Revista Brasileira de Recursos Hídricos
Brazilian Journal of Water Resources
Versão On-line ISSN 2318-0331
RBRH, Porto Alegre, v. 23, e18, 2018
Scientific/Technical Article

http://dx.doi.org/10.1590/2318-0331.231820170149

Comparison the monitoring data of an on-site stormwater detention (OSD) and the results in the use of theoretical methods for its design

Comparação entre os dados de monitoramento de um microrreservatório e os resultados no uso de métodos teóricos para o seu dimensionamento

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Received: September 10, 2017 - Revised: December 12, 2017 - Accepted: March 07, 2018

ABSTRACT

On-site stormwater detention – OSD has been constructed in big Brazilian cities, as a way to reduce the impact of urbanization on the drainage system. However, there are a few studies about its efficiency in real scale. This article aims to compare the monitoring data of an OSD built in a hospital in Belo Horizonte and the results determined by theoretical methods, commonly used in the design of this structure. Inside the OSD was installed a level sensor to monitor water level during rain events. The data was recorded on a data logger every 30 seconds during the period April 2015 to March 2017. It was analyzed the OSD filling during the occurrence of 48 precipitation events. In the maximum heights of water level comparison, it was found that the monitored values were higher than theoretical values and the results using Rational Method were closer to monitoring data than the results with SCS-HU Method. It was also found that the peak and recession time calculated with Rational Method represented better the water levels monitored.

Keywords: On-site stormwater detention (OSD); Source runoff control; Best Management Practices (BMP); Urban drainage.

RESUMO

Os microrreservatórios vêm sendo implantados nas grandes cidades brasileiras como forma de reduzir o impacto provocado pela urbanização no sistema de drenagem. Todavia, existem poucos estudos que avaliaram sua eficiência em escala real. Este artigo teve como objetivo comparar os dados de monitoramento de um microrreservatório implantado em um empreendimento hospitalar de Belo Horizonte, com os resultados determinados por meio de métodos teóricos, usualmente utilizados no dimensionamento dessa estrutura. No interior do microrreservatório foi instalado um sensor de nível para realizar as medições das alturas d'água durante os eventos de precipitação, sendo os dados registrados a cada 30 segundos durante o período de abril de 2015 a março de 2017. Analisou-se o enchimento do microrreservatório durante a ocorrência de 48 eventos de precipitação. Na comparação dos resultados das alturas máximas do nível d'água no interior da estrutura, verificou-se que, de maneira geral, os valores monitorados foram superiores aos teóricos e que os resultados obtidos com o método Racional foram mais próximos aos monitorados do que os obtidos com o método HUT-SCS. Verificou-se ainda que os tempos de pico e recessão dos linigramas calculados com o método Racional representaram melhor o linigrama monitorado.

Palavras-chave: Microrreservatório; Controle de escoamento na fonte; Técnica compensatória; Drenagem urbana.

INTRODUCTION

In the 1970's, Europe and North America countries developed a new approach for urban drainage problems. This approach is an alternative or compensatory drainage technologies that seek to neutralize the effects of urbanization on hydrological processes, with benefits for quality of life and environmental preservation (BAPTISTA; NASCIMENTO; BARRAUD, 2005).

The compensatory or alternative systems of urban drainage are opposed to the concept of rapid evacuation of rainwater. The alternative technologies are based on rainwater infiltration and retention, causing a decrease in the runoff volume and the temporal rearrangement of the flows (MOURA, 2004).

According to Baptista, Nascimento and Barraud (2005), on-site stormwater detention (OSD), also known as residential tank, is a type of urban drainage alternative technique based on stormwater source control. This technique aims to temporarily accumulate the additional volume of water generated by the increase of impervious area in lots, maintaining the peak flow close to the pre-development conditions.

Considering the recurrent problems of flooding, many Brazilian cities have created regulations requiring the construction of OSDs in new developments.

Despite being the first city in Brazil, regarding to require the OSD construction, the Belo Horizonte's regulation demands one of the lowest rainwater volumes for detention in the country, about 7 (seven) times lower than required in Porto Alegre regulation, as demonstrated by Drumond, Coelho and Moura (2011).

In Belo Horizonte Municipal (Law n° 9,959/10) (BELO HORIZONTE, 2010), it is allowed to have up to 100% of impervious area in new developments. However, it is necessary to build an OSD to store 30 liters of stormwater per square meter of impervious area that exceeds the limit established in the law. The Belo Horizonte regulation defines the maximum rate of 90% of impervious area for lots with area less than or equal to 360 m^2 and 80% of impervious area for lots with area bigger than 360 m^2.

According to Drumond, Coelho and Moura (2013), an experiment made in the laboratory showed that the storage volume of 1.08 m^3, calculated based on Belo Horizonte regulation for a lot of 360 m^2, is not sufficient to promote peak flow attenuation in a lot with 100% of impervious area.

The hydraulic OSD design has to be developed considering two main aspects: the maximum flow to be released in the public drainage system and the volume required to temporarily store the amount of excess water between the inlet hydrograph and the desired outlet hydrograph.

Currently, the Sudecap - Superintendência de Desenvolvimento da Capital (SUDECAP, 2009), a municipal authority responsible for the drainage system of Belo Horizonte, has required to designers of OSD to use the Rational method to calculate the peak flow and the Puls method for determination of peak flow attenuation.

Considering that OSDs have been built for decades in Brazilian cities and there are only a few studies that have evaluated their real performance, this paper seeks to compare the results obtained with the design methods and the monitoring data.

The Rational and Puls method, recommended by Sudecap, and the SCS-HU method, usually used in drainage projects, were evaluated.

Hydrological methods

According to Chow, Maidment and Mays (1988), the Rational method is based on three hypotheses:

- The entire catchment contributes to surface runoff, which means that the duration of precipitation should be equal to or exceed the time of concentration;
- Rainfall is uniformly distributed over the entire area of the catchment; and
- All losses are incorporated into the runoff coefficient.

The Rational method equation to calculate the maximum flow is:

$$Q_p = 0.278.C.I.A \quad (1)$$

where: Qp = peak flow (m^3/s); C = runoff coefficient (varies from 0 to 1); I = average rainfall intensity (mm/h); A = catchment area (km^2).

Defined the peak flows, the hydrographs are calculated considering that the peak and recession times are equal to the time of concentration, when the rainfall duration is equal to the time of concentration. In situations where the duration of the event is greater than the time of concentration the hydrograph of Figure 1 is considered, in which the constancy of the peak flow is observed until the end of the rainfall event.

Despite its simplicity, the use of the Rational method is strongly questioned. The main criticisms are related to the hypotheses of constancy of the runoff coefficient and the rainfall intensity during each event, as well as its uniform distribution over the entire drainage catchment. These hypotheses eventually overestimate the calculated flows, increasing the error as the size of the catchment increases.

Many authors differ on the upper boundary of the drainage catchment area for the use of the Rational method. The highest values, from 20 to 500 km^2, are recommended by Maidment (1993), while Wanielista and Ron (1997) suggested lower values, from 0.2 to 0.4 km^2.

Another method widely used in hydrological studies in urban areas is the Soil Conservation Service (SCS), now called Natural Resources Conservation Service (NRCS), unit hydrograph method (UH).

According to Babu and Mishra (2012), the UH-SCS method is very popular because it is simple, stable, easy to understand and to be used, and considers the characteristics of the catchment such as type and use of the soil, hydrological conditions and antecedent moisture condition. The model also considers the uniform rainfall in the entire catchment and a constant CN throughout the duration of the event.

Initially, the peak flow of the unit hydrograph is calculated using values of the contribution area, excess rainfall and peak time of the hydrograph.

The SCS unit hydrograph results in a triangular hydrograph in which the recession time is 1.67 times greater than the peak time, as can be observed in Figure 2.

Once the SCS synthetic unit hydrograph is determined, the resulting hydrograph for a precipitation event is calculated by

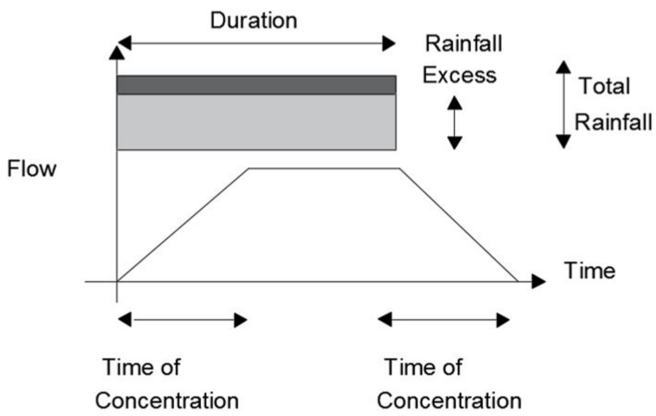

Figure 1. Trapezoidal hydrograph of Modified Rational method, with rainfall duration exceeding time of concentration.

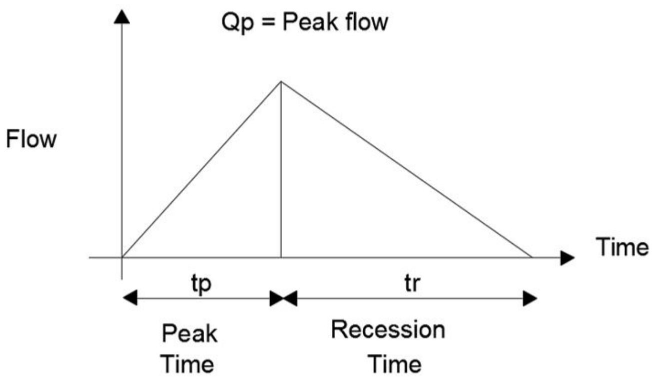

Figure 2. SCS Triangular Unit Hydrograph.

defining the excess rainfall for each time interval. To calculate the excess rainfall, the part of rainfall absorbed by the catchment and initial abstraction are removed from the total rainfall, as shown in the equation below.

$$P = P_e + F_a + I_a \tag{2}$$

where: P = total rainfall (mm); Pe = excess rainfall; Fa = rainfall absorbed by the catchment (mm); Ia = initial abstraction by of the catchment (mm).

The parameters of maximum absorption and initial abstraction of the catchment are based on the Curve Number (CN) values.

The CN parameter is a dimensionless value associated with the type and use of the soil and antecedent moisture conditions of the catchment.

If the rainfall temporal distribution is known, the rainfall excess over time is obtained, which is combined with the calculated unit hydrograph. This process is called convolution. In this process the unit hydrograph at each time increment is multiplied by the excess rainfall at the specified time. The final hydrograph is obtained by adding the hydrographs associated with each rainfall block.

According to Hawkins et al. (2002) apud Babu and Mishra (2012), the UH-SCS method has some limitations, as described below:

1. The effects of rainfall intensity and duration, which have a great impact on the runoff volume, are not considered in the UH-SCS method;
2. Although the initial abstraction is dependent on the antecedent moisture, this relationship is not considered in the method;
3. The relation between the CN value and the soil antecedent moisture is not realistic. The method allows large CN changes with variation of the antecedent moisture condition;
4. Fixing the initial abstraction rate at 20% of the maximum water retention potential in the soil may not represent reality.

Hawkins et al. (2009), based on literature, suggested that the value of the initial abstraction rate is 5% for general applications.

In a study of urban catchments, Cruz (2004) verified that small changes in the CN values could cause a great variation in the results of the flows.

According to Cunha et al. (2015), the results of the UH-SCS method for rainfalls of greater magnitude indicate a tendency to overestimate runoff volumes.

There is no consensus among researchers about the boundary of the catchment area for UH-SCS method application. Wilken (1978), McCuen (1982) and Genovez (2003) indicated that the method should be used in catchments with area less than 2600 km^2. Ramos, Barros and Palos (1999) suggested the use for catchments with areas between 3 and 250 km^2.

Reservoir model for flood routing

The peak flow attenuation provided by the OSD is estimated using reservoir routing models. According to Nascimento and Baptista (2009), the hydrological models of reservoir routing can be used in the conception and in the diagnostic phases.

One of the most used models to simulate reservoir routing is the method developed by Puls. This method uses the continuity equation and the relation between storage and flow is obtained considering the water level within the reservoir (NASCIMENTO; BAPTISTA, 2009).

If the equation of continuity is discretized and it is organized the known variables on one side and unknown on the other, the result is the following equation:

$$(I_1 + I_2) + (2S_1 / \Delta t - Q_1) = (2S_2 / \Delta t + Q_2) \tag{3}$$

where: I_1 = inflow at the beginning of the time interval; I_2 = inflow at the end of the time interval; S_1 = volume at the beginning of the time interval; S_2 = volume at the end of the time interval; Q_1 = outflow at the beginning of the time interval; Q_2 = output flow at the end of the time interval; Δt = duration of the time interval.

In Equation 3 the values of I_1, I_2, Q_1 and S_1 are known at any time t and the values Q_2 and S_2 are unknown. Akan (1993) suggested the following procedures for determination of the unknown variables and calculation of the flood routing:

1) The storage curve S_1 as a function of the outflow Q_1 can be obtained from the volume-stage and discharge-stage relations;
2) Select a time increment, Δt and calculate a graph of $[2S1/\Delta t + Q1]$ and the outflow, Q_1;
3) Calculate $[I_1 + I_2]$ of the inlet hydrograph and $[2S1/\Delta t - Q1]$ for the initial condition or the previous time for any time interval;
4) Calculate $[2S_2/\Delta t + Q_2]$ using Equation 3;
5) Determine Q_2 from the graph obtained in step 2, where this is the outflow at time t_2;
6) Calculate $[2S_2/\Delta t - Q_2]$ by subtracting $2Q_2$ from $[2S_2/\Delta t + Q_2]$ and return to step 3. The value of $[2S_2/\Delta t - Q_2]$ calculated at any time will be $[2S_1/\Delta t - Q_1]$ at the following time;
7) Repeat the same procedure until the outflow equals zero.

The calculated routing in the Puls method is related to the discharge structure that regulates the outflow. The discharge structure restricts the outflow in the OSD, which is a function of the hydraulic load and tube diameter.

The following equation is used to calculate the outflow through the orifices and nozzles, with free discharge at atmospheric pressure:

$$Q = C_d . A . \sqrt{2gh} \qquad (4)$$

where: Q = flow (m^3/s); Cd = discharge coefficient; A = cross-sectional area of the pipe (m^2); g = gravitational acceleration(m/s^2); h = total head on the axis of the tube (m).

The classical literature generally recommends the discharge coefficient value of 0.61 for orifices and 0.82 for nozzles.

According to Azevedo Netto et al. (1998), the discharge tubes can be classified by the relation between the length (L) and the diameter (D), as shown in Table 1.

The classification is not complex, although in most urban drainage projects, the discharge structure is considered as an orifice, no matter what is the L/D ratio (LENCASTRE, 1972). This incorrect classification can result in different outflows than designed, not causing reduction of the peak flow.

Aiken (1993) evaluated the effects of a steel plate with an orifice of an OSD on discharge coefficient values in Manly hydraulics laboratory in New South Wales, Australia for different configurations of the depth and wall distances, hydraulic loading and location of the solid waste retention screen. The value of discharge coefficient obtained was 0.62 for the diameters tested (50, 75 and 100 mm).

Pells and Miller (2004) tested the operation of steel plates with orifice diameters of 5, 10, 15, 20, 30, 40 and 50 mm in order to determine the minimum orifice size that is not affected by solid waste blockages. The results indicated that the discharge coefficient was 0.62 for diameters greater than 30 mm and for diameters smaller than 20 mm the flow rates were affected by solid waste.

According to Drumond et al. (2014), in laboratory tests with discharge pipes of 7 (seven) different diameters and lengths of 15 and 25 cm, classified as nozzles or short tubes, values of discharge coefficient close to 0.90 were obtained, value higher than recommended by the Sudecap.

In the study performed by Castro, Vianna and Ribeiro (2015) it was found that the discharge coefficient values for orifices installed in flocculator trays used in water treatment plants were greater than 0.61.

It is noticed the need to better evaluate the values of the discharge coefficients on a real scale. This parameter is essential for determination of the outflow to be released in the drainage system.

Therefore, this work will evaluate the value of 0.61 for the discharge coefficient, usually adopted in urban drainage projects and in OSD design, object of this study.

Design methods used by Belo Horizonte Municipality

In OSD design, SUDECAP (2009) recommends the use of Rational method for rainfall-runoff transformation, followed by the Puls method and the discharge equation of orifices, considering that the discharge structure works as an orifice.

To determine the peak flow it is recommended to use the value of 197.8 mm/hour for rainfall intensity, for a rainfall with 10 years average recurrence interval, 10 minutes duration and 5 minutes time of concentration. The runoff coefficient values for permeable and impervious area suggested are 0.45 and 0.95, respectively.

Regarding to UH-SCS method application, after Silva et al. (1995) study, Ramos (1998) classified Belo Horizonte soils into hydrological groups B and D.

Based on this soil classification, the Urban Drainage Master Plan (Plano Diretor de Drenagem Urbana - PDDU) of Belo Horizonte (2000) defined CN values for the two hydrological groups for permeable and impervious soil conditions, as can be observed in Table 2.

Table 1. Classification of discharge pipes.

Classification	L / D
Orifice	< 1.5
Nozzle	$1.5 < L/D < 3$
Short tube	$3 < L/D < 500$
Tube	$L/D \geq 500$

Note: based on data from Azevedo Netto et al. (1998).

Table 2. CN values defined by PDDU/BH (BELO HORIZONTE, 2000).

Soil condition	GH-B	GH_D
Permeable	69	84
Impervious	98	98

OSDs experiments carried out in Brazil

Agra (2001) evaluated the performance of an OSD with volume of 1 m^3 and two discharge tube configurations, one with two tubes with a diameter of 40 mm and other with one pipe with a diameter of 50 mm, to receive water from a roof contribution area of 337.5 m^2 (15 m x 22.5 m).

There were evaluated 8 (eight) events and the analysis indicated that the drainage device is efficient to reduce peak flows, as shown in Table 3. However, regarding the increase in response time in the catchment, it was observed that the structure was inefficient due to the small retention volume.

Regarding to the runoff coefficient of the drainage area, there was a variation from 0.83 to 0.95, resulting in an average of 0.90. It was also verified that in some events the peak flow of the outlet hydrograph was higher than the limit flow established at 4.7 L/s. It was raised the hypothesis that the discharge structure may operate as a nozzle and not as an orifice.

Also it was observed a rapid emptying of the OSD in the events. In some events it was not possible to perform the analysis because the reservoir overflowed. This happened because the storage volume was undersized to retain the generated flows. Therefore, it was verified that the OSD design should have been made using the critical duration methodology, indicated by Tucci and Marques (2000), instead of the methodology suggested with the Rational method, which predicts that rain duration is equal to or greater than the catchment time of concentration.

Cabral et al. (2009) evaluated the peak flow attenuation of an OSD with 200 m^3 of volume. The structure was built under the pavement of a street in the neighborhood of Espinheiro, in Recife/Brazil, with the objective of reducing flooding problems in the region for rainfall with return period of 2 years. The performance of the OSD was evaluated during two rain events that occurred in 2008. For the rainfall with approximately 13 years average recurrence interval, the OSD overflowed. However, the flooding was much less than before the OSD implementation. For the rainfall event with two years average recurrence interval the reservoir worked properly.

Another important study was developed by Campos (2007), who evaluated in the real-scale the hydrological features of three lots with 360 m^2 located in the city of Bertioga, SP/Brazil. The situations evaluated were:

Table 3. Reduction of peak flow events.

Event	Inlet flow	Outlet flow	Reduction of peak flow %
1	3.65 l/s	2.25 l/s	38
2	1.93 l/s	1.80 l/s	7
3	3.72 l/s	2.08 l/s	44
4	3.72 l/s	1.82 l/s	50
5	5.85 l/s	5.27 l/s	10
6	4.84 l/s	4.12 l/s	15
7	5.33 l/s	4.84 l/s	9
8	2.2 l/s	1.8 l/s	18

Note: data from Agra (2001).

i) Completely permeable land (natural lot);

ii) Land with 75% of impervious area and without flow control (conventional lot);

iii) Land with 75% of impervious area and two reservoirs of 750 liters receiving rainwater from a roof (sustainable lot).

To measure the runoff, in each lot was constructed a rectangular channel with triangular spillway.

The results of 15 rain events monitored showed that there was an average reduction of 81.1% of the peak flows in the sustainable lot in comparison to the conventional lot, with a minimum reduction of 45%. In comparison with the natural lot, the sustainable lot had an average flow rate of 1.41 higher and the conventional lot had a 20.7 times higher flow rate.

Thus, in order to evaluate the OSD performance in a real condition, it was decided to monitor an existing OSD in Belo Horizonte, and compare it with the theoretical results using methodologies usually adopted in drainage system projects. This type of analysis has not yet been made in the OSDs of Belo Horizonte.

MATERIAL AND METHODS

In order to verify the performance of an OSD built in Belo Horizonte, and designed according to the methodology recommended by Sudecap, it was decided to monitor the water level inside the structure through the use of a water level sensor, during two hydrological years from April 2015 to March 2017.

Details of the OSD monitored

The selection of the OSD to be monitored in this study was made after a consultation at Sudecap, in the department responsible for approving OSD projects in Belo Horizonte. The structure was chosen because of the ease of access to its interior, to the location to download the data, and to install the equipment.

The OSD was built in a hospital, located in the South-Central region of Belo Horizonte. According to the municipality regulation, the region is classified as a central zone, which is configured as a center of regional, municipal or metropolitan polarization.

The total contribution area of the OSD is 4,149.04 m^2, divided into 3,516.94 m^2 of building area and 632.10 m^2 of intensive green roof.

The monitored OSD is located under a sidewalk near the hospital area. The structure was constructed in concrete in a rectangular shape, and its dimensions are (Figure 3):

- Lenght: 12.10 meters;
- Width: 3.85 meters;
- Useful depth: 1.00 meters;
- Useful volume: 45.50 m^3;
- Diameter of the inlet pipe: DN 300mm;
- Diameter of the discharge tube: DN 200mm (1st hydrological year) 2 x DN 100mm (2nd hydrological year);

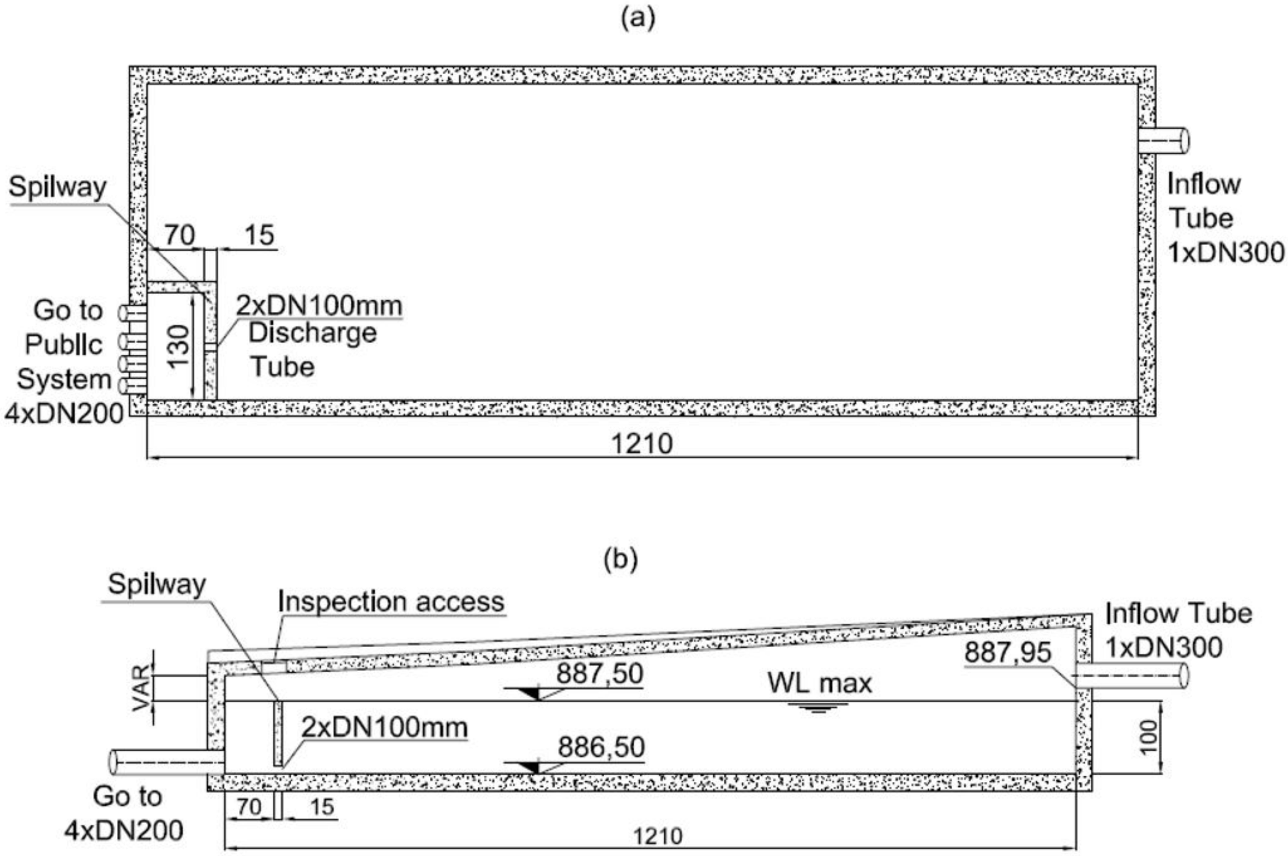

Figure 3. OSD monitored (a) plan view and (b) section view. Obs: The spillway width is 130 cm.

- Length of the discharge tube: 15 cm;
- Spillway length: 2.15 m (0.85m + 1.30m);
- Outlet tube to public system: 4 x DN 200.

The superficial runoff is guided to the OSD via the inlet tube indicated in Figure 3. The discharge structure, which regulates the outflow, was installed 11.25 meters away from the entrance. Due to the possibility of overflow, a free-flow spillway was constructed with a width of 2.15 m, located 1.0 m from the bottom of the structure. The water from the discharge tube and the spillway are guided to public drainage system.

Figure 4 shows the stage x volume relation of the OSD.

Although the discharge tube was designed with two pipes each one with 100 mm of diameter, it was verified during inspection that only one tube with a diameter of 200 mm was installed, i.e., two-times the discharge area of the initial design. Thus, it was decided to evaluate the performance of the OSD with a diameter of 200 mm in the first hydrological year, and in the second hydrological year the discharge structure was exchanged for two pipes of 100 mm diameter, as indicated in the design.

According to the hospital employees, who do the OSD maintenance, on November 6, 2015 it was removed a hose with 100 mm of diameter from the discharge tube. It was decided not to use the data obtained before that date, due to possible interference in the analysis.

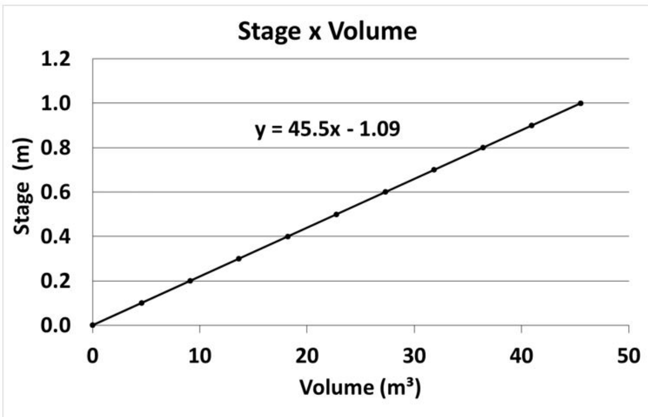

Figure 4. Water Level x OSD Volume.

OSD monitoring

After the hospital authorization, a water level sensor was installed, model SNS-400 of the brand Global Waters, with measurement range of 0 to 4 meters of water level and accuracy ± 0.1% FS. The monitored data was programmed to be recorded every 30 seconds and stored in the Global Waters model GL 500 data logger. The equipment was powered by a 12 volt and 7 amperes battery. The Figure 5 shows the installed sensor and the box protection with the data logger and battery.

The equipment were provided by Sudecap and calibrated in the CPH laboratory (Center for Hydraulic Research and Water Resources) of UFMG. The data were downloaded to a laptop every 14 days.

The water levels monitored inside the OSD were compared with the theoretical results calculated using the Rational and UH-SCS methods for rainfall-runoff transformation, according to the procedure described in the following items. In the theoretical calculations, it was used the same rainfall event recorded at the site.

Rainfall data

The rainfall data used was obtained from the records of two rain gauges, located about 1 km from the hospital. One rain gauge was operated by Inmet - National Institute of Meteorology and the other by Sudecap. Because the data from both rain gauges showed a good correlation (Figure 6), it was determined the average rainfall for each event.

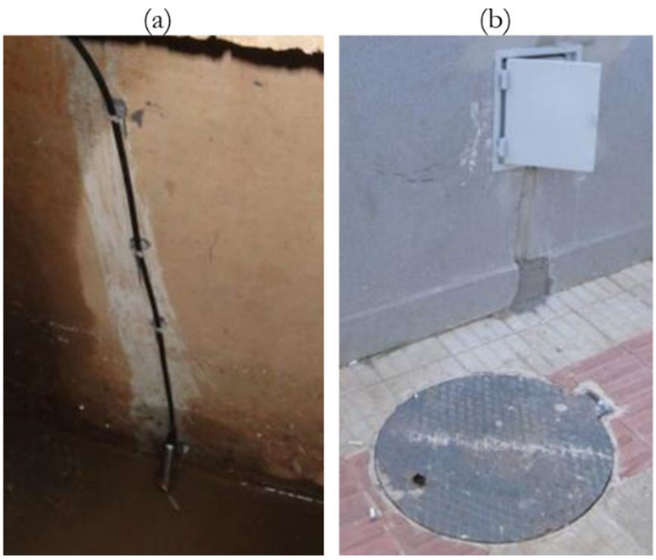

Figure 5. Equipment installed (a) Water level sensor and (b) Box protection to the battery and data logger, and inspection access.

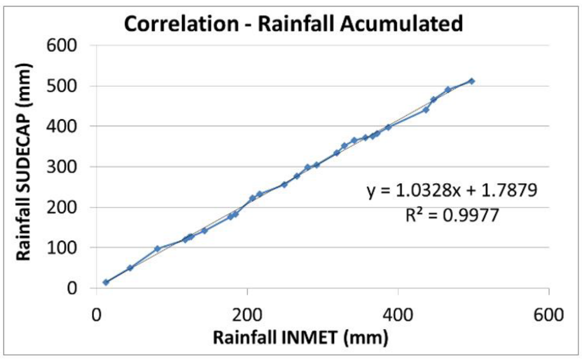

Figure 6. Correlation of rain gauges by the Double Mass method.

The interval time of rainfall record in each rain gauge was 10 minutes. Table 4 shows the total rainfall height (R), duration (D), intensity (I) and return period (TR) of the events used in the study. The events were organized based on results of the maximum water level monitored in the OSD, as shown in Figure 7.

Table 4. Rainfall data used.

Event	Date	R (mm)	D (min)	I (mm/h)	TR (years)
1	26/02/17	7.9	120	3.95	< 1.05
2 *	17/01/16	3.9	50	4.68	< 1.05
3	26/02/17	3.2	30	6.40	< 1.05
4 *	04/03/16	2.3	20	6.90	< 1.05
5 *	04/03/16	3.5	40	5.25	< 1.05
6 *	17/01/16	5.1	60	5.10	< 1.05
7 *	04/03/16	2.7	20	8.10	< 1.05
8 *	07/12/15	11.8	150	4.72	< 1.05
9	10/01/17	6.7	30	13.40	< 1.05
10 *	06/12/15	2.1	70	1.80	< 1.05
11 *	04/03/16	11.8	120	5.90	< 1.05
12 *	16/01/16	8.8	150	3.52	< 1.05
13	06/10/16	6.2	30	12.40	< 1.05
14 *	06/12/15	3.5	20	10.50	< 1.05
15	02/06/16	8.5	30	17.00	< 1.05
16 *	26/02/16	12.4	70	10.63	< 1.05
17	05/10/16	13.6	100	8.16	< 1.05
18 *	23/11/15	20.0	200	6.00	< 1.05
19	15/10/16	11.0	70	9.43	< 1.05
20 *	20/11/15	8.2	80	6.15	< 1.05
21 *	24/03/16	15.0	80	11.25	< 1.05
22 *	18/02/16	11.6	90	7.73	< 1.05
23	03/06/16	12.7	60	12.70	< 1.05
24	18/11/16	6.8	50	8.16	< 1.05
25 *	26/01/16	16.2	90	10.80	< 1.05
26 *	29/04/16	13.4	40	20.10	< 1.05
27	03/02/17	5.6	90	3.73	< 1.05
28	03/06/16	10.6	70	9.09	< 1.05
29 *	25/02/16	20.0	80	15.00	< 1.05
30 *	15/01/16	15.6	90	10.40	< 1.05
31	13/01/17	14.0	120	7.00	< 1.05
32 *	16/01/16	36.6	390	5.63	< 1.05
33	05/02/17	7.8	120	3.90	< 1.05
34	12/01/17	13.0	70	11.14	< 1.05
35 *	19/11/15	15.2	70	13.03	< 1.05
36 *	10/12/15	20.5	150	8.20	< 1.05
37	05/02/17	7.1	30	14.20	< 1.05
38	08/03/17	22.9	90	15.27	< 1.05
39	25/02/17	18.4	30	36.80	< 1.05
40 *	20/11/15	10.8	70	9.26	< 1.05
41 *	25/02/16	22.3	150	8.92	< 1.05
42 *	24/03/16	31.6	50	37.92	~ 1.25
43 *	08/12/15	18.7	40	28.05	< 1.05
44	01/06/16	17.6	30	35.20	< 1.05
45	06/02/17	38.5	250	9.24	< 1.05
46	19/03/17	60.8	110	33.16	~ 10
47 *	18/11/15	52.2	70	44.74	~ 5
48 *	12/02/16	89.0	80	66.75	~ 200

Obs: * Events that the discharge tube was a diameter of 200 mm.

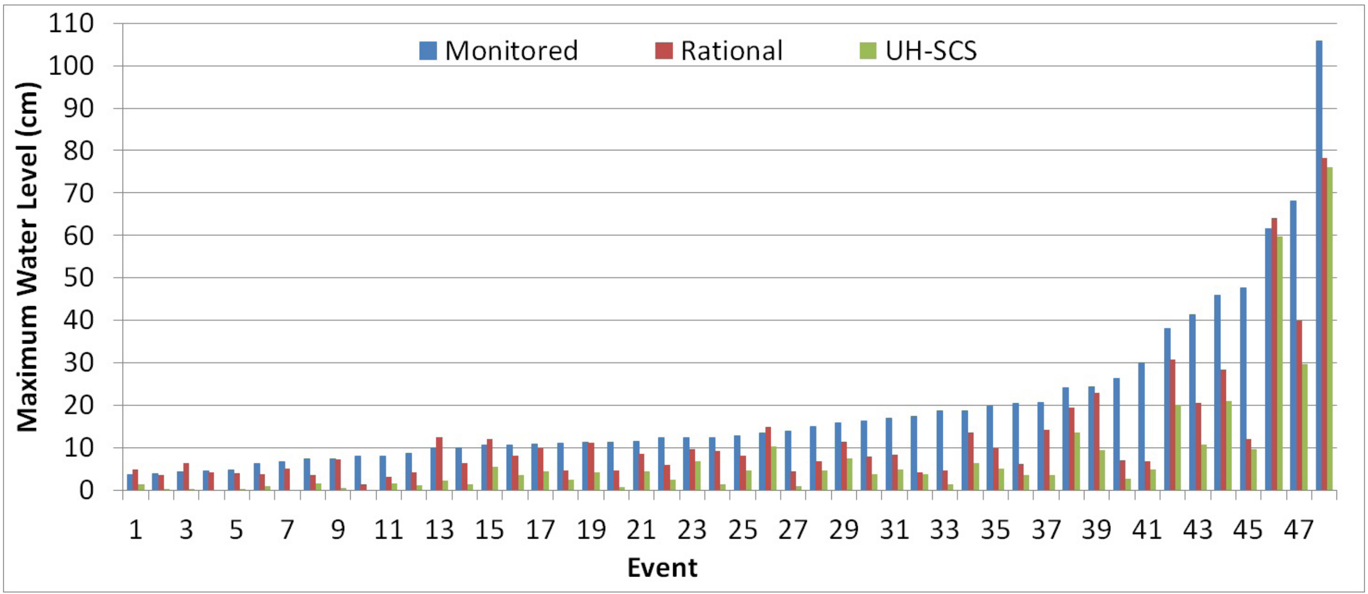

Figure 7. Comparison between maximum water levels monitored and calculated using the Rational and SCS-HU methods.

Theoretical calculations

The hydrological methods used in this evaluation were the Rational method, adopted in design of the structure, and the UH-SCS method, usually used in urban drainage projects.

The theoretical calculations of the OSD performance were made using the Puls method, determining the water levels inside the structures and the peak flow attenuation.

Rational method

In calculation of the peak flow by the Rational method, there were used the runoff coefficient values of 0.95 for the impervious area and 0.70 for the green roof area (based on its functioning as a green roof). According to Ladeira et al. (2017), in a green roof the runoff coefficient values can range from 0.25 to 0.93.

The rainfall intensity used was the mean calculated for each event as presented in Table 4. The time of concentration considered to determine the OSD inlet hydrograph was 5 minutes.

SCS-UH method

As the OSD is located in the region of Hydrological Group D, in UH-SCS method simulations was used the value of 98 for the impervious area and 84 for the green roof, determining the weighted average for the CN (95.87). However, the antecedent moisture condition was not evaluated, since the soil has a large amount of impervious area.

The time of concentration adopted was 5 minutes to determine the unit hydrograph. The rainfall data obtained in the rain gauges was used in the hyetographs, with a discretization of 10 minutes.

Puls method

After setting the inlet hydrographs with the Rational and UH-SCS methods, the reservoir flood routing modeling was made in the OSD for each event.

To determine the stage-flow curve, the occurrence of free surface flow was considered up to the height of the discharge tube diameter. From that point until the height of the spillway, the flow was considered under pressure and above the spillway height, it was defined the occurrence of overflow in the OSD.

Thus, the outflow in the OSD was determined according to the following situations:

- WL (water level) < Tube diameter: use of the equation of a simple tubular culvert without the occurrence of hydraulic load downstream, with a value of 0.015 for the Manning coefficient, slope of 1% and culvert length of 15 cm;
- Tube diameter < WL <100 cm: use of the orifice equation with a value of 0.61 for the discharge coefficient;
- WL > 100 cm: use of the Francis equation for a rectangular spillway, with a length of 2.15 m, plus the use of the orifice equation, for WL equal to 100 cm.

After determining the stage-volume and stage-flow curves, the Equation 3 was used to determine the volume in the OSD at each time interval and consequently the height of the water level.

RESULTS AND DISCUSSION

The results of the maximum water levels obtained in monitoring of the OSD and in the simulations carried out with the Rational/Puls and UH-SCS/Puls methods are presented in Figure 7.

The results showed that, in general, the monitored water level were generally higher than those defined by the Rational/Puls and UH-SCS/Puls method. In the Rational/Puls methods the maximum water levels were on average about 33% lower than those monitored, while in the UH-SCS/Puls method were approximately 73% lower.

In events 1, 3, 13, 15, 26 and 46 the maximum water levels determined by the Rational/Puls method were higher than the monitored value. In comparison with the UH-SCS/Puls method, the maximum height of the theoretical water level was not higher than the monitored height in any event.

The differences observed between the monitored data and the theoretical results, which underestimated the water levels, opposite to what was expected, may have occurred mainly due to:

- The discretization time defined in the Rational method, that uses the average rainfall intensity, while there were intervals during the event in which intensities exceed the average;
- The values of absorbed rainfall and initial abstraction defined in the UH-SCS method, which were significant, especially in the low rainfall events.

After changing the dimensions of the discharge structure, in which the area was reduced by half, as defined in the OSD design, the differences between the monitored maximum water levels and the theoretical water levels were reduced, as shown in Table 5.

It was observed that the trend of the Rational method results to be significantly higher than those of the UH-SCS occurred mainly in the events of reduced precipitation. In the events 46, 47 and 48, with return periods of 10, 5 and 200 years, respectively, the maximum water levels of the two methods were similar. This indicates that the absorbed rainfall and initial abstraction defined in the UH-SCS method are significant for reduced rainfall events.

Figure 8 presents the water levels of the rainfall events with different return periods: TR 1.25 years (Event 42), TR 10 years (Event 46), TR 5 years (Event 47) and TR 200 years (Event 48).

Note that in the water levels calculation using the theoretical methods, the small precipitations occurred after the main event were not considered but presented in Figure 8. The rainfall durations considered in the calculations were 50, 110, 70 and 80 minutes for events 42, 46, 47 and 48, respectively.

The comparison of the peak and recession times between the monitored water levels inside the OSD and the hyetograph

Table 5. Average of the percentage differences between the maximum water levels monitored and theoretical.

Discharge device	Rational/Puls Method %	SCS-HU/Puls Method %
DN 200	-43	-76
2 x DN 100	-19	-68

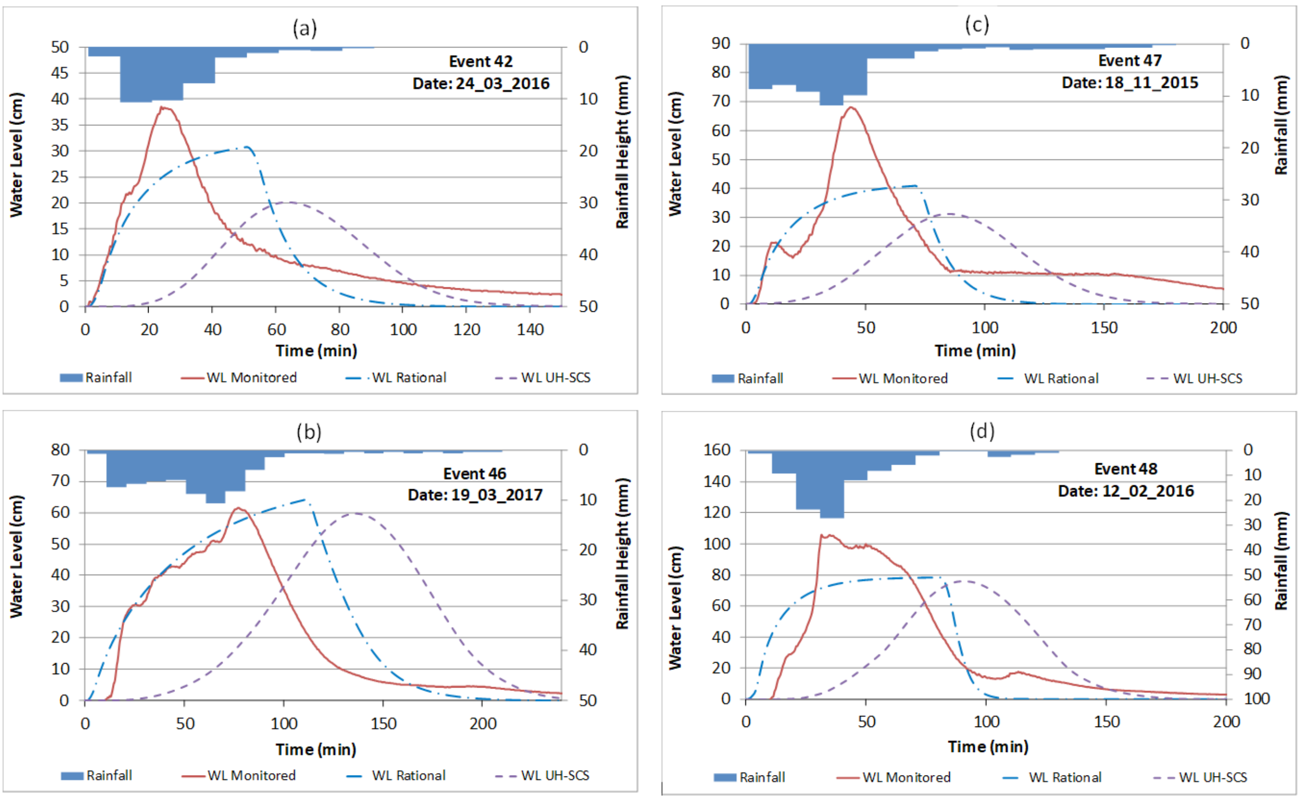

Figure 8. Comparative graphics of the water levels (WL) monitored and calculated with the Rational and UH-SCS methods (a) Event 42 (b) Event 46 (c) Event 47 and (d) Event 48.

indicates that durations of the events were close to the duration of the water levels rise and fall in the OSD. This can be explained by the small size of the contribution area, which provides a fast response time and the inability of the reservoir to control the peak flow, as will be discussed later.

The results indicate that the duration of the hydrograph in the lot can be determined according to the recommendation of the Rational method: the peak and recession times are equal to the time of concentration, or in cases where the duration of the events is greater than the time of concentration, according to the hydrograph of Figure 1.

The peak time of the water levels calculated with the UH-SCS method was considerably higher than the monitored water levels. In the UH-SCS method the peak time of the unit hydrograph is defined as half of the rain duration plus 60% of the time of concentration. Besides, the UH-SCS peak time formula be greater than that defined by the Rational method, the values for the initial abstraction and rainfall absorbed in the catchment adopted in the method could be higher than the reality of urban lots, resulting in low excess rainfall values, especially at the beginning of the rainfall events.

Because the contribution area is small, the use of UH-SCS method with smaller intervals in the rainfall discretization could improve the representation of the effects of rainfall intensity and duration on superficial runoff. It was not possible to do this analysis, since the data of discretization intervals of the in the rain gauge were 10 minutes.

The monitoring water levels in Figure 8d indicates that an overflow happened during the event and this was not observed in the theoretical methods results.

It was found that the monitored and calculated areas under the water levels x time graphs, which are proportional to the volumes stored in the evaluated period, were not the same. These differences may have been caused by some factors:

- Partial obstruction of the discharge tube, what may have caused a greater time of emptying the rainwater from the reservoir;
- Difference between the real values of the runoff coefficients and CN, and those used in the theoretical simulations;
- Difference between the values of the real discharge coefficients and those used in the theoretical simulations;
- Divergence between rainfall recorded in rain gauges and the occurred in the hospital contribution area;
- Level sensor measurement error.

Considering the results obtained with the Rational method were closer to those monitored, it was also evaluated if the OSD could have reduced the peak flows of the events monitored.

The inlet peak inflow was determined by the equation of the Rational method and the outlet peak flow using the Puls method with discharge coefficient of 0.61. The results are shown in Table 6.

The results of the theoretical calculations indicated that there was, in general, a small reduction of the inlet peak flows. The highest efficiency in reducing the peak flow was 22.9%, occurred in event 39. In addition, the sensitivity of the discharge coefficient was evaluated in the efficiency estimation, but for the discharge coefficient values tested (from 0.60 to 0.70), the calculated efficiencies remained low.

For the rainfall with a 10-year average recurrence interval (Event 46), the same recurrence defined in calculation of the

Table 6. Peak flow attenuation efficiency (Qp).

Event	Qp Inlet (L/s)	Qp Outlet (L/s)	Efficiency %
1	4.15	4.13	0.58
2	4.92	4.86	1.20
3	6.73	5.59	16.84
4	4.13	4.13	0.00
5	5.52	5.35	3.02
6	5.36	5.33	0.48
7	7.25	5.86	19.16
8	4.96	4.96	0.00
9	7.57	6.29	16.91
10	1.89	1.89	0.09
11	8.51	6.93	18.62
12	5.87	5.87	0.00
13	13.03	10.63	18.43
14	11.04	8.92	19.16
15	17.87	14.38	19.50
16	11.17	11.15	0.19
17	8.58	8.53	0.58
18	6.31	6.31	0.00
19	9.91	9.70	2.14
20	6.46	6.46	0.02
21	11.82	11.81	0.08
22	8.13	8.13	0.03
23	13.35	12.80	4.10
24	8.58	8.12	5.28
25	11.35	11.35	0.03
26	21.12	20.61	2.45
27	3.92	3.90	0.58
28	9.55	9.35	2.04
29	9.37	9.37	0.00
30	10.93	10.93	0.03
31	7.36	7.31	0.58
32	5.99	5.85	2.28
33	4.10	4.07	0.58
34	11.71	11.43	2.38
35	13.69	13.67	0.19
36	8.62	8.62	0.00
37	14.92	12.41	16.84
38	16.05	15.88	1.04
39	23.12	17.81	22.97
40	9.73	9.71	0.20
41	15.76	15.75	0.08
42	39.85	38.58	3.19
43	29.48	28.39	3.68
44	36.99	30.02	18.84
45	9.71	9.63	0.85
46	34.85	31.53	9.53
47	47.02	46.37	1.39
48	70.15	70.06	0.13

rainfall intensity used by the Sudecap in calculation of the OSD, there was a 9.53% reduction of the peak flow.

The low efficiency of peak flow reduction indicates that the structure does not perform the function for which it was designed.

It stands out that in the simulations it was used the discharge coefficient as an orifice, with a value of 0.61. However, in the experiments of Drumond, Coelho and Moura (2014) and Castro, Vianna and Ribeiro (2015) indicated that the discharge coefficient value could be higher than that adopted.

In order to improve the evaluation of the OSD performance, it would be necessary to monitor the inflow and outflow, verifying the entire system performance. Besides that, the discharge coefficient values of the orifice equation could be checked.

However, because the OSD is underground and there is no physical space for flowmeters installation, it was not possible to record the inlet and outlet flows in the studied reservoir.

CONCLUSIONS

The comparison between OSD monitoring data and the water levels calculated with the methods usually used in urban drainage projects showed that the results with Rational method were closer to those monitored than the results obtained with the UH-SCS method.

In addition, the results related to the peak and recession times of the water levels calculated with the Rational method represented better the monitored water levels. However, the use of rainfall data with discretization less than that adopted in this study can improve the results with the UH-SCS method. Nevertheless, it is recommended to use the Rational method in design of these structures.

The Rational method simulations indicate that the OSD presents low efficiency for peak flow reduction, using the discharge coefficient value of 0.61. As this value may not represent reality, the outflows can be even higher.

New studies monitoring the inflow and outflow and the water level in the OSDs it is recommended, in order to verify the maximum flows generated, the efficiency of peak flow attenuation, the peak and recession times of the hydrographs and the discharge coefficient values.

ACKNOWLEDGEMENTS

The authors wish to thank Sudecap – Superintendência de Desenvolvimento da Capital, CNPq - Conselho Nacional de Desenvolvimento Científico e Tecnológico, Capes - Coordenação de Aperfeiçoamento de Pessoal de Nível Superior (Projeto Capes – Cofecub) and Fapemig - Fundação de Amparo à Pesquisa do Estado de Minas Gerais for their financial support granted for the scientific and technological development of the performed research.

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Authors contributions

Pedro de Paula Drumond: OSD monitoring; article structure and writing; data processing and results analysis.

Priscilla Macedo Moura: principal advisor of the work, contributed in the methodology, discussion of the results and final review.

Márcia Maria Lara Pinto Coelho: co- advisor of the work, contributed in the methodology, discussion of the results and final review.