



Impact of Installing Rainwater Harvesting System on Urban Water Management

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Abstract

The urban-scale impacts of the use of Rainwater Harvesting Systems (RWHS) are little explored within the Brazilian urban and climatic reality. Thus, the objective of this work is to verify the impacts of these systems in reducing runoff and the demand for drinking water, through a case study carried out in a neighborhood in the city of Goiânia-GO. In order to carry out the study, the land use characteristics of the studied region were studied from satellite images, with three batch patterns being defined as study scenarios. The daily residential demand for non-drinking uses of water was calculated from the standard lots and the annual rainfall pattern in the city of Goiânia, which was also characterized by taking a 30-year historical series of daily rainfall data. With these data in hand, daily water balances were calculated using Excel, for commercial reservoirs of 1, 5 and 10 m³, in addition to reservoirs with ideal volumes obtained from the NETUNO software. Two balance models were tested and compared and, from them, analyzes were made of the potential for reducing the annual consumption of drinking water in the neighborhood and hydrological simulations were run to verify the flow damping, with the support of the SWMM program. In general, the RWHS use has positive aspects: it reduces the drinking water demand in the neighborhood (up to 37%). However, when analyzing the reduction of peak flows in the drainage system, it is only significant in scenarios with larger reservoirs, confirming its marginal role in stormwater management.

Keywords Rainwater harvesting systems · Water conservation · Low impact development · Alternative water supply

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1 Introduction

The use of rainwater harvesting systems (RWHS) is widely described in the literature as a complementary alternative water supply that satisfactorily meets the demands for non-potable water. However, studies in the field of drainage are still incipient (Farooq et al. 2022; Snir and Friedler 2021; Campisano and Lupia 2017; Kim et al. 2012). The works that exist on the subject point out the modest contribution of RWHS in reducing runoff (Hu et al. 2017; Jenkins et al. 2012; Pauleit and Duhme 2000; Herrmann and Schmida 2000). They also established that the efficiency of this system, in both supply and drainage terms, is linked to local characteristics, such as land use and cover, climate, reservoir capacity and demand (Snir and Friedler 2021; Fernandes et al. 2020; Jing et al. 2018; Jing et al. 2017; Rostad et al. 2016; Steffen et al. 2013; Walsh et al. 2014).

In most studies, the water behavior and efficiency of RWHS are described by means of continuous simulations based on mass balance theory (Jing et al. 2017). Continuous water models, which describe the behavior of drainage systems, have been used to assess the impact of using RWHS to reduce runoff. These models allow more complex analyzes that take into account the variation of rainfall events (Dornelles 2012; Herrmann and Schmida 2000), with the Storm Water Management Model (SWMM) being one of the most used (York et al. 2015).

This scenario highlights the need to discuss the adequacy of the drainage and urban supply in the city within the public sphere, in order to seek solutions and alternatives that are consistent with the resumption of the pre-urbanized hydrological conditions.

Thus, the objective of this work is to evaluate the impacts of different scenarios of the adoption of RWHS in controlling runoff and reducing the demand for drinking water in a residential neighborhood in the city of Goiânia-GO.

2 Methodology

2.1 Choice of Study Area

A residential neighborhood in the city of Goiânia-GO was initially chosen to assess the impacts of using RWHS in damping runoff flow and reducing drinking water consumption. This choice of area was guided by three criteria: 1) it should consist predominantly of single-family dwellings; 2) it should be a neighborhood that had already been consolidated, with built subdivisions, street infrastructure and public areas; and 3) the buildings should be relatively homogeneous, so that the characteristics of the lots could be more easily replicated. Thus, because it contains all of these characteristics, the Parque Atheneu housing complex was chosen (Fig. 1).

Conceived in the 1980s, this housing development is located in the southeastern region of Goiânia. The development makes up a significant portion of the sub-basin area of the São José Stream, which is part of the Meia Ponte River basin.

The sector is divided into ten Urban Units, permeated by collecting routes and an urban ring, the Parque Atheneu Avenue (SEPLAN 1980). The total area of the land is 1.86 km² and the estimated population is 14,068 inhabitants, with an income between R\$ 2,300.00 and R\$ 3,000.00 (Instituto Brasileiro de Geografia e Estatística 2017a; SEPLAN 2015).

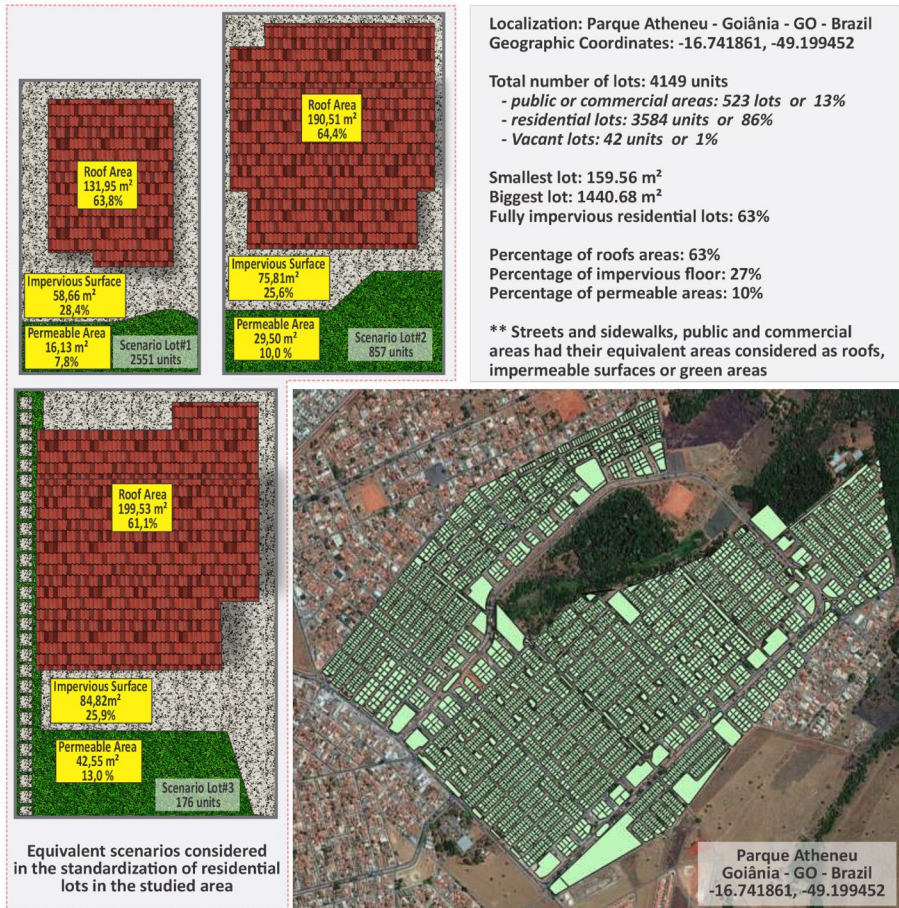


Fig. 1 Location and characterization of the Parque Atheneu neighborhood in the metropolitan region of Goiânia and remote sensing image overlaid with vector files representing the lots in the neighborhood

2.2 Characterization of Lots and Land Cover

To calculate the RWHS' collection, storage, utilization and drainage capacity in the study area, it was necessary to characterize the coverage and occupation pattern of the built lots, identifying the green areas and the waterproofed areas. To this end, the following procedures were followed to delimit the land's occupation profile:

Using the computer program QGIS 2.18 (2017), images from Google's remote sensing images and vector files of the Geographic Information System of Goiânia (SIGGO) were superimposed (SEPLAN 2015).

Public areas were divided into two groups: an exclusive layer of public green areas was created to represent squares, flowerbeds, parks, etc., with other layers representing public buildings, impermeable areas and built areas. The remaining areas were considered to be green areas.

The vacant lots started to constitute a layer of permeable grass and the parking lots started to constitute a layer of impermeable concrete.

The areas assigned to commercial activities were identified using Google Earth Pro (GOOGLE 2018) and converted into allotments with a roof (roof).

For residential subdivisions, two layers were created: one covering the roof and the other a permeable grass area.

The value of the impermeable external areas of the residential lots was obtained by subtracting the roof and grass areas from the total area of the lot.

A layer of grass was also created by drawing polygons on the green spots outside the lots.

Finally, the subtraction of the total area of the other layers from the total area of the neighborhood resulted in the area value of the paved streets.

To define a pattern of residential lots, the related vector layers were extracted to an Excel spreadsheet in CSV format, in order to perform statistical analyzes. In this spreadsheet, the mode, median, mean, variance and standard deviation of the residential lot, garden, roof and waterproofed patio areas were calculated, as well as the total area of these elements in the neighborhood. Accumulated percentage distribution plots of the areas of these elements were also constructed. Then, the values of the areas of the lower and upper limits of the standard deviation interval for roofs, gardens and waterproofed patios were calculated. The values of the elements in the upper, lower and average limits were added to obtain the lot area in these limits and the averages.

Depending on the pattern of distribution of the areas of the elements, three kinds of lots were defined, in order to preserve the sum of the total areas of the elements in the neighborhoods as much as possible whilst, at the same time, model the variation in the size of the lots in the neighborhood more accurately. This would not be possible within the limitations of an average lot.

The lots that had total areas between the lower limit and the total average formed the group of lots categorized as L1. The area pattern was obtained from the average of the roof, garden and impermeable patio areas of the lots in the interval. The same reasoning was applied to the L2 and L3 standards, in the intervals between the total average and the upper limit, and in the interval between the upper limit and the largest lot, respectively.

2.3 Modeling of the Rainfall Pattern

The modeling of the rainfall pattern considered the volume of rainwater that entered the system and that could be used. This volume was obtained from daily rainfall data over a historical series of 30 years (between 01/01/1988 and 12/31/2018), measured at a rain station of the National Meteorological Institute (INMET 2018) located in the city of Goiânia.

The average annual, monthly and daily rainfall was calculated by using electronic spreadsheets. For the days of the historical series that did not present any rainfall data, the average daily rainfall value in the relevant month was adopted.

The average annual precipitation for the city of Goiânia was 1617.31 mm. It is possible to observe the seasonal behavior of the rains in the region, with two distinct seasons: the rainy season from October to April and the dry season from May to September.

2.4 Definition of Rainwater Demand

Rainwater demand was computed, with reference to Campos and Pacheco (2016) and NBR 15527 (ABNT 2019). The following uses were considered: toilet flushing, floor cleaning and garden irrigation. The number of inhabitants was defined based on population data by neighborhood, obtained from IBGE (2017b).

For the definition of the per capita consumption profile, it was considered that each resident uses the toilet six times a day and that the volume consumed in each flush is 6.8 L. For garden irrigation, 2.4 L were considered per m^2 at a frequency of three times a week (Mondays, Wednesdays and Fridays). The yard is assumed to be cleaned every day, except on Sundays. On weekdays, a damp cloth is used, with a consumption of 0.5 L / m^2 and on Saturdays the cleaning is done with a squeegee and water, consuming 1 L / m^2 (Campos and Pacheco 2016). The water demand for irrigation and washing the patio was neglected in the rainy months, as it was assumed that this consumption only occurred in the months when the average monthly rainfall was less than 200 mm.

The demand for rainwater in the standard lots is shown in Table 1. In the rainy season, the rainwater consumption was the same for the standard lots, since the demand is solely due to the flushing of the sanitary basins. It was in the dry months that the demand varied for the different lots, since the garden and outdoor patio areas varied.

In total there was a percentage increase in relation to the demand for L1 of 6% and 10%, considering the demands of L2 and L3, respectively. The percentage increase in annual demand between L2 and L3 was 4%.

Considering only the dry months, the percentage variation in relation to the demand for L1 was 9% and 37%, considering L2 and L3, respectively. The percentage increase in demand in the dry months between Lots 2 and 3 was 7%.

Relating the demand to the rainfall pattern verified one of the challenges in meeting demand. In the months when there is greater demand, there is less rainwater available to supply it.

2.5 Water Balance of RWHS

From the daily rainfall and demand data and the sizes of the standard lots, an electronic spreadsheet was compiled of the mass balance for reservoirs with different volumes.

It was decided that the water balance would be conducted using commercial reservoirs of 1, 5 and 10 m^3 . The balances of the reservoirs with the ideal volumes indicated by the Netuno 4 software (Ghisi and Cordova 2014) were studied for the three standards lots. This is based on behavioral models in which the variables are known, to simulate the mass balance of rainwater harvesting systems.

The ideal reservoir volume would be defined as the one which would generate a difference in water-saving potential less than or equal to 1%, with simulations being performed every 0.5 m^3 for reservoirs between 0.5 m^3 and 100.0 m^3 . The input values were the same as those used to study the water balance and the runoff coefficient adopted for the simulations was 0.8. The initial disposal was 2 mm. An upper reservoir of 0.5 m^3 was used, in which the system would be repressed whenever the upper reservoir had half its volume consumed.

Two balance models were tested: a more simplified one, in which the volume of the upper reservoir is added to the volume of the lower reservoir, and a second one, in which

Table 1 Estimated daily rainwater demand for Lots analyzed

	Months	Weekdays	Toilet flushing (L/person)	External cleaning (L/person)	floor	Watering garden (L/person)	Total (L/person)
ALL LOTS	November to March	Sunday	40.80	0.00		0.00	40.80
		Monday					
		Tuesday					
		Wednesday					
		Thursday					
		Friday					
		Saturday					
L1	April to October	Sunday	0.00	0.00	40.80		
		Monday	5.87	7.74	54.41		
		Tuesday	5.87	0.00	46.67		
		Wednesday	5.87	7.74	54.41		
		Thursday	5.87	0.00	46.67		
		Friday	5.87	7.74	54.41		
		Saturday	11.73	0.00	52.53		
L2	April to October	Sunday	0.00	0.00	40.80		
		Monday	7.58	14.16	62.54		
		Tuesday	7.58	0.00	48.38		
		Wednesday	7.58	14.16	62.54		
		Thursday	7.58	0.00	48.38		
		Friday	7.58	14.16	62.54		
		Saturday	15.16	0.00	55.96		
L3	April to October	Sunday	0.00	0.00	40.80		
		Monday	8.48	20.42	69.71		
		Tuesday	8.48	0.00	49.28		
		Wednesday	8.48	20.42	69.71		
		Thursday	8.48	0.00	49.28		
		Friday	8.48	20.42	69.71		
		Saturday	16.96	0.00	57.76		

the settlement intervals are computed in the system. In both models, precipitation occurs first. The volume of rainwater captured by the system is given by Eq. (1):

$$V_{cap} = (P - DI) \times A \times Rf \tag{1}$$

where

Vcap volume of harvested rainwater (L);

P	rainfall index (mm / m ²);
DI	first flush (mm / m ²);
A	catchment area (m ²);
Rf	runoff coefficient.

The volume that is actually stored in the reservoir corresponds to the waiting volume (Eq. (2)) of the reservoir if it is less than the capped volume, or equal to the capped volume if the waiting volume is higher.

$$V_{esp} = V_{res} - V_{fim} \quad (2)$$

where

V_{esp}	waiting volume (L);
V_{res}	reservoir volume (L);
V_{fim}	water volume in the reservoir at the end of the previous day (L).

In this way, before the rainwater consumption occurs, the volume of water in the reservoir (Eq. (3)) is equal to the sum of the volume of water in the reservoir the previous day (V_{fim}) plus the volume of water stored in the system after precipitation.

$$V_{in} = V_{fim} - V_{arm} \quad (3)$$

where

V_{in}	water volume in the reservoir after precipitation (L)
V_{arm}	water volume stored after precipitation (L)

After the precipitation and storage of water in the lower reservoir, consumption occurs. The volume consumed is equal to the demand if V_{in} is greater than the demand, or equal to V_{in} if the demand is greater than the initial volume. So, the final volume in the reservoir is given by Eq. (4).

$$V_{fim} = V_{in} - V_{cons} \quad (4)$$

where

V_{cons} volume of water consumed (L)

In the second model, the calculations of the captured, waiting, initial and final volumes in the lower reservoir follow the same procedure. The change is in the consumption time step. Consumption in the lower reservoir occurs whenever the upper demand is for pumping.

From the simulations, the average daily, monthly and annual values of the potential for saving drinking water, the volume of rainwater consumed, the volumes of water waiting, stored, discarded, disposed of in the network, capitulated and overflowed, and the percentage of demand for rainwater (completely, partially and unanswered), are all generated. Taking these results as a basis, it was also possible to verify the permanence in time of the different waiting volumes and to estimate the annual reduction in drinking water consumption in the neighborhood.

The reduction in drinking water consumption in the neighborhood was estimated for four scales of installation of RWHS, by considering their adoption in 25%, 50%, 75% and 100% of dwellings. The annual volumes of rainwater consumed by the households were used as a reference for the four different reservoir patterns. These volumes were deducted from the annual volume of drinking water consumed in the neighborhood.

The volume of drinking water consumed in the neighborhood was estimated by multiplying the total population by the average per capita water consumption in the city of Goiânia in 2017. This value was obtained from the National Sanitation Information System (BRASIL 2019).

2.6 Urban Hydrological Model

The verification of the impacts of RWHS on the drainage network was carried out using hydrological models designed in the Storm Water Management Model computer program (EPA 2018). This software, based on water balances, allows the forecasting of the volume of rainfall and peak flow in urban areas from specific rainy events or within a time interval. Since the rainfall data obtained were daily, it was decided to evaluate the behavior of the basin submitted to a design rain with a return time of 25 years.

The soil parameters required for the simulations (hydraulic conductivity, capillary suction, porosity, field moisture content and saturation moisture content) were adopted from the work of Carvalho (2013). The Manning coefficients adopted for each type of surface were obtained from Rossman (2010).

Using the SWG files obtained from SIGGO, the neighborhood was divided into 19 sub-basins in the AutoCAD program (AUTODESK 2018). For this, the limits of the Urban Units and the main roads were respected. In this way, it was possible to describe the coverage that generates flow in more detail, without generating an excessively large volume of data to be entered into the program. Two sub-basins were also generated, to represent the central area of the neighborhood where the stream that receives all the rainwater flow is located.

In QGIS, all of the layers generated and described in the ‘Characterization of lots and soil cover’ step were related to their respective sub-basin. Flow lines were also drawn on the contour lines of the neighborhood, in order to define the direction of rainwater flow from one basin to another. From the flow lines and contour lines, the thalweg lines of each sub-basin were drawn in AutoCAD and the approximate slopes were calculated. The slope was obtained by randomly choosing at least three sections of the flow lines between two contour lines of each sub-basin. The mean slope for these stretches was adopted.

The sub-basins were designed and characterized in the SWMM. All flows were directed to a single drainage discharge at the end of sub-basin 2, which received the sum of the flows from the others. The design rain was calculated from the rain equation for the city of Goiânia (Eq. (5)), for a return time of 25 years and a concentration time of one hour, corresponding to the times used for dimensioning manholes in basins between 1 km² and 5 km². Then, the design hyetogram was drawn from the Alternating Blocks Method (Costa et al. 2007).

$$i = (64.3044 \times T^{0.1417}) / (t + 24.8)^{0.974711} \quad (5)$$

where

- i maximum rain intensity (mm / min)
- T period of return (years)
- t duration of precipitation (min)

Five different urban deployment scales were considered. In the first scenario, no system was implemented, in the others, the installation of different reservoir volumes in 25%, 50%, 75% and 100% of residences was considered, making a total of 17 scenarios. For each of

the 17 scenarios, the design project hyetograms were changed according to the contribution of each sub-basin to decreasing the intensity of precipitation. For this, the total waiting volumes per sub-basin were calculated in each scenario, adopting the volume of residential waiting that had 90% of permanence. Then, the total waiting volumes were divided by the areas of their respective sub-basins, resulting in the values of retained rainfall. The retained precipitation values were, in turn, subtracted from the discretized precipitation values used in the construction of the project hyetograms. From the new precipitation values, the intensities in the new hyetograms were calculated. Since the design rain had its intensity discretized in six blocks of 10 min, the results returned by the SWMM program were also discretized for 10 min time intervals.

3 Results and Analysis

The results obtained from the characterization of the neighborhood, from the RWHS simulations and from the urban water model, are presented and analyzed in three main sections.

3.1 Characterization of Lots and Land Cover

From the layers built in the QGIS, it was found that residential lots accounted for most of the land use in the neighborhood (46%) and the second largest area was covered by streets (30%).

80% of the neighborhood was covered by impermeable surfaces (concrete 14%, asphalt 30% and ceramic tiles 36%). It should be noted that the roof areas of the residences, which can act as rainwater catchment areas, corresponded to 29% of the neighborhood's coverage.

As for the subdivision, the neighborhood had a total of 4,149 lots, of which only 1% were vacant, with no apparent use, and 86% were exclusively residential.

3.2 Performance of RWHS

The ideal storage volumes defined by the NETUNO program were 30.5 m³, 28.5 m³ and 29.5 m³ for L1, L2 and L3, respectively. The Supplementary Material presents the average behavior of the RWHS for the two tested water balance models.

In general, the model that considers the elevated tank separately is more conservative, both in the use of rainwater and in the waiting volume, since the simplified model adds the entire volume of the elevated tank to the harvesting volume, while in the method with a lift system, the increase in the harvesting volume is variable and depends on whether lifting occurs or not. Thus, in order to deepen the analysis and apply the simulations in the SWMM program, it was decided to work with the most conservative model.

Once the definition was made, the system analysis started. The volume of rainwater caught followed the annual precipitation curve, and so July and December had the lowest and the highest average daily catchment values. The average annual catchment volume varied in the same proportion as the percentage variation of the roofs of the residences in the standard lots. In this way, L2 and L3 caught volumes 44% and 51% higher than L1, respectively, and L3 caught a volume 5% greater than L2.

The volume of rainwater that was actually stored was strongly influenced by the volume of the tank and demand, since these variables have an impact on the waiting volume. In

the most severe drought months (June to September), for tanks with volumes equal to or greater than 5 m^3 , the average daily volume stored reached values equal to the values of the average daily volume captured. This indicated that the reservoirs are, in those months, capable of absorbing all the rain captured on the roofs, which results in zero overflowing values. In the rainy season, it was observed that the average volumes stored were very close to the average volumes consumed, indicating that the waiting volumes in this period resulted from daily consumption. In the Supplementary Material, we present the behavior of the tanks for the three lots.

In terms of consumption and the potential to meet demand, we observed that the seasonality of rainy seasons has a strong influence, followed by the size of the tank and, to a lesser extent, by the greater demand in the dry season. Table 2 presents the behavior of tanks with ideal volumes for L1.

The total volume of drinking water consumption estimated for the neighborhood was $777,309.05 \text{ m}^3 / \text{year}$. The estimated values of the potential savings in drinking water consumption in the neighborhood for the different rates of RWHS installation are presented in Table 3.

The results indicate that the use of RWHS as a supply source for non-drinking uses in dwellings has a great potential to reduce the demand on the public drinking water supply system in Goiânia. Under the assumption that tanks capable of supplying all the demand for non-potable water could be installed in all residences, throughout the drought period, the potential for reducing drinking water consumption in the neighborhood would reach 46%. On the other hand, considering the ideal volume tank (30 m^3) installed in 100% of the neighborhood houses, this would result in a 37.6% reduction in drinking water consumption.

It is worth noting the existence of a more evident scaling effect in larger volume tanks; that is, the larger the tanks, the greater the influence of the installation scale in reducing drinking water consumption. This is demonstrated by the greater differences between the values that describe the relationship between the scale of installation and the reduction in the consumption of drinking water for tanks of larger volumes.

The volume of precipitation that can be retained by the sub-basin for each of the 17 installation scenarios, taking the waiting volumes with 90% permanence as a starting point, is shown in Table 4.

Taking sub-basin 10 as an example, which can retain the largest volume among the others, the dependence of the volume retained in relation to the installation rates of the systems in the neighborhood is illustrated in Fig. 2a. For all volumes, there is a linear relationship between the increase in retention capacity and the installation rate in the neighborhood. However, the greater the tank volume, the more the increase in the installation rate was noticed to be higher, which is evidenced by greater inclinations in the curves of the tanks.

The volume of precipitation in one hour, for a design rain with 25 years of return time, was 80 mm. From this precipitation, the hyetogram of the scenario without installation was built (Fig. 2b).

From the hyetogram and the reductions in the calculated precipitation volumes, the project hyetograms by sub-basin were constructed. Since the buffer effect of the tanks is related to the rainwater that is captured at the beginning of the precipitation, the precipitated volume was reduced starting from the first block of the base hyetogram and, when necessary, advancing on the others. By applying the different hyetograms to the urban hydrological model designed in the SWMM, it was possible to analyze the 17 scenarios from the hydrographs of the outlet flow in the drainage discharge

Table 2 Average flow behavior in the ideal volume tank for the LI

Month	Rate x saving (%)	Mean volume of rainwater consumed (L/day)	Mean waiting volume (L/day)	Mean volume caught (L/day)	Mean volume stored (L/day)	Mean volume overflowed (L/day)	V stored/V caught (%)
1	100.0%	204.00	1887.14	715.08	247.69	467.39	34.6%
2	100.0%	204.00	885.11	752.04	233.85	518.20	31.1%
3	100.0%	204.00	638.91	774.17	198.03	576.14	25.6%
4	100.0%	251.08	1912.29	402.73	149.92	252.81	37.2%
5	100.0%	248.62	6244.66	83.58	75.39	8.20	90.2%
6	100.0%	250.11	12130.59	28.48	28.48	0.00	100.0%
7	100.0%	249.57	19483.82	5.67	5.67	0.00	100.0%
8	84.2%	211.10	26462.77	31.42	31.42	0.00	100.0%
9	53.9%	134.82	29043.03	132.01	132.01	0.00	100.0%
10	82.3%	205.31	26183.26	428.07	427.84	0.23	99.9%
11	100.0%	204.00	16629.11	670.13	622.39	47.74	92.9%
12	100.0%	204.00	4748.81	820.28	454.82	365.45	55.4%
Mean	92.8%	214.34	12242.28	402.07	216.99	185.07	54.0%
Annual mean		80898.90		151753.02	81900.34	69852.67	

Table 3 Potable water savings potential in the neighborhood with respect to installation rate

Rainwater tank (m ³)	Annual volume of rainwater consumed in the neighborhood (m ³)	Installation rate in the neighborhood			
		25%	50%	75%	100%
1	157488.5	5.1%	10.1%	15.2%	20.3%
5	199801.8	6.4%	12.9%	19.3%	25.7%
10	218892.8	7.0%	14.1%	21.1%	28.2%
30	292242.4	9.4%	18.8%	28.2%	37.6%

In general, the impact on decreasing the peak flow did not reach 1%, when commercial reservoirs were used but, for the scenario in which 100% of households adopted reservoirs with ideal volumes, the reduction in peak flow reached 16.28% (Table 5).

The slopes of the curves (Fig. 3) show that the greater the tank volume, the greater the influence of the installation rate on the reduction of peak flow.

It was also observed that flow reduction occurred in the ascending part of the hydrograph (Fig. 4) providing, in some cases, a delay of up to 20 min at the beginning of the runoff (Table 5). The delay at the beginning of the runoff coincides with the advance of the reductions in the intensity of the rain for the middle blocks of the hyetograms. Thus, a reduction in the intensity of rainfall in the most intense precipitation interval only began discreetly in the most favorable storage scenario (tanks with ideal volumes implemented in 100% of the houses); it was expected that the peak flow would remain in the same interval (1h).

It is important to note that the reductions in peak flow in the simulations conducted are only valid for the considerations made in this study; they may also suffer different influences that can both increase and decrease their impact.

Flooding episodes in Goiânia are related to very intense isolated rainfall events in drought months and to rain events that occur in rainy months, when the city soil is already saturated due to continuous rainfall in the preceding days (Rego and Barros 2014). Starting from the first hypothesis, there is a probability that the impact of reducing the peak flow may be greater, since the waiting volumes in the dry season tend to be higher than those adopted. Considering the second hypothesis, for rains of the same intensity, there is a likelihood that the impact of a reduction in peak flow may be less, since the waiting volumes in the rainy season tend to be low.

The design rainfall adopted for this work was also a conservative consideration since, for the 30 years studied, on the days when there was precipitation, the daily average volume value precipitated was 18.26 mm. This shows that the rain in question is an extreme occurrence and most precipitation events in the city have lower intensities and precipitated volumes than those adopted. Knowing this, it is expected that, in general, the impact will be greater than is simulated here because, for more intense rainfall, the impact of using RWHS tends to be less (Walsh et al. 2014).

It should also be considered that the results obtained are for an urbanized and, predominantly, residential neighborhood, so the scenario in question is favorable regarding the proportion of the catchment area by the total area of the neighborhood. On the other hand, the highly impermeable ground surfaces in a neighborhood has an impact on the amount of runoff generated, decreasing the power of interference on the roofs of residences.

Table 4 Rain volume retained by each sub-basin for each scenario (mm)

Tank	1m ³				5m ³				10m ³				Ideal volume			
	25	50	75	100	25	50	75	100	25	50	75	100	25	50	75%	100
Rate of installation	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%
SB1	0.2	0.3	0.5	0.7	0.5	1.1	1.6	2.2	1.4	2.8	4.2	5.7	3.4	6.8	10.3	13.4
SB2	0.3	0.6	0.8	1.1	0.9	1.8	2.8	3.7	2.4	4.8	7.2	9.6	5.8	11.7	17.5	22.9
SB3	0.2	0.4	0.7	0.9	0.7	1.4	2.1	2.8	1.9	3.7	5.6	7.4	4.5	9.0	13.5	17.7
SB4	0.2	0.4	0.6	0.8	0.7	1.3	2.0	2.7	1.7	3.5	5.2	7.0	4.3	8.6	13.0	14.4
SB5	0.3	0.6	0.9	1.2	1.0	1.9	2.9	3.9	2.5	5.1	7.6	10.1	6.1	12.2	18.3	24.4
SB6	0.2	0.4	0.6	0.9	0.7	1.4	2.1	2.8	1.8	3.7	5.5	7.3	4.4	8.9	13.3	17.1
SB7	0.3	0.6	0.9	1.2	1.0	2.0	3.0	4.0	2.6	5.2	7.8	10.4	6.3	12.6	18.9	24.4
SB8	0.2	0.4	0.6	0.8	0.6	1.2	1.9	2.5	1.6	3.3	4.9	6.6	3.9	7.9	11.8	15.8
SB9	0.2	0.5	0.7	1.0	0.8	1.6	2.4	3.2	2.1	4.2	6.2	8.3	5.1	10.1	15.2	19.5
SB10	0.3	0.7	1.0	1.4	1.1	2.2	3.3	4.4	2.9	5.8	8.7	11.6	7.0	14.0	21.1	28.1
SB11	0.2	0.4	0.7	0.9	0.7	1.4	2.1	2.8	1.8	3.6	5.4	7.1	4.5	9.0	13.6	15.9
SB12	0.2	0.3	0.5	0.7	0.5	1.1	1.6	2.2	1.3	2.6	3.9	5.3	3.5	6.9	10.4	12.7
SB13	0.2	0.4	0.6	0.7	0.6	1.2	1.8	2.4	1.5	2.9	4.4	5.9	3.8	7.6	11.4	14.6
SB14	0.3	0.5	0.8	1.0	0.8	1.7	2.5	3.3	2.1	4.3	6.4	8.5	5.3	10.6	16.0	21.3
SB15	0.2	0.4	0.7	0.9	0.7	1.4	2.2	2.9	1.8	3.7	5.5	7.3	4.6	9.2	13.9	16.3
SB16	0.2	0.5	0.7	0.9	0.7	1.4	2.2	2.9	1.8	3.7	5.5	7.4	4.6	9.2	13.8	18.0
SB17	0.2	0.3	0.5	0.7	0.5	1.1	1.6	2.2	1.4	2.7	4.1	5.5	3.5	7.1	10.6	12.2
SB18	0.2	0.4	0.6	0.8	0.7	1.3	2.0	2.6	1.6	3.3	4.9	6.5	4.2	8.5	12.7	15.4
SB19	0.3	0.5	0.8	1.0	0.8	1.7	2.5	3.4	2.2	4.3	6.5	8.6	5.3	10.7	16.0	20.5

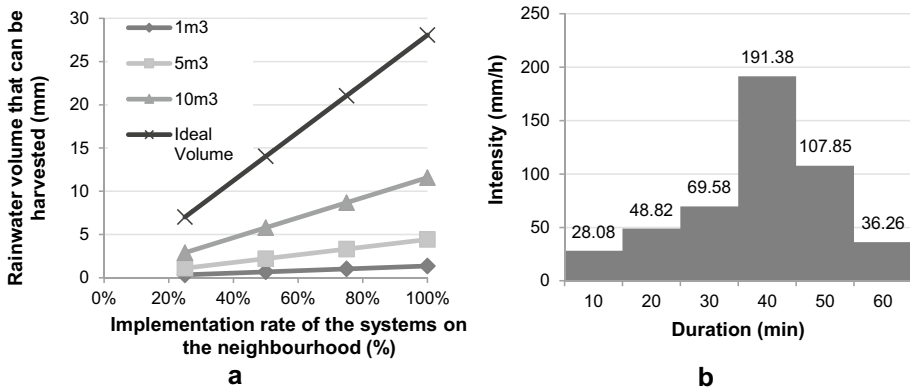


Fig. 2 **a** Precipitated volume that can be harvested in relation to the installation rate of the systems. **b** Base project hyetogram

Table 5 General behavior of outflow flows

Scenarios	Peak flow at the Drainage discharge(m ³ /s)	Decrease of the peak flow (%)	Flow start moment	
Without RWHS	52.65	-	0:20:00	
1m³	25%	52.64	0.02%	0:20:00
	50%	52.62	0.06%	0:20:00
	75%	52.6	0.09%	0:20:00
	100%	52.59	0.11%	0:20:00
5m³	25%	52.6	0.09%	0:20:00
	50%	52.54	0.21%	0:20:00
	75%	52.47	0.34%	0:20:00
	100%	52.47	0.34%	0:20:00
10m³	25%	52.5	0.28%	0:20:00
	50%	52.27	0.72%	0:30:00
	75%	51.92	1.39%	0:30:00
	100%	51.43	2.32%	0:30:00
Ideal volume	25%	52.13	0.99%	0:30:00
	50%	50.86	3.40%	0:30:00
	75%	48.08	8.68%	0:40:00
	100%	44.08	16.28%	0:40:00

Fig. 3 Peak flow reduction due to different installation rates

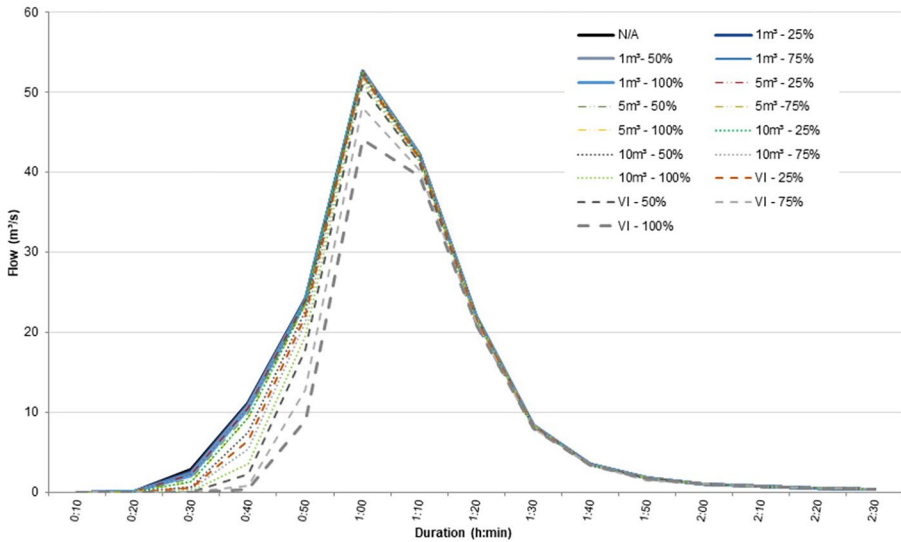
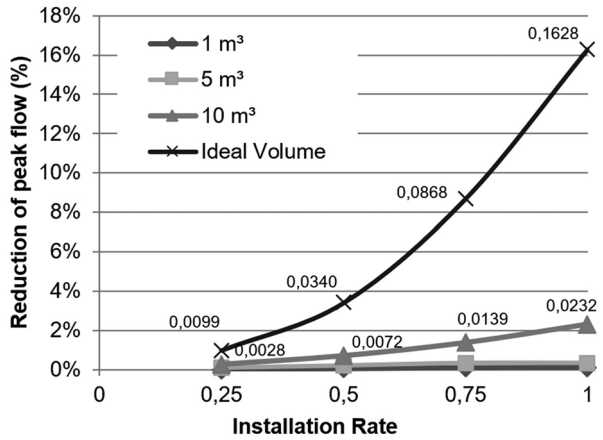


Fig. 4 Runoff flow hydrograph at the drainage discharge

4 Conclusion

By analyzing the results of the water balances conducted for different lots and tank sizes, it was found that the seasonality of rainfall has an effect on both the capacity to meet demand and the runoff volume at the system.

In the drought months, when the greatest demand occurs and precipitation volumes fall, the ability to meet demand decreases and the ability to reduce overflow can reach 100%. The first rain events of the rainy season, in the final months of the year, also benefit from the buffer effect as the tanks are depleted by drought. In the same period, the variation in the size of the tanks was a notably important factor for the perception of a buffer effect, since larger water tanks absorbed more rainwater and for a longer period.

However, in the wettest months, which precede the drought season, while the demand can be fully met, little reduction is seen in overflow, since the waiting volumes correspond

almost solely to the volumes of the demand met on the previous day. Thus, in the months in question, there is little variation in the overflowed volume among the different water tanks in the same lot pattern.

Seasonality also had a strong influence on the volume of the ideal tank which, in order to better meet demand, needs to be dimensioned for a long period of drought, thus requiring volumes far above commercial volumes.

The volume of the tanks was a determining factor in meeting demand and in the total rainwater retention capacity. However, while the variation in the waiting volume for the same level of prevalence tends to increase with the increase in the volume of the tank, the variation in the capacity to meet demand decreases as the volume of the tank increases.

It was also found that the unit value of demand per roof area had a significant influence on the value of the waiting volume and, consequently, on the total overflowed volume. The higher the unit demand, the lower the leakage values.

Regarding the reduction in drinking water consumption in the neighborhood, the use of RWHS showed up to a 37% supply substitution potential in the most favorable scenarios. This result is strongly dependent on the scale of installation and the volume of the tank adopted for the households. From these results, we can conclude that RWHS, when deployed on a large scale, can act as an important auxiliary supply system, alleviating the problem of scarcity and saturation of the city's centralized supply system.

Taking into account the urban hydrological model, it was found that the higher the rate of installation of the systems in the neighborhood, the greater the influence of the increase in the tank volume in increasing the harvesting capacity.

For the simulation variables considered in this study, it was noticed that, even if RWHS are implemented and work in all residences, the volumes of the commercial tanks most commonly used are not able to generate a significant impact on reducing peak flow in the drainage discharge. However, for larger volumes designed to meet existing demand, the impact on reducing peak flow reached values of up to 16.28%.

The use of RWHS also implied a delay in the beginning of the flow in some cases but there was no delay in the peak flow in any of them. As the decrease and the delay in the beginning of the flow coincide with the advance of the reductions in the intensity of the rain for the medium blocks of the hyetogram, it is expected that less intense precipitation will be experienced by the blocks of the hyetograms most affected and, therefore, the impacts in decreasing the flow are more evident.

It has been shown that the RWHS designed to meet the demand for homes under the climatic conditions of the city of Goiânia have a good potential to act as complementary systems to the urban supply and drainage system. However, policies that encourage the use of these systems are necessary for their adoption to be widespread and the benefits to be significant.

In order to better understand the impacts of using RWHS in the city of Goiânia, it is suggested that similar studies be carried out in neighborhoods with different occupation patterns and for projected rainfall with shorter return times. Research in this direction would make it possible to ascertain the real role of these variables in the potential for saving drinking water, controlling runoff and defining which scenarios have more or less added benefits from the installation of RWHS.

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Declarations

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