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Damping of the Rainwater Runoff by Small Underground Reservoirs in Subdivision Lots

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Abstract: *The present work aims to evaluate the effect of small flow dampening reservoirs installed in the front of the lots in subdivision, regarding the reduction of the discharge peak and eventual decrease of the diameters of the drainage galleries, in relation to a conventional drainage system. After a review of the literature on the subject, the calculation of the drainage network of a certain street is carried out in the conventional way, with the presentation of the respective dimensioning worksheet. In sequence, the Routing Simulator Application is used in order to verify the reductions in the flows caused by the damping devices, with the presentation of the new dimensioning worksheet with damping. Finally, the flow values for gallery sections and diameters are compared, considering both scenarios. Lot reservoirs were responsible for a reduction rate of 21% to 35% in flow rates, with a decrease in diameter in 50% of the evaluated galleries.*

Keywords: *On-site stormwater detention, urban drainage, reservoir routing, floods, sustainable development*

1. Introduction

Drainage systems in expanding metropolitan areas are becoming insufficient as population rises, necessitating more occupation. The growth in impermeability has a direct effect on the peak and volume of water discharged superficially, necessitating work to extend the drainage system. Because of the enormous expenditures and physical constraints, this extension is frequently impracticable. The Urban Drainage Master Plans have identified issues and pointed to solutions incorporated within urban basins, with the goal of resolving them as near to the source as feasible. One of the plans' recommendations is to prevent the increasing of natural flow at the exit of the lots by applying control techniques at the source. The use of this type of measure has some objectives such as dampening the flood peak, by improving infiltration and storage conditions [1].

This decrease has a significant beneficial effect on downstream communities by lowering the peak discharge and postponing the arrival of floods. Reservoir routing is a mathematical approach for calculating the amount and form of a flood wave transition via a water retention facility over time [2].

As stated by [3], the absence of urban planning, along with chaotic population expansion and a rise in surface runoff, has considerably contributed to flooding concerns in metropolitan areas. The existing drainage system, which is primarily concerned with plumbing, is built on a rapid flow of rainfall downstream, which adds to increased peak flows and exacerbates the situation. As a result, alternative strategies for pluvial water management are required.

According to [4], a more recent concept of urban drainage has been used, based on the principle of storing and delaying the surplus in order to provide a better flow distribution over time. The main sustainable measures at source have been detention in lots, with the use of small reservoirs, and infiltration into the soil.

During a rain event, the detention tanks collect and store storm water runoff from roofs, pavements, and other impervious surfaces. The collected water will be discharged into the downstream drainage system at a regulated rate, lowering the peak discharge [5].

As per [6], low impact development strategies are used across the world to offset the effects of urbanisation on the hydrological cycle. Aside from their widespread use, the public's understanding

of these strategies and stormwater management is limited. People's awareness of flood control approaches and use of those techniques leads to greater acceptance and involvement.
In [7] a short script is suggested with general guidelines for the incorporation of detention reservoirs to new projects, in order to guide and facilitate the use of these devices, improving their acceptance by society.

2. Applications: Theoretical and Field Situations

Reference [5] introduces an article where 88 detention tanks are supposed to be uniformly spaced throughout the section, collecting stormwater from 44 ha of effective impervious area (one detention tank every 0.5 ha of impervious surfaces on average).
Using a mathematical hydrological model of precipitation flow, six types of reservoirs implanted in typical and subjected to increases in impermeable regions lots of a city were analysed [8]. The required volumes of implantation and maintenance expenses were determined using the typical precipitations of micro drainage and the hydraulic behaviour of each device. The results revealed that for the biggest simulated lot, 600 m2, with 100% waterproofing, quantities in the range of 2.5 to 3.0 m3 would be required, and for 50% waterproofing, volumes in the region of 1.0 to 1.5 m3 would be required.
A work is presented in [9], in which the volume of the damping reservoir required for a lot with 360 m2, fully waterproofed was 3.24 m3. For less impermeability values the volume of the reservoir was 0.97 m3. The study carried out showed that it is possible to obtain a reduction in peak flows from 13.31% to 40.69% through variations in dimensions of the outlet pipes.
As stated by [10], underground stormwater detention chambers (USDC) are a stormwater detention and treatment technology that can eliminate the thermal difficulties associated with sun-exposed detention facilities while still offering a similar degree of stormwater pollution treatment services. A field study of an USDC was undertaken to characterise its treatment performance and effect on water temperature. It was found that the USDC provided results, in terms of level of stormwater treatment, similar to those of the wet detention ponds. Outlet maximum temperatures were 5 °C colder than intake maximum temperatures on average, and outlet water temperatures stayed within the thermal range for cold water fish habitat across the study period.
Reference [11] presents an article in which they compare the monitoring data of an on-site stormwater detention device (OSD) installed in a hospital with the findings derived by theoretical approaches often employed in the construction of this type of structure. The OSD filling was studied during 48 precipitation occurrences. The measured values were greater than theoretical values in the maximum heights of water level comparison, and the results using the Rational Method were closer to monitoring data than the results using the SCS-HU Method.
The use of OSD in aspects of regulations, technical details and management matters is evaluated in [12]. Based on it, all administrations, in general, have the same management challenges. There is no regulation for the quantity of OSD installed in cities or even their condition. As a result, the authors provided several recommendations for improving OSD policy.
In order to evaluate OSD performance on a real scale as well as its hydrological and hydraulic parameters, two devices were monitored, with results described in [13]. Based on measuring data analysis, it was discovered that the OSD mean efficiencies in discharge attenuation were greater than 50%; the storage quantity per catchment area was close to 25 L/m2; the short pipes discharge coefficients were around 0.90; and the shape of inflow hydrographs was similar to hyetographs. The Unit Hydrograph Modified Rational Method was used to accurately portray the rain-runoff transition. The findings might be utilised in OSD design to calculate the inlet hydrograph, pre-sizing the storage capacity based on contribution area, and selecting the appropriate discharge coefficient value for the output device.
Reference [14] investigates the attenuation impact of household rainwater storage reservoir implementation in a typical square using Visual Basic for Applications (VBA) programming, where two damping volumes for the storage device in lot condition settings were built. The household reservoir was sized for initial rains of 05 and 10 year return period and tested for maximum project rains of 02, 05, 10, 15, 20, 25, 50, and 100 year return period, of which the temporal distribution used was the largest storm recorded at that place. The detention reservoir with a damping volume

of 3.25 m3 provided attenuation, at the end of the simulated stretch, of 37.38, 24.02, 21.74, 20.99, 18.51, 16.70, 11.73, and 7.52 percent, for the rains 02, 05, 10, 15, 20, 25, 50 and 100 year return period, respectively, while for the detention reservoir with a damping volume of 5.015 m3 the attenuation was 64.72, 65.10, 63.96, 63.23, 62.71, 62.32, 61.11, and 59.98 percent. In a subsequent work [15], another home detention reservoir, this one with a volume of 1.5 m3, was added to the study mentioned in [14], whose efficiency was 15.47% attenuation in the peak flow of the respective maximum rainfall.

A study that aimed to simulate and evaluate the impact of implementing a sustainable drainage system, more specifically distributed detention facilities, in a small contribution area, is presented in [16]. The simulations were carried out in a subdivision, from which a block was delimited for the implementation of the reservoirs, in order to assess the impact on the gallery system. Two blocks influenced by 24 lots were selected, being subdivided into 6 sub-basins with areas of 0.16 hectares each, covering 4 lots, with the reservoirs provided with 35 mm outlet orifices. Data were obtained from two series of precipitation with different intensities and durations, in order to verify the behaviour of the detention reservoir in different situations. For the simulations, the EPA SWMM software was used, which made it possible to implement the scenarios under study as well as the compensatory measure. The results showed that the distributed detention basins were efficient in mitigating the peaks of surface runoff in a sustainable way, from almost 45.0 L/s to approximately 7.5 L/s, in addition to increasing the peak time of the area considered in the simulations.

A research presented in [17] addresses the flow dampening microreservoirs, seeking to size and assess their impact at the lot and subdivision level and, subsequently, at the macrodrainage scale. In the subdivision used in the study, for a 10-minute rainstorm, the peak flow dampening was 63%, while for a 60-minute rainstorm and for the critical rain, it was reduced by 54% and 44%, respectively.

A study described in [18] evaluated the effect that the implementation of control measures at the source can provide in the abatement of flood peaks in densely occupied urban areas. The analyses were done through 209 scenarios, using a decision support system for SWMM, and contemplated permeable pavements, green roofs, and rain gardens as alternatives for retention.

In accordance with [19], an on-site stormwater detention system was constructed in a house's car porch with a 4.40 m x 4.70 m x 0.45 m tank filled with precast-concrete modular units with an effective storage volume of 3.97 m3. The system received water from a 95 m2 house roof via 0.1 m diameter pipe, discharged water via 0.05 m diameter pipe. It had been recorded six observed storm events, that consisted 20 - 50 mm peak hourly rainfall, 0.7 - 1.8 L/s inflow, 0.5 - 1.2 L/s outflow, and 0.21 - 0.47 m water level. Previous four historical storm events were sourced to augment the analysis. A computer model developed using the storm water management model was calibrated and verified using the six observed events. As such, the calibrated and verified model was used to simulate the historical storm events with 40 - 50 mm peak hourly rainfall and produced 1.0 - 1.3 L/s inflow, 0.72 - 0.76 L/s outflow, and 0.41 - 0.45 m water level.

A similar study, mentioned in [20], focuses on the probability of OSD below a residential vehicle porch. The space given in the vehicle porch area can be utilised by installing an OSD beneath it to temporarily store rainwater from the roof while raining in the hopes of decreasing surface runoff. The OSD is exposed to 15-minute, 10-year return periods interval design rains. SWMM is used to illustrate this process in urban hydrology. The performance of the OSD is further examined by varying the number of orifice exits. According to modelling efforts, one orifice exit is ideal, resulting in a 95% discharge decrease at the outfall.

An article published in [21] describes the findings of a study comparing the efficiency of three retention devices with different hydraulic systems: the standard single-chamber reservoir, the modified multi-chamber reservoir with an accumulation and flow compartment implemented as a channel with overflow, and a reservoir that works together with a drainage system via a certain degree of retention capacity availability of its channels itself. The simulation study's research revealed that the usage of ordinary single-chamber reservoirs is the less efficient approach. A contrast of the functions of various hydraulic systems of retention reservoirs under equal circumstances revealed that the required retention volume of a single-chamber reservoir is many times bigger than that of highly efficient alternatives, and it can account for up to 582% of the reservoir's capacity when used in conjunction with the channel retention system. Simultaneously, it has been proved that using channel retention is not the most efficient approach for all hydraulic circumstances in a drainage network or for all hydrological conditions. Furthermore, the research presents a set of retention performance measures that may be used to evaluate particular rainwater storage technologies in the prospective.

Reference [22] states that it is well known that OSD can have adverse effects when it is installed at inappropriate locations, ending up exacerbating the problems of floods. Issues about relying on OSD for regional hydrology control are increased by parameter uncertainty and the requirement for a statistical method to hydrograph development. It introduces research that serves to spread awareness of these concerns as well as providing a realistic solution to the issue. Using interconnected modules, a hydrologic framework for Monte Carlo simulation of regional watershed hydrographs was built. A sample of ten regional watersheds was modelled with the scenarios of current situation, with plots of land of varying sizes and urbanised points in different locations within the regional catchment basin, and with such urbanised points containing OSD. The results have been focused on the discovery and evaluation of two important factors that affect the peak runoff of regional watersheds, namely the size and position of the urban areas land parcel.

The article presented in [23] expresses concern that many existing OSD systems created using the singular temporal pattern for creation storms can fail to meet their claimed aims when tested against a range of other temporal patterns. Following an investigation of the performance of twenty genuine OSD systems, it was determined that expanding the number of temporal patterns for the design and evaluation of OSD systems improved the success rate of accomplishing objectives. In practice, as many different temporal patterns as feasible should be explored as a proposed solution.

In [24], a design technique is suggested that establishes certain new criteria that connect impervious portions of the lots to tank design parameters. The efficiency concept was developed on the assumption that the tanks should offer the restoration of flows from an impervious region to its pre-urbanization situation. This was determined to be 70% of the local maximum discharge. Based on flow routing simulations using the Puls Method, the ideal geometric properties of the tanks (volume, area, water depth, and orifice diameter) were determined to ensure a decrease in the peak. When compared to the results of the municipal legislation plan, the new technique proved to be more efficient, with a 24% reduction in storage tank.

The efficiency of rainwater detention tanks with specific design configurations (insertion into the rain sewerage system; capacity per impermeable area) and operating circumstances (constant and irregular emptying criteria) was analysed using an integrated methodology presented in [25]. Different performance measures have been used to quantify the decrease of pollution impact on the natural environment, the reduction of maintenance and management costs for the urban drainage system, the preservation of regular purifying reliability, and the restriction of expenses at the treatment system. The impact of the primary parameters of the urban catchment and the drainage system (area of the basin and system inclination) on the performance of different design and operational approaches has also been investigated. According to it, stormwater detention tanks combined with discharge controls demonstrated positive results in terms of environmental damage: adequate performance metrics can be achieved with relatively low flow rates of flow regulators (0.5 - 1.0 L/s per hectare of impervious area) and tank volumes of about 35 - 50 m3 per impervious surface. Constant emptying ensured the least amount and length of overflows, but discontinuous operation reduced the amount sent for purification, lowering costs and chances of degradation in the plant's regular treatment efficiency. Generally, simulation results demonstrated that the extent of the watershed and the slope of the drainage system have little effect on efficiency indicators.

3. Methodology

The aim of this article is to present a theoretical comparative study, in terms of peak flow and required pipe diameters, between a conventional drainage system and another one equipped with small underground reservoirs for rainwater runoff attenuation, located at the front of the lots in an urban subdivision. Six stretches of pluvial gallery, receiving contributions from sub-basins delimited by the bottom of the lots and by the drain inlets, which follow each sequence of 03 lots on the same side of the street (06 on both) were considered. The lots have a frontage of 15 metres and a depth of 32 metres. The cross section of the street and sidewalks is 12 metres, and the longitudinal slope of the street, which coincides with that of the drainage galleries, is 0.01 m/m. The system described is schematized in Figure 1.

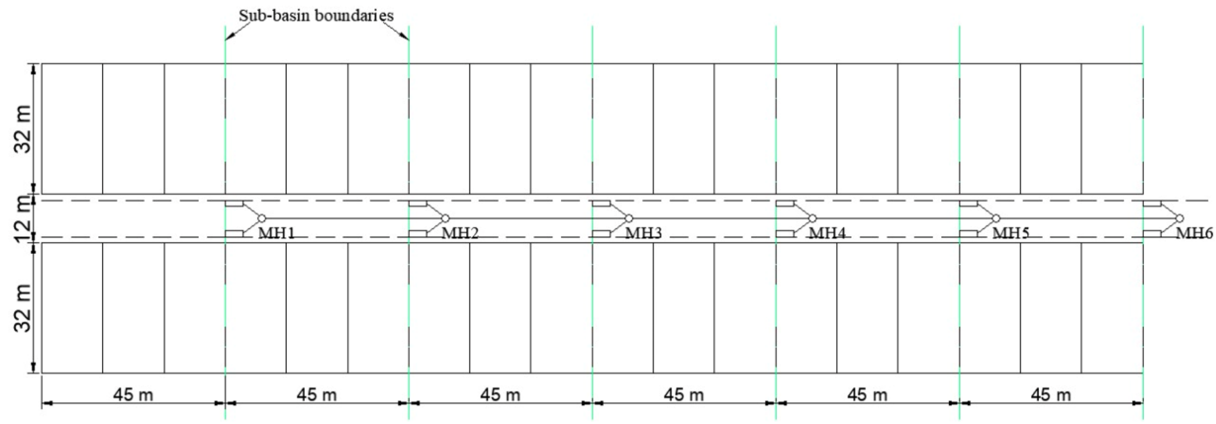

Fig. 1. Schematic of the conventional rainwater drainage system

The process adopted for flow rates calculation was the Rational Method, with time of concentration of 12 minutes in the headwater sub-basin for residential areas with gutter slope less than 03% [26], and 18-year recurrence time, according to that recommended for micro-drainage works (05-20-year). Intensity-duration-frequency equation for the city of Juiz de Fora was used. The runoff coefficient C is 0.5. A uniform steady state is assumed in sections between manholes, and the hydraulic calculations were made using the Manning equation, with a relative roughness coefficient of 0.013. For the hydraulic sizing of pipes, a water depth of at most 85% of the diameter is allowed, in accordance with technical standards. Figure 2 illustrates the worksheet for calculating the drainage gallery, inspired by the worksheet for the dimensioning of sanitary sewage networks [27].

UPSTREAM RUNOFF											DOWNSTREAM GALLERY									
Manhole	A lot (ha)	A st+sw (ha)	A_{accum} (ha)	DC	tc (min)	I (mm/h)	C	Q lot (L/s)	Q st+sw (L/s)	Q total (L/s)	S (m/m)	D_{theor} (m)	D_{real} (m)	K1´	h/D_{real}	h (m)	V (m/s)	L (m)	T_{ti} (min)
1	0,288	0,054	0,342	1	12,00	158,7	0,5	63,5	11,9	75,5	0,01	0,27	0,40	2,266	0,42	0,168	1,50	45	0,50
2	0,288	0,054	0,684	1	12,50	156,6	0,5	62,7	11,8	149,9	0,01	0,35	0,40	1,751	0,63	0,250	1,81	45	0,41
3	0,288	0,054	1,026	0,9962	12,91	154,9	0,5	61,8	11,6	223,3	0,01	0,41	0,50	1,885	0,55	0,275	2,02	45	0,37
4	0,288	0,054	1,368	0,9541	13,28	153,5	0,5	58,6	11,0	292,9	0,01	0,45	0,50	1,703	0,66	0,331	2,13	45	0,35
5	0,288	0,054	1,71	0,9227	13,64	152,1	0,5	56,2	10,5	359,6	0,01	0,49	0,50	1,577	0,78	0,392	2,18	45	0,34
6	0,288	0,054	2,052	0,8978	13,98	150,8	0,5	54,2	10,2	423,9	0,01	0,52	0,60	1,779	0,61	0,365	2,36	45	0,32

Fig. 2. Calculation worksheet of the conventional rainwater drainage system

The worksheet in Figure 2 lists, for each sub-basin and the respective downstream gallery, the areas of the lots and the street and sidewalks; the accumulated area from the headwater sub-basin; the distribution coefficient, which takes into account the irregular distribution of rainfall; the time of concentration; the rainfall intensity; the runoff coefficient; the flow rates of lots, of the street and sidewalks, and total; the slope of the rainwater gallery; the theoretical diameter, for a water depth of 85% of the diameter; the real diameter, immediately superior to the theoretical one; an auxiliary coefficient; the relationship between water height and the chosen diameter; the height of the water depth; the average velocity of the flow; the length of the gallery; and the travel time.

The flow rates from the rains on the lots had to be treated separately from those generated from the streets and sidewalks, due to the fact that, as the reservoirs are placed in the lots, only the flows originating there will have their flow rates attenuated.

It is also important to keep in mind that the damped flow rates of the lots must be those that occur in the same time of concentration considered for the flow rates of the streets and sidewalks, since they will be added.

Figure 3 illustrates a scheme similar to that of Figure 1, however with the small underground damping reservoirs in the front part of the lots. The reservoirs were given the full dimension of the width of the lot, 15.0 m, and a length of 1.0 m, making up an area of 15.0 m2. Each reservoir is provided with a 75 mm orifice at the bottom, for the discharge of the flows.

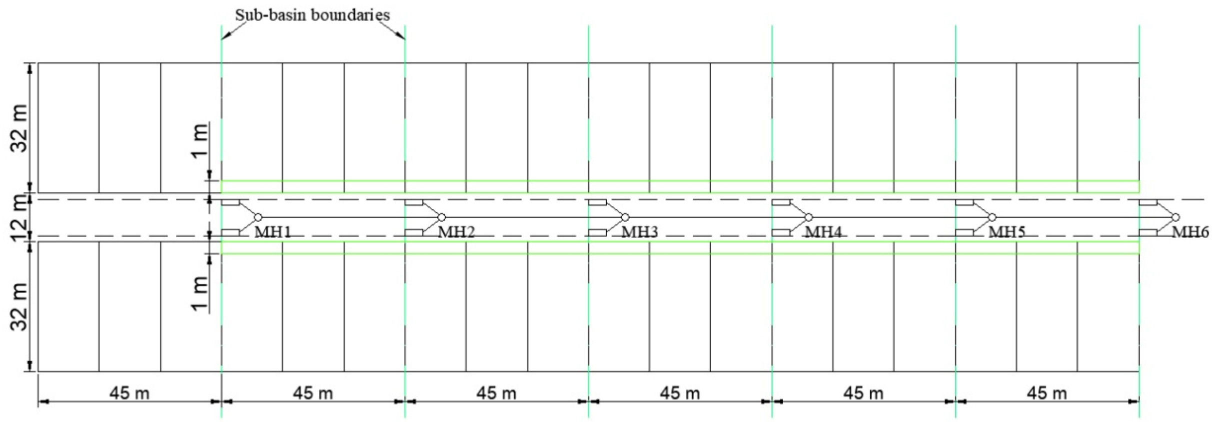

Fig. 3. Schematic of the rainwater drainage system with damping

Using the Routing Simulator Application, introduced in [28, 29], it is possible to determine the flows damped by the lot reservoirs, in each of the sub-basins. Consequently, the total flows of each section will be reduced in relation to the scheme without damping, and it is possible that the diameter previously designated for each section between manholes (MH) can also be reduced in relation to the conventional drainage system, still meeting the criterion that the maximum depth of water is 85% of the diameter.

Figures 4 and 5 show a simulation by the Routing Simulator of the lots flow rate attenuation, from 62.7 L/s to 30.6 L/s, referring to the second upstream sub-basin, which contributes to the gallery section MH2-MH3, the values being in m3/s.

Fração	Tempo Inicial [s]	DT [s]	QA_i	QA_f	QEi	$V_{i\,[m^3]}$	V_f	QE_f	K_1	K_2	Tentativa
1	60	60	0.00	0.01	0.00	0.00	0.08	0.0026	0.27	0.16	21
2	120	60	0.01	0.01	0.00	0.08	0.32	0.0051	0.27	0.47	10
3	180	60	0.01	0.02	0.01	0.32	0.72	0.0077	0.27	0.95	7
4	240	60	0.02	0.02	0.01	0.72	1.28	0.0102	0.27	1.59	11
5	300	60	0.02	0.03	0.01	1.28	2.00	0.0128	0.27	2.38	7
6	360	60	0.03	0.03	0.01	2.00	2.88	0.0153	0.27	3.34	13
7	420	60	0.03	0.04	0.02	2.88	3.92	0.0179	0.27	4.46	14
8	480	60	0.04	0.04	0.02	3.92	5.12	0.0204	0.27	5.74	14
9	540	60	0.04	0.05	0.02	5.12	6.48	0.0230	0.27	7.17	9
10	600	60	0.05	0.05	0.02	6.48	8.00	0.0255	0.27	8.77	9
11	660	60	0.05	0.06	0.03	8.00	9.68	0.0281	0.27	10.53	13
12	720	60	0.06	0.06	0.03	9.68	11.52	**0.0306**	0.27	12.44	15
13	780	60	0.06	0.06	0.03	11.52	13.27	0.0329	0.27	14.26	15
14	840	60	0.06	0.06	0.03	13.27	14.69	0.0346	0.27	15.73	15
15	900	60	0.06	0.05	0.03	14.69	15.81	0.0359	0.27	16.89	16
16	960	60	0.05	0.05	0.04	15.81	16.66	0.0369	0.27	17.76	16
17	1020	60	0.05	0.05	0.04	16.66	17.25	0.0375	0.27	18.38	14
18	1080	60	0.05	0.04	0.04	17.25	17.60	0.0379	0.27	18.74	15
19	1140	60	0.04	0.04	0.04	17.60	17.73	0.0380	0.27	18.87	17
20	1200	60	0.04	0.03	0.04	17.73	17.65	0.0379	0.27	18.78	14
21	1260	60	0.03	0.03	0.04	17.65	17.37	0.0376	0.27	18.50	14
22	1320	60	0.03	0.03	0.04	17.37	16.90	0.0371	0.27	18.02	14
23	1380	60	0.03	0.02	0.04	16.90	16.26	0.0364	0.27	17.35	14
24	1440	60	0.02	0.02	0.04	16.26	15.46	0.0355	0.27	16.53	12
25	1500	60	0.02	0.02	0.04	15.46	14.51	0.0344	0.27	15.54	13
26	1560	60	0.02	0.01	0.03	14.51	13.43	0.0331	0.27	14.42	13
27	1620	60	0.01	0.01	0.03	13.43	12.22	0.0316	0.27	13.17	13
28	1680	60	0.01	0.01	0.03	12.22	10.90	0.0298	0.27	11.80	13
29	1740	60	0.01	0.00	0.03	10.90	9.48	0.0278	0.27	10.32	11
30	1800	60	0.00	0.00	0.03	9.48	7.98	0.0255	0.27	8.75	10

Fig. 4. Routing Simulator numeric output screen

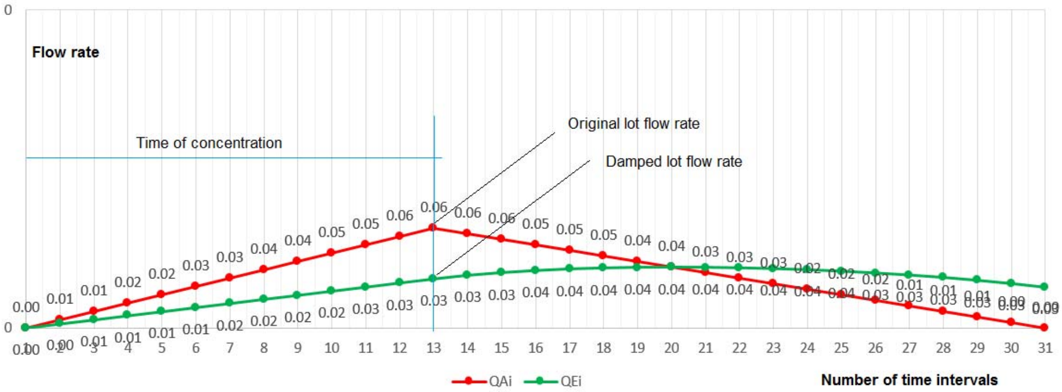

Fig. 5. Routing Simulator graphical output screen

Assigning the damped values to a spreadsheet as in Figure 2, a new spreadsheet is generated, representing the proposed drainage system, shown in Figure 6.

UPSTREAM RUNOFF											DOWNSTREAM GALLERY								
Manhole	A lot (ha)	A st+sw (ha)	A_{accum} (ha)	DC	tc (min)	I (mm/h)	C	Q lot (L/s)	Q st+sw (L/s)	Q total (L/s)	S (m/m)	D_{theor} (m)	D_{real} (m)	K1´	h/D_{real}	h (m)	V (m/s)	L (m)	T_{ti} (min)
1	0,288	0,054	0,342	1	12,00	158,7	0,5	63,5	11,9	75,5	0,01	0,27	0,40	2,266	0,42	0,168	1,50	45	0,50
2	0,288	0,054	0,684	1	12,50	156,6	0,5	30,6	11,8	117,8	0,01	0,32	0,40	1,917	0,54	0,214	1,72	45	0,44
3	0,288	0,054	1,026	0,9962	12,94	154,9	0,5	30,4	11,6	159,8	0,01	0,36	0,40	1,71	0,66	0,262	1,83	45	0,41
4	0,288	0,054	1,368	0,9541	13,35	153,2	0,5	29,4	11,0	200,2	0,01	0,39	0,40	1,571	0,79	0,316	1,88	45	0,40
5	0,288	0,054	1,71	0,9227	13,74	151,7	0,5	28,4	10,5	239,1	0,01	0,42	0,50	1,838	0,57	0,287	2,05	45	0,37
6	0,288	0,054	2,052	0,8978	14,11	150,3	0,5	27,8	10,1	277,0	0,01	0,44	0,50	1,739	0,63	0,317	2,11	45	0,36

Fig. 6. Calculation worksheet for the damped rainwater drainage system

Figure 6 expresses the flow values in L/s, unlike the Routing Simulator, where they are represented in m3/s.

4. Results and Discussion

As verified in the simulations, the maximum level reached by the water in the damping reservoir was 20.0 cm. A recess in the bottom is suggested to accommodate the 75 mm discharge orifice, to be protected with mesh to prevent the ingress of solids, with consequent flow to the street drainage gallery. In this way, the reservoirs could have a total depth of 30 cm.
Table 1 summarises the values of the worksheet in Figure 6 and shows the total flow rates in each section between manholes and their respective diameters of the rain gallery, respecting the maximum depth limits of 85% of the diameter, both for the conventional drainage system and for the drainage system with damping in the lot reservoirs.

Table 1: Flow rates and diameters of conventional and damped systems

Gallery section	Q (L/s) conventional system	Q (L/s) damped system	D (mm) conventional system	D (mm) damped system
1 - (MH1-MH2)	75.5	75.5	400	400
2 - (MH2-MH3)	149.9	117.8	400	400
3 - (MH3-MH4)	223.3	159.8	500	400
4 - (MH4-MH5)	292.9	200.2	500	400
5 - (MH5-MH6)	359.6	239.1	500	500
6 - (MH6-MH7)	423.9	277.0	600	500

Flow data were expressed graphically and are represented in Figure 7.

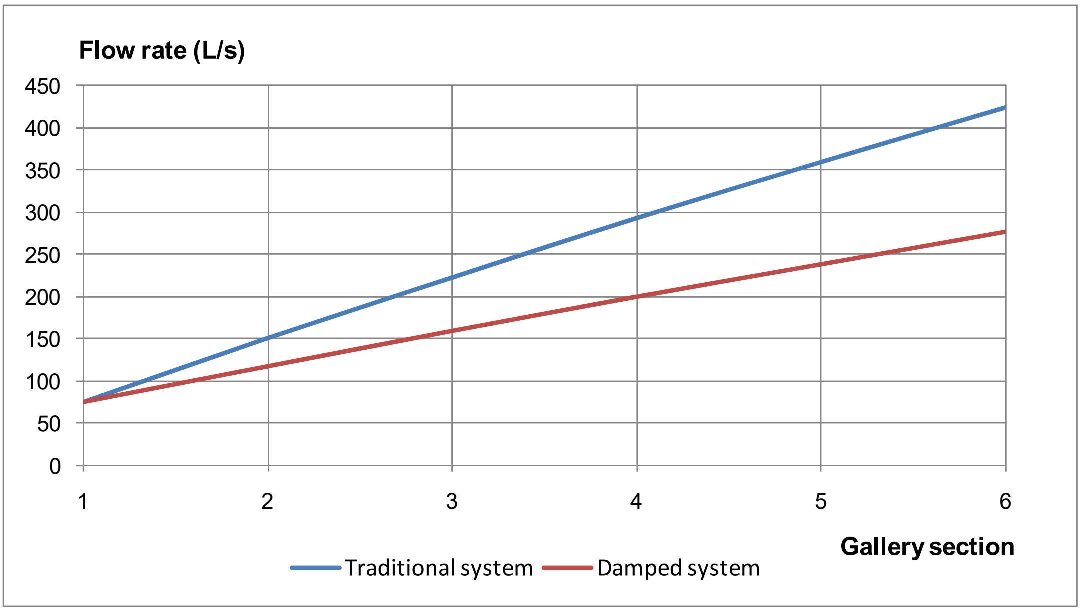

Fig. 7. Evolution of flow damping per section of rainwater gallery

It was found that, with the adoption of damping reservoirs, the flow rates were reduced from 21% to 35% in relation to those of traditional systems, with such percentages of reduction increasing as one went from upstream to downstream. This behaviour suggests that there could be even greater damping if the number of sections were greater than that represented here.
The design of the drainage galleries using the traditional system resulted in diameters of 400 mm for the two initial sections, 500 mm for the three following sections, and 600 mm for the last section. Using the proposal of damping reservoirs in the lots, the design indicated diameters of 400 mm for the first four sections and 500 mm for the last two sections. That is, with the methodology, there was a gain in relation to the diameter reduction in the third and fourth sections, from 500 to 400 mm, and in the last section, from 600 to 500 mm.

5. Conclusion

A work was presented and discussed on the installation of small reservoirs in the lots to dampen the flows, with an impact on reducing the diameters of the drainage galleries. Simply aiming at a matter of financial advantage, it would be enough for the person responsible for the subdivision's infrastructure to verify if it would be more interesting, financially speaking, to save the amount referring to the reduction of the diameters of the galleries and bear the cost of building the damping units, or proceed from the traditional way.
However, this issue can be presented in a much more complex way, where not only the micro level, internal to the subdivision, is the one to be evaluated, but also the macro level, that of the city where it is inserted. Although the issues inherent to the subdivision, referring to the correct drainage of rainwater can be fully satisfied, greater or lesser flow rates may be released in the already consolidated galleries of the city, providing conditions for the occurrence or not of flooding. The matter may involve social policies and urban development interests. It is not just a necessarily punctual approach. This can, and perhaps should, also be seen as an environmental, public health, welfare and social responsibility issue.

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