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Predesign of an Urban Rainfall Drainage Network with Genetic Algorithms

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Abstract: The pre-design of an urban storm drainage network is proposed with an optimization method consisting of two parts: in one part, the connection of the manholes is proposed so that there is little water flow in the pipes; in the other part, it is based on a genetic algorithm to find the lowest cost of acquisition of the pipes and of the excavation necessary for their installation, complying with several hydraulic and constructive restrictions recommended in construction manuals of networks of this type. The described procedure is made from the linking of the pipes that make up the network (network layout) trying to obtain the smallest sum of the lengths. It is considered that the conduits are of circular section, where their diameters only take commercial values. The slope of each pipe in the network is calculated so that the hydraulic depth of the water flow is 80% of the pipe diameter, in order to approximate the flow conduction capacity of the liquid within it. Finally, its application to a real case is demonstrated, and the hydraulic performance of the pre-designed network is evaluated by simulating the non-permanent free surface flow in the network using the EPA-SWMM 5.1.015 model, in order to ensure that the hydraulic restrictions and assumptions made for permanent flow are met.

Key words: design; drainage network; rainwater; sewerage; genetic algorithm; non-permanent flow in free surface

1. Introduction

A method is proposed to pre-design or rehabilitate a storm sewer network in an urban area. The method consists of two parts: in the first part, several options are proposed for the connection of the pipes along the streets (layout of the pipe network) and in the second part, the diameters and slopes for each pipe in the network are obtained in order to minimize the total cost of acquiring the pipes and excavating for their installation, while adhering to conditions related to pipe velocity and filling, as well as construction standards set for this type of network, thereby ensuring adequate hydraulic performance.

The method considers several network arrangements and chooses one of them. For reasons of economy, the proposed sewerage networks are designed so that the pipes have a slope similar to that of the ground surface. The layout starts at one or more discharge sites of the network, from which the connections of collectors and emitters are defined.

The depth at which the pipelines are laid is also taken into account, so that the sum of the excavation costs of the installation and those of the purchase of the pipes is as low as possible. The application to a real case is presented. In addition, the hydraulic performance of the pre-designed network is checked to ensure that it complies with the hydraulic requirements established in the design manuals for this type of network.

In this method, it is aimed that the water flow occupies 80% of the cross-section of the pipes, so that the flow approximates the largest possible flow that occurs in circular section pipes. Additionally, this leaves a free space to ensure that the pipes are not obstructed by solids that could move with the water.

1.1 Development

Pre-design of storm drainage networks requires hydrological information for a specific design return period. Therefore, the records of pluviometric stations near and within the area where it will be built are considered. Stations with the most years of daily rainfall data are chosen. The homogeneity and seasonality (Salas & Obeysekera, 2014) of the mentioned records are also reviewed to perform reliable statistical analysis.

For each pluviometric station, maximum daily rainfall in each year is found to obtain the parameters of several probability functions that approximate the annual distribution of this rainfall. To estimate the hourly variation of daily rainfall, different procedures can be used, such as the one based on the convective ratio (R) between the maximum annual rainfall in 1 hour and the mean of the maximum annual daily rainfall (Reich, 1963). This relationship is shown in Figure 1, which takes into account the one-hour rainfall and two-year return period, the average annual rainfall number and the mean annual maximum rainfall.

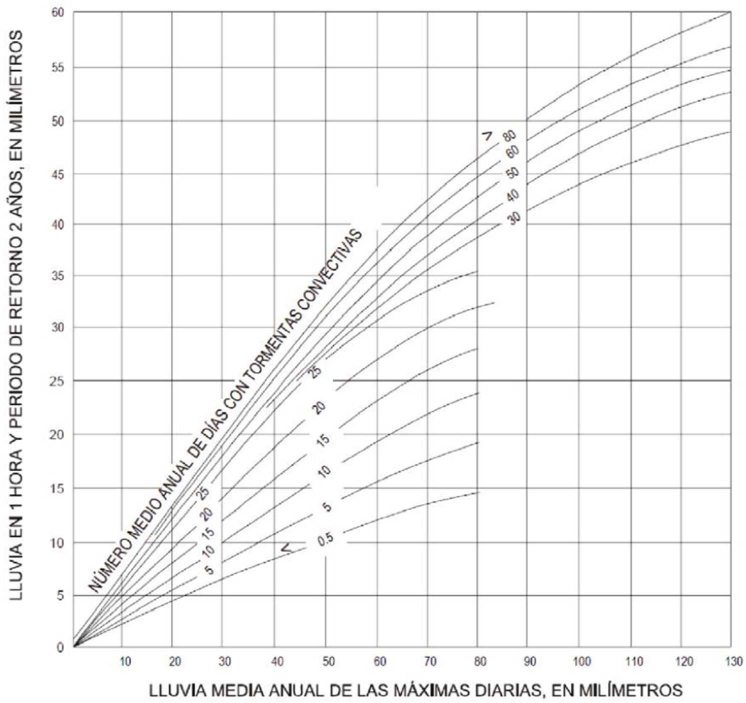


Figure 1. Convective ratio (R) between the maximum annual rainfall in 1 hour and the mean annual maximum daily rainfall.

It is considered that the value of the ratio R remains constant for other return periods, so that one-hour rainfall sheet for return period *T*:

$$hp_T^{1h} = R hp_T^{24h} \tag{1}$$

In order to have the necessary data to draw the rainfall-duration-return period (hp-d-T) curves, the following equation is used:

$$hp_T^d = 10^{\left\{\left[\frac{\log(d)-\log(1)}{\log(24)-\log(1)}\right]\left[\log(hp_T^{24h})-\log(hp_T^{1h})\right]+\log(hp_T^{1h})\right\}} \tag{2}$$

Where:

d = duration of rainfall (hours).

hp_T^{1h} = one-hour rainfall for the return period (T).

hp_T^{24} = 24-hour rainfall for the return period (T).

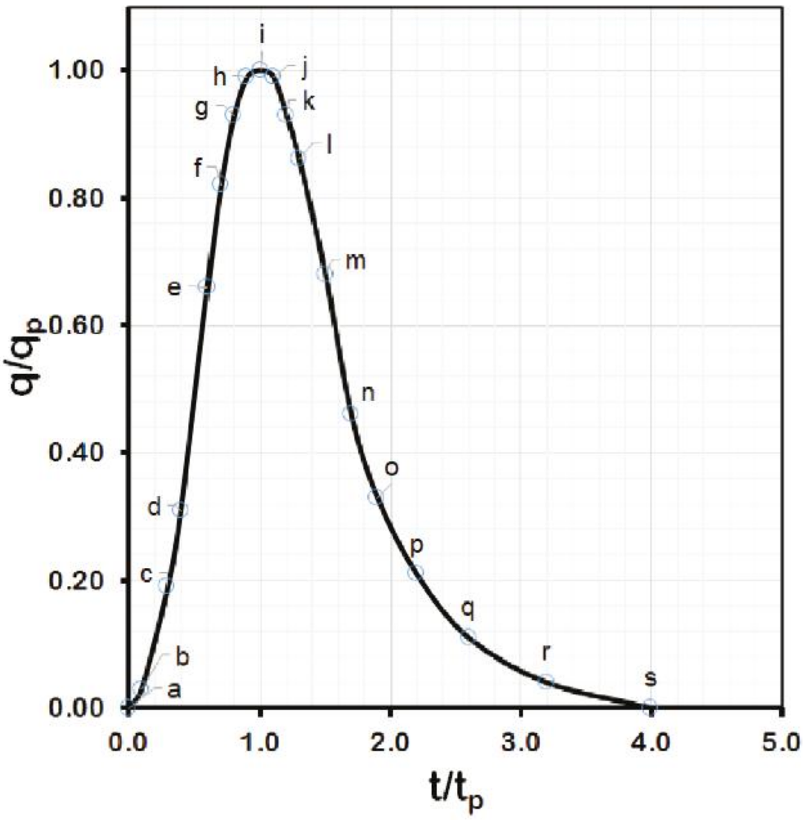
For durations of 5, 10, 15, 15, 30 and 45 minutes, it is considered that $hp_T^{d\,min}/hp_T^{1h}$ is equal to 0.30, 0.45, 0.57, 0.79, 0.91, respectively.

The rainfall intensity is estimated from the rainfall heights by:

$$i_T^d = \frac{hp_T^d}{d} \tag{3}$$

Runoff can be obtained with the rational formula which assumes that rainfall intensity begins instantaneously and continues indefinitely until the time of concentration is reached.

To obtain the hydrographs, the dimensionless unit hydrograph proposed by the Soil Conservation Service of the United States (Mockus, 1957) was used, shown in Figure 2.



Punto	t/tp	q/qp
a	0.00	0.00
b	0.10	0.03
c	0.30	0.19
d	0.40	0.31
e	0.60	0.66
f	0.70	0.82
g	0.80	0.93
h	0.90	0.99
i	1.00	1.00
j	1.10	0.99
k	1.20	0.93
l	1.30	0.86
m	1.50	0.68
n	1.70	0.46
o	1.90	0.33
p	2.20	0.21
q	2.60	0.11
r	3.20	0.04
s	4.00	0.00

Figure 2. Dimensionless unit hydrograph of the SCS.

The runoff hydrograph is defined by multiplying the values of the ordinates and abscissae, by the peak flow and time, by q_p and t_p, respectively.

The Manual of Drinking Water, Sewerage and Sanitation (MAPAS) (Conagua, 2019) states that the following restrictions must be met in urban drainage networks:

a) To avoid deposits of solids inside the pipes or to reduce possible wear of the inner wall due to friction, the average flow velocity (V) inside the duct must be greater than 0.30 m/s and less than 3.0 m/s.

b) It is considered that the flow is uniform to relate the diameter of the conduit and the slope with the flow.

c) It is sought that the hydraulic depth (y) of the liquid flow in the pipes occupies 80% of its diameter (D), to approach the maximum flow rate of conduction with uniform flow at free surface in a circular section pipe (Figure 3).

d) The depth in the ground at which the pipes are installed must be greater or equal to a minimum (cushion) established to protect them against live loads on the ground surface (Figure 4).

e) When pipes are joined, the diameter of the downstream pipe must be equal or greater than that of the upstream pipe.

f) It is considered that, in the union of consecutive pipes, the elevation of the key of the downstream pipe should be equal to the elevation of the key of the upstream pipe to avoid steps.

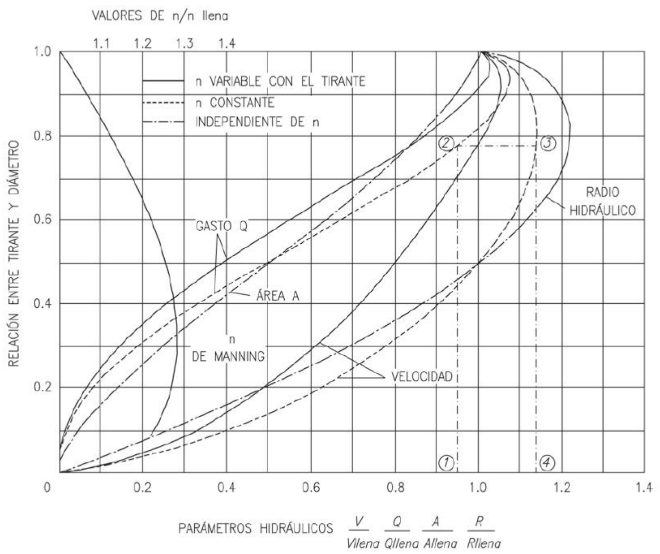


Figure 3. Relationships between hydraulic variables of a circular section pipe with uniform free surface flow (Chow, 1959).

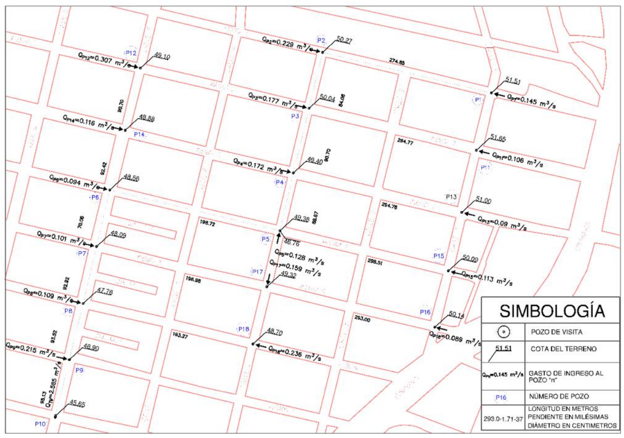


Figure 4. Study area of the San Reforma locality in Tapachula, Chiapas, Mexico.

The proposed procedure for the pre-design of the urban storm drainage network is based on the main idea of associating each pipe diameter with a slope (S) for its template, according to the flow rate (Q) of the liquid flowing inside it. To estimate this slope, it is considered that this flow is free surface and uniform, so that the hydraulic depth or flow (y) is of the order of 80% of the diameter (D) with the intention of approaching the maximum conduction capacity of pipes of circular cross section (Figure 3), which according to Manning's formula (Chow, 1959) complies with the following equation:

$$\frac{Q\,n}{D^{8/3}\,S^{1/2}} = 0.30466 \tag{4}$$

The hydraulic area (A) inside the circular section duct for a hydraulic depth equal to 0.80 D, is obtained from the relation:

$$\frac{A}{D^2} = 0.6736 \tag{5}$$

Thus, the average velocity (V) of the flow is obtained by dividing the flow rate by the hydraulic area. To ensure that the water flow inside the ducts is subcritical, the flow rate must be less than the critical flow rate (Q_C) which for the flow to diameter ratio of 80 % is given by:

$$\frac{Q_C}{\sqrt{g}D^{5/2}} = 0.6177 \qquad (6)$$

In this way, each pipe diameter in the network is related to the flow rate, slope and average velocity.

In drainage networks, the connection method of pipes is that at the end of one pipe, it is only connected to another pipe, forming a series of pipes called branches. It is important to consider that each branch of the network starts from a pipe in the highest topographic elevation area, which requires its installation depth to ensure that the key at its upstream end meets the minimum depth to prevent damage from live loads on the ground above.

The urban storm drainage network pre-design method requires the following information:

a) The layout of the network. According to the configuration of the streets, the topographic elevations and the place where the water from the network is discharged, the manholes and the connections between conduits of the drainage network are proposed.

b) Eligible commercial diameters for the network pipes.

c) Roughness coefficients of the pipe material.

d) Excavation costs per cubic meter according to the type of material in the area where the drainage network pipes will be installed.

e) Acquisition costs of the pipe per unit length (meter) according to its diameter and material.

The pre-design method consists of two main parts; in the first, the layout of the network is proposed and in the second, a genetic algorithm is used to obtain the diameters, the slopes of the pipes and the depth at which they should be installed, so that the sum of the acquisition and excavation costs for installation of the conduits is minimal, in addition to complying with the hydraulic and construction conditions imposed.

To define the network layout, a graph is used, which consists of a scheme showing a set of sites (nodes) connected by lines (arcs). When considering the distance between the nodes at the ends of each arc, it is possible to choose nodes to join with a specific node, specifically those with the shortest distance to it. This can be done with algorithms from operations research network theory.

In the case of the drainage network, nodes and arcs correspond respectively to manholes and pipes. In addition, the arcs are considered to have a direction of travel, going from the manhole with the highest topographic elevation at its rim to the manhole with the lowest elevation of this element. In this article, a variant of Dijkstra's (1959) algorithm was proposed, where the sum of the lengths of the arcs is sought to be the smallest possible.

To obtain the network trace, the first step is to designate the current node as the exit node of the network and label the remaining nodes as unvisited; subsequently, a routine is employed to select the manholes from downstream to upstream, and this routine consists of the following steps:

- The unvisited node with the lowest topographic elevation of its rim, which has a connecting arc reaching it with the shortest length, is selected.
- The node and arc that were selected are marked as visited node and arc.
- When there are no visited nodes left to select, step 4 is continued; otherwise, the unvisited node is chosen as the current node, since it has the lowest topographic elevation at its base (if there is more than one, the closest one is chosen) and returns to step 1.
- Sequences of arcs (network branches or routes) are identified from nodes of lower topographic elevation to nodes of higher elevation, until reaching a node that no longer has connections directed to it.

Genetic algorithms can be used to optimize functions. They are based on the survival process of living organisms. Over the course of several generations, populations evolve according to the principles of natural selection and the strongest are maintained according to Darwin's postulates. By imitating this process, genetic algorithms are used to obtain the global maximum or minimum of functions of some variables that comply with different specific restrictions.

The genetic algorithm is established as a repetitive method, where a certain number of values of the function to be optimized correspond to individuals in the population of a generation. From these individuals, another population of descendants is created, consisting of stronger individuals. Then, the individuals of the original generation are replaced with their descendants, thus establishing a repetitive process. Over the course of generations, this process increases the strength of the descendant populations.

In the genetic algorithm, a certain degree of strength is assigned to each individual of the population in a generation by means of a function known as fitness, where the strongest individual is identified. After several generations, the fittest individuals are identified and the fittest one is found among them (Fuentes et al, 2004). In the case of pre-design, the drainage network for each individual corresponds to a set of pipes that are part of a drainage network and the fittest individual to the drainage network that has the lowest acquisition and installation cost, which also meets the hydraulic and constructive conditions of interest (Hernández, 2007).

Since the flow circulating within each pipe of the network is known, it is possible to assign a diameter and a slope to each pipe. Subsequently, the pipes that make up each branch of the network are considered in order to determine the depth at which they will be installed, taking into account the minimum depth restriction. In this way, the pre-design of the drainage network is established, which represents an individual within the population of a generation.

The suitability of a pre-designed pipe network is considered equal to the reciprocal of the sum of the pipe acquisition and excavation costs. Subsequently, from all the networks of a generation, the one with the highest suitability is located.

The method begins with an initial population consisting of a set of randomly selected individuals. In the context of drainage systems, this corresponds to a set of pipe networks where the diameter of each pipe is randomly chosen from a list of commercially available diameters. Based on the maximum flow rate in each pipe, a diameter is randomly assigned, which is then associated with a slope and the depth of the upstream end of the pipe. Subsequently, using the diameter, slope, and key elevation of the upstream end of the pipes, the volume of excavation required for installation is calculated.

After several generations, it is found that the maximum suitability of the successive generations does not change, which means that the optimal pre-design of the network was made, since the diameters and slopes of the pipes were defined, as well as the depths at which they will be located, so that the minimum sum of the acquisition and excavation costs comply with the hydraulic and constructive restrictions established as a requirement. The suitability function was proposed as the reciprocal of the sum of the acquisition costs (C_{Ti}) and the excavation costs for installation (C_{Ei}) of each storm drainage network conduit *i*. The network cost (C_T) was calculated as the sum of the acquisition costs (C_{Ti}) and the excavation costs for installation (C_{Ei}) of each storm drainage network conduit. The cost of the network (C_T) is equal to the sum of the costs of its pipes:

$$F_A = \frac{K}{\Sigma(C_{T_i} + C_{E_i})} \qquad (7)$$

Where K is a constant that allows us to count the values of the objective function, which are easy to appreciate and, above all, reduce the rounding error of the decimal figures.

For the mixing of individuals, the proportional fitness selection process or roulette was used. A crossover probability equal to 0.7 was used, considering that, for the problem in question, the mutation probability equal to 0.02 was the one that best matched the hydraulic performance. These values were set so as to achieve an adequate variety of results (Fuentes et al., 2011).

2. Discussion and Analysis of Results

The proposed method was applied to a real case. The town of San Reforma in the municipality of Tapachula, Chiapas, Mexico, was chosen. Figure 4 shows the studied area, marked with manholes and streets where rainwater drainage network pipelines can be installed.

The following information was considered:

a) The time of concentration, the areas (tributary) that receive the rainfall entering each of the manholes and the runoff coefficients corresponding to these areas.

b) The hydrographs of water entering the manholes calculated from rainfall with a return period of 10 years. For this purpose, the intensity, duration and return period of the rainfall in each tributary area of the drainage network were taken into account.

c) Daily rainfall records from 5 climatological stations covering the area of influence for the storm drainage network were used.

d) The missing data from the records were completed and the records at the stations were checked to ensure that they were homogeneous and stationary.

e) With the daily rainfall, the maximum annual rainfall was obtained and the parameters of several probability functions were adjusted. To choose the best fit, the minimum standard error of adjustment (SEA) was obtained.

The intensity-duration-return period curves for the area under study were obtained and are shown in Figure 5.

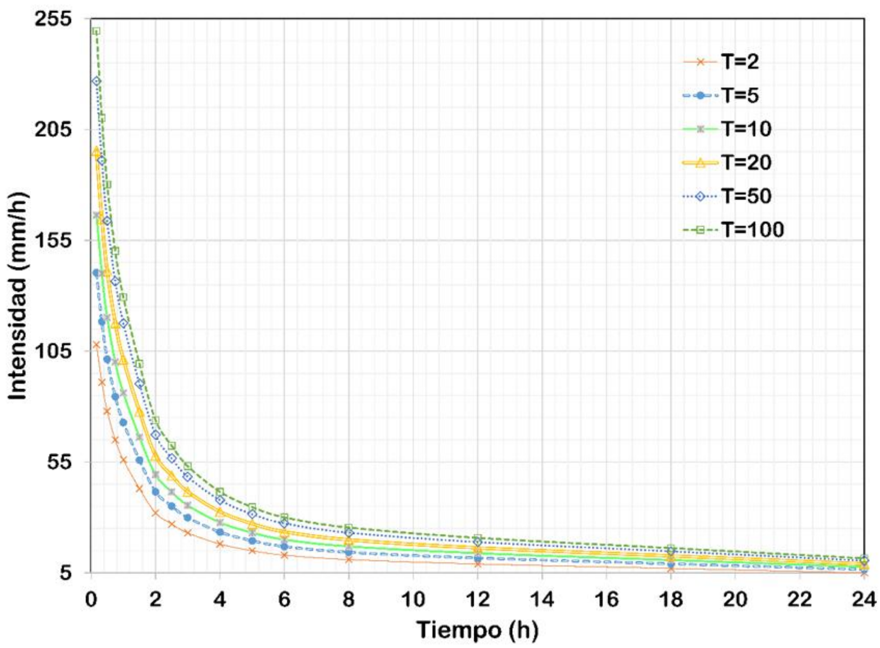


Figure 5. Return period intensity-duration curves (I- d- Tr) in the study area.

Based on the physiographic characteristics of the area under study, it was estimated that the peak time is 3.25 hours, and furthermore, for a 10-year return period, the rainfall intensity was found to be 41.7 mm/h. The maximum discharge in each of the study area for a 10-year return period was obtained using the rational method, considering that the runoff coefficient is 0.62. Table 1 shows the tributary areas of the study area of Tapachula, Chiapas, and the maximum inflow to their manholes.

Table 1. Tributary areas and peak inflow rates to the manholes

Tributary area	Surface (ha)	Peak flow (m^3/s)	Tributary area	Surface (ha)	Peak flow (m^3/s)
1	2.02	0.145	10	1.77	0.128
2	3.17	0.229	11	1.30	0.094
3	1.47	0.106	12	1.23	0.089
4	2.45	0.177	13	2.20	0.159
5	4.26	0.307	14	1.40	0.101
6	1.24	0.090	15	3.27	0.236
7	2.38	0.172	16	1.50	0.110
8	1.61	0.116	17	2.98	0.215
9	1.56	0.113	18	3.27	0.236

With the dimensionless unit hydrograph of the Soil Conservation Service of the United States (Mockus, 1957), the hydrographs of runoff entering each manhole were obtained and are shown in Figure 6.

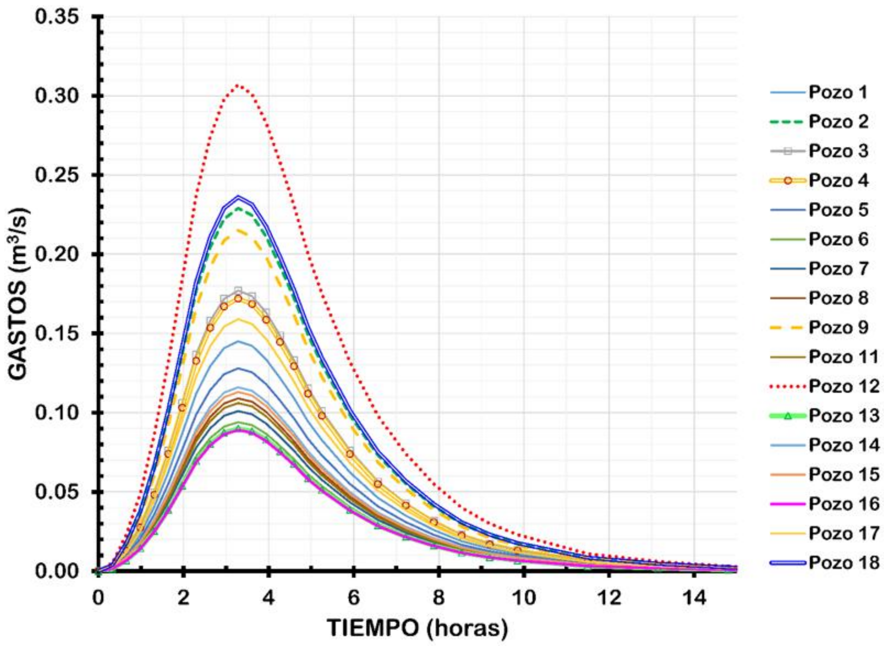


Figure 6. Hydrographs of runoff for the 10-year return period entering the manholes.

Based on the location of the manholes in the urban plan of the area under study (Figure 4) and the streets where the pipes could be installed, the graph shown in Figure 5 was formed. The following graph shows the arcs linking the nodes (manholes) as directed segments going from the manhole with the highest threshold elevation to the manhole with the lowest threshold elevation (Figure 7).

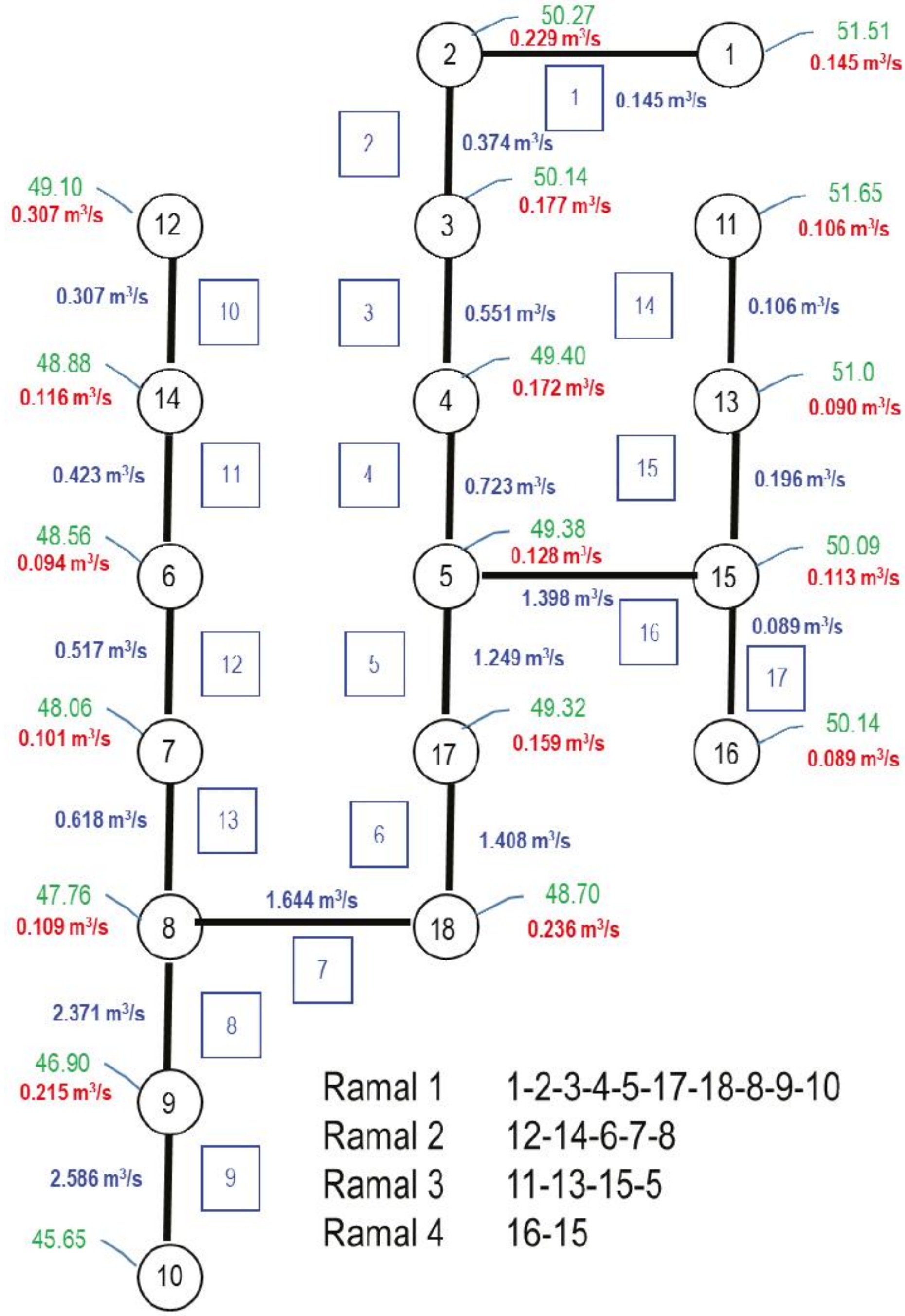


Figure 7. Supporting graph for the layout of the storm drainage network.

The process to obtain the layout of the urban storm drainage network is done from downstream to upstream. We started with node 10, which was only linked to node 9, and then moved on to node 8 because it was the only one associated with node 9. Nodes 10, 9 and 8 were visited. The unvisited node with the lowest elevation was node 7, which was chosen and became a visited node. The next lowest elevation unvisited node was 6, which was changed to visited. It was followed by 18 as unvisited, and was linked to 8 as it was the closest of the visited ones. In this way, nodes 6, 14, 12, 17, 5, 4, 3, 15, 13, 6, 2, 1 and 11 were linked together: Branch 1 with nodes 1, 2, 3, 4, 5, 17, 18, 8, 9 and 10. Branch 2 with nodes 12, 14, 6, 7 and 8. Branch 3 with nodes 11, 13, 15 and 5. Lastly, branch 4 with nodes 16 and 15. Figure 8 shows the layout of the storm drainage network on which the genetic algorithm is based to obtain its optimum pre-design.

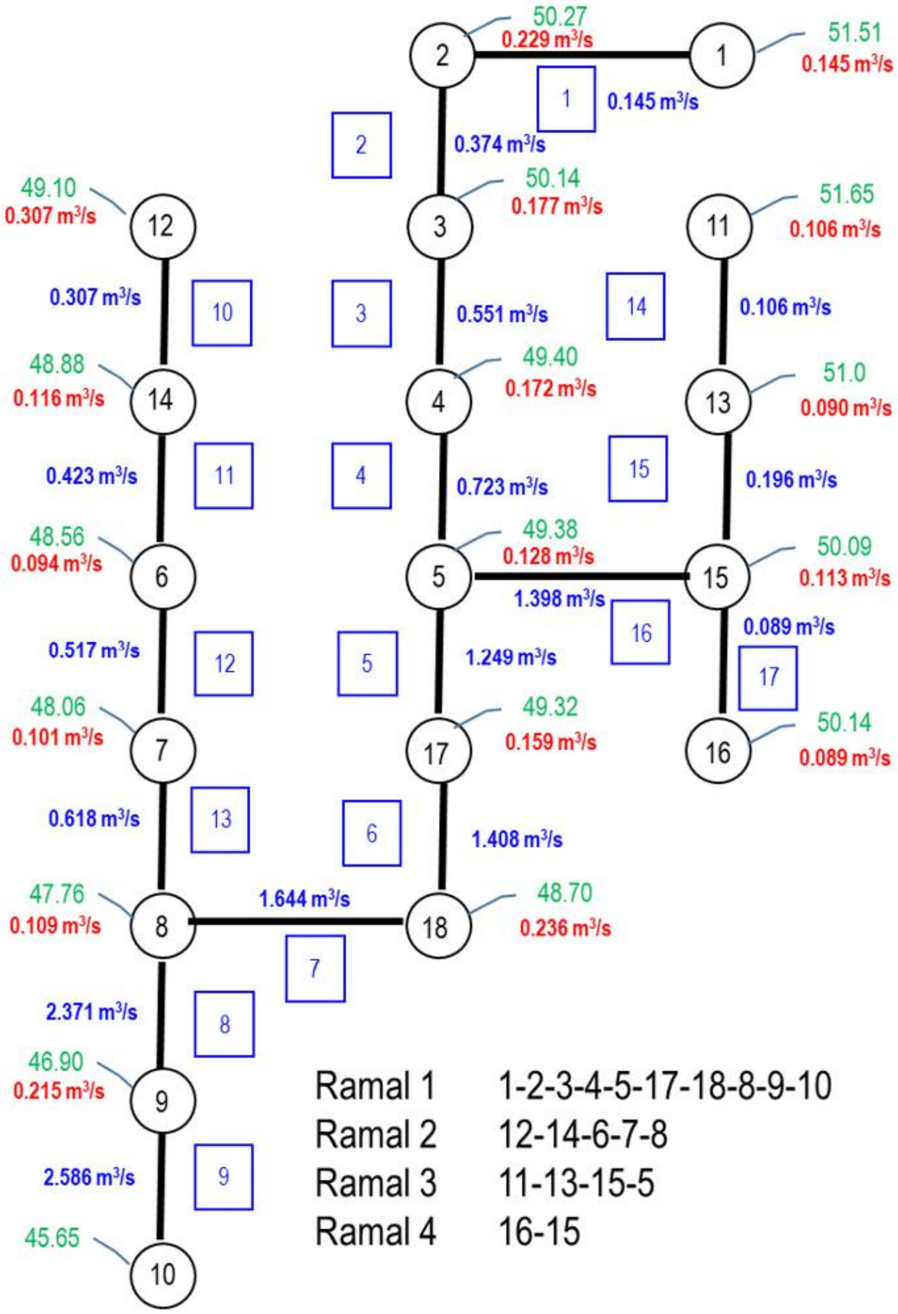


Figure 8. Layout of the storm drainage network

The largest flows circulating in each pipe of the drainage network shown in Table 2 were obtained by adding the peak flows entering the manholes according to the network layout shown in Figure 8.

Table 2. Expenditures flowing in the storm drainage network pipes

Branch	Pipe	Well		Length (m)	Ground elevation		Flow (m^3/s)
		Initial	Final		Initial (m)	Final (m)	
1	1	P1	P2	274.90	51.51	50.27	0.145
1	2	P2	P3	84.10	50.27	50.14	0.374
1	3	P3	P4	90.70	50.14	49.40	0.551

Branch	Pipe	Well		Length (m)	Ground elevation		Flow (m^3/s)
		Initial	Final		Initial (m)	Final (m)	
1	4	P4	P5	88.70	49.40	49.38	0.723
1	5	P5	P17	70.06	49.38	49.32	1.249
1	6	P17	P18	92.99	49.32	48.70	1.408
1	7	P18	P8	193.00	48.70	47.76	1.644
1	8	P8	P9	93.50	47.76	46.90	2.371
1	9	P9	P10	95.10	46.90	45.65	2.586
2	10	P12	P14	90.70	49.10	48.88	0.307
2	11	P14	P6	92.40	48.88	48.56	0.423
2	12	P6	P7	70.10	48.56	48.06	0.517
2	13	P7	P8	92.90	48.06	47.76	0.618
3	14	P15	P5	298.50	50.09	49.38	0.106
3	15	P11	P13	90.10	51.65	51.00	0.196
3	16	P13	P15	88.74	51.00	50.09	0.398
4	17	P16	P15	70.43	50.14	50.09	0.089

High density corrugated polyethylene (HDPE) pipe shall be used. Eligible commercial pipe diameters are given in Table 3. For all pipes the roughness coefficient of Manning's formula is 0.01 $s/m^{1/3}$.

Table 3. Commercial diameters and costs per meter for storm drainage network pipes

Diameter (m)	Cost ($/m)
0.37	450
0.45	510
0.61	630
0.76	845
0.91	1,250
1.07	1,610
1.22	2,000
1.52	3,210
1.83	4,620
2.13	6,120
2.44	7,944

According to the maximum flow conducted in each pipe and the hydraulic operation at free surface with uniform flow and hydraulic depth, the commercial diameters of the pipes that would comply with the conditions of average velocity and subcritical flow shown in Table 4 were chosen.

Table 4. Eligible diameters for each of the pipes of the storm drainage network

Branch	Pipe	Eligible diameters (m)							
1	1	0.45	0.61	0.76	0.76	0.76	0.76	0.76	0.76
1	2	0.61	0.76	0.91	1.07	1.22	1.22	1.22	1.22
1	3	0.76	0.91	1.07	1.22	1.52	1.52	1.52	1.52
1	4	0.76	0.91	1.07	1.22	1.52	1.83	1.83	1.83
1	5	1.07	1.22	1.52	1.83	2.13	2.44	2.44	2.44
1	6	1.07	1.22	1.52	1.83	2.13	2.44	2.44	2.44
1	7	1.07	1.22	1.52	1.83	2.13	2.44	2.44	2.44
1	8	1.22	1.52	1.83	2.13	2.44	2.44	2.44	2.44
1	9	1.52	1.83	2.13	2.44	2.44	2.44	2.44	2.44
2	10	0.61	0.76	0.91	1.07	1.22	1.22	1.22	1.22
2	11	0.61	0.76	0.91	1.07	1.22	1.22	1.22	1.22
2	12	0.76	0.91	1.07	1.22	1.52	1.52	1.52	1.52
2	13	0.76	0.91	1.07	1.22	1.52	1.52	1.52	1.52
3	14	0.37	0.45	0.61	0.61	0.61	0.61	0.61	0.61
3	15	0.45	0.61	0.76	0.91	0.91	0.91	0.91	0.91
3	16	0.61	0.76	0.91	1.07	1.22	1.22	1.22	1.22
4	17	0.37	0.45	0.61	0.61	0.61	0.61	0.61	0.61

Table 5 shows some results of the genetic algorithm, the highest values of the fitness function, the sum of the excavation and pipe acquisition costs for the fittest individual of each generation.

Table 5. Process for approximating the minimum total cost of the storm drainage network that meets hydraulic and construction conditions

Generation	Individual	Function aptitude	Excavation cost (pesos)	Pipeline cost (pesos)	Total cost (pesos)
1	1	0.2079	1,887,311	2,922,411	4,809,722
1	4	0.223	1,750,509	2,733,877	4,484,386
1	9	0.2245	1,732,965	2,721,958	4,454,923
1	38	0.2341	1,697,728	2,573,819	4,271,547
2	13	0.2604	1,491,587	2,348,175	3,839,762

Generation	Individual	Function aptitude	Excavation cost (pesos)	Pipeline cost (pesos)	Total cost (pesos)
3	39	0.2744	1,421,644	2,222,415	3,644,059
4	22	0.2799	1,397,988	2,174,499	3,572,487
5	24	0.2902	1,352,795	2,092,998	3,445,793
9	20	0.2984	1,325,277	2,026,416	3,351,693
11	30	0.2999	1,329,839	2,005,009	3,334,848
14	21	0.3013	1,313,255	2,005,192	3,318,447
32	34	0.3055	1,298,705	1,974,331	3,273,036
35	28	0.3059	1,295,398	1,973,151	3,268,549
38	11	0.3067	1,291,386	1,968,925	3,260,312

Note that the savings between the cost of the first alternative and the last one is 32 %, ((4,809,722 - 3,260,312)/ 4,809,722 = 0.32).

Table 6 shows the main characteristics of the storm drainage network arrangement with the proposed method.

Table 6. Characteristics of the urban storm drainage network for 10-year return period

Pipe	Well initial	Well Final	Length (m)	Flow rate (m^3/s)	Diameter (m)	Mattress initial (m)	Mattress end (m)	EIB (m)	EFB (m)	Slope (m/m)
1	1	2	274.90	0.145	0.45	2.35	1.55	49.16	48.72	0.0016
2	2	3	84.10	0.374	0.61	1.71	1.76	48.56	48.38	0.0021
3	3	4	90.70	0.551	0.76	2.47	1.86	47.67	47.54	0.0014
4	4	5	88.70	0.723	0.76	1.86	2.06	47.54	47.32	0.0024
5	5	17	70.06	1.249	1.07	2.37	2.39	47.01	46.93	0.0012
6	17	18	92.99	1.408	1.07	2.65	2.17	46.67	46.53	0.0015
7	18	8	193.00	1.644	1.07	2.72	2.17	45.98	45.59	0.0020
8	8	9	93.50	2.371	1.22	2.98	2.32	44.78	44.58	0.0021
9	9	10	95.10	2.586	1.52	3.80	2.62	43.1	43.03	0.0008
10	12	14	90.70	0.307	0.61	1.80	1.71	47.3	47.17	0.0014
11	14	6	92.40	0.423	0.61	1.78	1.71	47.1	46.85	0.0027
12	6	7	70.10	0.517	0.76	2.27	1.86	46.29	46.2	0.0012
13	7	8	92.90	0.618	0.76	1.99	1.86	46.07	45.9	0.0018
14	11	13	90.10	0.106	0.37	1.90	1.47	49.75	49.53	0.0024
15	13	15	88.74	0.196	0.45	2.20	1.55	48.8	48.54	0.0029
16	15	5	298.50	0.398	0.61	1.71	1.71	48.38	47.67	0.0024
17	16	15	70.43	0.089	0.37	1.47	1.54	48.67	48.55	0.0017

Note: EIB: Elevation initial bottom; EFB: Elevation final bottom

Figure 9 shows the arrangement of the urban storm drainage network obtained with the proposed method.

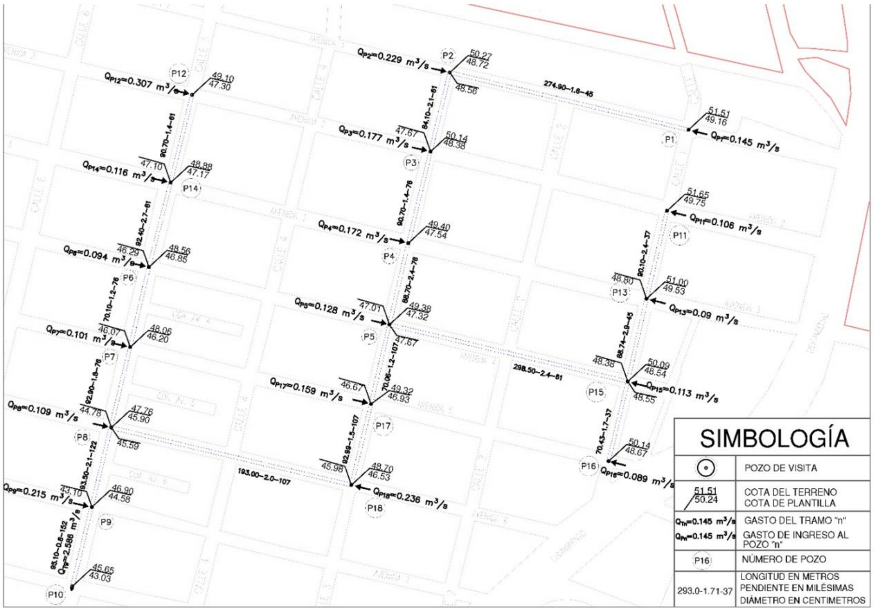


Figure 9. Urban storm drainage network with minimum cost that meets hydraulic and constructive constraints for rainfall with a return period of 10 years.

To verify that the flow of the liquid inside the network is adequate and that the hypotheses of hydraulic operation and water flow in the pipes were complied with, as well as compliance with the hydraulic restrictions of the drainage network, a simulation was made with unsteady flow of the network using the model: EPA SWMM 5.0 of the U.S. Environmental Protection Agency (Rossman, 2005).

In the manholes, the inflow hydrographs shown in Figure 6 were considered. Figures 10 to 13 show the profiles of the maximum flow rates in the subcollectors, showing that they were close to 80% of the diameter, thus complying with the pre-design method.

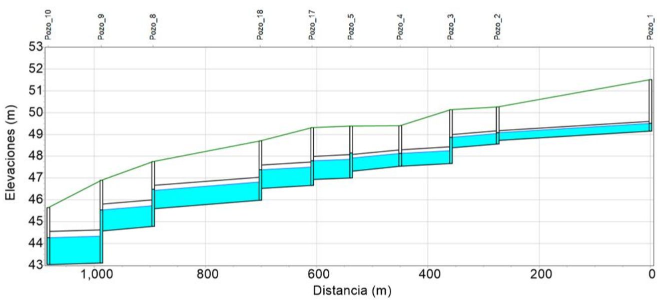


Figure 10. Flow profile of Branch 1.

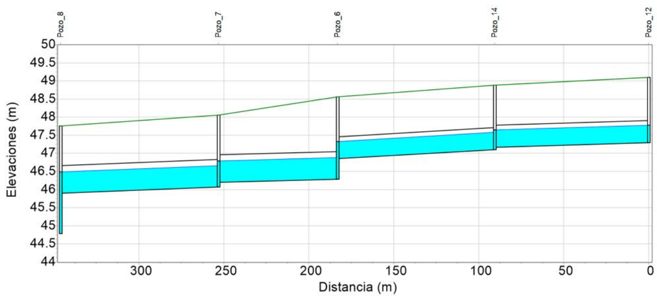


Figure 11. Flow profile in Branch 2.

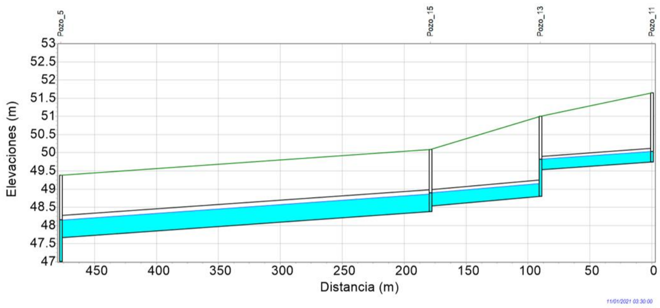


Figure 12. Flow profile in Branch 3.

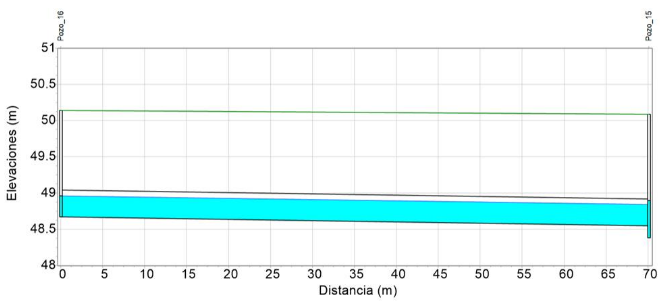


Figure 13. Flow profile in Branch 4.

Table 7 shows the flow rates, flow to diameter ratio (fill ratio), velocities, critical flow rate and normal flow rate in each pipe obtained with the mathematical method of unsteady flow SWMM 5.0.

Table 7. Results of the hydraulic review of the performance of the resulting urban storm drainage network - maximum values

Section	Tension (m)	Speed (m/s)	Y/D ratio (%)	Q (m^3/s)
T1	0.36	1.07	80	0.145
T2	0.48	1.5	79	0.373
T3	0.60	1.43	79	0.55
T4	0.60	1.87	79	0.721
T5	0.87	1.6	81	1.246
T6	0.85	1.83	79	1.405
T7	0.86	2.13	80	1.639
T8	0.96	2.39	79	2.365
T9	1.25	1.62	82	2.578
T10	0.49	1.23	80	0.307
T11	0.49	1.69	80	0.423
T12	0.60	1.35	79	0.516
T13	0.60	1.61	79	0.617
T14	0.30	1.15	80	0.106
T15	0.36	1.44	80	0.196
T16	0.49	1.59	80	0.397
T17	0.30	0.96	80	0.089

3. Conclusions

The layout of an urban storm drainage pipe network is crucial in the design process of an urban storm drainage system. The diameters of the pipes selected to connect the manholes within the drainage network were based on commercially available options. The slopes of the pipes are subcritical, and the installation depth of the pipes was minimized, as the designed network aimed to minimize the total cost of pipe acquisition and excavation, while adhering to the hydraulic and construction restrictions outlined in urban drainage network manuals. The optimal drainage network achieved was reviewed through mathematical simulation of unsteady flow within its conduits, utilizing the one-dimensional mathematical modeling software EPA SWMM 5.0 developed by the U.S. Environmental Protection Agency (Rossman, 2005).

The results of the simulation of water flows within the conduits of the drainage network indicated that the flow filling ratios in the pipes (i.e., the ratio of flow to pipe diameter) were approximately 80% (see Figures 10 to 13). Furthermore, the permanent flow rate assumed for each pipe in the pre-design phase did not exceed the maximum flow rate predicted by the mathematical simulation of time-varying water flows. The simulation of the hydraulic operation with non-permanent flow conditions in the network demonstrated its adequacy, thereby confirming that the process employed corresponded to a method for designing an urban storm drainage network.

The savings between the cost of the initial alternative considered and the final one obtained was 32%, indicating that the proposed method resulted in pre-designs of storm drainage networks with reduced costs. Of course, there is room for improvement in several aspects of the methodology, such as the consideration of head losses in manholes and the determination of runoff hydrographs.

It is recommended that, after designing this type of infrastructure, a review of the hydraulic operation with non-permanent flow be conducted to ensure its adequacy and compliance with hydraulic operating restrictions in each of its conduits. The obtained sewerage network comprises 17 pipes, with 10 commercially available diameters and 10 possible slopes for each section. For the design of the network, there are a total of 1,017 potential solution combinations. Since the genetic algorithm was applied multiple times and yielded similar results, it is highly probable that a solution close to the optimal one has been identified.

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Conflicts of Interest

The author declares no conflicts of interest regarding the publication of this paper.

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