Innovative Approaches to Flood Control in Hydraulic Engineering

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Abstract

Drainage system managers face significant challenges due to the increase in extreme rain events occurring in various parts of the world. These events lead to hydraulic overload in drainage systems, causing floods. Adapting existing infrastructure to handle extreme rains without negatively impacting city inhabitants has become a necessity. This research introduces a new method to improve drainage systems with minimal investment costs, utilizing a novel methodology that includes the addition of hydraulic control elements to the network, installation of storm tanks, and pipe replacement.

The proposed methodology employs the Storm Water Management Model for hydraulic network analysis and a modified Genetic Algorithm for optimization. This algorithm, called the Pseudo-Genetic Algorithm, uses integral chromosome coding and has been applied in previous hydraulic optimization studies. This study evaluates both the cost of necessary infrastructure and the damage caused by floods to determine the optimal solution. The main conclusion is that incorporating hydraulic controls can reduce the cost of network rehabilitation and decrease flood levels.

Keywords: *Hydraulic control, rehabilitation, drainage networks, optimization*

INTRODUCTION

Discussions have been generated about the pressure that climate change can generate on water supply and drainage systems. Recent research has shown that climate change is a preponderant factor and affects the future availability of water resources. Additionally, other studies show that due to anthropogenic climate change, the climate cannot be considered unchanging and presents spatially heterogeneous trends in both mean behavior and variability. This shows that the increase in extreme rains in certain parts of the world is evident, and there is a need to take measures. Floods in urban areas are mainly due to the increase in the impervious surface and the increase in the intensity of extreme rains. This increase and the impervious surface due to the urban growth of cities reduces the time of concentration of the water and produces a rapid accumulation of water on the surface that drainage network systems cannot evacuate. All these effects have a consequence: the appearance of increasingly frequent and intense floods.

Floods in urban areas are one of the problems of greatest concern. The damage associated with this type of disaster is expected to increase in the future. Although rain floods generate less economic losses than river floods, they are much more frequent, so their accumulated cost could be higher in one year than another.

There are different methods or technologies that can be applied to face the consequences of excess runoff in urban areas. A proven measure to control urban runoff is the installation of infrastructure that allows water to be temporarily retained and stored during rain events. One of the best structures to achieve this goal is Storm Tanks (STs). In this field, one of the first works was developed by Howard, who presented a theoretical method to evaluate the storage efficiency of a retention tank in combination with a treatment plant using probabilistic methods based on precipitation data. More recently, studied effect of climate change on ST performance. They relied on Intergovernmental Panel on Climate Change (IPCC) medium- high emission scenarios for the city of London. The results of their work indicate that significantly larger storage volumes are required to maintain the same level of flood protection. One of the most prominent studies on this topic was conducted by Andrés- Doménech et al. who studied the ability of STs to regain their efficiency. With this objective, they presented a probabilistic analytical model to assess the volumetric efficiency of STs according to the new climatic scenarios and urban catchment.

Some study presented a method of optimizing the location of STs in two modules trying to reduce flooding, cost of tanks and available total solids load. The first module evaluates and classifies flood nodes with the Analytical Hierarchy Process (AHP) using two indicators: flood depth and flood duration. The second is an iterative module that provides the optimal scheme for the location of the tanks through the method of searching for generalized patterns.

In addition, Cunha et al. presented an optimization method based on a previous disposition of STs to size them and the outflow orifice. They concluded the importance of a good dimensioning of the orifice since it reduces the output flow of the storage unit that regulates the descending flows, allowing flow control and a reduction of floods in the whole network.

Better results are obtained by the combination of STs and the replacement of pipes with others with higher diameters. Some applied two different approaches to determine the optimal location and size of the storage units through the application of Simulated Annealing and Genetic Algorithms. This research validated the use of STs for peak flow reduction in urban areas. It also showed the advantages of considering the replacement of certain pipes and the installation of STs instead of replacing the whole pipe infrastructure.

Presented a methodology to improve storm water systems using an optimization model that incorporates a Pseudo Genetic Algorithm (PGA). The model includes as decision variables the replacement of pipes, the location and sizing of STs and the initial state, and starts and stop levels in the case of systems with pumping stations. The study demonstrated the economic benefits of the joint installation of STs and the replacement of pipes with others of greater capacity. Following this line of research, Ngamalieu-Nengoue et al. present a methodology that uses the aforementioned postulates seeking rehabilitation of drainage networks and shortens the calculation time through a process of Search Space Reduction (SSR). Their methodology is divided into two parts. The first one uses a PGA and aims to reduce the search space for solutions. The second one optimizes the multiple objectives of the new scenario generated in the first part using a Non-dominated Sorting Genetic Algorithm (NSGA-II). Continuing with his research, Ngamalieu-Nengoue et al. present in their work a way to calculate the flood level using the ponded area of the Storm Water Management Model (SWMM). The optimization model assumes the definition of this area in each node to convert flood volume into flood level. The method developed by Ngamalieu-Nengoue et al.

considers reducing the diameter of certain pipes and concludes that it is necessary to include resistance elements that generate a head loss equivalent to that which would be caused by the installation of smaller diameters.

The use of hydraulic controls in drainage networks is considered by different authors as a tool to limit the flow of water in drainage networks, promoting their retention and storage within the network. An outstanding study is the one prepared by Dziopak, who proposed, as an alternative to ST, using the sewerage network as a temporary water storage unit. To achieve this goal, this author designed a retention channel with interior partitions in the form of cameras with an opening at the bottom of the channel acting as an orifice. In this way, the conduit becomes a retention channel.

For this reason, this methodology proposes including the use of hydraulic control in the optimization of drainage networks. In this work, two classes of cost functions have been defined, one associated with the investment cost to improve the network (installation of tanks, replacement of pipes and elements for hydraulic control) and the other related to the cost of the damage that flooding can cause. To find the optimal solution, the model uses a PGA connected to the SWMM hydraulic simulation model through a Toolkit. For this reason, the objective of this work is to include additional energy losses to help retain water in the network, decrease the levels of flooding in the networks and minimize the size of the necessary protection structures.

FORMULATION OF THE PROBLEM

A. Initial Assumptions

The methodology consists of performing the hydraulic analysis of the network using the SWMM model and with the help of a PGA to find the best solutions to adapt the system to extreme rain events. The possibilities to improve the performance of the network are changing pipe diameters, installing ST and including devices for hydraulic control of flow. Presented in this way, the following hypotheses are considered:

- A spatially static design storm is considered for the entire network. This design storm contemplates the most unfavorable operating scenario. The hydrologic study and the runoff model are beyond the scope of this work. Usually, for design or assessment purposes in small, urban areas, flash floods are used.

- The SWMM is used as a network analysis tool. Dynamic wave model was used because it is the model that best represents both the pressurized flow and floods.
- The mathematical model of the network must be calibrated and simplified as much as possible without losing accuracy in the results.
- The actions considered are the renovation of pipes with others of greater diameter, the installation of STs and the installation of hydraulic controls. Changes in the topology of the network are not included in this work.
- STs are considered installed on-line, and their depth is the same as that of the existing manhole. Therefore, the cost of STs is proportional to the depth, and the area of STs is defined as a decision variable.
- The optimization problem is analyzed in terms of costs. The objective function must be established based on the hydraulic variables and includes the cost of renovating pipes, the cost of installing STs, the cost of hydraulic controls and the damage costs caused by floods.

B. Hydraulic Control

A hydraulic control element is a device that allows the control of flow in the network. In this work, the hydraulic controls are installed in the pipes that come out of the STs (Figure 1). The purpose of including this device is to generate a local loss to slow down the flow of water, inducing it to accumulate upstream and using the network as temporary storage. This alternative is presented as a novel option that can contribute to improving the levels of flooding of the networks and minimizing the size of the necessary protection infrastructure.

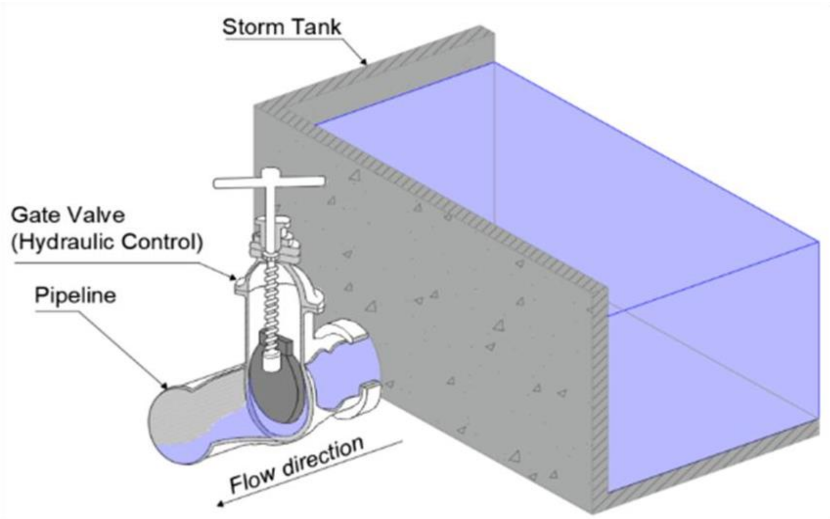

Figure: 1 Gate valve installed as a hydraulic control

In the results obtained in the study carried out by Ngamalieu-Nengoue et al. the reduction of the diameter is observed in certain pipes leaving ST. These smaller diameters act as hydraulic control in the system. It would then be thought that the inclusion of a local head loss in the initial part of the pipe that leaves the ST produces similar results to those obtained by these researchers. This statement needed to be proven.

To find the best option for including the hydraulic controls, the network used by Ngamalieu-Nengoue et al. was used. In the final solution presented by these authors, it was observed that two pipes of the solution (named as P04 and P10) had smaller diameters than the original ones. Reducing diameters during a rehabilitation program in a sewage network seemed unrealistic. An alternative solution consisted of the use of some type of hydraulic control of flow in these pipes. To verify that similar results could be obtained, the network was considered with the optimized solution proposed by the authors but keeping the original diameters of pipes P04 and P10. In these pipes, a hydraulic control based on including local head losses was used. This local loss was modeled by installing a valve or a gate in the initial part of the pipes that came out of the ST. The area of the through-hole is variable, allowing setting the opening according to the demands of the system. To represent the head loss generated by the control element, an expression to calculate the coefficient of losses in valves was needed. In this work, the expression was used (Equation (1)):

$$k=2g\Delta H/v^2 \qquad (1)$$

Where, g is the acceleration of the gravity, ΔH is the loss of energy in the gate and V is the average flow velocity through the gate. The values of the coefficient k as a function of valve travel are shown in Figure 2.

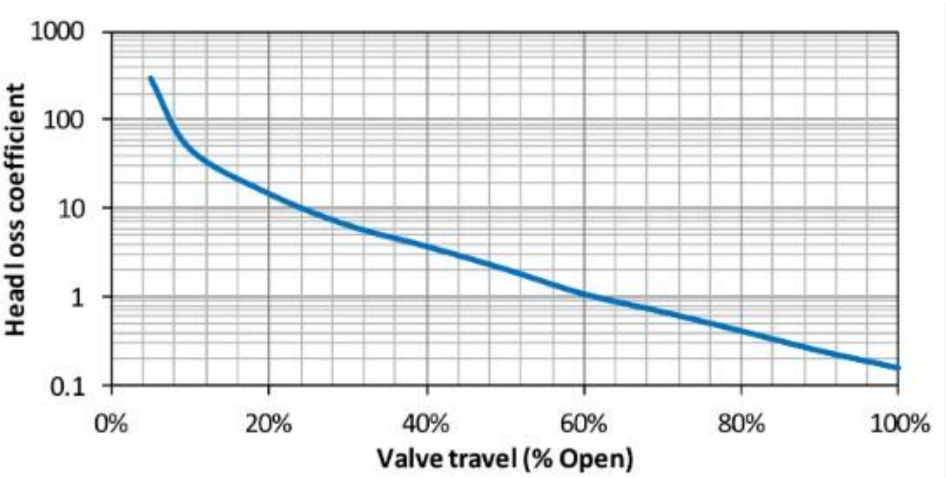

Figure: 2 Head loss coefficient as a function of valve travel

The head loss coefficient represented in Figure 2 can be mathematically adjusted to the following Equation (2):

$$k=C1\theta C2 \quad (2)$$

Where C1 and C2 represent the adjustment coefficients and θ is the valve opening percentage. For this work, the value used of coefficient C1 was 0.2736 and the value for the exponent C2 was −2.395.

The local head loss defined in Equation (1) was calculated based on the flow that the pipes with the smaller diameter would transport. Once these local head losses were obtained, they were included in pipes P04 and P10. Under these conditions, a hydraulic analysis of the network was performed in the SWMM model, obtaining the flow rates in these pipes for the entire simulation period. Figure 3 compares the flow rates of the pipes P04 and P10 in the solution presented by Ngamalieu-Nengoue et al. to the flow rates that were obtained including a local head loss in the pipes.

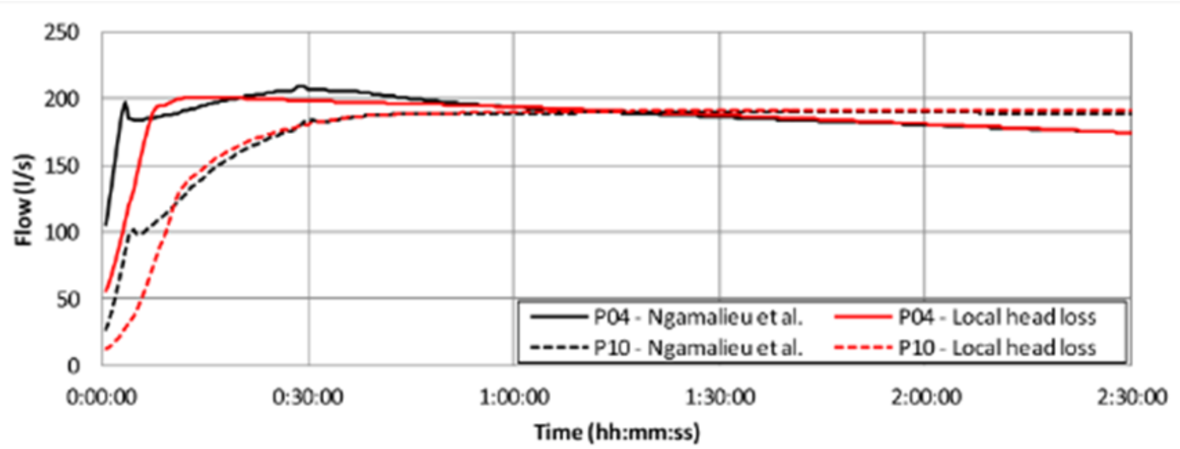

Figure: 3 Evolution over time of the flow in pipes P04 and P10

The results indicated that both alternatives (a pipe with a reduced diameter and a local head loss) could be used as Hydraulic Control in a network. It is interesting to analyze the use of a local loss since it would be much easier to implement than a pipe change and economically more advantageous. In the work presented in this paper, a modification of the SWMM connection Toolkit was done to allow the modification of the local head loss coefficient within any pipe.

DECISION VARIABLES

In this investigation, the drainage network optimization problem considers three types of decision variables. The first type is the diameter of the pipes; the optimization model searches for the best combination of network diameters to minimize flooding. This decision variable can range from a value of 0 (the pipe is not replaced) to a maximum established value. The value 0 implies that the capacity of the pipe is enough to carry the analyzed flow, while a different value indicates the need to increase the capacity of the pipe. It is necessary to define the following parameters for the analysis of this decision variable: NC represents the number of pipes in the network, and ms is the number of pipes selected for replacement and can vary from 0 to NC. Each pipe candidate to be replaced can adopt a diameter of a defined range. Therefore, ND is defined as the range of available diameters, and this range can have a reduced number of diameters ND0 or a full range NDmax.

The second type of decision variable considered by the optimization model is the storage capacity of the nodes. The optimization model searches for the best location of the STs in the network and their lowest volume to reduce flooding. When considering the problem in an urban environment, the excavation is limited to the current depth of the manholes, defining the cross-section of the ST as a decision variable. Decision variables related to STs can take values from 0 (ST is not required in the node) to a previously defined maximum value based on the available space. The model defines the following parameters to analyze this decision variable: NN is the number of nodes in the network; ns is the number of nodes where ST will be installed and whose value can vary from 0 to NN. SWMM defines the cross-section of an ST using Equation (3):

$$S= ASyBS+ CS \quad (3)$$

Where AS, BS and CS are adjustment coefficients for the tank section and y is the water level of the node. For tanks of constant section, the coefficient AS represents the cross-section, while the coefficients BS and CS are null. This cross-section S must necessarily be discretized. For this purpose, the maximum cross-section Smax is divided into a range of partitions N that can take values from N0 to Nmax.

The last group of decision variables is those that consider the loss coefficients introduced in certain pipes in the network. A local loss can be caused by a rapid change in magnitude or direction of velocity (bends, contractions or extensions in the pipe geometry). In this work, a gate valve with different opening degrees is installed in the initial part of the pipes that come out of the ST. The action of this local head loss is considered as hydraulic control. The number of pipes in which hydraulic controls are installed is represented by ps. θ is defined as the opening range that the gate valve can adopt. The range of opening values that the gate valve can take is represented as Nθ.

METHODOLOGY

The methodology of this work is to perform the network analysis using the SWMM model. Then, the results of this analysis are analyzed with the help of an algorithm to find the best solutions according to the values of the objective function. This work used a PGA based on an integer coding of the solution instead of traditional binary coding. A PGA solution is represented by a chromosome comprising a series of genes, and each gene is identified with a decision variable through an integer coding. These algorithms can find many local optimums that can give a final solution far from the optimum.

In this case, every individual has a genome that codes the diameter of selected conduits, the size of the ST and the setting of the hydraulic controls installed in the network. In all three cases, if the gene is 0, this is interpreted as there being no tank, conduit or control to be installed or modified in such a location. Then, an evaluation of investment costs is made using Equations. Finally, a hydraulic analysis is performed, and the results are extracted using a programming library so that the flood cost can also be evaluated. More details about the whole process might be found in.

To improve the efficiency of the algorithm in the optimization process, a convergence criterion must be defined. This convergence criterion determines the number of iterations that the algorithm must perform when evaluating the objective function to find a satisfactory final solution.

To define this convergence criterion, it is considered that a solution very close to the final solution has been found, in which only one gene on the chromosome has not yet reached its optimal value. This value is reached by mutation, so the probability of occurrence of this change must be calculated. Equation (9) shows the expression to calculate this probability.

F=τ1∑i=1msCD (Di) + τ2∑i=1nsCV (Vi) + τ3 ∑i=1psCv (Di) + τ4∑i=1NNCy (yi) ------ (9)

CASE STUDY

Description of the Network

The proposed methodology was tested for various drainage networks. In order to show its application, E-Chico network located in Bogotá (Colombia) was selected as a case study (Figure 5). This network covers an area of 51 hectares and limits with the Andes in its East end. The difference in elevation between the highest and lowest points is 39 m in height. Figure 6 shows a digital elevation model of the area. In this figure, it is easy to realize the role that the hills play in the network. Near the hills, the slope of the terrain is very steep. The slopes of the network pipes vary from 7.22% as maximum value (near the Andes) to 0.16% as minimum value, close to the outlet. The area is divided into 35 hydrological sub-basins. The network has a total length of 5000 m. Pipes' diameters have values from 300 mm to 1400 mm. Further details of this network were presented in the work of Ngamalieu-Nengoue et al., and its use allows comparison of results. Data used for the case study is attached to this article as Supplementary Materials.

Figure: 4 Representation of the network in the Storm Water Management Model (SWMM) mode

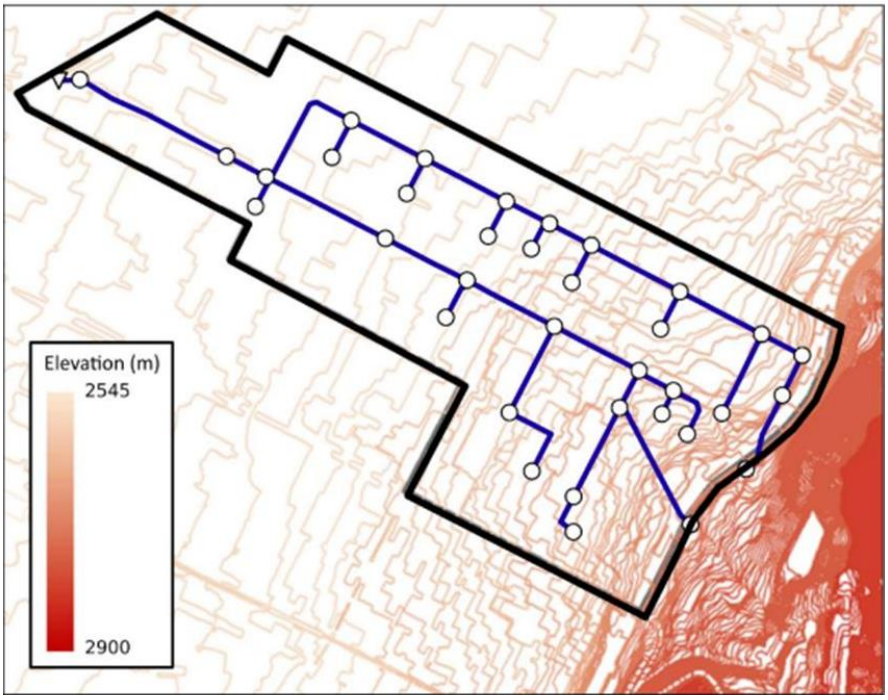

Figure: 5 Digital elevation model of the area for the network.

For the evaluation of the problem, a design storm was used (Figure 5), calculated by the alternate blocks method with 5 min intervals and previously defined by Ngamalieu-Nengoue et al. This design storm was calculated from an IDF curve for a return period of 10 years and a duration of 55 min. To avoid extremely high intensities, the intensity was limited to a maximum intensity of 118 mm/h corresponding to a duration of 10 min. The urban basin generates a runoff volume of 20,123 m3 and presents the nodes of the network flooded with a flood volume of 3834 m3, which is 19.07% of the runoff volume.

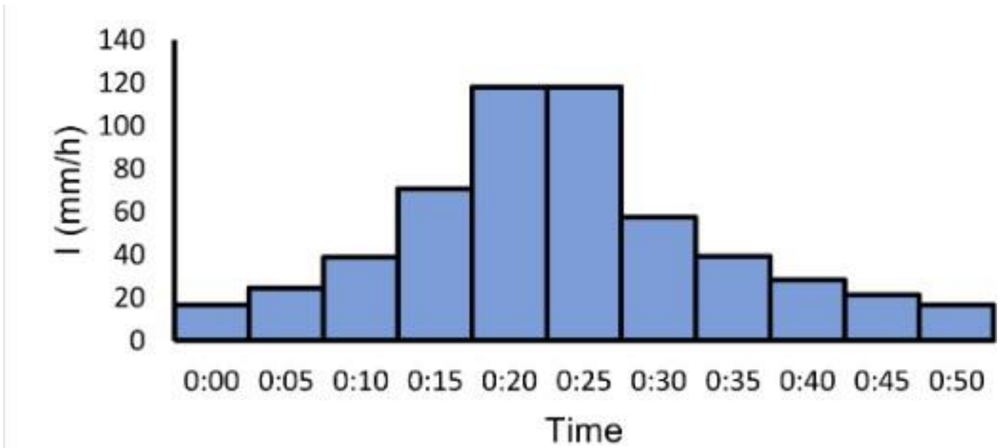

Figure: 6 Design storm for the E-Chico network

The network in the actual conditions and considering the design storm previously presented for the hydraulic analysis presents flooding problems in many nodes of the network, so it needs to be rehabilitated to recover its benefits and provide the security that cities require. Table 1 shows the nodes with flooding problems and the cost of the damages caused by these floods.

Table: 1 Flood on nodes and cost of damage in the current state of the network.

Node	Flood Volume (m^3)	Flood Area (m^2)	y (m)	Cost (€)
N02	123.56	1240	0.100	135,857.00
N04	132.56	930	0.413	181,375.00
N06	501.79	1890	0.265	875,502.00
N07	23.95	1250	0.019	6644.00
N09	1.82	1130	0.002	45.00
N10	385.12	700	0.550	646,838.00
N11	25.83	820	0.032	11,288.00
N23	949.54	450	2.110	569,922.00
N32	36.65	1500	0.024	12,727.00
N33	469.82	3030	0.155	671,908.00
N34	1181.87	3270	0.361	2,131,929.00
		TOTAL		5,244,035.00

RESULTS

The results of the final optimization are as follows. It is required to replace the current P02 pipe with a diameter of 400 mm with another with a diameter of 500 mm. Three STs must be installed in the nodes N04, N10 and N23. Table 2 shows the required volume of the STs. Finally, it is required to install three hydraulic control elements must be installed in pipes P04, P10 and P23. The local head loss that these elements must generate is detailed in Table 3. Figure 9 shows the elements to be installed and their location. With these actions, the objective function has a value of 203,859.69 € made up of the following terms: cost of pipe renovation 6583.81 €, cost of installation of STs 181,540.69 €, cost of installation of hydraulic control elements 7671.75 € and the cost of flood damages 12,701.00 €.

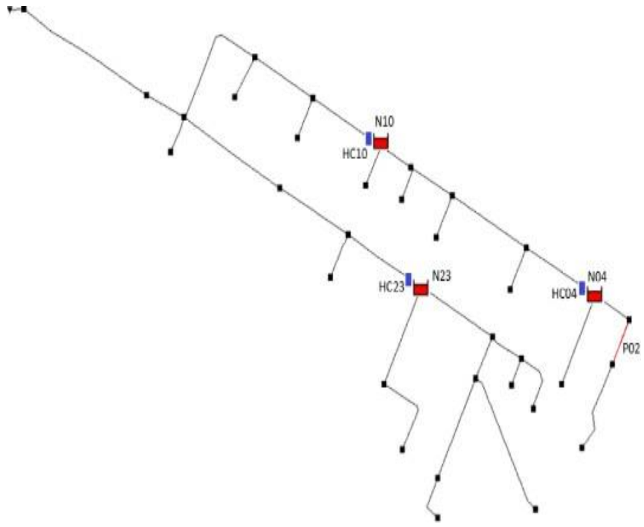

Figure: 7 Infrastructure to be implemented in the network after final optimization

Table: 2 Area and volume required in STs.

Node	Area (m^2)	Volume (m^3)
N04	550.00	946.00
N10	1050.00	2299.50
N23	1050.00	274.50

Table: 3 Loss coefficient k required in hydraulics control elements.

ID	Location	k	θ
HC04	P04	14.73	18.93%
HC10	P10	6.64	26.41%
HC23	P23	161.01	6.97%

CONCLUSIONS

Extreme rains and impermeable ground due to population growth have caused many cities to experience flooding. Drainage networks were not designed for these new conditions, so adaptation is necessary. Implementing tanks to retain flow peaks is a well-studied alternative that has proven to have advantages over other methods in extreme rain events. In the literature, there are many techniques to rehabilitate drainage networks and reduce damage caused by floods. However, none of them were based on hydraulic simulation and considered at the same time the replacement of pipes, the installation of STs and the introduction of hydraulic control.

One of the main contributions of this work is the inclusion of hydraulic control as a complementary rehabilitation strategy to the replacement of pipes and the STs Installation. For this, it has been necessary to represent in economic terms all the parameters involved in the process, including investment costs and costs associated with flood damage. In this sense, another contribution has been the inclusion of hydraulic control and its valuation in economic terms. The participation of hydraulic control has been possible to make it compatible with the part of the objective function previously defined. This formulation of the objective functions in monetary units is very useful for decision-makers in the development of a rehabilitation project.

The proposed method also includes improvements in the solution space exploration capabilities. In this way, the SSR methodology has been generalized to be compatible with the presence of potential hydraulic control devices. Likewise, the specific convergence criterion Gmax of the optimization model has been formulated explicitly in terms of the probability of success.

One of the main limitations of the method, derived from the initial working hypotheses, is the impossibility of modifying the topology of the network. This means that it is not possible to add new pipes. Indirectly, this means that all the STs added to the network must be installed in on-line mode. Undoubtedly, one of the improvements of this work lies in allowing the installation of off-line STs with new pipes that connect these tanks to the network and included in the methodology the determination of the devices that must be activated for the filling and emptying these tanks. In any case, obtaining this improvement is something that seems a natural consequence of the results obtained in this work.

Finally, the results obtained comprise a feasible solution for a defined problem with a defined rainfall. Depending on this rainfall, different solutions will be obtained. Hence, there is no optimal solution, and any problem must be solved in accordance with budget availability, design criteria, etc. This an open field for future investigations.

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