PRIVATE LOT FLOOD PEAK ATTENUATION BY STORMWATER DETENTION TANKS

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ABSTRACT

Changes in the hydrological cycle due to climate change and urbanization augment and accelerate runoff and flooding, degrade the urban environment, and cause human and material losses. Thus, it is important to implement measures that ensure urban hydrological conditions are kept as close as possible to pre-urbanization conditions, preventing floods. In addition to the conventional major and minor systems, cities may establish criteria for percentage of permeable area as well as stormwater management practices such as stormwater detention tanks, a type of low impact development technology (LID). The present study evaluates the adequacy of current practices in private lot detention tank design. It analyses time to empty, total detention time and flood peak abatement provided by detention tanks designed according to Curitiba's (Brazil) Bylaw 176/2007. Based on the results obtained, modifications were suggested to existing legislation to increase the efficiency of the detention tanks and, thus, reduce urban flooding and adapt to climate change. The proposed methodology can be applied elsewhere to guide detention tank design.

Keywords: flood control mechanisms, low impact development technologies (LID), sustainable drainage, sustainable hydrology, urban drainage.

1 INTRODUCTION

Urbanization drastically changes the hydrological characteristics of urban agglomerations. Balance is altered, water is polluted, and human populations increase, fostering urban growth to the detriment of local ecosystems [1]. In addition, hydrological changes associated with climate change have been observed and documented globally. These indicate that greater water storage can balance the water cycle and offset vulnerabilities related to flooding and water resource availability [2]. Increases in frequency and magnitude of flood events are expected due to climate change, with tropical and subtropical regions being more susceptible to flood events [3]. As a result, cost of water related services are expected to increase, as well as the cumulative risk of water shortages, floods and water quality degradation [4]. Accordingly, adaptation to climate change should consider measures of resilience [5], taking into account population spatial distribution and their access to urban infrastructure, public policies, among other factors [6]–[8].

To analyze the effects of intense rainfall and the performance of stormwater management facilities, hydrological models are used to estimate runoff distribution and intensity [8]–[10]. According to Hong [11], the importance of storing excess precipitation water in private flood holding reservoirs has been recognized in recent decades. These reservoirs can reduce the level or even eliminate the occurrence of floods. On-site stormwater detention (OSD) is a component of urban drainage systems [12] and a low impact development (LID) technology that helps reduce runoff and avoid overloading public stormwater infrastructure [13]. It can also contribute to the reduction of spatiotemporal variability in both local and river basin scales [2], and mitigate the effects of climate change in urban watersheds [14]. Furthermore, in water scarce contexts, these reservoirs can be used to harvest rainwater [15], [16].

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LID encompasses changes in the design and use of buildings and infrastructure to minimize impacts, being an important aspect of the sustainable management of stormwater [17], [18]. Considering the hydrological cycle, actions focus on individual lots and overall improvements are expected as a cumulative impact across the urbanized area. LID also seeks to keep impermeable surfaces to a minimum, and stormwater within the lot as long as possible [1].

The present study focuses on the application of stormwater tanks in the city of Curitiba, in southern Brazil. The city has a subtropical climate and most of its area has separate sewage and stormwater systems. Many low-income settlements are located in flood plains, making this part of the population particularly vulnerable to flooding. The city has been subjected to flooding since 1911, even before the consolidation of urban areas, and the situation has only worsened over the years. For this reason, legislation has existed since 1991 to mandate the construction of stormwater detention tanks on private properties to control runoff. These tanks decrease peak flows and complement macro drainage structures in the six main rivers that flow through the city. The legislation was revised in 2007, increasing the scope of buildings that can contribute to flood control in the municipality. This legislation establishes the minimum permeable area per lot, minimum detention tank volumes, and the diameter of the flow regulating orifice (FRO) but does not indicate the corresponding detention time, although it proposes a minimum of 20 minutes. Thus, a new revision is needed to promote ways to increase the efficiency of individual reservoirs on private lots to mitigate the effects of climate change. Limits for maximum specific outflow and detention times vary by municipality. Porto Alegre legislation [19] indicates a maximum specific outflow of 20.8 $L.s^{-1}.ha^{-1}$. In the city of São José do Rio Preto, the legislation [20] establishes a maximum outflow of 13 $L.s^{-1}.ha^{-1}$, while Curitiba's Drainage Master Plan [21] indicates 27 $L.s^{-1}.ha^{-1}$. No technical justifications are presented for these limits.

Most combinations of tank volumes and Flow Regulating Orifices (FRO) diameters established in Decree 176/2007 [22] are capable of holding peak floods for at least 20 minutes. Other municipalities recommend in their legislation detention times of one hour [23]–[26]. In other localities, systems that allow the infiltration of stormwater in the soil to reduce runoff and promote aquifer recharge are also prioritized [27]–[30].

The objective of the present study was to model peak flow reduction and detention times associated with prescribed tanks and FROs in order to propose changes to existing legislation and increase the efficiency of these devices.

2 METHODS

In order to model the efficiency of different tank sizes and FRO diameter combinations, tank emptying time and flood peak attenuation was calculated. The present methodology focuses on the tank volumes and FRO diameters established by Decree 176/2007 [22] and the local precipitation intensity but can be widely applied.

2.1 Tank emptying time

Unsteady flow analysis was performed to determine tank emptying times considering the volume ranges covered by the decree and above as they may be needed to promote lower peak discharges. Volumes analysed were between less than 2 m^3 and 20,000 m^3. Respective FRO diameters were based on commercially available PVC diameters, between 25 mm and 500 mm. Fig. 1 shows plan and section views of a typical detention tank. A vertical wall, the septum, separates the tank proper from an adjacent inspection chamber. These are connected at the bottom by the FRO, which is usually built with bricks or concrete blocks. Septum thickness generally varies between 12 and 19 cm, and an average 15.5 cm value is considered here. According to Finnemore and Franzini [31], if a wall has a thickness of less than 1.5 times the orifice diameter it can be considered thin, otherwise thick. Thus, for FRO diameters of up to 10 cm the septum is considered a thick wall, while above this FRO diameter the septum becomes a thin wall. This leads to a discharge coefficient, Cd, equal to 0.86 for tanks with FRO diameters equal to or less than 100 mm, and 0.61 otherwise [31].

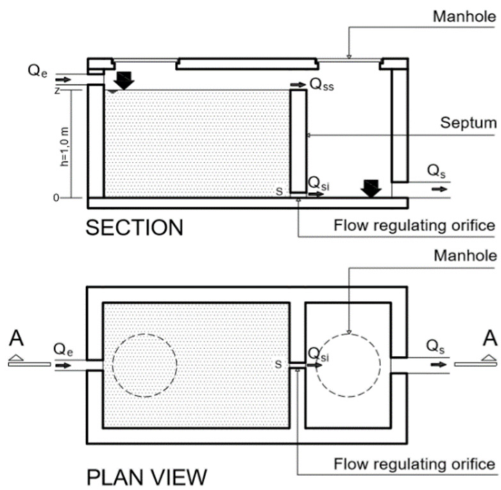

Figure 1: Plan and section views of a typical stormwater detention tank.

The discharge coefficient must be corrected for the position of the orifice (incomplete contraction weir) (eqn (1)). Here, a correction factor k = 0.5 was used since the orifice is usually installed at the bottom to allow full discharge, or in the middle or near one of the side tank walls [32], [33].

$$C_{d_{corrected}} = C_d.(1 + 0.13.k), \tag{1}$$

where:
C_d = Discharge coefficient with complete contraction;
Correction factors for incomplete contraction:
k = 0.25 orifice near the bottom or near a wall;
k = 0.5 orifice near the bottom and near one wall;
k = 0.5 orifice near the bottom and two walls.

After this correction, the discharge coefficient becomes 0.92 for FRO diameters up to 100 mm and 0.65 otherwise.

Septum height is usually limited by the invert elevation of existing stormwater infrastructure, in order to guarantee gravity flow, with pumping as a last resort. Motor-pump assemblies are vulnerable to power outages as they can cause disturbances such as underground flooding in parking garages, impairment of drinking water tanks and fire prevention equipment, among others. Such assemblies also require frequent maintenance. Internal tank height (septum + overflow space) should be sufficient to allow at least one person for sediment removal, FRO clearing and tank maintenance. The recommendation for closed tanks is at least 80 cm of internal total height, the same minimum measure indicated in Decree 176/2007 for dry access chamber for maintenance [22]. Septum heights of 1, 1.2, 1.5, 1.8 and 2 m were considered, as these are the most often applied in existing projects and usually lead to feasible designs.

Emptying times were calculated considering a full tank with no inflow. These were obtained from eqn (2) (adapted from [31]).

$$\Delta t = \frac{A_r}{S} \cdot \left(\frac{1+C_d}{2g}\right)^{1/2} \cdot \frac{h^{1/2}}{1/2}, \tag{2}$$

where:
Δt = tank emptying time (s);
A_r = tank surface area (m^2);
h = septum height or tank depth (m);
S = cross-sectional area of the flow regulating orifice (m^2);
g = acceleration of gravity 9.81 (m.s^{-2});
C_d = FRO discharge coefficient (0.92 and 0.65).

2.2 Flood peak attenuation

The analysis detention tank levels based on inflow applied eqns (3) to (8). The same variables represented in Fig. 1 were used. A septum height of 1.00 m was considered, since this is the most employed in projects, as it facilitates gravity flow to public drainage pipes, which, as a rule, are at a depth of 1.20 m.

Precipitation intensity is specific to each city and it his case was estimated using eqn (3) established by Parigot de Souza for the municipality of Curitiba [34], [35].

$$i = \frac{5950.T_R^{0,217}}{\left(t+26\right)^{1,15}}, \tag{3}$$

where:
i = maximum rainfall intensity (mm.h^{-1});
T = return period (years);
t = rain duration time (min).

Return periods of 2 and 10 years were considered, usual for microdrainage works [36], [37]. A rain duration of 10 minutes was analysed, which is the magnitude of the time of concentration in the region analyzed [38].

The rational method was applied to estimate the outflow hydrograph, dependent upon peak lot outflow and time of concentration (T_c).

Considering that (T_c) generally depends on drainage basin factors and this study focuses on the operation of detention tanks, several T_c equations were tested. These were Hataway, Kirpich Tenessee, Kirpich Pennsylvania, FAA [39], NRCS (velocity), Kinematic wave [40], Temez, Bransby-Williams [41], Dooge, SCS (lag time), Izzard, Kirpich corrected [41], Ventura, Giandotti, Picking, Ven te Chow, George Ribeiro, Schaake et al. [42], Arizona DOT [44], FAA (2006), Papadakis-Kazan [45], Williams, Johnstone-Cross, Simas-Hawkins and Haktanir [46], UFCD [27], MPCA [28].

According to McCuen et al. [39], the non-conformity of a basin parameter for which the time of concentration equation was obtained is not a reason to discard the equation. A more comprehensive assessment of the study site is required. Results obtained by Silveira [47] showed that some equations obtained for rural basins showed good results in urban settings, even without the application of correction coefficients. Some equations are indicated for small drainage basins, but there is no consensus as to the maximum size that defines a small basin [27], [36], [42], [48]. Still, some of the small basin equations have also presented good results for large basins.

It should be noted, however, that T_c equations were not obtained for areas as small as individual urban lots, as is the case in the present work. Thus, no existing T_c equation is adequate, a priori, for the present case, and results need to be analyzed before being used. The time of concentration is used in urban drainage mainly for design. Overestimates lead to undersized drainage works, and vice versa. The present work focuses on urban lots, their drainage being the responsibility of the owners, and an attempt should be made not to penalize private property owners to pay for oversized drainage elements. Thus, the minimum value of 6.0 minutes recommended by the USDA [40] was adopted as a limit for the time of concentration.

Results below the minimum were discarded as well as those that were more than one standard deviation above the average. By applying the minimum concentration time, peak reduction results are more conservative. Lower concentration times would lead to steeper hydrographs and higher peak reduction would be attributed to the reservoirs. Eqn (4) was adopted [44] since it resulted in values closer to the average of the remaining results.

$$T_c = 3,258.\left(\frac{L_c}{S_c}\right)^{0,5}, \tag{4}$$

where:
T_c = time of concentration (min);
L_c = talweg length (km);
S_c = slope (m.m^{-1}).

Tank peak inflow was calculated by eqn (5).

$$Q_e = C.i.A, \tag{5}$$

where:
C = runoff coefficient (Rational Method);
i = precipitation intensity (eqn (2)) (mm.h^{-1});
A = catchment area (m^2).

Outflow through the FRO was given by eqn (6).

$$Q_{si} = S.C_{d\ corrected}\sqrt{2 \times 9.81 \times h}, \tag{6}$$

where:
$C_{d\ corrected}$ = corrected discharge coefficient (eqn (1));
h = water level in the tank above the FRO (m).

Tank level variation (eqn (7)) over a time interval, Δt, was calculated by considering the difference between inflow and the outflow. When the bottom of the tank is permeable, the infiltration flow, Qi, must also be considered.

$$\Delta z = \frac{(Q_e - Q_{si} - Q_i).\Delta t}{A_r}. \tag{7}$$

When the tank level exceeds the height of the septum overflow occurs, and eqn (8) is used.

$$Q_s = Q_{si} + Q_{ss} + Q_i \,, \tag{8}$$

where:
Qs = outflow ($m^3.s^{-1}$);
Qsi = outflow through the FRO ($m^3.s^{-1}$);
Qss = overflow (over the septum) ($m^3.s^{-1}$);
Qi = infiltration in the soil ($m^3.s^{-1}$).

Using eqns (3) to (8), an outflow hydrograph can be obtained. Post-urbanization hydrographs, with and without detention tanks were compared to the pre-urbanization situation, considering the lot covered with vegetation.

Eqn (9) is currently employed to calculate tank volume [22]:

$$V = k.i.A \tag{9}$$

where:
V = minimum tank volume (m^3);
k = dimensionless constant = 0.2;
i = precipitation intensity = 80 $mm.h^{-1}$;
A = impervious area on the lot (m^2).

The dimensionless constant k corresponds to the expected minimum detention time (1/3 of an hour, according to the current project practice in the municipality) multiplied by the difference between the impermeability coefficients adopted: current (0.9) and pre-urbanization (0.3).

3 RESULTS AND DISCUSSION

A typical hydrograph obtained through the procedures summarized by eqns (7) and (8) is shown in Fig. 2. It shows a lot in pre-urbanization and post-urbanization conditions, with and without detention tanks. The dashed line represents the hydrograph before urbanization with 75% grass cover, 0 to 2% slope, 10 years return period precipitation, soil type C and runoff coefficient of 0.25 [42]. The dotted line represents the surface runoff for concrete and roof surfaces, 10 years return period precipitation, type C soil, and runoff coefficient of 0.83) [42]. The continuous line represents the surface runoff with the implementation of a detention tank [22].

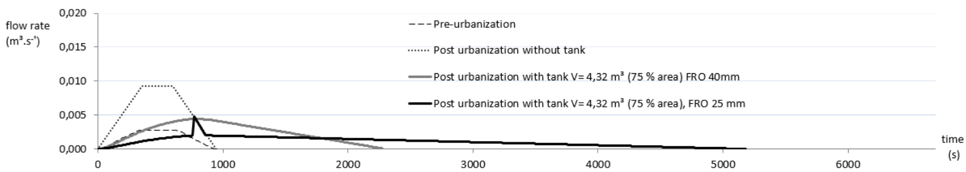

Figure 2: Hydrograph for precipitation with T_2 and t_{10}, FRO of 40 and 25 mm, and tank volume calculated on 75% of area.

The precipitation intensity of 80 $mm.h^{-1}$ considered in Decree 176/2007 [22] is below the T_2 to T_{10} values recommended by standards for urban drainage [36], [37]. The present study compared scenarios with t_2, t_{10} and T_{10}, calculated according to eqn (3). With T_2, and t_{10}, the precipitation intensity results in $i_{2,10}$ = 112.23 $mm.h^{-1}$, while T_{10} gives $i_{10.10}$ = 159.14 $mm.h^{-1}$. An increase of 20% in rainfall intensity was considered, as suggested by the

Table 1: Total detention times and average outflow as a function of tank volume and flow regulating orifice (FRO) diameters recommended here.

	FRO diameter (mm)	Tank volume (m^3)	Septum height (m). h = 1.00 Time (min.)	h = 1.00 Average outflow ($L.s^{-1}$)	h = 1.20 Time (min.)	h = 1.20 Average outflow ($L.s^{-1}$)	h = 1.50 Time (min.)	h = 1.50 Average outflow ($L.s^{-1}$)	h = 1.80 Time (min.)	h = 1.80 Average outflow ($L.s^{-1}$)	h = 2.00 Time (min.)	h = 2.00 Average outflow ($L.s^{-1}$)
C_d 0.92	25	≤ 3.0	≤ 64	0.79	≤ 58	0.86	≤ 52	0.96	≤ 47	1.05	≤ 45	1.11
	40	3.1 to 15.9	26 to 132	2.01	23 to 129	2.20	21 to 108	2.46	19 to 98	2.70	18 to 93	2.84
	40	7.24	60	2.01	55	2.20	49	2.46	45	2.70	42	2.84
	40	10.24	85	2.01	77	2.20	69	2.46	63	2.70	60	2.84
	50	16.0 to 35.9	85 to 190	3.14	77 to 174	3.44	69 to 155	3.85	63 to 142	4.22	60 to 135	4.44
	75	36.0 to 63.9	85 to 151	7.07	77 to 138	7.74	69 to 123	8.66	63 to 112	9.48	60 to 107	10.00
	100	64.0 to 155.9	86 to 207	12.57	78 to 189	13.77	69 to 169	15.39	63 to 154	16.86	60 to 146	17.77
C_d 0.65	150	156.0 to 275.9	85 to 151	30.47	78 to 138	33.38	70 to 123	37.32	64 to 112	40.88	60 to 107	43.09
	200	276.0 to 620.9	85 to 191	54.17	78 to 174	59.34	69 to 156	66.35	63 to 142	72.68	60 to 135	76.61
	300	621.0 to 1,104.9	85 to 151	121.89	78 to 138	133.52	69 to 123	149.28	63 to 113	163.53	60 to 107	172.37
	400	1,005.0 to 1,725.9	85 to 133	216.69	78 to 121	237.37	69 to 108	265.39	63 to 99	290.72	60 to 94	306.44
	500	≥ 1,726	≥. 85	338.57	≥. 78	370.89	≥. 69	414.67	≥. 63	454.25	≥. 60	478.82

Intergovernmental Panel on Climate Change [49] for the Curitiba region (Magrin et al., 2007; Marengo, 2008), resulting $i_{2,10}$ + 20% = 134.67 mm.h^{-1} and $i_{10,10}$ + 20% = 190.96 mm.h^{-1}.

For precipitation intensity 112.23 mm.h^{-1}, increasing the reservoir volume increases the detention time by 12%, while changing the FROs more than doubles the detention time, even with some overflow (FRO 25 mm). Thus, combining reservoir volumes with the appropriate FROs leads to better results than simply increasing reservoir volumes. The higher the volume of the tank, the lower the probability of septum overflow. Peak flow control is most efficient when there is no overflow. However, by simply increasing FRO diameter to avoid septum overflow, detention time is reduced, and rainfall is released to the receiving water body sooner, which can reduce flood control efficiency. Thus, considering the inherent septum height limitations, the best condition is achieved by adequate combination between tank volume and FRO diameter, in order to optimize flood peak abatement.

Table 1 presents combinations of tank volumes, FRO diameters, and septum heights recommended here. A minimum emptying time of one hour is achieved for all combinations, except for a 25 mm FRO combined with septa higher than 1.00 m. FRO diameters below 25 mm can be easily clogged and compromise tank operation, since water could remain inside the tank, lacking space to accommodate the next rainfall. This would also lead to high detention times, conducive to the development of aquatic organisms such as mosquito larvae. Thus, 25 mm FROs require more frequent maintenance, and are not generally recommended.

Table 2 shows detention tank volumes and the corresponding FRO diameters in accordance with Decree 176/2007 and the proposed changes. These tank size – FRO diameter combinations ensure flood peak reductions for all cases studied.

Table 2: Suggested tank volume and FRO diameter combinations.

Tank size (m^3) Decree 176/2007	Tank size (m^3) Proposed	FRO (mm)
≤ 2	≤ 3	25
3 to 6	3.1 to 15.9	40
7 to 26	16.0 to 35.9	50
27 to 60	36.0 to 64.9	75
61 to 134	65.0 to 155.9	100
135 to 355	156.0 to 275.9	150
356 to 405	276.0 to 620.9	200
406 to 800	621.0 to 1.104.9	300
801 to 1,300	1,105.0 to 1,725.9	400
1,301 to 2,000	≥ 1,726.0	500

4 CONCLUSIONS

Notwithstanding the fact that Decree 176/2007 was supposedly conceived considering a rainfall intensity of 80 mm.h^{-1} and a 20 minute detention time, results obtained here show that tank volume and FRO diameter combinations prescribed by this decree are effective for higher rainfall intensities and produce longer detention times. The only exception is the tank volume range for 25 mm FROs.

Stormwater detention tanks smaller than 3.0 m^3 are employed in small lots, usually in low-income housing developments [50]. The combined effect of several such tanks is not negligible [35], but they require careful maintenance, as 25 mm FROs are easily obstructed. Thus, the use of other LID technologies such as permeable pavement may increase flood control efficiency. Achieving the total detention design volume through the combination of smaller tanks is another valid option. It might facilitate runoff collection due to lot topography and reduced costs. Partitioning the volume may also help to avoid very long detention times, which could lead to the proliferation of disease vectors. The results presented herein indicate that stormwater detention tanks can be used to reduce the effects of urbanization in the hydrological cycle, since they abate flood peaks and also lengthen the base of the hydrograph. However, stormwater detention tanks, even if they promote the abatement of flood peaks, cannot be considered the only solution to control runoff. It is possible, and necessary, to combine their use with that of other LID technologies that promote evapotranspiration and infiltration of stormwater into the ground to recharge aquifers. Green corridors (linear parks), containment basins, rooftop detention systems (green roofs, blue roofs), and others, are among such technologies that, in addition to flood control, contribute to pollutant removal and water quality improvement in urban water bodies.

The method proposed and demonstrated herein to adjust flood control legislation in the city of Curitiba can be used to perform similar analysis elsewhere. In such cases, rainfall intensities would assume different values, and local soil conditions would have to be considered to account for different infiltration rates. With those changes, the overall method remains the same, and unsteady flow analysis is employed to determine detention times and the corresponding hydrographs.

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