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Analysis of the Implementation of Detention Tanks in Urban Lots

Análise da Implantação de Reservatórios de Detenção em Lotes Urbanos

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Abstract: The waterproofing of watersheds increases surface runoff and decreases water infiltration into the soil. In this context, urban floods occur more frequently and with greater magnitudes, which can have negative impacts on the environment. This study evaluated flood attenuation through detention tanks in urban lots installed in the city of Crato, Ceará, Brazil. An urban drainage system was sized and simulations were performed using the Storm Water Management Model software. Peak flow from sub-basins, peak flow and water depths in the stormwater pipes, peak flow and volume discharged from the system outfalls were evaluated under two scenarios: (1) without the presence of attenuation structures and (2) with detention tanks. The results showed that detention tanks were able to reduce peak flow in the sub-basins and stormwater pipes by an average of up to 83% and 71%, respectively, and 52% of the maximum relative water depth in the stormwater pipes. The discharged volume and peak flow at the outfalls had maximum reductions of 68% and 69%, respectively. Therefore, the installation of these structures enabled better management of the watershed's water balance, with a forecasted decrease in flood occurrences.

Keywords: Urbanization; Detention tanks; LID; SWMM.

Resumo: A impermeabilização das bacias hidrográficas aumenta o escoamento superficial e diminui a infiltração de água no solo. Nesse contexto, as inundações urbanas ocorrem com maiores frequências e magnitudes, e podem provocar impactos negativos no meio. Este trabalho avaliou o amortecimento de cheias por meio de reservatórios de detenção nos lotes urbanos instalados na cidade do Crato, Ceará, Brasil. Um sistema de drenagem urbana foi dimensionado e simulações foram realizadas no *software Storm Water Management Model*. A vazão de pico de sub-bacias, vazão de pico e lâminas d'água nas galerias, vazão de pico e volume escoado nos exutórios do sistema foram avaliados a partir de dois cenários: (1) sem a presença de estruturas de amortecimento e (2) com reservatórios de detenção. Os resultados mostraram que os reservatórios de detenção foram capazes de reduzir a vazão de pico nas sub-bacias e nas galerias em média com valores máximos de 83% e 71%, respectivamente, e 52% da lâmina d'água relativa máxima nas galerias. O volume escoado e vazão de pico nos exutórios tiveram reduções máximas de 68% e 69%, respectivamente. Logo, a instalação dessas estruturas possibilitou um melhor gerenciamento do balanço hídrico da bacia hidrográfica, com previsão de diminuição de ocorrências de inundações.

Palavras-chave: Urbanização; Reservatórios de detenção; LID; SWMM.

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1. Introduction

The process of urbanization has increased the impermeabilization of watersheds (FRIAS; MANIQUIZ-REDILLAS, 2021). This process, along with climate change, is affecting the hydrological cycle (BAEK et al., 2020). In the water balance, there is an increase in the proportion of surface runoff (FRIAS; MANIQUIZ-REDILLAS, 2021) and peak flow, and a decrease in water infiltration into the soil (UCHIYAMA; BHATTACHARYA; NAKAMURA, 2022). As a result, there is an increased probability of urban flooding occurrences that can cause economic losses and threats to life (ZHOU et al., 2022), and a decrease in the supply of water resources in adequate quantity and quality (TUCCI, 2016; NOWOGOŃSKI, 2020).

In order to mitigate the effects of urban floods, conventional urban drainage measures are generally taken, referring to the hygienist conception of the system, which aims to quickly convey stormwater from a location to a receiving water body (ECKART; MCPHEE; BOLISETTI, 2017), through microdrainage elements such as curbs, gutters, catch basins, connecting pipes, pipes, along with macrodrainage elements such as rivers, open channels, energy dissipaters, and culverts. However, the emergence of new impermeable areas can lead to a failure in the current system (CIMORELLI et al., 2016; QUICHIMBO-MIGUITAMA et al., 2022).

To correct the effects caused by the failure, elements of the drainage system are usually replaced and/or modified to meet the new hydraulic condition, requiring high financial costs (DRUMOND, 2012), which are common practices adopted in Brazil.

Currently, studies are being developed with the aim of making the urbanized watershed more similar to natural hydrology (XIAN et al., 2021) through structural measures, essentially characterized by engineering works; and non-structural measures, such as the development of stormwater management plans, soil management, flood insurance, and environmental education (LA LOGGIA; PULEO; FRENI, 2020).

Sustainable practices such as Low Impact Development (LID) (ZHUANG; LI; LU, 2023), Water Sensitive Urban Design (WSUD), Integrated Urban Water Management (IUWM), Sustainable Drainage Systems (SuDS), Best Management Practices (BMPs), Alternative Techniques (ATs), Source Control (SC) (FLETCHER et al., 2015), and Green Infrastructure (GI) (ZHANG; CHUI, 2020) are commonly employed in managing the water balance of the watershed.

Among the structural measures to make the watershed more similar to natural hydrology, green roofs, permeable pavements, rainwater harvesting reservoirs for non-potable uses, infiltration trenches, bioretention cells (rain gardens), swales and infiltration pits, and rainwater detention tanks stand out. These measures, implemented individually or in combination, enable an increase in infiltration, evaporation, transpiration, retention, and detention, decrease in surface runoff with consequent reduction in peak flow, and possible non-potable uses of rainwater (QIN, 2020).

The Storm Water Management Model (SWMM) by the U.S. Environmental Protection Agency (EPA) stands out as a tool for hydrological modeling (PANOS; WOLFAND; HOGUE, 2020). Variables such as size; number of units; location in the watershed; choice and combinations of LID practices; and installation, operation, and maintenance costs can be evaluated with the assistance of this and other computational tools, significantly facilitating decision-making (ECKART; MCPHEE; BOLISETTI, 2017), as well as the automation of detention processes through so-called "smart systems" (DI MATTEO et al., 2019) and real-time control (ALTOBELLI; EVANGELISTI; MAGLIONICO, 2024).

This study aimed to analyze flood attenuation through detention tanks in urban lots installed in the city of Crato, Ceará, Brazil, by composing two scenarios. The first scenario consisted of a conventional urban drainage system, and the second scenario included the addition of low impact development (LID) such as detention tanks installed in urban lots.

2. Methodology

The municipality of Crato is located in the southern region of the state of Ceará, Brazil (7°14’03’’ S and 39°24’34’’ WGr), with an area of 1138.15 km² and a resident population of 131,050 inhabitants (IBGE, 2023). It has a mild semi-arid tropical climate and a warm subhumid tropical climate, with an average annual precipitation of 1091 mm, average annual potential evapotranspiration (EP) of 1428 mm, and average temperatures ranging between 24 and 26 °C. Close to the Chapada do Araripe, Crato presents soils of the Argisol, Latosol, and Neosol types, and vegetation including Thorny Scrub, Deciduous Thorny Tropical Forest (Arboreal Caatinga), Semi-Deciduous Tropical Rainforest (Dry Forest), Semi-Deciduous Xeromorphic Tropical Forest (Cerradão), and Semi-Evergreen Tropical Rain-Cloud Forest (Wet Forest) (IPECE, 2012).

The Mirandão neighborhood was chosen as the research subject due to its distance from urbanized areas and the existence of academic studies already conducted, such as that of Feitosa (2015).

Using contour lines and street mapping imagery (Figure 1), the Mirandão neighborhood was divided into 8 parts (Figure 2). Each part may consist of several sub-basins and classified into two groups: Urban Block (in this study, referring to the number of urban lots that produce sufficient runoff to fill the gutter section) and Asphalt Paved Streets. The delineation of each sub-basin was performed manually in AutoCAD with the assistance of the UFC8 software belonging to the UFC System (BEZERRA; CASTRO, 2009).

Figure 1 – Street Mapping of Mirandão Neighborhood, Crato, Ceará, Brazil.
Source: Google Earth Pro (2020).

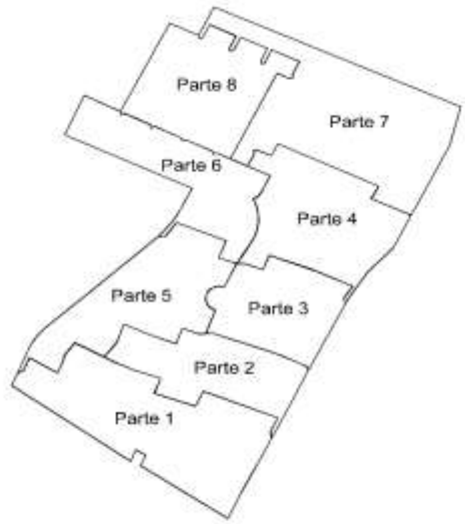

Figure 2 – Parts of Mirandão Neighborhood.
Source: The author (2021).

The hydraulic and hydrological parameters were defined for each sub-basin depending on its group (Table 1). These parameters are generally related to surface permeability, and specifically, the runoff coefficient (C) and Curve Number (CN) are dimensionless factors corresponding to land cover, while the initial abstraction (I_a) refers to a percentage of the maximum potential cumulative infiltration (S_R) from which the assessment proceeds.

Table 1 – Hydraulic and hydrological parameters adopted.

Group	**Watershed parameters**		
	Runoff Coefficient, C	**Curve Number, CN**	**Initial Abstraction, I_a**
Urban Block	0,60	80	$0{,}2 \cdot S_R$
Asphalt Paved Streets	0,95	98	$0{,}2 \cdot S_R$

Source: Tucci (1995).

After delineating each sub-basin, the UFC8 program (BEZERRA; CASTRO, 2009) was used to calculate morphometric parameters, such as: area (A_B), length of the main river in kilometers (L), maximum elevation difference in meters (ΔH), width of the surface flow in meters, and the concentration time in minutes using the Kirpich equation (t_c) which is valid for areas less than 1 km² (Eq. 1).

$$t_c = 57 \cdot \left(\frac{L^3}{\Delta H}\right)^{0{,}385} \quad (1)$$

The Rational Method was chosen to calculate the peak flow of the sub-basin (Eq. 2) due to the areas being less than 1 km².

$$Q_P = \frac{C \cdot i \cdot A_B}{3{,}6} \quad (2)$$

In which:

Q_P = peak flow of sub-basin, in m³/s;
C = runoff coefficient, dimensionless;
i = rainfall intensity, in mm/h;
A_B = area of sub-basin, in Km².

The rainfall intensity was determined by the Intensity-Duration-Frequency (IDF) Equation (Eq. 3).

$$i = \frac{31{,}86 \cdot (Tr - 2{,}08)^{0{,}0896}}{(t_d + 9{,}8325)^{0{,}7939}} \tag{3}$$

In which:
i = rainfall intensity, in mm/h;
Tr = return period, in years;
t_d = rainfall duration, in minutes.

The concentration time of each sub-basin (Eq. 1) was less than 10 minutes; therefore, the design rainfall duration was considered to be 10 minutes, as recommended by the National Department of Transportation Infrastructure (BRASIL, 2006). The choice of the return period is related to the analysis of the frequency with which a rainfall event can be equaled or exceeded, thus determining the scale of the mitigation work. The recurrence of a system's demand varies inversely with the return period. Macrodrainage systems are designed to handle higher flows, which should occur less frequently, i.e., longer return periods, and the reduction in flows indicates a reversal of this analysis, in which case shorter return periods are applied for microdrainage structures. Therefore, being a microdrainage system, the chosen return period was 10 years (SÃO PAULO, 2012).

Subsequently, the manual layout of concrete connecting pipes connected the outlets of the sub-basins to the concrete pipes. Manholes were inserted automatically. The Saint Venant equations of the dynamic wave model (Eq. 4 and 5) were used in the sizing of the conduits.

In the sizing tab of the UFC8 System, the minimum and maximum slopes of the stormwater pipes were adopted as 0.5% and 7.5%, respectively; maximum velocity in the conduit equal to 5 m/s; minimum step equal to 5 cm; maximum depth in the conduits equal to 100%; minimum clearance between the conduit and the obstacle equal to 15 cm; minimum cover equal to 1 m; and not allowing the reduction of diameters downstream.

$$\frac{\partial A}{\partial t} + \frac{\partial Q_v}{\partial x} = 0 \tag{4}$$

$$\frac{\partial Q_v}{\partial t} + \frac{\partial \left(\frac{{Q_v}^2}{A}\right)}{\partial x} + g \cdot A \cdot \frac{\partial Y}{\partial x} + g \cdot A \cdot (S_f - S_0) = 0 \tag{5}$$

In which:
x = distance, [L];
t = time, [T];
A = flow cross-sectional area, [L^2];
Q_v = volumetric flow rate, [$L^3.T^{-1}$];
$H = (Z + Y)$ = hydraulic head of water in the conduit, [L];
Z = conduit invert elevation, [L];
Y = conduit water depth, [L];
S_f = friction slope (head loss per unit length);

$S_0 = -\frac{\partial z}{\partial x}$ = conduit slope;
g = gravitational acceleration, [L.T^{-2}];

The Storm Water Management Model (SWMM) is a dynamic (distributed) model with applications, for example, in the design and sizing of drainage systems, flood control, water quality, and performance assessment of compensatory techniques.

With the urban drainage system sized by UFC8, simulation files in SWMM were generated for each of the eight parts (Figure 2). In SWMM, the water balance in the sub-basin is carried out through the nonlinear reservoir model and considers uniform flow in the sub-basin towards a rectangular channel with width (W), height ($d - d_S$), slope (S), and Manning's roughness coefficient (n) (ROSSMAN, 2015).

Therefore, by conservation of mass, the variation of depth (d) over time (dt) in the sub-basin is given by the nonlinear differential equation (Eq. 6):

$$\frac{dd}{dt} = p - e - f - \frac{S^{1/2} \cdot W}{n \cdot A_B} \cdot (d - d_S)^{5/3} \quad (6)$$

In which:
p = rainfall rate per unit area $\left(\frac{m}{s}\right)$;
e = surface evaporation rate per unit area $\left(\frac{m}{s}\right)$;
f = infiltration rate per unit area $\left(\frac{m}{s}\right)$;
n = Manning's roughness coefficient $\left(\frac{s}{m^{1/3}}\right)$;
A_B = area of sub-basin (m^2);

The infiltration model chosen was based on the Curve Number (CN) value developed by the Natural Resource Conservation Service (NRCS), with values input for each sub-basin (Table 1). The propagation of flow in nodes and conduits is carried out by solving the one-dimensional Saint Venant equations (Eqs. 4 and 5).

In SWMM, detention tanks can be represented only by the storage layer, along with the drain and possible overflow, without the presence of rainfall directly over the area, as it is closed, and the bottom layer is impermeable (Figure 3).

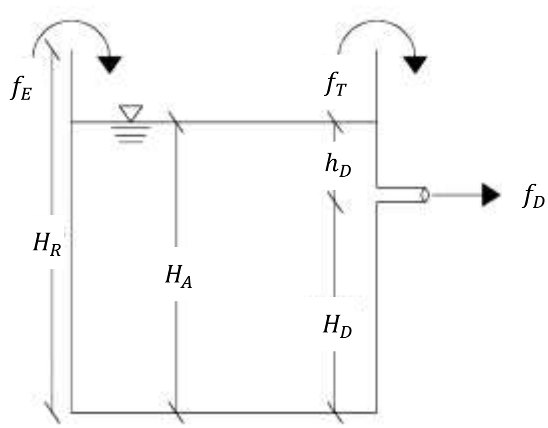

Figure 3 – Detention tanks simulation model in SWMM.
Source: The author (2021).

The water balance in the detention tanks is given by Eq. 7:

$$\frac{dH_A}{dt} = f_E - f_T - f_D \quad (7)$$

In which:
H_A = depth of water in the tank, in mm;
dt = time interval, in hours;
f_E = inflow into the tank, in mm/h;
f_T = overflow through the tank, in mm/h;
f_D = outflow through the drain, in mm/h;
H_R= tank height, in mm;
H_D = drain displacement height, in mm;
h_D = water height that will produce drainage through the drain, in mm.

The operation of the tank is determined by the following conditions:

- $h_D = 0$ to $H_A \leq H_D$;
- $h_D = H_A - H_D$ to $H_D < H_A \leq H_R$.

The outflow through the drain (f_D) in time interval (Δt) is limited by the volume of water stored in the tank (Eq. 8):

$$f_D = min\left[f_D, \frac{H_A}{\Delta t}\right] \quad (8)$$

SWMM allows the drain to be closed before a rain event and then opened after a specified number of hours following its end. The inflow into the tank (f_E) is defined by Eq. 9:

$$f_E = min\left[q_0, \frac{(H_R - H_A)}{\Delta t} + f_D\right] \quad (9)$$

Finally, overflow (f_T) occurs when the captured quantity of runoff from the treated area (q_0) is greater than the inflow (f_E) (Eq. 10):

$$f_T = max[0, q_0 - f_E] \quad (10)$$

The outflow through the drain (f_D) is given by (Eq. 11):

$$f_D = \alpha \cdot h_D{}^{\beta} \quad (11)$$

In which:
α = parameter of the drain, with dimension: $\frac{mm^{(1-\beta)}}{h}$;
β = exponent of the drain, dimensionless.

When the exponent of the drain is equal to 0.5, it functions as an orifice or tube; therefore, we have (Eqs. 12 e 13):

$$\alpha = C_d \cdot \left(\frac{A_D}{A_R}\right) \cdot \sqrt{2 \cdot g} \quad (12)$$

$$f_D = \alpha \cdot {h_D}^{0.5} \quad (13)$$

In which:
A_R = tank base area, in m²;
A_D = drain opening area, in m²;
g = gravitational acceleration, in mm/h²;
C_d = coefficient of discharge of the drain, adopted 0.6 in the EPA technical manual.

Through equations 12 and 13, it is observed that the outflow through the drain is characterized by the discharge per unit area of the tank base. In the SWMM LID control editing window, a diameter of 1.5 m was chosen for the detention tank, justified by its availability in the civil construction market. The tank height (H_R) was simulated from 0 to 3 m, at intervals of 0.5 m, with a value of 0 m representing the drainage system without the tanks. The drain parameter (α) and exponent (β), displacement height (H_D), and time the drain remains closed were set to 0. The rationale for these choices is that the tank will receive contributions from the Urban Block sub-basin, and over time, the water level in it may rise to the maximum height (H_R) (Figure 3). If there is no more storage capacity, runoff will be directed to the sub-basin outlet by overflow (f_T), following equation 7. The percentage of treated area parameter was set to 100, so that runoff from all impervious areas of the sub-basin would be directed to the tank.

Thus, the hydrograph of the sub-basins and stormwater pipes, peak flow, and volume discharged at the system outfalls were evaluated for the scenario composed of a conventional urban drainage system and the scenario with the addition of low impact development (LID), such as detention tanks installed in urban lots.

3. Results and discussion

After delimiting and selecting the hydrological parameters for each sub-basin, the drainage system was sized, and the input file for simulations in SWMM was generated. Subsequently, detention tanks in the lots of Urban Block type sub-basins were inserted (Table 2).

Table 2 – Characteristics of the parts.

Parts	Total area (hectares)	Total sub-basins	Total number of Urban Block type sub-basins	Urban Lots with tank
Part 1	4,98	81	32	175
Part 2	2,62	44	16	97
Part 3	2,55	33	6	42
Part 4	3,61	43	19	119
Part 5	3,50	53	19	117
Part 6	2,95	47	14	83
Part 7	5,55	76	27	173
Part 8	3,36	51	21	160

Source: The author (2021).

3.1 Sub-basins

The detention tank heights of 300 cm, 250 cm, 200 cm, 150 cm, and 100 cm reduced peak flow from sub-basins on average in the range of 75% to 83%, 65% to 83%, 36% to 79%, 14% to 41%, and 2% to 10%, respectively. The 50 cm height was considered ineffective as the percentage reduction in peak flow from sub-basins was zero.

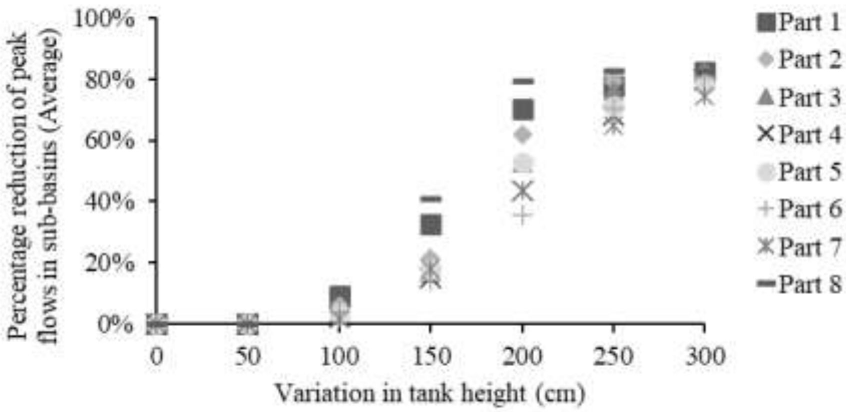

Figure 4 – Percentage reductions in peak flow in the sub-basins.
Source: The author (2021).

Regarding the 100 cm height, the maximum and minimum reductions were identified in Parts 8, and 4 and 7, respectively. Concerning the tank height of 150 cm, Part 8, again, showed the highest percentage reduction compared to other subareas, with the minimum value found in Part 6. The tank with a height of 200 cm reduced peak flow by percentages above 40% in subareas, except in Part 6 with a value of 36%. The 250 cm height reduced peak flow by percentages equal to or greater than 70%, except in Parts 4 and 7 with 68% and 65%, respectively. Part 8 provided the highest percentage reductions compared to other subareas. This can be justified by it having approximately 47 urban lots with tanks per total area.

3.2 Stormwater Pipes

Regarding peak flow in the stormwater pipes, the results showed that the smallest reductions were found in Part 3 with values equal to or less than 11%, justified by having a ratio of 16 urban lots with tank per total area, the lowest value among the others. Excluding it, the reductions varied between minimum and maximum limits by 49% to 71%, 44% to 71%, 34% to 70%, 16% to 51%, 6% to 15%, 0% to 2% for heights of 300, 250, 200, 150, 100, and 50 cm, respectively (Figure 5).

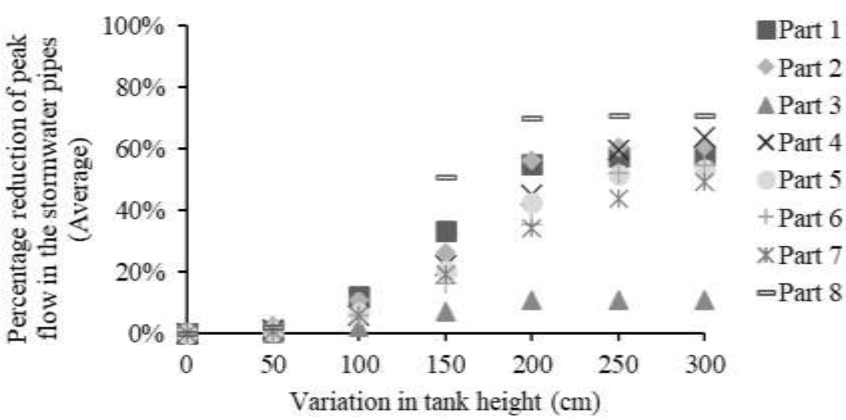

Figure 5 – Percentage reductions of peak flow in the stormwater pipes.
Source: The author (2021).

The smallest percentage reductions in maximum relative water depth in the stormwater pipes (Figure 6) were found in Part 3, with reductions equal to or less than 8%. Compared to other parts studied, the percentage reductions in maximum relative water depth ranged between minimum and maximum limits of 36% and 52%, 34% and 52%, 23% and 52%, 11% and 37%, 4% and 11%, 0% and 2% for heights of 300, 250, 200, 150, 100, and 50 cm, respectively.

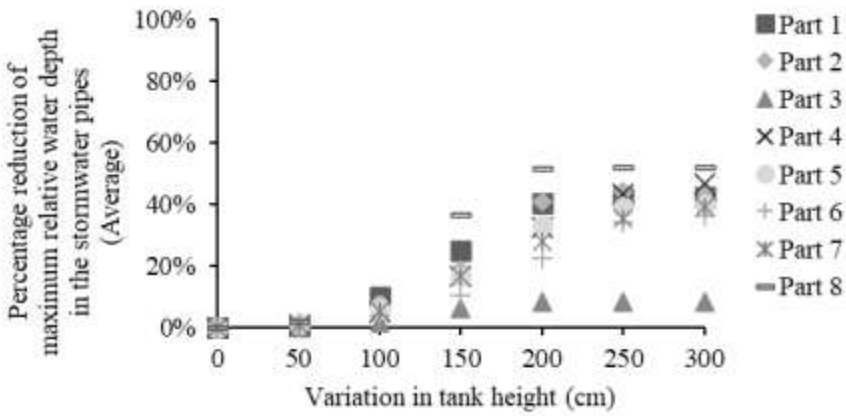

Figure 6 – Percentage reductions of maximum relative water depth in the stormwater pipes.
Source: The author (2021).

The results indicate that the implementation of tanks in urban lots allows for a reduction in the diameter when designing the stormwater pipes. The extent of the diameter reduction depends on the choice of tank size for the site under study. In subareas 1, 2, 3, and 8, a tank height of 200 cm can be considered optimized, as heights of 250 and 300 cm reduced peak flow and maximum relative water depth in the stormwater pipes by a maximum of 5%. Following the same criterion, a height of 250 cm is adopted as optimized in subareas 4, 5, 6, and 7. Therefore, it is observed that the water level in the detention structure has stabilized. Optimized heights indicate that for such limits, larger tanks are unnecessary, as they no longer represent changes in reducing water depth and peak flow. However, it is emphasized that the choice of tank height depends on the efficiency to be achieved.

3.3 Outfalls

The hydrographs at the outfalls (Figures 7 to 14) clearly showed a reduction in peak flow (Figure 15) and runoff volume (Figure 16). The time to peak without detention structures was 9, 8, 8, 10, 9, 9, 11, and 9 minutes for sub-areas 1 to 8, respectively. With the tanks, the time to peak at the outfalls ranged from 8 to 13 minutes, with a maximum damping of 2 minutes. In situations where the water level in the tank was stabilized (Parts 1, 2, 3, 6, and 8: Tank height 250 and 300 cm), capturing all the rainfall on the impermeable area of the urban lot, the flow at the outfalls was due to runoff in the Asphalt Paved Streets sub-basins. For these cases, the peak time at the outfalls occurred at the same time as the peak time for the Asphalt Paved Streets sub-basins.

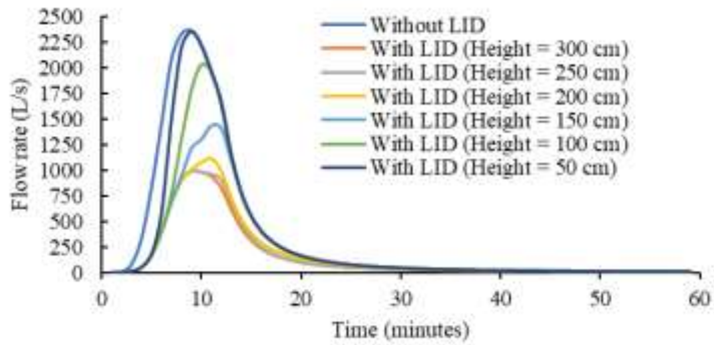

Figure 7 – Hydrographs at the outfall of Part 1.
Source: The author (2021).

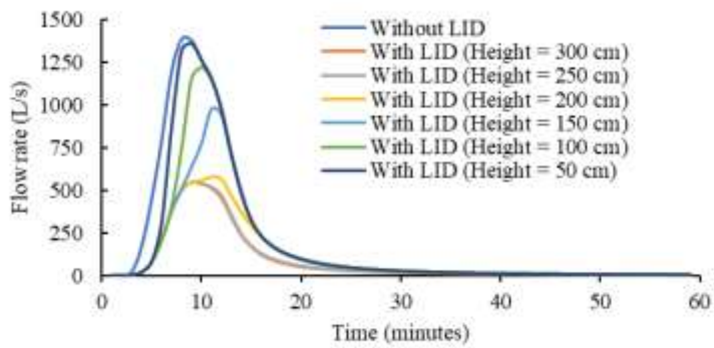

Figure 8 – Hydrographs at the outfall of Part 2.
Source: The author (2021).

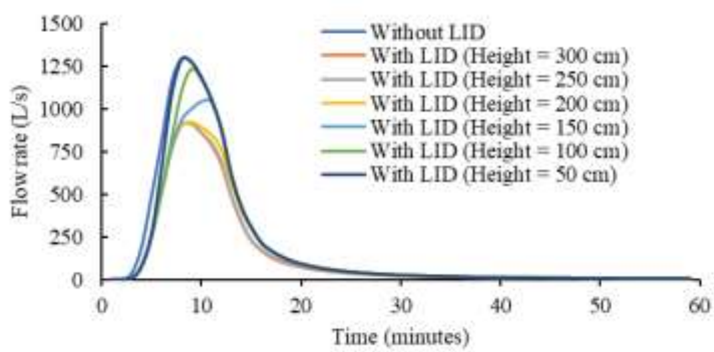

Figure 9 – Hydrographs at the outfall of Part 3.
Source: The author (2021).

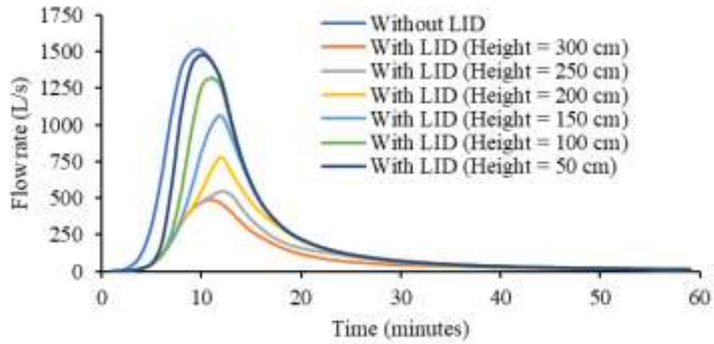

Figure 10 – Hydrographs at the outfall of Part 4.
Source: The author (2021).

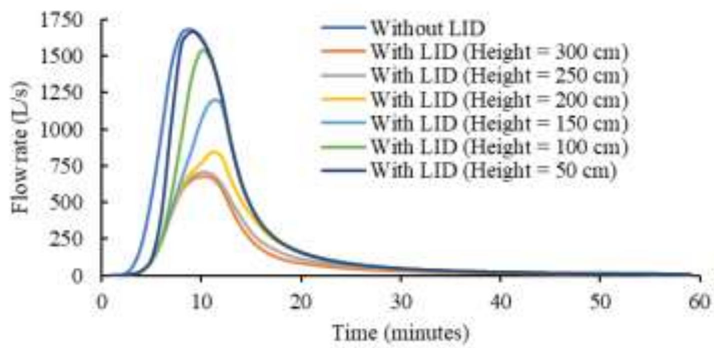

Figure 11 – Hydrographs at the outfall of Part 5.
Source: The author (2021).

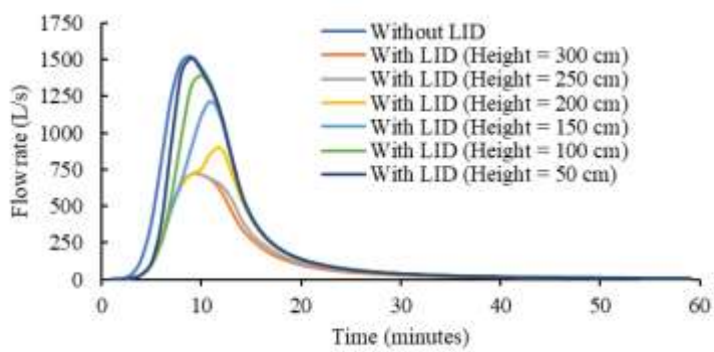

Figure 12 – Hydrographs at the outfall of Part 6.
Source: The author (2021).

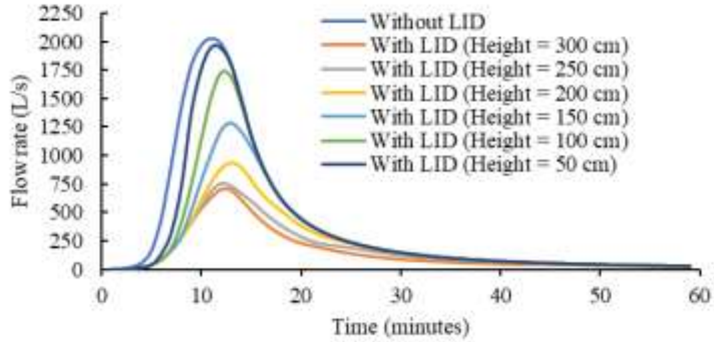

Figure 13 – Hydrographs at the outfall of Part 7.
Source: The author (2021).

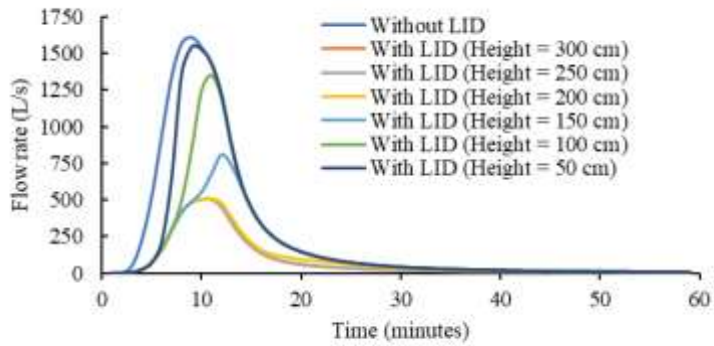

Figure 14 – Hydrographs at the outfall of Part 8.
Source: The author (2021).

Peak flow at outfalls 1 to 8 in the situation without implementation of detention tanks were equal to 2388.89; 1405.83; 1304.29; 1511.37; 1686.39; 1534.74; 2031.33; and 1617.41 L/s, respectively. In the scenario of implementing LID detention tanks, peak flow at outfalls 1 to 8 ranged between the minimum limit (tank height of 300 cm) and maximum limit (tank height of 50 cm) from 995.90 to 2359.53 L/s; 547.42 to 1376.84 L/s; 920.60 to 1297.61 L/s; 490.03 to 1475.35 L/s; 680.55 to 1672.44 L/s; 724.29 to 1513.33 L/s; 714.41 to 1957.72 L/s; 506.73 to 1561.05 L/s, respectively. The percentage reductions in peak flow (Figure 15) for outfalls 1 to 8 were equal in the range of 1% to 58%; 2% to 61%; 1% to 29%; 2% to 68%; 1% to 60%; 1% to 53%; 4% to 65%; 3% to 69%, respectively.

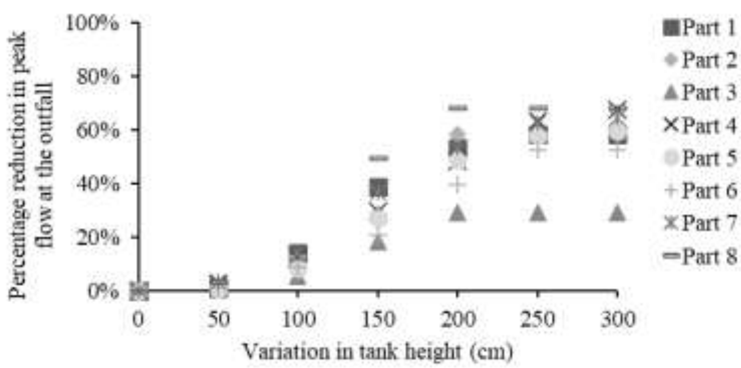

Figure 15 – Percentage reductions in peak flow at the outfall.
Source: The author (2021).

With the installation of detention tanks on the urban lots, as shown in Figure 16, the runoff volume at outfalls of parts 1 to 8 decreased in the range of 13% to 56%; 12% to 59%; 6% to 29%; 12% to 63%; 11% to 58%; 9% to 49%; 11% to 61%; 6% to 68%, respectively, with minimum values referring to a height of 50 cm and maximum value of 300 cm.

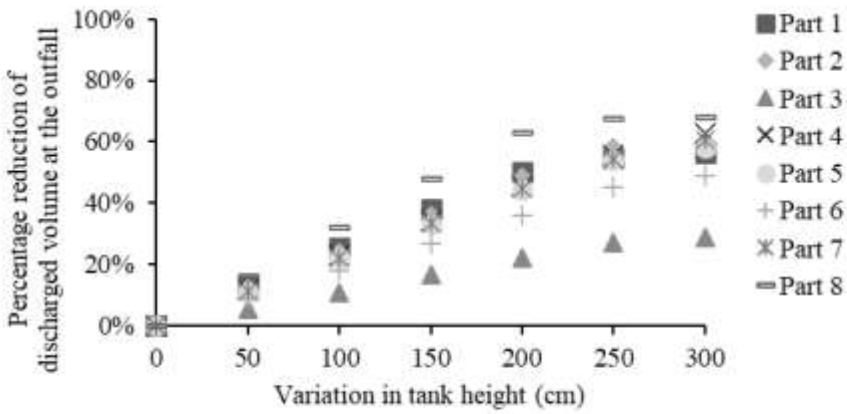

Figure 16 – Percentage reductions in volume discharged at the outfall.
Source: The author (2021).

The analyses revealed that in the outfalls, the efficiency of each studied part was not altered in some situations of increased tank height, as the water level therein stabilized, with the tank dampening all runoff from the impervious area of the urban lot, with flow directed to the outlet, mainly due to the Asphalt Paved Streets. Therefore, the presence of a trend plateau indicated the optimized tank height from a hydraulic perspective. Thus, the optimized heights are 200, 200, 200, 250, 250, 250, 250, and 200 for parts 1, 2, 3, 4, 5, 6, 7, and 8, respectively.

Feitosa (2015) analyzed the implementation of infiltration trenches with a square cross-section of 1.00x1.00 m in the public sidewalk in the Mirandão neighborhood, Crato, CE. The results of the simulations demonstrated that the use of this LID technique reduced peak flow on the urban lots by at least 70%, in addition to increasing aquifer recharge. However, for the proper functioning of the system, strict maintenance control must be developed to reduce the risks of clogging. Pochwat and Pizzo (2022) conducted simulations in SWMM to evaluate the reduction in the volume of stormwater runoff to the drainage system with the use of automated detention tanks connected in series or parallel, and concluded that, in most laboratory analyses, these structures reduced the amount of water by 40%, 67%, and 83% for tank areas of 2 m², 4 m², and 10 m², respectively.

Despite executive differences with infiltration trenches, detention tanks appear as an alternative in controlling urban water runoff, as demonstrated by the results of this research and Pochwat and Pizzo (2022), with the magnitude of the system's efficiency influenced mainly by the area of stormwater capture and the design rainfall.

4. Conclusions

The present study compared two urban stormwater runoff scenarios in the Mirandão neighborhood, city of Crato, Ceará, Brazil. The first scenario consisted of a conventional urban drainage system, while the second scenario involved the addition of Low Impact Development (LID) measures such as detention tanks installed within urban lots. These LID structures attenuated peak flow in the sub-basins, reduced the maximum relative water depth in the stormwater pipes, as well as the runoff volume and peak flow at the outfalls. Implementation, operation, and maintenance costs were not evaluated.

The use of LID techniques such as detention tanks within urban lots proved effective from a hydraulic standpoint. Consequently, lesser water depths and volumes are discharged into the drainage network, allowing for better management of the watershed's water balance and reducing potential flood occurrences. Additionally, drainage systems can be designed with smaller flows, leading to reduced costs.

Hydrodynamic simulations in SWMM can contribute to stormwater management, including the definition of minimum detention and infiltration volumes and combinations of different sustainable structures within urban spaces. As recommendations for future work, further analysis of cost-benefit, accountability criteria for implementation, operation, and maintenance of systems, and the location of structures could be evaluated.

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