Distributed vs. Concentrated Storage Options for Controlling CSO Volumes and Pollutant loads

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Abstract: The Best Management Practices (BMPs), the volume control and pollutant mass removal operated with a stormwater tanks are investigated in the present work. Data collected in 2007 during two monitoring campaigns was used to this aim. During the first monitoring campaign were observed six rainfall-runoff events, and was collected hydrological and quality data that were used to calibrate and to validate the SWMM (Storm Water Management Model – by USEPA) parameters. Ten rainfall-runoff events were observed during the second monitoring campaign, and the data were used to simulate, with SWMM, five different scenarios. Each scenario was characterized by a different tanks distribution on urban catchment surface. The results was analyzed in terms of overflow volume and overall pollutant mass in the receiving water body. The results obtained indicated a good efficiency of CSO volume control and pollutant mass removal for the scenarios characterized by multiple storage tanks implemented at specific locations.

Keywords: Combined sewer overflows (CSO), Rainfall-runoff, BMPs, Treatment, Urban hydrology, Pollution

INTRODUCTION

The issue of wet-weather flow management is becoming a crucial topic in the field of environmental protection. One of the main objectives in the urban drainage management is the control of suspended solids concentrations in stormwater and combined sewer overflows (CSOs) and of their discharge into the receiving waters. This objective may be attained through a series of interventions that begin with source controls, continue with control by Best Management Practices (BMPs) and end with controlling discharges into the receiving waters. The detention and retention tanks are commonly used to control total suspended solids (TSS) in combined sewage systems, while for separate sewage systems the infiltration and wetlands are largely adopted (Marsalek et al., 2002).

Many methods of wet-weather flow management have been developed to achieve low environmental impact systems, and may be divided into two categories. The first one comprises interventions that attempt to restore the original catchment hydrologic cycle, prior to the urbanization, by means of measures such as the permeable pavement systems, vegetated swales, infiltration and biofiltration systems, and infiltration trenches (Lloyd et al., 2002; Dierkes et al., 2002).

The second category comprises structural measures implemented at strategic points of the catchment in the form of flow storage in retention and detention tanks. It is known that such measures require large storage capacities, if the entire overflow volume is to be stored, in order to reduce the number of overflow events to a certain number per year. Therefore, it is evident that for the selection of a correct treatment measure, it is not only necessary to define the nature of the pollutants to be removed, but also to evaluate the space available for implementation of control/treatment measures.

The results obtained by Piro et al. (2007a) demonstrate that if the main constituent to be controlled is COD (chemical oxygen demand), the selected treatment strategy should involve a physical treatment unit, because the COD₇ load (particulate chemical oxygen demand) generally greatly exceeds the COD₄ load (dissolved chemical oxygen demand) and represents as much as 70% of the total COD load.

The above described tanks may be located throughout the catchment and convey the stored runoff volume to a sedimentation tank, before discharging flow into the final tank. This sequence of
structures represents a ‘treatment train’ (Wong et al., 2006). In fact, the ‘treatment train’ is a sequence of structural BMPs that promotes a distributed approach to the management across a catchment (Lloyd et al., 2002).

On the other hand, it is important to take into consideration that although the sustainable development of urban areas is favoured by the distributed measures, like detention ponds, the land developers often consider such measures as a loss of land that could be potentially developed as well as an extra cost due to the construction and operation and maintenance of stormwater facilities. Furthermore, the detention ponds may occupy valuable urban land, and hence, it is desirable to minimize the land cover by these facilities, while satisfying the contemporary water quality control objectives. For this reason, a methodology for optimizing pond geometries in a single catchment and in a set of parallel catchments was studied (Papa et al., 1997) and the cost of using distributed treatment technologies was investigated, particularly by defining the break-even point at which a distributed system is cost-competitive with the traditional centralized treatment approaches (Norton et al., 2006). Thus, although the water industry has successfully used for years central treatment facilities to meet water quality goals, a deeper understanding of water quality degradation and greater capabilities of small-scale water treatment technologies has been recently achieved. The optimal water-treatment strategy is changing from single centralized treatment facilities to distributed treatment technologies (Norton et al., 2006).

The main objective of this paper is to compare distributed vs. concentrated storage options for a study catchment served by combined sewers, by means of modelling with calibrated models, and using overflow volumes and pollutant mass discharged as objective functions. In order to mitigate the impact of urban wet-weather flow discharges onto the receiving waters, a set of scenarios composed of a varying number of detention tanks, distributed over the studied catchment, were modelled. These CSO control and treatment scenarios were applied to the Liguori Channel (LC) catchment in Cosenza, Italy, and compared, in order to study behaviour of such systems under the following two conditions: (a) a concentrated storage system comprising one larger tank with a properly chosen volume, and (b) a series of distributed smaller tanks, located at strategic points of the catchment and releasing smaller, distributed discharges causing smaller impacts on the receiving waters. It was assumed that the only form of treatment in these tanks was balancing of pollutant concentrations due to complete mixing. This latter alternative reflects a modern drainage philosophy and strategy, which postulates that drainage schemes exerting controls closer to the source avoid transfer of problems downstream and should result in cost-effective, affordable and sustainable solutions (Andoh et al., 1997).

STUDY METHODS

Study site and data collection

The catchment studied is the experimental Liguori Channel (LC) catchment, located in Cosenza (Italy). The catchment area is 414 ha, of which 48% is densely urbanized and the remainder (52%) has a pervious cover, represented mostly by vegetated surfaces (Figure 1). More details can be found in Piro et al. (2001).

The catchment is drained by a combined sewer system that conveys the entire dry weather flow to the wastewater treatment plant. During the most intense rainfall events, wet-weather flows occasionally exceed the capacity of the sewer system and the excess flows escape from the sewer system, via an overflow structure, as combined sewer overflows (CSOs). Such overflows are discharged into the receiving water body, the Crati River, without any treatment (Piro, 2007).
Some of the field data used in the present study were collected during the study of flows and their chemical composition in the LC catchment during the period from 1995 to 2003 (Piro, 2007). The monitoring station was located at the network outfall. CSO flow rates were derived from the flow depths measured by an ultrasonic sensor, and the flow rating curve of the control cross-section. Rainfall depths were recorded by a tipping-bucket raingauge. A data acquisition logger, installed by the outlet, recorded the rainfall depths and water levels at 1 minute intervals.

Observations of sewer flow quality have been conducted at the outfall of the LC catchment since 2004, with the objective of obtaining a sufficient number of representative samples for characterization of wet and dry weather flows. Discrete grab samples were collected manually, in duplicates, at 15-minute time intervals, using wide-mouth bottles. The samples were analyzed in the Acquedotti & Fognature Laboratory of the University of Calabria for the following water quality constituents of interest: oxygen-demand substances, measured as chemical oxygen demand (COD), total suspended solids (TSS), pH and conductivity (Sansalone et al., 2007). Suspended solids (TSS) and COD were analyzed using protocols 2540D and 5220B, respectively, of the Standard Methods (APHA 1998).

Analysis of the CSO volumes is carried out using a stochastic approach, assuming a well-fitting lognormal distribution displayed in Figure 2 (Piro et al., 2007b). The goodness of fit was tested by linear regression, which yielded a correlation coefficient \( R^2 = 0.98 \) (Figure 2b). By using this stochastic approach, it is possible to define the overflow volume that occurs with a specific a probability of non-exceedance, and hence, one can determine the storage volume of the tank needed to avoid the discharge of CSOs into the receiving waters for a chosen probability value. The value of 5000 m\(^3\), equivalent to 25 m\(^3\)/ha (Calomino et al., 2005), meets probability of non-exceedance of 60%, on the fitted log-normal distribution (Figure 2a). Details of the hydrologic data of the storm events used to determine the log-normal distribution were reported elsewhere (Piro et al., 2007b).

**Model used and event delivery mass analysis**

For prediction of general performance of a series of treatment structures (i.e., a treatment train), one can use mathematical models that provide sophisticated descriptions of the hydraulic behaviour and pollutant transport in the watershed studied. Such an analysis is very useful, because the design of storage tanks requires not only the specification of the flow peak time, peak flow rate and the total runoff volume, but also the determination of hydrographs and pollutographs in every section of the drainage system at certain time intervals. In order to simulate the response of the LC drainage system to storm events, the SWMM model was used (*Storm Water Management Model*, USA EPA, 2005).
To ensure reliable results, environmental models require calibration and verification. In the present study, the input data and parameters required by the SWMM model to simulate drainage flow hydrographs are physiographic characteristics of the catchment (e.g., area and slope), geometric and hydraulic characteristics of the sewer pipes (diameter, length, slope and roughness coefficient), and the hydrological/hydraulic parameters such as the width of the subcatchments, catchment roughness described by the Manning coefficient, surface depression depths, and the infiltration rates for pervious areas.

In this study, the calibration parameters taken into consideration in flow modelling were: surface roughness of the conduit \((n)\) and of the impervious \((N-Imperv)\) and pervious \((N-perv)\) surfaces in the catchment, and the depths of surface depressions on impervious \((Dstore-Imperv)\) and pervious \((Dstore-Perv)\) areas. For quality modelling, there were four more calibration parameters: \(P_1\) = the maximum potential build up (mass per unit of area), \(P_2\) = build up rate constant, \(P_3\) = wash-off coefficient, and \(P_4\) = wash-off exponent. These parameters were calibrated using the events recorded in the Liguori Channel catchment on the following dates: 15/02/2007, 04/04/2007, and 26/04/07. Finally, the calibrated model was verified against the storms monitored in the LC catchment on 14/04/2007, 27/4/2007 and 18/05/2007 (Piro, 2007) (Table 1).

Table 1: General SWMM parameters used in the model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>(n_{IMP}) ((m^{1/3}/s))</th>
<th>(n_{PER}) ((m^{1/3}/s))</th>
<th>(d_{storeIMP}) (mm)</th>
<th>(d_{storePER}) (mm)</th>
<th>(n) ((m^{1/3}/s))</th>
<th>(P_1) (Kg/ha)</th>
<th>(P_2) (1/day)</th>
<th>(P_3)</th>
<th>(P_4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>0.02</td>
<td>0.25</td>
<td>2.0</td>
<td>2.0</td>
<td>0.011</td>
<td>150</td>
<td>0.01</td>
<td>0.025</td>
<td>8</td>
</tr>
</tbody>
</table>

- \(n_{IMP}\): Manning’s \(n\) for overland flow over the impervious portion of the subcatchment
- \(n_{PER}\): Manning’s \(n\) for overland flow over the pervious portion of the subcatchment
- \(d_{storeIMP}\): Depth of depression storage on the impervious portion of the subcatchment
- \(d_{storePER}\): Depth of depression storage on the pervious portion of the subcatchment
- \(n\): Manning’s roughness coefficient of the conduit
- \(P_1\): The maximum potential build up (mass per unit of area)
- \(P_2\): Build up rate constant
- \(P_3\): Wash-off coefficient
- \(P_4\): Wash-off exponent
Characteristics of calibration and verification storm events are presented in Table 2; measured flow rates and rainfall intensities, normalized by the respective maximum values (i.e., $Q(t)/Q_p$ and $i(t)/i_{\text{max}}$), and normalized modelled flow rates are displayed in Figure 3 for the events listed in Table 2.

**Table 2: Characteristics of the storm events used in calibration and verification of the SWMM model**

<table>
<thead>
<tr>
<th>Event</th>
<th>start</th>
<th>Rain</th>
<th>D</th>
<th>$h_{\text{tot}}$</th>
<th>$i_{\text{avg}}$</th>
<th>$i_{\text{max}}$</th>
<th>start</th>
<th>Runoff</th>
<th>D</th>
<th>$V_{\text{runoff}}$</th>
<th>$V_{\text{rain}}$</th>
<th>$Q_p$</th>
<th>$t_d$</th>
<th>$t_p$</th>
<th>$t_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>hh:mm</td>
<td>min</td>
<td>mm</td>
<td>mm</td>
<td>mm/h</td>
<td>mm/h</td>
<td>hh:mm</td>
<td>mm</td>
<td>mm/h</td>
<td>m$^3$</td>
<td>m$^3$</td>
<td>m$^3$/s</td>
<td>min</td>
<td>min</td>
<td>min</td>
</tr>
<tr>
<td>15-Feb-07</td>
<td>14.08</td>
<td>79</td>
<td>3.6</td>
<td>2.7</td>
<td>12</td>
<td>14.28</td>
<td>123</td>
<td>2723</td>
<td>14904</td>
<td>1.10</td>
<td>156</td>
<td>81</td>
<td>61</td>
<td></td>
<td></td>
</tr>
<tr>
<td>04-Apr-07</td>
<td>5.29</td>
<td>205</td>
<td>9</td>
<td>2.6</td>
<td>4</td>
<td>5.55</td>
<td>179</td>
<td>2993</td>
<td>37260</td>
<td>1.22</td>
<td>395</td>
<td>191</td>
<td>165</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14-Apr-07</td>
<td>15.50</td>
<td>80</td>
<td>2</td>
<td>1.5</td>
<td>4</td>
<td>16.34</td>
<td>78</td>
<td>416</td>
<td>8280</td>
<td>0.37</td>
<td>126</td>
<td>71</td>
<td>27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>26-Apr-07</td>
<td>10.34</td>
<td>96</td>
<td>2.6</td>
<td>1.6</td>
<td>2.4</td>
<td>10.34</td>
<td>294</td>
<td>2302</td>
<td>10764</td>
<td>0.59</td>
<td>251</td>
<td>97</td>
<td>97</td>
<td></td>
<td></td>
</tr>
<tr>
<td>27-Apr-07</td>
<td>17.46</td>
<td>42</td>
<td>1.8</td>
<td>2.6</td>
<td>12</td>
<td>18.03</td>
<td>88</td>
<td>646</td>
<td>7452</td>
<td>0.49</td>
<td>135</td>
<td>29</td>
<td>12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18-May-07</td>
<td>17.49</td>
<td>14</td>
<td>1</td>
<td>4.3</td>
<td>24</td>
<td>18.10</td>
<td>58</td>
<td>571</td>
<td>4140</td>
<td>0.45</td>
<td>82</td>
<td>26</td>
<td>5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Following the calibration of modelled flows, water quality subroutines in the SWMM model were also calibrated, using parameters $P_1$-$P_4$ listed in Table 1, and assuming these four parameters and other characteristics of the catchment (land use, physical propriety of the catchment, pollutant inputs due to precipitation, build up and wash-off rates) to be constant for all storms.

In the latter case, the parameters of interest include the maximum build up of pollutants on the catchment surface (max build-up mass), and the washoff coefficient and exponent. The goodness of fit of modelled hydrographs and pollutographs is described in Table 3 by the linear regression coefficient $R^2$.

**Table 3: Characteristics of the storm events used in calibration and verification of the SWMM model**

<table>
<thead>
<tr>
<th>Event</th>
<th>Quantity Simulations</th>
<th>Quality Simulations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$R^2$</td>
<td>$R^2$</td>
</tr>
<tr>
<td>Calibration</td>
<td>15 February 2007</td>
<td>0.74</td>
</tr>
<tr>
<td></td>
<td>4 April 2007</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>26 April 2007</td>
<td>0.79</td>
</tr>
<tr>
<td>Verification</td>
<td>14 April 2007</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>27 April 2007</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>18 May 2007</td>
<td>0.90</td>
</tr>
</tbody>
</table>

The data in Table 3 and Figure 3 indicate that the agreement between the observed and modelled hydrographs and pollutographs is fairly good. Such results confirm the validity of the calibrated SWMM model and its ability to serve as a good prediction tool.

The calibration of the SWMM model allowed identification of the optimal model parameters to be used in simulations of pollutographs for storm events for which it was not possible to obtain experimental data, as well as to mimic the behaviour of the drainage network, in terms of both, runoff volumes and pollutant loads, for the different types of treatment applied.
Figure 3: Characteristics of calibration and verification storm events

As an alternative to deterministic modelling of the constituent fluxes with the SWMM model, it is possible to adopt a conceptual model expressing the normalized cumulative constituent flux as a function of the normalized cumulative flow, introduced into stormwater event analysis by Marsalek (1976) for describing stormwater quality variations during events and detecting the occurrence of high concentrations (the first flush) during the initial parts of runoff. This function can be used to analyze the catchment response with respect to the first flush phenomenon (Marsalek, 1976; Bertrand-Krajewski et al., 1998; Deletic, 1998; Sansalone et al., 2004). Individual events then can be classified as the events of which pollutant load is limited by the flow volume (i.e., flow limited) and the events of which pollutant load is limited by the available pollutant mass (i.e., mass limited).

Defined the cumulative volume at time \( t \), \( V_t = \sum \nabla_j \), and the cumulative mass at time \( t \), \( M_t = \sum \nabla_j \cdot \text{TSS} \), where \( \nabla_j \) and \( m_j \text{(TSS)} \) are, respectively, the flow volume and the mass of TSS at time \( j \), it possible observe that the first case emphasizes a linear relationship between \( V_t(t) \) and \( M_t(t) \), and while the second case is described by an exponential relationship (Sansalone and Cristina, 2004). The storm events listed in Table 4 were analyzed by the above mentioned method and the results are shown in Figure 4.
### Table 4: Hydrologic data for storm events used in simulations

<table>
<thead>
<tr>
<th>Event</th>
<th>Start rain event</th>
<th>Rainfall D</th>
<th>( h_{\text{tot}} )</th>
<th>( i_{\text{avg}} )</th>
<th>( i_{\text{max}} )</th>
<th>Start runoff D</th>
<th>Runoff D</th>
<th>( V_{\text{runoff}} )</th>
<th>( V_{\text{rain}} )</th>
<th>( Q_p )</th>
<th>( t_d )</th>
<th>( t_p )</th>
<th>( t_r )</th>
<th>( \Delta )</th>
<th>a.d.d.</th>
</tr>
</thead>
<tbody>
<tr>
<td>27 Jul 06</td>
<td>15.30</td>
<td>61</td>
<td>10.2</td>
<td>10.0</td>
<td>48</td>
<td>15.45</td>
<td>59</td>
<td>11255</td>
<td>42228</td>
<td>6.6</td>
<td>73</td>
<td>25</td>
<td>10</td>
<td>0.30</td>
<td>8</td>
</tr>
<tr>
<td>10 Feb 07</td>
<td>8.08</td>
<td>162</td>
<td>11.2</td>
<td>4.1</td>
<td>12</td>
<td>8.55</td>
<td>185</td>
<td>14756</td>
<td>46368</td>
<td>3.3</td>
<td>231</td>
<td>69</td>
<td>22</td>
<td>0.58</td>
<td>4</td>
</tr>
<tr>
<td>01 May 07</td>
<td>6.59</td>
<td>862</td>
<td>37.0</td>
<td>2.6</td>
<td>24</td>
<td>7.35</td>
<td>714</td>
<td>52032</td>
<td>153180</td>
<td>6.5</td>
<td>749</td>
<td>153</td>
<td>36</td>
<td>0.52</td>
<td>3</td>
</tr>
<tr>
<td>28 May 07</td>
<td>2.27</td>
<td>393</td>
<td>29.6</td>
<td>4.5</td>
<td>72</td>
<td>2.39</td>
<td>381</td>
<td>50937</td>
<td>122426</td>
<td>15.3</td>
<td>392</td>
<td>22</td>
<td>10</td>
<td>0.66</td>
<td>4</td>
</tr>
<tr>
<td>11 Oct 07</td>
<td>15.24</td>
<td>56</td>
<td>12.2</td>
<td>13.1</td>
<td>96</td>
<td>15.38</td>
<td>50</td>
<td>7416</td>
<td>50508</td>
<td>7.4</td>
<td>63</td>
<td>18</td>
<td>4</td>
<td>0.27</td>
<td>4</td>
</tr>
<tr>
<td>21 Oct 07</td>
<td>0.56</td>
<td>3325</td>
<td>33.8</td>
<td>0.6</td>
<td>24</td>
<td>1.41</td>
<td>3728</td>
<td>50125</td>
<td>139932</td>
<td>2.75</td>
<td>3772</td>
<td>1350</td>
<td>305</td>
<td>0.31</td>
<td>1</td>
</tr>
<tr>
<td>09 Nov 07</td>
<td>13.05</td>
<td>862</td>
<td>18.4</td>
<td>1.3</td>
<td>72</td>
<td>13.25</td>
<td>1341</td>
<td>27896</td>
<td>76176</td>
<td>5.27</td>
<td>1360</td>
<td>651</td>
<td>63</td>
<td>0.49</td>
<td>2</td>
</tr>
<tr>
<td>13 Nov 07</td>
<td>0.01</td>
<td>3846</td>
<td>88.6</td>
<td>1.4</td>
<td>48</td>
<td>0.35</td>
<td>4255</td>
<td>197711</td>
<td>366804</td>
<td>11.16</td>
<td>4288</td>
<td>1836</td>
<td>1802</td>
<td>0.43</td>
<td>1</td>
</tr>
<tr>
<td>03 Dec 07</td>
<td>22.34</td>
<td>895</td>
<td>24.4</td>
<td>1.6</td>
<td>36</td>
<td>22.55</td>
<td>1419</td>
<td>54356</td>
<td>101016</td>
<td>5.45</td>
<td>1439</td>
<td>61</td>
<td>40</td>
<td>0.57</td>
<td>3</td>
</tr>
<tr>
<td>09 Dec 07</td>
<td>21.16</td>
<td>919</td>
<td>30.2</td>
<td>2.0</td>
<td>84</td>
<td>21.43</td>
<td>977</td>
<td>56002</td>
<td>125028</td>
<td>12.44</td>
<td>1003</td>
<td>603</td>
<td>576</td>
<td>0.50</td>
<td>4</td>
</tr>
</tbody>
</table>

**Rain D** = Rainfall duration  
**\( h_{\text{tot}} \)** = Total rainfall depth  
**\( i_{\text{avg}} \)** = Average rainfall intensity of the event  
**\( i_{\text{max}} \)** = Maximum rainfall intensity of the event  
**Runoff D** = Runoff duration  
**\( V_{\text{runoff}} \)** = Total runoff volume  
**\( V_{\text{rain}} \)** = Total rainfall volume  
**\( Q_p \)** = Runoff peak flow  
**\( t_d \)** = Event duration  
**\( t_p \)** = Time to \( Q_p \) (measured from start of rainfall)  
**\( t_r \)** = Time to \( Q_p \) (measured from start of runoff)  
**\( \Delta \)** = Shape parameter for \( M(V) \) curve  
**a.d.d.** = Antecedent dry days

In each chart of Figure 4 are shown with dashed line the relationships between the normalized cumulative mass of TSS, \( M(t) = \frac{M_T(t)}{M_T(t_d)} \), and the normalized cumulative volume, \( V(t) = \frac{V_T(t)}{V_T(t_d)} \), and the shaded shapes depict the normalized hydrographs (i.e., the normalized flow rate, \( Q/Q_p \), versus the normalized elapsed time, \( t/t_d \), where \( t_d = \) event duration).

When examining the events in Figure 4, it can be noted that there are differences in pollutant export of various mass-limited events. To be able to distinguish the flow limited or mass limited behaviour, it is required to simulate individual events continuously. The determination of the event type (i.e., mass or flow limited) is important for choosing the appropriate treatment.

In order to interpret different characteristics of the studied mass-limited events, a shape parameter \( \Delta \) of the curve \( M(V) \) is introduced and defined as:

\[
\Delta = \int_0^1 (M(V) - M_b) \, dV
\]

where \( M \) = the normalized cumulative mass, \( V \) = the normalized cumulative volume, \( t \) = the elapsed time, \( M(V) \) is a function of the normalized cumulative volume of the normalized cumulative mass, and \( M_b \) is the line \( M(t) = V(t) \). Values of \( \Delta \) are reported in Table 4 for all the simulated events together with their main hydrologic characteristics.

The \( \Delta \) shape parameter allows making a quantitative evaluation of mass-limited events; values of \( \Delta \) greater than 0.4 indicate mass-limited behaviour characterized by mass transport in the very early phase of the event. Values of \( \Delta \) close to 0 or negative indicate flow-limited behaviour.
Proposed sustainable storage scenarios
In order to analyze the behaviour of the Liguori Channel sewer network in terms of mass and volumes discharged into the Crati River, five different scenarios were considered and characterized.
by the number of tanks, located in series, along the network. Each tank has a capacity proportional to the wastewater and stormwater volumes (i.e., wet-weather sewage) that reach it. The total volume of the tanks considered in different scenarios was constant and equalled 5,000 m$^3$.

**Scenario 0.** In order to evaluate the existing conditions in the sewer system, simulations were carried out without any tank storage in place (see Figure 5).  **Scenario 1.** In this scenario, six prismatic tanks were proposed, with the upstream ones located on the main channel and the downstream ones located on the secondary sewer lines.  **Scenario 2.** This scenario is characterized by ten prismatic storage tanks located downstream of the secondary sewer lines.  **Scenario 3.** This scenario is characterized by eight prismatic tanks, with three located in the downstream area by the main sewer system nodes and the remaining five located upstream on the secondary pipes.  **Scenario 4.** Four storage tanks are located downstream of the major confluence nodes. Finally, the schematic of the tank system relevant to Scenarios 1-4 is shown in Figure 6.

![Figure 5: Sustainable scenarios studied (designated and presented in the order 0, 1, 2, 3, and 4, from left to right)](https://iwaponline.com/wpt/article-pdf/5/3/wpt2010071/382660/71.pdf)

![Figure 6: Schematic of the detention tanks located in downstream sections of the network)](https://iwaponline.com/wpt/article-pdf/5/3/wpt2010071/382660/71.pdf)

It’s important to note that the volume stored in the detention tank is released directly into the main sewer network.

The last Scenario 5 is shown in Figure 7; it is characterized by a concentrated detention volume located just upstream of the sewer network outfall. At this tank, flows below the wastewater
treatment plant (WWTP) capacity are diverted to the WWTP; excess flows are diverted to the storage tank, and once the tank is full, excess flows overflow into the receiving water, the Crati River.

Figure 7: Scenario 5

The schematic of the sewer system related to Scenario 5 is shown in Figure 8; it should be noted that overflows are discharged directly into the receiving water. Further analysis will focus on practical solutions for emptying the tank.

Figure 8: Schematic of the detention tank located downstream of the sewer network (flow diverted to WWTP = 0.6 m³/s)

RESULTS AND DISCUSSION

The differences in terms of hydraulics and environmental impacts of different alternative scenarios were determined in order to evaluate under which conditions the distributed storage volume system becomes more preferable than the concentrated storage volume system. As mentioned earlier, it was assumed in this study that each tank basically works as a completely mixed reactor, in which the effect of dilution under steady conditions gradually reduces the concentrations in the outflow (Metcalf et al., 1991). In Figure 9 below, the effect of pollutant mass routing through one tank located downstream of the network is shown.

Furthermore, the hydrographs and pollutographs for each scenario applied to the Sept. 11, 2007 event are shown in Figure 10, in order to compare the performance of various scenarios. It can be observed in Figure 10 that the distributed system discharged into the receiving water body lower flow rates and pollutant concentrations than the concentrated system. Such results are very interesting with respect to the potential receiving waters benefits; in fact, it can be observed that the scenarios with distributed storage are characterized by reduced pollutant concentration (i.e., compared to the peak concentration from the concentrated storage system) and this is very important for sustaining self-purification processes of the receiving waters, which can better cope with smaller mass concentrations associated with lower discharges.
Quantity and quality simulations for six rainfall-runoff events were carried out to evaluate the relative performance of concentrated and distributed detention storage systems. The comparison among the different scenarios was carried out on the basis of the pollutant mass removal efficiency of each scenario. For each event, the volume and pollutant mass discharged were evaluated for all the scenarios; the pollutant mass was described by the total suspended solids (TSS) loads.

Figure 11 displays the TSS mass and the CSO volume discharged in each scenario and normalized with respect to the TSS mass and the CSO volume discharged in Scenario 0 ($V_{T(0)}$ and $M_{T(0)}$) without any storage tanks.

In Figure 11, in which mean values and standard deviations of $V_T/V_{T(0)}$ and $M_T/M_{T(0)}$ are shown, it is possible to observe for the total storage volume of 5,000 m$^3$, the following:

- Similar results, in terms of CSO volume, were obtained for all the five scenarios, without showing substantial variations in the efficiency of reducing the discharged volume, with the average reduction of 30%, and about 40% at most, for the fifth scenario with the concentrated storage volume by the sewer network outfall,
- The beneficial effect of tank placement is clearly evident; the more efficient scenarios are those with only one tank (Scenario 5) or with tanks distributed along the main sewer conduit (Scenario 4); the tanks located downstream of the secondary pipes of the network (Scenario 2) showed the worst performance in terms of average CSO volume reduction.
In Figure 11, the values of the TSS loads, normalized by the maximum load value \(M_{T(0)}\) obtained in Scenario 0 (no storage tanks), and displayed as the mean values and standard deviations, are presented for all the scenarios.

![Figure 11: Normalized TSS mass and CSO volume discharged for different scenarios](image)

It is possible to observe relatively good efficiency in controlling TSS mass released with overflows; good reductions, up to 40% (i.e., 1.0 – minus 0.6) in the case of Scenario 1, but less than 30% in the case of Scenario 5, were noted.

The analysis of two groups of storm events, defined according to the characteristics of pollutant transport and specifically in relation to the value of parameter \(\Delta\), turns out to be particularly interesting. The study presented herein shows that during the monitoring period, the most critical events in terms of overflow volumes and pollutant loads discharged into the receiving water body are those classified as mass limited. For some of the events, for which \(\Delta > 0.4\), the beneficial effects of storage and treatment are clearly evident; for the others, with \(\Delta < 0.4\), the mass is released more gradually in time than the flow volume. Thus, if one divides the storm events according to the two categories mentioned above, it can be observed:

- Specific behaviour of the events with \(\Delta > 0.4\) (Figure 12) tends to favour the selection of distributed systems,
- Characteristic behaviour of the events with \(\Delta < 0.4\) (Figure 13) does not show particular advantages of any of the analyzed scenarios.

![Figure 12: Normalized TSS masses and volumes discharged for various scenarios applied to the events with \(\Delta > 0.4\)](image)
These results emphasize a better performance of the distributed tank systems, especially for events characterized by $\Delta$ greater than 0.4. The results further indicate that the volumetric control efficiency of prismatic tanks in series is likely higher than that of a single tank with large volume, for more than one reason. First, the hydraulic residence time of a distributed system is likely greater than that of a concentrated system: in the distributed system the volume of the conveyance pipes between individual tanks increases the residence time, thus extending the travel path of the pollutant particles. The appurtenances of these structures have also another positive effect on providing a lower impact on the environment than a larger tank. Furthermore, the storage volume in tanks in series is used more efficiently; in fact, dead space can potentially be reduced and, in addition, in terms of maintenance it is clear that a small tank is easier to clean than a large tank. In theory, a series of small tanks in a catchment system begins to approach a plug-flow reactor model of the catchment drainage system.

Figures 12 and 13 show a better performance of the distributed tank storage, especially with respect to the TSS mass removed. In fact in this case, it is possible to observe a very low sensitivity of the assessment of different scenarios and consider the removal efficiency of both systems (i.e., concentrated and distributed) quite similar. For this reason, in an urban catchment without available land, it is better to use the distributed system than the concentrated system. However, the choice of a system to be adopted depends on different factors, such as costs, available space, environmental sustainability and efficiency of different systems in mitigating the impacts on the receiving waters. The results of this research show substantially similar behaviours of different systems studied in terms of treatment efficiencies; so the system chosen for implementation should be the one that best fits the criteria of environmental sustainability, in terms of planning and economic investment.

It’s possible to have a better removal capacity for all scenarios if the last tank is constructed like a clarifier. This result was observed through previous research work (Calomino et al., 2005) where the removal efficiency improved about of the 20%. A series of experimental tests on real models are carried out in order to have more information on these important topics.

**CONCLUSION**

A study of the mitigation of the impacts of CSOs from the Liguori Channel catchment on the receiving waters indicated that a distributed system of storage tanks in series is a preferred solution, from the sustainable development point of view. The distributed storage systems turned out to be more efficient, in comparison to the traditional interventions employing large storage tanks. Such efficiency was analyzed on the basis of simulations of treatment processes for six scenarios using
the SWMM model. The results obtained indicated a good efficiency of CSO volume control and pollutant mass removal for the scenarios characterized by multiple storage tanks implemented at specific locations. The scenario with only one tank downstream of the network turned out to be the optimal scenario. However, it has to be recognized that if using tanks with less volume than needed to solve issues regarding environmental sustainability, and depending on the availability of land, it is possible to opt for a distributed system that guarantees a good CSO volume and pollutant mass reduction efficiency, as demonstrated in this study. Since the choice of the system depends on different factors, including construction costs, the availability of urban land, and the environmental sustainability of various interventions, this study offers a new methodology for assessing behaviour of different scenarios and, in particular, for comparing them in terms of their efficiency. Hence, it is possible to determine the differences in CSO mass and volume reduction obtained by using a particular scenario. Finally, in the absence of representative cost data for the maintenance and operation of the distributed systems, the analysis of such issues is deferred to the future research.

LIST OF REFERENCES


