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Selection of the Optimal Design Rainfall Return Period of Urban Drainage Systems

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Abstract

The aim of this work is to define a methodology to identify the optimal rainfall return period for the design of urban drainage systems. The choice of the optimal return period is made minimizing the total costs of the system: the sewer network is dimensioned for a set of possible design rainfall return periods, and the corresponding construction, maintenance and operation costs are evaluated. For each scenario, the total expected damage from flooding caused by rainfall events with return period greater than the design one is then estimated by hydraulic simulation. This methodology has been applied to a small urban catchment in Palermo (Italy).

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1. Introduction

The assessment and management of flood risk in urban areas have assumed an increasingly central role in the context of both research and engineering practice, also due to the climate changes, causing the increase of frequency and intensity of rainfall events, and to the raising urbanization [1]. The need to optimize the investments related to urban drainage systems has prompted the researchers to investigate design approaches different from the conventional ones based on the a priori assumption of the return period of the design rainfall.

Even if the choice of the return period can be based on the hydrologic-hydraulic characteristics of the catchment [2] this approach is nowadays outdated. The current trend is actually aimed at minimizing the investment costs and

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the costs of managing and operating the networks, identifying the less expensive technical solution by focusing on the concept of risk of flooding.

The aim of this work is to define a methodology to identify the optimal rainfall return period, $T$, for the design of an urban drainage system, taking into account the construction, maintenance and operation costs of the system and its benefits, both in socio-environmental terms and in terms of avoided economic losses from possible flooding due to the system surcharges.

The optimal return period is therefore deduced by a minimization analysis of total costs, as formerly suggested by Paoletti [3]; the proposed methodology is based on an economic analysis of the effects of the system surcharges, thus avoiding the adoption of design standards non related with the characteristics of the urban area and, more generally, of the socio-economic environment [4].

Such analysis is performed by dimensioning the sewer network for a set of possible design rainfall return periods, $T_i$, and assessing the construction costs associated with each value of $T$. For each scenario, the total expected damage caused by rainfall events with return period greater than the design one, during the entire operation period of the infrastructure, is then evaluated by hydraulic simulation.

The estimation of building, operation and maintenance costs of the infrastructure are carried out adopting the life cycle cost analysis (LCCA) concepts. The urban drainage system operation is simulated using the well know EPA SWMM hydrologic-hydraulic model [5]. The expected damage resulting from the inadequacy of the urban drainage system is quantified in terms of both damage to the buildings [6] and economic damage from traffic congestion.

The proposed methodology has been applied to a small urban catchment near the city of Palermo (Italy), in order to verify its applicability and identify its criticality.

In the following sections, the methodology is first proposed (section 2); then, the applications to the case study is presented (section 3) and conclusions are eventually drawn (section 4).

2. Methodology

2.1. Methodology for the selection of the optimal rainfall return period

The idea, proposed in early stage by Paoletti [3], is based on the identification of the optimal return period, $T$, of the design rainfall of a drainage network through the minimization of the sum of the present values of construction, maintenance and operation costs and of the estimated total cost of the damages, caused by flooding due to the network surcharges, during the system operating lifetime.

As disclosed above in section 1, such analysis can be performed by designing the network, even if roughly, for a set of conveniently selected design rainfall return periods, $T_i$, and assessing the corresponding construction costs. For each of these possible network configurations, it is then necessary to evaluate the total expected value of the damages caused by rainfall events with return period greater than the design one, occurring during the entire operating lifetime of the system. The optimal return period of the design rainfall is the one yielding the minimum sum of construction, maintenance and operation costs and total expected damage. The design rainfall return periods $T_i$ chosen in this work are 2, 5, 10, 15 years.

The proposed methodology, summarized in the flowchart of Fig. 1, is based on three major simplifying hypothesis:

- the return period of the damage is equal to the return period of the rainfall event from which it arises;
- the damage can be assessed by assuming that it depends solely on the maximum depth locally reached by the water [6];
- all the consequences of flooding can be quantified in monetary units.

For the application of the proposed methodology three types of data are required:

- physical data of the urban area, that is morphological and topographical data, land uses and zonation, typology and characteristics of buildings (number of floors, presence or absence of basements, construction materials, preservation status, etc.);
- hydrologic and hydraulic data, such as rainfall events characteristics, hydrologic and hydraulic characteristics of the urban catchments and sub-catchments, water depths reached with respect to ground surface;
- economic data of the urban area and information about the presence of monuments or artworks to protect [6].
2.2. Assessment of construction, maintenance and operation costs

The evaluation of the overall cost of the network has been performed according to the LCCA philosophy, considering its entire life cycle. The overall cost of an infrastructure is in fact represented by the cost of construction as well as all the expenses that would be supported for its use and its maintenance during its useful life. The residual value of the infrastructure at the end of its useful life has not been considered in this work.

The total costs of the infrastructure are ultimately given by the sum of investment costs $C_i$ and maintenance and operation costs $C_{oper}$.

The investment cost $C_i$ is composed of three items: the purchase cost of the pipeline $AC$, the cost of work $MAN$ and the cost of installation and connections $POS$. The purchase cost of the pipes $AC$ has been determined on the basis of the new Price List for Public Works of Sicily Region, published in March 2013. The overall cost of work $MAN$ is obtained by assuming the employment, for 8 h/day, of a work team formed by: one skilled worker (hourly cost 25.95 €/h), three ordinary workers (hourly cost 21.78 €/h), one excavating machine with operator (hourly cost 54.68 €/h). The daily productivity of the work team in m/day has been assumed to be inverse function of the pipe diameter. Installation cost and connections cost $POS$, in the present analysis, is constituted by:

- Excavation and transportation of waste material: cutting of road pavement; excavation of the trench (the costs depend on the depth and width as well as on the characteristics of the soil); transportation and disposal of waste materials.
- Restorations: pipe bedding; backfilling; restoration of road pavement.

The cost of manholes and customers' connections are not included in this analysis while largely not dependent on the return period of design rainfall.

Operation and maintenance costs $C_{oper}$ are the sum of: maintenance cost $C_m$, inspection costs $C_{in}$, cleaning costs $C_e$ and any replacement costs of the pipes $C_r$ (which are not considered in this analysis).

The maintenance costs $C_m$ depend on four factors: the correct evaluation of all factors and variables in the design phase; the good implementation of the project; any tampering to the pipeline due to customers’ connections; erosion
and corrosion of the pipes. In the present study a maintenance cost of 2.57 €/m, basically not dependent on the pipe diameter, has been considered; this value is referred to HDPE pipes [7].

Inspection costs are highly dependent on the pipe diameter and on the frequency of the inspection itself. For the evaluation of inspection costs the authors referred to [7].

Cleaning costs depend essentially on: diameter of the pipes, type of drainage network (unitary or separate sewer system) and slope of the pipeline. On these variables it also depends the definition of the most appropriate frequency of cleaning.

The annual costs of maintenance and operation of the network must be discounted over a period equal to the useful life of the network. The estimated useful life of HDPE pipes is about 40-50 years [8], and a value of 50 years has been assumed here.

2.3. Hydraulic simulation model

For a given sewer network topology, the pipes’ diameters for the different return periods have been designed using the well-known EPA SWMM model in Steady Flow routing, by assuming that within each computational time step the flow is uniform and steady.

Once the network has been dimensioned for a given return period T_i, the hydraulic response of the catchment has been simulated for rainfall events having return periods T_j > T_i.

On the catchment surface, the roads are assumed to be dry until a sewer surcharge occurs; when water levels exceed the ground level, the overload volumes flow on the road network. Following the dual drainage approach [9], the pipe network routing has been simulated using the 1-D dynamic wave, coupled to a 1-D dynamic wave for the road network (surface canals). The modeling of this interaction with EPA SWMM is carried out assuming that the flow between the two systems may be exchanged only through the manholes, hence neglecting the presence of trap-doors. The manholes are represented by broad crested weirs: the length of the weir is equal to the perimeter of the manhole while the elevation of the weir crest correspond to that of the roadway.

2.4. Evaluation of total expected damage caused by rainfall events with return period greater than the design one

For the evaluation of the expected economic damage the procedure proposed by Penning-Rowsell and Chatterton [10] has been adopted. For a given network configuration, the expected annual damage, EAD, is calculated combining:

- the storm-probability curve, resulting from hydrological analyses, which links the return period T of the design rainfall to the discharge Q produced by the design rainfall itself;
- The flood-depth curve, derived by hydraulic simulations, that shows the water depths on the road network due to the flow Q produced by the precipitation of return period T. The return periods T_i of rainfall events chosen in this work to test the alternative network configurations are 2, 5, 10, 15, 20 and 25 years.
- The depth-damage curves, which express the expected total damage of the elements at risk (buildings, vehicles, infrastructures, etc.) depending on the depth of flooding. The damage curves depend on the physical and economic characteristics of the site considered and can be absolute or relative. The former relate the total damage to the depth of flooding, the latter connect the relative damage (that is the percentage of damage compared to the value of element at risk) with the depth of flooding.
- Flood damage-probability curves which links the return period of the rain that generated a certain distribution of water levels (i.e. the probability of occurrence of the event) to the corresponding total damage. The area under the curve represents the expected annual damage and takes into consideration both the overall frequency of all possible rainfall events that cause flooding and the resulting total damages.

For each alternative design configuration of the sewer network and for each hydrological input, the water depths, h, in correspondence of the road network nodes have been calculated. The spatial distribution of maximum water depths for each T_j has been obtained interpolating the corresponding maximum water depths by the geostatistical Kriging method and represented through water level contours. As maximum water depths are known, the total surface
of flooded zone can be appraised, as well as each portion of flooded zone where water depth is higher then $h_k$ and less then $h_{k+1}$. The average water depth is considered in each portion and used to assess the damage; the total damage is given by the sum of the damages in each portion.

In order to estimate the costs of traffic congestion, the variation of traffic parameters due to the inhibition of one or more arcs of the road network, whose transport capacity is assumed close to zero as a result of the flooding, is evaluated. The parameters of interest are the passengers·hour (calculated multiplying the vehicles·hour value by the load factor of the vehicles) and vehicles·km: in particular it is relevant their variation along the entire road network compared to the standard situation (traffic parameters of the peak hour of the average winter working day).

The damage due to the traffic congestion is calculated multiplying the increase of passenger-hour by the hourly average value of time per passenger, VOT [€/passenger·h].

The increase in the number of vehicles·km also determines environmental externalities such as the increase of: pollutants emissions and in particular of CO$_2$, consumption of energy resources, noise, and (albeit very limited) of climate change. The costs associated with air pollution and the impacts on climate change considered in this paper are those of the study by CE Delft [11]: for CO$_2$ emissions an average value in the urban area of 1.27 €cent/vehicle·km is adopted, while for the impact on the climate change a value of 0.94 €cent/vehicle·km is assumed.

The damage-frequency curve is then derived point by point: each value $f = 1/T_i$ of the probability of occurrence of the damage is associated with the value of the damage $D_i$ generated by the rainfall event with return period $T_i$.

With the increase of the return period of the design rainfall the expected annual damage reduces due to the increased flow capacity of the drainage network. However, increasing the return period of the project implies the realization of a larger network with higher construction, maintenance and operating costs. The optimal solution is the one which minimizes the total annual costs during the entire useful life of the infrastructure.

3. Numerical application

3.1. Case study

The proposed methodology has been applied to a portion of the urban catchment of Mondello in Palermo (Italy). The area has an extension of 2.7 km$^2$, Fig. 2 shows the scheme of the drainage network which consist of 112 pipes and 113 nodes (including the outfall).
The topology of network does not vary with the return period of the design rainfall; the only parameter that may change is the slope of the main sewers. It was chosen a combined sewer system with HDPE pipes (Manning’s roughness coefficient equal to 0.0125 s/m$^{1/3}$) with a minimum laying depth of 2.50 m (for pipes with diameter $\leq 500$ mm) up to 3÷ 3.50 m (for pipes with diameter $\geq 500$ mm). Only in one case of the principal main sewer, in order to ensure a slope of at least 0.5%, the maximum laying depth is set at 5 m. The Manning roughness coefficient of the road section has been set equal to 0.016 s/m$^{1/3}$.

The parameters of the pluviometric probability curves $h(t) = at^n$ are reported in Table 1.

<table>
<thead>
<tr>
<th>$T$ [years]</th>
<th>$a$</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>21.370</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>32.046</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>40.122</td>
<td>0.305</td>
</tr>
<tr>
<td>15</td>
<td>44.847</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>48.198</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>50.798</td>
<td></td>
</tr>
</tbody>
</table>

The concentration time of the catchment is almost half an hour: a design rainfall with constant intensity and 30 min duration was therefore adopted; the drainage networks so designed have been then stressed with Chicago hyetographs (rainfall duration 30 min, time to peak = 0.5, time step 6 min).

3.2. Results

For each network configuration and for each rainfall event EPA SWMM provided the water depth at road network nodes. The spatial distribution of maximum water depths for each $T_j$ has been obtained using a specific software package and represented through water level contours, so determining the areas with constant average water level.

As previously said, suitable damage curves, which are highly site-specific, have been employed, namely the “relative” damage curve developed by Oliveri and Santoro [6] for the historical center of Palermo with reference to the structural damage to the buildings (Fig. 3) and the “absolute” curve developed by Freni et al. [1] for the historical center of Palermo with reference to the damage caused to the vehicles and to the contents of the buildings in the area at risk. The curves of Freni et al. have been modified in order to obtain damage per unit area. The built-up surface of the studied area, where Art Nouveau buildings and detached houses surrounded by a small garden can be mainly found, has been calculated; it was assumed that the built-up area is made of 50% of Art Nouveau villas and the remaining 50% of detached houses with garden; both typologies have been schematized as a 4 m high one story building. The value of each type of building per unit area (€/m$^2$) has been calculated from the values provided by Oliveri and Santoro [6] using appropriate inflation adjustment factors. It is also assumed that the building damages begin when the water depth reaches 25 cm, and that the damage to the vehicles and to the buildings content begins when the water reaches a 10 cm level above the road surface.

The changes in traffic parameters due to the inhibition of the road arcs, made "non-viable" by the flooding (a water depth threshold of 20 cm was assumed), has been calculated through a specific software package.

The road network model has been extrapolated from the road network model of Palermo developed by the University of Palermo in the preparation of the new transport plan of the city. The only street taken as a reference is the main and central street of the studied area. The traffic of this arc is then redistributed in the road network causing an increase of vehicles·km and vehicles·hour. Traffic disruption costs have been evaluated adopting an occupancy rate of vehicles equal to 1.30 and considering that the average VOT in the urban area of Palermo is about 5.40 €/passenger-hour [12].
Fig. 3. Depth-damage curves employed in the study.

The present value of construction, maintenance and operation costs of the drainage network dimensioned for each design return period and the corresponding present values of the expected annual damages and the present total costs are reported in Table 2 and Fig. 4. Observing Fig. 4, the optimal solution for this case study is the adoption of a design rainfall period of 10 years.

Table 2. Variation of the present value of the total cost of the drainage network with design rainfall return period, T.

<table>
<thead>
<tr>
<th>T [years]</th>
<th>Construction and operation costs [€]</th>
<th>Expected annual damages [€]</th>
<th>Total cost [€]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>9,467,182</td>
<td>9,029,822</td>
<td>18,497,005</td>
</tr>
<tr>
<td>5</td>
<td>10,766,538</td>
<td>3,037,309</td>
<td>13,803,847</td>
</tr>
<tr>
<td>10</td>
<td>12,182,190</td>
<td>252,476</td>
<td>12,434,666</td>
</tr>
<tr>
<td>15</td>
<td>13,332,078</td>
<td>52,781</td>
<td>13,384,859</td>
</tr>
</tbody>
</table>

4. Conclusions

The aim of this work is to define a methodology to identify the optimal rainfall return period, T, for the design of an urban drainage system, taking into account the construction, maintenance and operation costs of the system as well as damages from flooding due to the system surcharges.

The optimal return period is deduced by a minimization analysis of total costs. Such analysis is performed by dimensioning the sewer network for a set of possible design rainfall return periods, T, and assessing the construction costs associated with each value of T.

For each scenario, the total expected damage caused by the sewer surcharge in case of rainfall events with return period greater than the design one is then evaluated. A hydrologic-hydraulic physically based model (EPA SWMM) has been used to estimate water depths. Suitable depth-damage curves and traffic models have been used to calculate the damages to the buildings and their contents, as well as the damages to the vehicles and those due to the traffic congestion.

The application of the proposed methodology to a real life case study proved that it is actually capable of selecting the most effective solution and is viable in technical practice.
Fig. 4. Variation of the present value of the total cost of the drainage network with design rainfall return period, T.

References