graphs in figure 9, corresponding to an specific discharge of 17.4 cubic meters per drained Ha, which is presented below, this difference is quite evident.

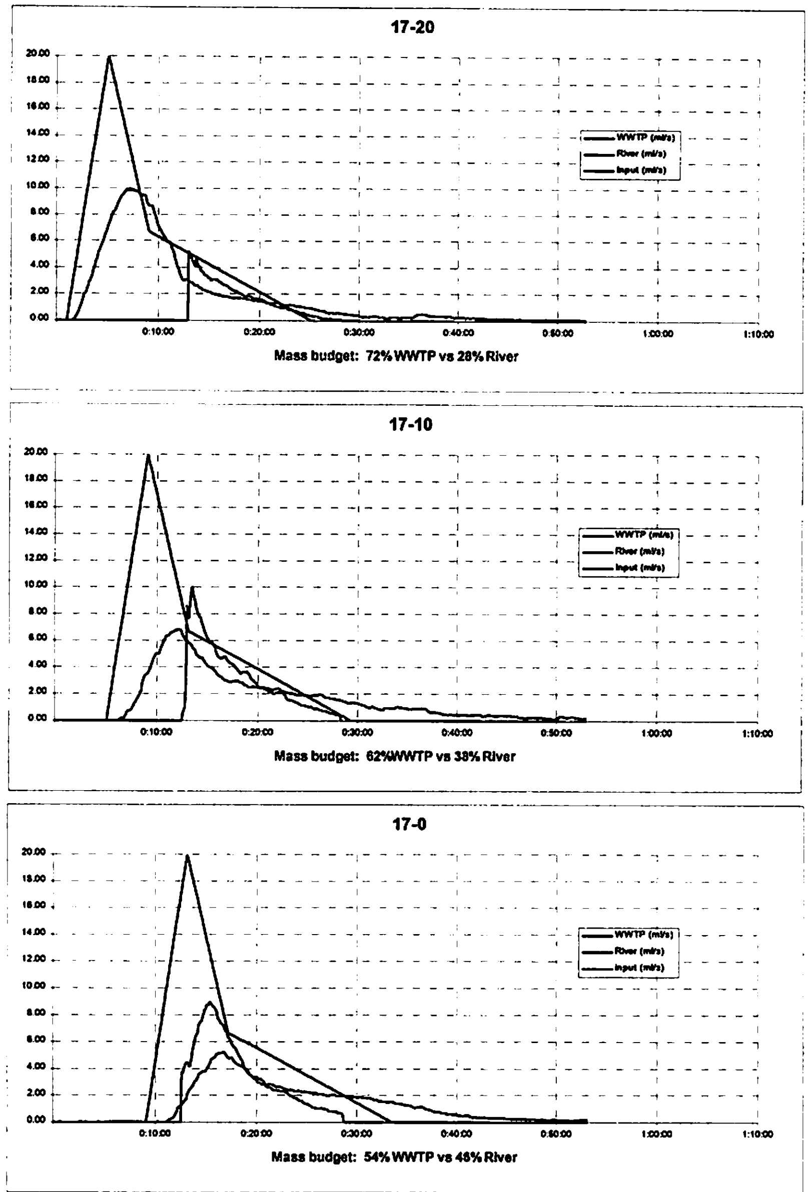


Fig. 9.- Effect of the first flush over the efficiency of the tank

As it can be easily seen, the greater the delay between the pollutograph and the hydrograph is, the more efficiently the tank works. For the same values in discharges and mass of pollutant, a difference of 18% of efficiency can be noticed from the first test to the last one. That fact could be expected, if it is remembered that these tanks are designed so as to catch the first water of the hydrograph. However, if the series of tests with small specific volumes (4.35 or even 8.7 cubic meters per drained Ha) are analysed, this trend will not be so clearly noticed for a 10 minutes delay (as can be seen in table 3). That is due to the fact that this volume is too small even for catching the peak of the first flush (with a 10-min delay), and if any mass is driven to the WWTP is mainly because the capacity of the WWTP pipe grows as the level in the main tank generates a higher hydraulic gradient.

If we consider the variations in volume, the efficiency ratios can be stablished as a function of the specific volume, for a given first flush delay. So, if we consider the series of figures below, for a 20 min delay and specific volumes of 4.35, 8.7, 17.4 and

27.2 cubic meters per drained Ha, the evolution of the volumes ratio can be easily noticed.

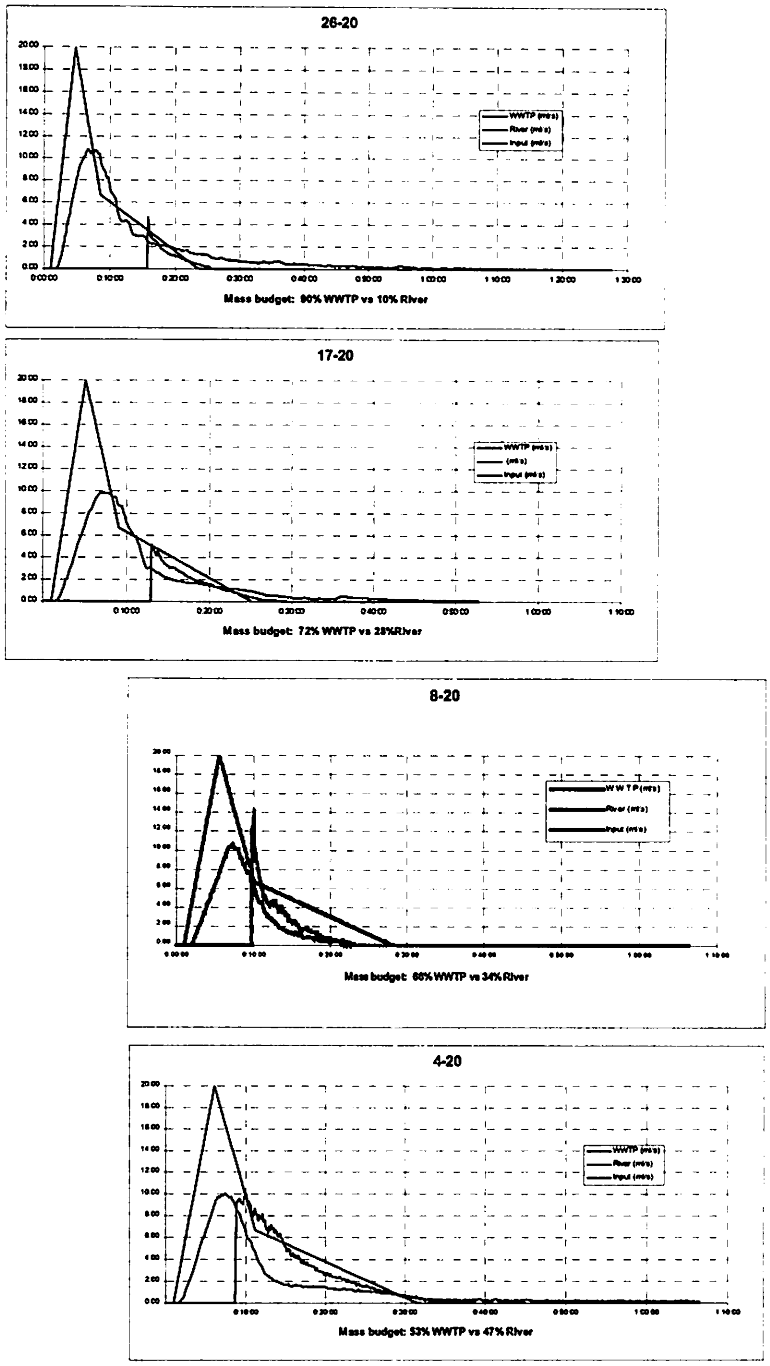


Fig 10.- Effect of the tank volume over its efficiency

Table 3.- Mass of pollutant released to the river (%) as a function of the tank volume and the delay of the first flush

	4.35	8.7	17.4	26.1
0	47	34	28	10

10	62	58	38	14
20	58	46	46	24

As a final remark, velocity sensors located in the first flush tank give an idea of recirculation in the tank (Figure 11). Provided this Froudian model can not reproduce the actual velocity fields due to shear stresses, it can be noticed however that the raise in volume implies a raise in the kinetic energy. It is very noticeable that the maximum peak discharge (corresponding to the 4-0 test) is about 47 l/s, while the actual discharge using a Reynolds model would be over 400 l/s for the same specific volume. It is quite obvious that the velocity modulus for this discharge would be much higher, the washing up much more possible and the settling capability much lower.

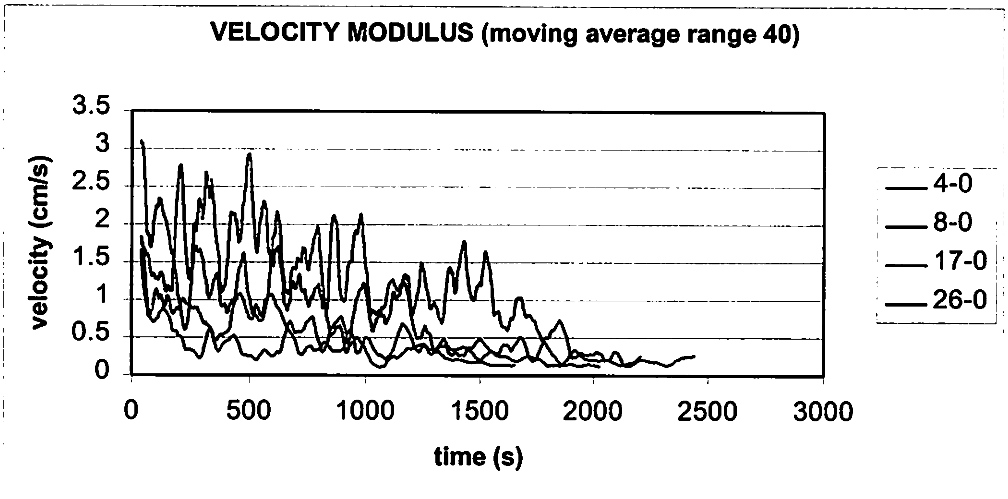


Fig. 11.- Velocity level as a function of the tank volume

Conclusions

First flush tanks show a good performance when their specific volume is over 17 m3/drained/Ha. For smaller specific volumes, their efficiency depend strongly on the delay between the pollutograph peak and the hydrograph peak.

Small tanks do not catch the peaks of pollutographs. However, a certain amount of pollutant mass is driven to the WWTP, and this is mainly by the surcharge effect in the main chamber.

Some washing-up seems to be possible in the first flush tanks. Further analysis must be done in order to confirm this hypothesis.

References

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