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HYDROLOGICAL AND HYDRAULIC MODELING AND EVALUATION OF FLOOD DAMPING SCENARIOS WITH THE IMPLANTATION OF MICRORESERVOIRS IN LOTS IN THE NEIGHBORHOOD OF TIJUCA, RJ

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Abstract: This work aims to evaluate a specific methodology for the implantation of microreservoirs in urban lots in the neighborhood of Tijuca, in the city of Rio de Janeiro, based on hydrological and hydraulic modeling with the SWMM *software*. The methodology consists of selecting micro-drainage areas and assessing the operating conditions of existing drainage networks, with and without the use of detention micro-reservoirs in the lots. The results indicate potential flood dampening with an average reduction of 37% in peak flows in the studied network for a fully urban basin with no affluence from adjacent sub-basins.
Keywords: Urban drainage. Rainwater. Mitigation measures. Damping flood peaks.

INTRODUCTION

Floods are responsible for 10% of the total amount of damage and losses from all disasters in Brazil, reaching an annual average of 1.3 billion, in the 25-year study period between 1995 and 2020(CEPED UFSC; FAPEU; BANCO MUNDIAL, 2020).

Brazil recorded about 2,500 deaths between the years 1996 and 2016 and Rio de Janeiro is ahead in the statistics of all Brazilian states, with 1,542 deaths caused by landslides, storms and floods (PORTINARI, 2022).

The problems of flooding on public roads and streets are directly related to the lack of micro-drainage structures, causing impacts associated with public order, such as transport and the proliferation of water-borne diseases, in addition to damage to property and disruption to the day-to-day life of the community. city (BONAVITA; FONSECA, 2019) and (SOUZA; OTTONI, 2015).

In general, urban microdrainage systems (SMU) are structures for capturing and conducting and volumes of surface runoff. These systems collect water from buildings, urban lots, squares, streets and conduct it through pipes after runoff through gutters and drains into drains. At the end, the collector tubes lead the runoff to the manholes (PVs), interspersed along the drainage galleries, which in turn flow into rivers and canals.

These structures are susceptible to problems of lack of maintenance, obstructions (trash, tree leaves, deposited sediments and others) and vandalism. Inadequately, most of these pipes also receive the flow of sanitary sewage, which contributes to the problems of diffuse pollution in microdrainage systems. In addition, the disorderly occupation of urban land contributes to increased waterproofing and erosion processes, with a greater frequency and impact of floods in cities.

The increase in the frequency of urban floods has made it increasingly important to incorporate tools that assist in the decision-making process in the management of urban drainage (MELLER; PAIVA, 2007).

In order to avoid flooding and flooding, studies on urban drainage and rainwater management have been widely carried out by the academy, but little discussed within the scope of public policies, with a lack of integrated management between society and power. public (OHNUMA JR, 2008).

With the development of cities and the reduction of green areas, the increase in surface runoff causing problems in urban drainage, the possible solutions are compensatory techniques for managing rainwater (DUTRA JÚNIOR; MENEZES FILHO, 2022).

Cities around the world have adopted complementary approaches to the basic drainage structure, based on concepts such as sponge cities (FOGEIRO, 2019), nature-based solutions (GOMES NÉTO et al., 2020), low-impact solutions (AMANTHEA; NASCIMENTO, 2015), in addition to other methodologies that favor the use of techniques to increase interception, infiltration, and damping. Such devices are preferably located close to the origin and generation of these runoffs, and also provide an improvement in the quality of the water to be released into the receiving water bodies.

Among these techniques, green roofs, rain gardens, detention and retention basins, permeable pavements and the use of damping reservoirs stand out. The capture and storage of rainwater within lots in microreservoirs (RMs) have also been studied as flood damping devices, in addition to constituting an alternative water supply, by providing the replacement of part of the consumption of potable water with the use of stored rainwater for non-potable purposes.

This work aims to evaluate a specific methodology for implementing RMs in urban lots in the neighborhood of Tijuca, in the city of Rio de Janeiro, based on hydrological and hydraulic modeling with the *software Storm Water Management Model* (SWMM, 2012).

The impacts of the drained volumes were evaluated in terms of damping in the microdrainage network, with the implementation of RMs for floods with a 10-year return period.

The differentiated approach of this work is the simulation, mainly the improvement of hydraulic conditions, considering all the singularities of an existing underground drainage network, in the region of Tijuca, RJ, the site has few works with the flood peak attenuation approach with MRs.

METHODOLOGY

THE STUDY AREA

The study area is located in the city of Rio de Janeiro, in the Tijuca neighborhood, in the Joana River sub-basin, belonging to the Mangue channel basin (Figure 1).

The Joana River has its source next to ``Pico do Andaraí,`` in the Grajaú Forest, at an elevation of 600 m, being formed by the rivers Perdido and Jacó. The watercourse of the Joana River runs for a total length of 8.0 km to its mouth on the Maracanã River (PMSB, 2015).

The study region is located at latitude 22º 55” 2.81’ and longitude 43º 14” 7.95” (Figure 1), in Grande Tijuca, on the boundary between Rua Dona Zulmira and Rua Santa Luzia, perpendicular to Rua Felipe Camarão.

The area has residential and commercial occupation, with houses, villas, restaurants and small businesses. With an approximate area of 5 hectares, this region is located between the flood detention reservoirs of Praça Varnhagen and Praça Niterói.

The Tijuca basin was adopted as a study area because it covers an extensive territory, with formal occupation and consolidated urbanization, in addition to presenting several drainage problems with the frequent occurrence of floods and floods that occur in part because of the saturation of systems and networks of microdrainage in the region (OHNUMA JR. et al., 2018), such as the one observed in the event on March 12, 2016 (GLOBO 2016).

The process of urban occupation in Tijuca caused impacts on the environment, bringing changes to the hydrological cycle, related, among others, to: a) reduction of vegetation cover; b) intense soil sealing; c) increase in erosion processes and d) inadequate management of liquid and solid urban waste.

The studied area has a high degree of waterproofing, with 75.4% occupancy (OLIVEIRA; BOTELHO, 2014).

MODELING WITH SWMM

The necessary data were obtained, including the longitudinal profile of the drainage network, and the floor plans and sub-basins referring to the Tijuca study area (Figure 2), according to the drainage network registration of ``Fundação Rio-Águas``, second file number 4-3-D-898 pages 5/10 and

Figure 1– Map of the Joana River sub-basin and the Mangue Canal, in the Tijuca region of the study area, Rio de Janeiro. Source: (OLIVEIRA; BOTELHO, 2014) and (ARAUJO; NEVES; FRAZÃO, 2017) and image from Google Earth Pro - Adapted by the authors.

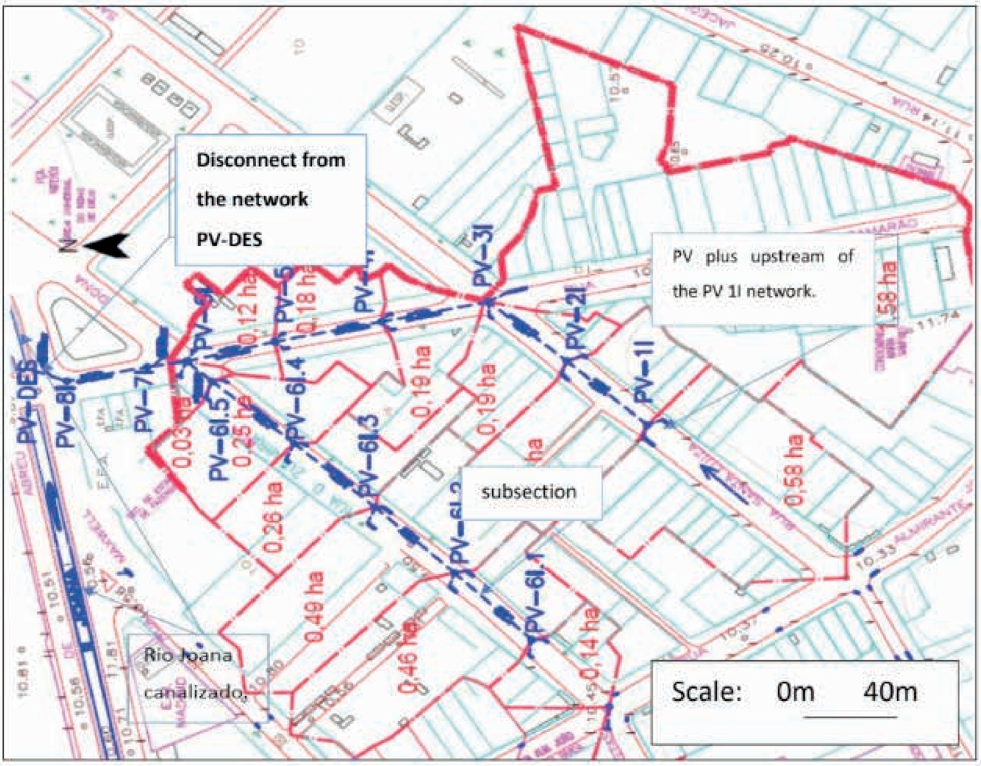

Figure 2 – Basin delimitation plan and contribution sub-basins. Source: Adapted from Fundação Rio Águas (COHIDRO, 2010).

9/10 (COHIDRO, 2010), in June 2010.

These data were used for characterization, contextualization and identification of the existence of flooding points, due to a possible insufficiency of the local micro-drainage system.

It must be noted that Fundação Rio Águas already has a hydraulic and hydrological project/modeling for these networks, for uniform flow conditions, using a model based on the Rational Method, using Excel® spreadsheets.

In the present work, a hydraulic and hydrological modeling was carried out with the aid of the programs *Storm Water Management Model* (SWMM), *Google Earth Pro* e *AutoCad®* for assessing the flow conditions of the current drainage system and identifying problems in the existing drainage network, as well as the flows and volumes of surface runoff.

As a compensatory measure, the installation of RMs in the lots was adopted, according to the selected roof area. The hydrological and hydraulic modeling was carried out from scenarios with and without RMs implanted in the intralots, having as a boundary condition to avoid that the existing galleries work drowned or as an orifice, for a return time of 10 years (RIO-ÁGUAS, 2019).

The precipitation data used in the model were obtained from the rainfall station in Tijuca, with historical series analysis between 1997 and 2021 (ALERTA RIO, 2022).

The modeling adopted in this study contemplates routing with the Saint-Venant kinematic wave, available as a calculation methodology in the SWMM model.

Based on the identification of flooding points, a calibration of the model was carried out, based on evidence of flooding and the height of precipitation in the drainage network, during flooding events.

HYDRAULIC AND HYDROLOGICAL CRITERIA FOR OBTAINING SCENARIOS WITH AND WITHOUT MRS IN BATCHES

The hydrological calculation methodology considers the determination of project flows, defined according to the contribution areas in the drainage network, runoff coefficients and rainfall intensity.

According to the Technical Instructions for the Elaboration of Hydrological Studies and Hydraulic Sizing of Urban Drainage Systems, of the Rio de Janeiro City Hall (RIO-ÁGUAS, 2019), for a micro-drainage network and surface drainage devices, the stormwater galleries must be dimensioned for a recurrence time of 10 years.

Another criterion adopted by RIO-ÁGUAS (2019) used in this work was the filling ratio Y/D, where Y is the tie rod and D is the diameter or height of the gallery, in which Y/D must be less than or equal to 0.85 for circular galleries and 0.90 for rectangular galleries.

The Manning roughness coefficients (n) adopted were: a) 0.013 for galleries made of reinforced concrete tubes or for circular galleries, and b) 0.015 for concrete galleries with wooden formwork or rectangular sections (CHOW, 1998). For permeable stretches, n equal to 0.022 was adopted referring to grassy areas (CHOW, 1998).

According to data obtained from ``Fundação Rio Águas``, the dimensioning of the drainage network in the evaluated location presented flow conditions in the existing galleries that met the design criteria corresponding to the closed galleries. Of the total, only 5 (five) stretches lacked the necessary free edge, with a Y/D ratio <= 0.90 (less than or equal to 0.90).

Among all networks with records available at Fundação Rio Águas for the adopted area, network "I" (Figure 2) was the one with the most stretches close to the limits of the filling criterion Y/D <= 0.90. As a result of these data, referring to the stretches at the limit of the maximum depth criterion, the selection of network "I" was determined as a case study.

HYDRAULIC AND HYDROLOGICAL MODELING

The topographic data of the region and of the microdrainage systems used in the modeling were provided by Fundação Rio Águas based on projects prepared in June 2010.

The rainfall equation for Posto 30 - Tijuca, in the Tijuca neighborhood, was used (BRAGA et al., 2018).

For the region studied, a runoff coefficient of 0.80 was considered, a value close to the average of commercial areas and similar to that adopted by RIO-ÁGUAS (2019).

An average slope of 1% of the microdrainage sub-basin was considered, corresponding to the areas of contribution of the basin studied according to Table 1.

The infiltration model used in the SWMM used the number curve (CN) equal to 80, with average antecedent humidity and hydrological group A, composed of sandy soils with low total clay content, less than 8%, with no rock or clayey layers, and not even densified to a depth of 1.5 m (CHOW, 1993).

The SWMM model has three types of flow propagation: a) the uniform regime; b) kinematic wave and c) dynamic wave (ROSSMAN, 2015). For the present study, the dynamic wave model was adopted.

CARGO LOSSES

The hydraulic premises of the network were adopted according to consolidated bibliographies (PORTO, 2006 and AZEVEDO NETTO; FERNÁNDEZ, 2015).

The head losses at the entrance and exit of the galleries were estimated according to the express methodology, according to the SWMM manual (ROSSMAN, 2015), in which equation (1) represents the localized head loss.

$$\Delta h = \sum k \frac{v^2}{2g} \tag{1}$$

Where: Δh – head loss in (m); v – average speed in the section (m/s); k – head loss coefficient (dimensionless); g - acceleration due to gravity (9,81 m/s²).

The SWMM manual proposes several values of k for head losses at the input. Due to the lack of knowledge of the geometric details of the manholes (PV), k equal to 0.2 was adopted, referring to concrete pipes with an adaptation section by lateral narrowing or height narrowing (ROSSMAN, [s.d.]).

The head loss coefficient at the exit of the galleries is suggested by the SWMM model as being equal to 1 (one), but this value is considered high due to the transitions between the PV's with upstream and downstream sections appearing less abrupt, generating k values less than 1.

Thus, the output load losses were estimated according to the methodology *Hydraulic Desing Criteria* (HDC, 1987), in order to increase the accuracy of the calculations in the hydrodynamic model.

The head loss coefficients at the exit of the galleries were defined according to equation (2).

$$k = \left[1 - \frac{A_1}{A_2}\right]^2 \tag{2}$$

Onde: k – Coeficiente de perda de carga por expansão (adimensional); A_1 – área da seção da galeria de montante (m²); A_2 – área da seção da galeria de jusante (m²).

The outflow has as an initial downstream contour condition the normal water level throughout the projected system, with the calculation of the risers in the gallery from the backwater and the propagation of the control flow at the exit of the gallery.

Sub-Basin Number	PV receiver	Drainage area (ha)	Waterproofing rate (%)	Basin axial length (m)	Sub-basin slope (%)
1	PV-1I	0,58	80	112,00	1
2	PV-2I	0,36	80	101,25	1
3	PV-3I	0,19	80	62,00	1
4	PV-4I	0,19	80	70,00	1
5	PV-3I	1,58	80	150,00	1
6	PV-5I	0,18	80	62,50	1
8	PV-6I	0,12	80	62,50	1
9	PV-6I	0,03	80	43,00	1
10	PV-6I.5	0,25	80	57,70	1
11	PV-6I.4	0,26	80	72,70	1
12	PV-6I.3	0,49	80	125,00	1
13	PV-6I.2	0,46	80	117,00	1
14	PV-6I.1	0,14	80	60,00	1

Table 1– Subbasin input data in the SWMM model

Source: Elaborated by the authors.

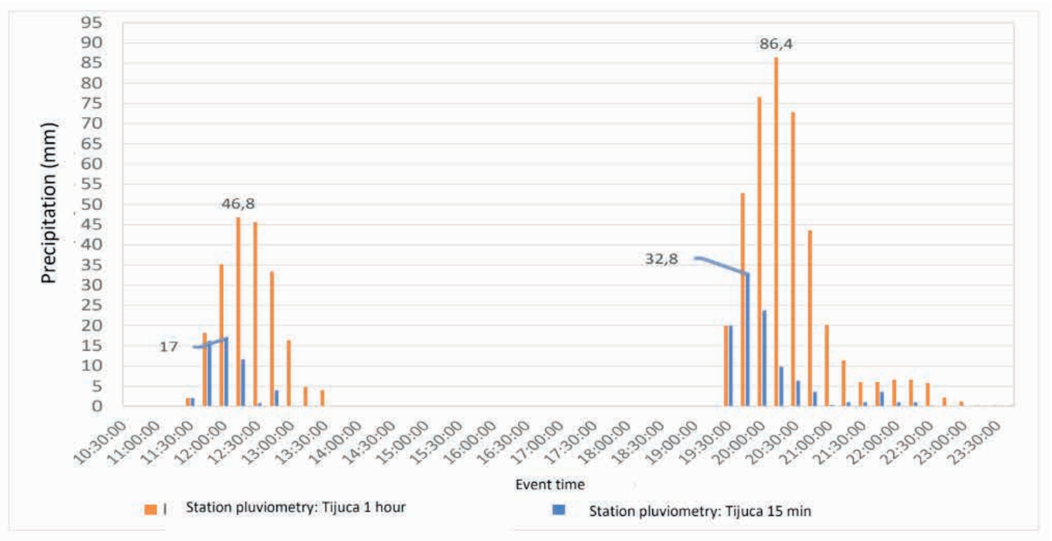

Figure 3 – Graph of the precipitation height that occurred on 03/12/2016 every 15 min and 1 hour.

Source: (ALERTA RIO, 2022)

CALIBRATION OF HYDRAULIC AND HYDROLOGICAL MODELING WITH SWMM IN THE STUDY AREA

For modeling validation, its calibration is of fundamental importance. The existing microdrainage network in the Tijuca region studied is underground, which makes it difficult to calibrate it with surface runoff flow velocity measurement devices.

The occurrence of a specific rainfall and flooding event in the region was adopted as a measure for adjusting the model parameters, based on data from the news and precipitation records obtained from the Tijuca rainfall station.

It must be noted that even after interventions to build large reservoirs to dampen floods in Greater Tijuca, the studied region still has drainage problems.

Subdaily events were observed, with impacts on the microdrainage network on March 12, 2016: rain of 17 mm in 15 minutes and 46.8 mm between 11:30 and 12:30 (Figure 3); 32.8 mm of rainfall in 15 minutes with about a 10-year recurrence period; and 86.4 mm of rain between 7:15 pm and 8:15 pm with about 23 years of recurrence period.

The nocturnal subdaily events presented flooding in the region, especially near Rua Dona Maria, upstream of the study area, as reported in the newspaper RJ, 1st Edition, of March 12, 2016 (GRAEL, 2016).

Located upstream of the study area and close to the outflow of the studied drainage network, and having the capacity to store up to 58,000 m^3, the Praça Niterói reservoir was already in operation.

According to the rainfall data presented, the calibrations of the SWMM model were carried out for the studied region. For the calibration, it was necessary to change the parameters of the SWMM model, in the initial modeling, according to the data obtained by the Rio Águas model in the existing network.

In order to calibrate the model, the maximum runoff coefficient for the region was increased from 80% to 90% depending on the asphalt and concrete surface, as it is a highly urbanized region, with characteristics of dense residential occupation (RIO WATERS, 2019).

The analyzed streets had their runoff coefficients altered, according to the treetop areas present in the sub-basins. For example, Rua Felipe Camarão had a small amount of tree area (1011 m^2 sub-basin area in the 20600 m^2 section); a waterproofing coefficient of 90% was adopted, while Rua Santa Luzia had an average amount (1752 m^2 sub-basin area in the 11300 m^2 stretch), adopting 85% waterproofing.

For Rua Dona Zulmira, which had a large number of trees (2484m^2 sub-basin area in the 16000 m^2 stretch), a waterproofing coefficient of 80% was adopted. The trees on the streets presented an area consisting of a greater amount of trees and larger crowns. The trees within the lots were disregarded due to their small quantity and size.

With the data adjusted for the runoff coefficient, the modeling of the sections was carried out, showing the overflow of PV's 1I and 2I upstream on Rua Santa Luzia, close to the flooding of March 12, 2016 (Figure 4), due to heavy rains in the region in this day.

On a visit to the study region, flood spots were observed on the walls of the houses in the order of 30 cm in relation to the sidewalks with dimensions similar to or equal to the PVs, and that most of the houses had their thresholds above the level of the sidewalk, the which justifies the region susceptible to flooding.

In order to calibrate the model, it was decided to adopt, in the same event, a hydraulic load of 30 cm in PV 1I, in order to characterize that the flooded street obtained this quota, since no records of flood spots were obtained. in the studied area.

For the simulation, an overload on the water level was adopted in PV1I (elevation El 10.10 m), in the order of 30 cm (N.A=El 10.40), and the simulations without and with calibration were compared (NA = El 10.02m), noting that flooding lasted about 1 hour.

The graph in Figure 4 shows the result of the simulations.

With the help of Google Earth Pro (2021) and AutoCad® (Auto Desk - 2021) *software*, a survey of the roof areas in the study area was carried out (Figure 5).

146 roof areas in lots were selected, with a total of 19,311 m² of roof area. These areas were deducted from the areas of the sub-basins in the stretches studied. Roofs smaller than 50 m² were discarded due to their reduced quantity and respective contribution to the sub-basin.

Based on the survey of the roof areas, the flow to be collected was calculated using the Rational Method.

The contribution areas with values lower than 2 km² (TUCCI, 2009), the Rational Method is considered for determining the contribution flows referring to the roofs, according to equation (3).

$$Q = 2{,}77810^{-4}CIA \qquad (3)$$

Where: Q = flow, in (l/s); C = runoff coefficient (dimensionless) for roofs adopted 0.90; I = rainfall intensity, in (mm/h); A = area of the roof, in (m²).

The “reservoir” cell of the SWMM program can receive a hydrograph or flow as inflows.

In this work, it was decided to insert the flow as inflow in the reservoir cell so that the intensity of rain in the network is in the same time interval in all RMs, from the transformation of the model rain *versus* flow.

The dimension versus area ratio was used for a 5 m³ microreservoir (MR) model (Table 2).

A simulation was carried out with this reservoir in order to obtain the volumes of storage and use of rain throughout the network.

With the purpose of restoring the flow network exceeding the capacity of the MR, an outlet with a diameter of 0.15 m was designed in the upper part of the reservoir, with its generatrix less than 1.12 m from the bottom (Figure 6), with the flow calculated according to equation (4).

Due to the positioning of the orifice with its generator less than 1.12 m in the 5 m³ or 5,000 liters reservoir, the accumulated volume for use of rain at the maximum MR in the modeling is 4,413 m³ or 4,413 liters.

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

Where: Q = orifice flow, in (m³/s or Qx1000 [l/s]); Cd= discharge coefficient = adopted 0.65, dimensionless; A = orifice area, in (m²); h = difference in heights through the hole, between the axis and the manhole in the network (m).

According to PORTO (2006), for an orifice of 0.15 m in diameter, with a concrete piping length L equal to 6 m and sharp edges, a discharge coefficient equal to 0.64 can be adopted. However, a more conservative coefficient of 0.65 was adopted due to the PVC piping and sharp edges, also considering the uncertainties throughout the network about the actual positioning of the reservoirs in each lot.

The RMs overflow into the existing drainage networks when they reach volumes above 4413 liters and when the flow inflow exceeds the accumulation capacity. In the present hydraulic modeling, the RMs have orifice and geometry, so as to be located high, with their bottom elevation above the upper elevation of the PV, making it impossible for the water levels in the drainage network to drown the outlet of the orifices, avoiding

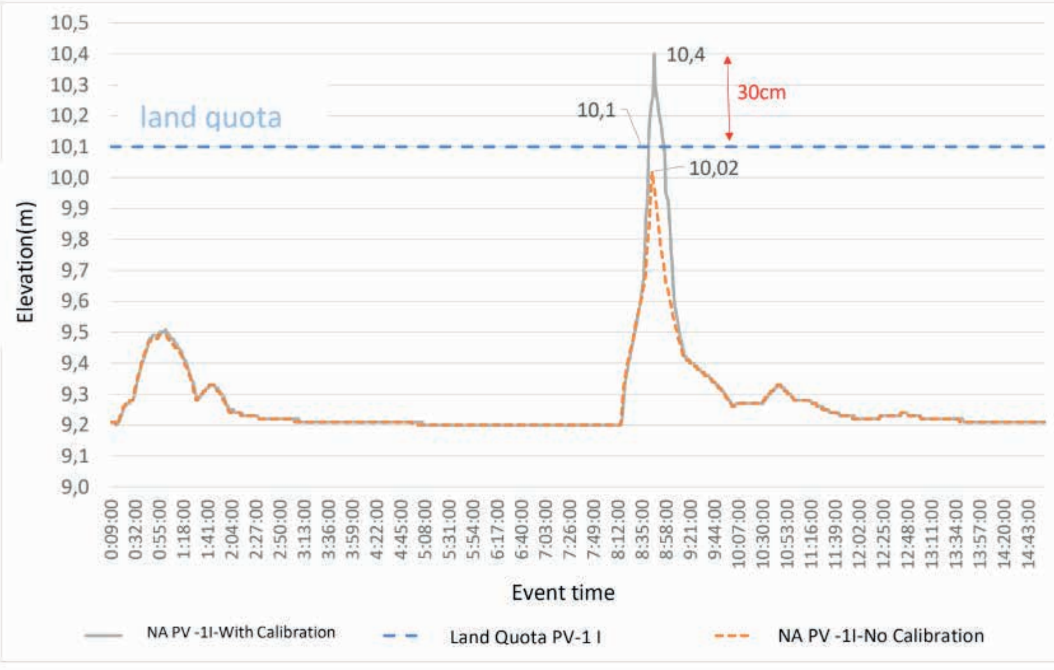

Figure 4 – Calibration of the SWMM model with the overflow of PV 1I for the rain of 03/12/2016. Source: Authors

Figure 5 – Roof areas in the Tijuca sub-basin region, Rio de Janeiro. Legend: Roof areas in yellow. Source: Based on Image from Google Earth Pro.

Depth (m)	Area (m^2)	Volume (m^3)
0,00	3,94	0,00
1,00	3,94	3,94
1,12	3,94	4,41
1,27	3,94	5,00

Table 2- Ratio depth versus area and volume of the RM

Source: Authors.

controls downstream hydraulic systems and making it possible to carry out damping in the sub-basin.

RESULTS AND DISCUSSIONS

For the damping alternative in the drainage network, highlighting the 13 existing manholes (PV), the implementation of 146 MRs upstream of the PV's was evaluated, based on modeling with SWMM (Figure 7), in the diagrams without and with the MRs, respectively

From the hydraulic modeling, there is a reduction in water levels, in the filling of the network and in the PV's in the simulation with the implementation of intralot RMs, as shown in Figure 8.

The results indicate a 31% reduction in the affluent levels to the microdrainage network, considering the implementation of intralot RMs in the study area as a compensatory measure for the effects of urbanization in compliance with the design criteria for a 10-year TR.

DEPTHS AND FLOWS IN EXISTING GALLERIES

According to the criteria of RIO-ÁGUAS (2019), in studies of the impacts of RMs on the micro-drainage of the Tijuca sub-basin, the recurrence adopted was 10 years (for a minimum time of 10 min, intensity of 145.46mm/ h and precipitation height of 24.24mm and maximum time of 20 min, 117.69mm/h and precipitation height of 39.25mm (BRAGA et al., 2018)), also considering for closed galleries the Y/D ratio not greater than 0.90 (90% filling) for rectangular sections and 0.85 for circular sections.

Figure 9 presents the comparative quotagram (left) and the comparative hydrograph (right), of the upstream and outflow stretch (last stretch of the network), of the studied network, with and without the use of RMs.

Figure 10 shows the maximum depths of water lines in the studied stretches, with and without the use of RMs

The depth results obtained with and without RMs demonstrate that with the addition of RMs in the drainage system of the study area, there is a significant reduction in the depth of the drainage waterline along the entire piping system, contributing to the effective compliance with the criterion of depth Y/D less than or equal to 0.90 for rectangular galleries and Y/D less than or equal to 0.85 for galleries with circular sections.

Section 6I.4-6I.5 did not meet the Y/D criterion less than or equal to 0.90 with a Y/D ratio equal to 0.983 (RIO-ÁGUAS, 2019). The results expressed with the addition of the MRs captured by roofs in the drainage system of the study area show a significant average depth reduction in the entire system of about 31%.

To evaluate the attenuation of the flood peak for a TR of 10 years of recurrence in the galleries with the implantation of RMs and the damping of the flow, the results are shown in Figure 11.

The results obtained with the use of intralot RMs in the study area show an average flow reduction of about 37%. in the whole system

For rainfall with a return period of 20 years, the studies by (FRANCISCHET, 2012) showed efficiencies of 34.0%, 20.19% and 15.47%, adopting reservoirs with a capacity of 5.015 m³ 3.25 m³ and 1.50 m³, respectively, that is, with reservoir size and percentages compatible with those obtained in this research.

VOLUMES STORED IN MRS

The resulting total roof area was 19,380 m², with a flood damping volume of 351,639 liters or 351,639 m³, while the volume below the generatrix lower than the overflow hole,

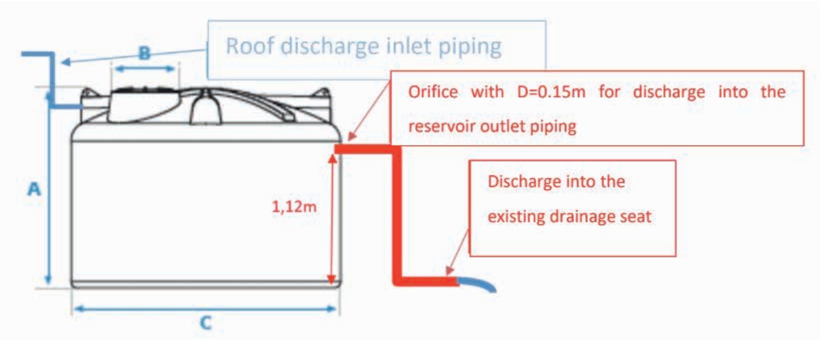

Figure 6 – Schematic sketch of the positioning of the hole in the MR
Source: Adapted (FORTLEV, 2022).

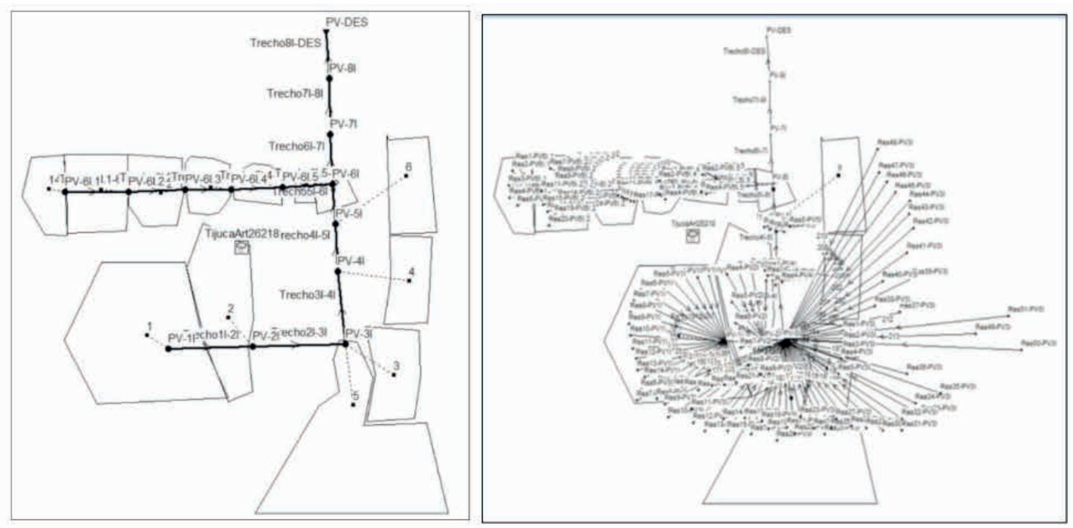

Diagram in SWMM without MRs (on the left) and with the 146 MRs (on the right). Source: Authors.

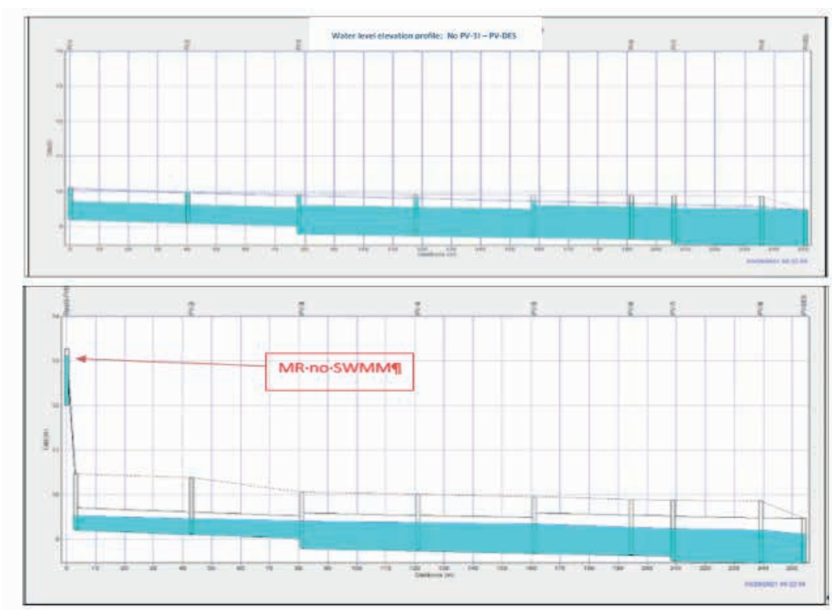

Figure 8 - Waterline profile (SWMM), for section PV-1I to PV-DES, without MRs (above) and with MRs (below). Source: Authors

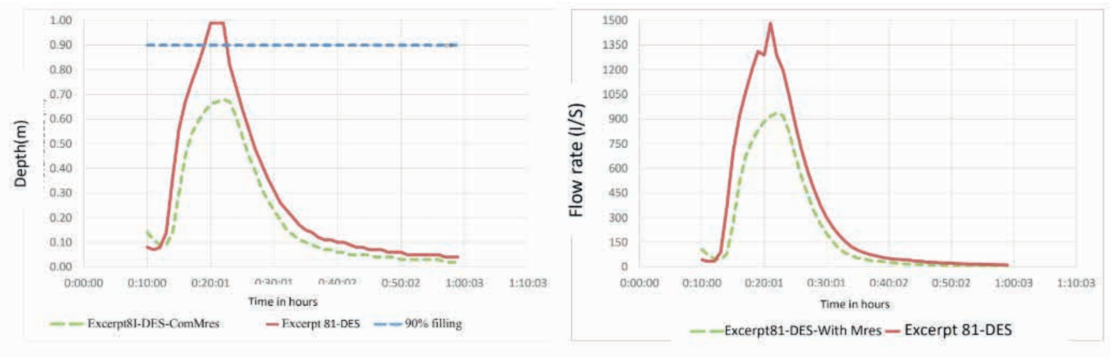

Figure 9- Waterline depth (on the left) and flow (on the right) of the upstream section of the network and discharge in the downstream section with (dashed line) and without MRs (solid line). Source: Authors

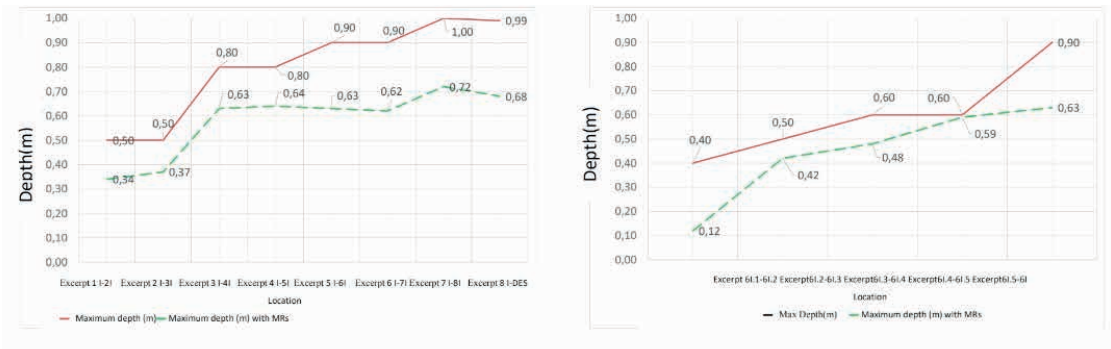

Figure 10– Maximum waterline depths in the network by section with (dashed line) and without MRs (solid line). Source: Authors

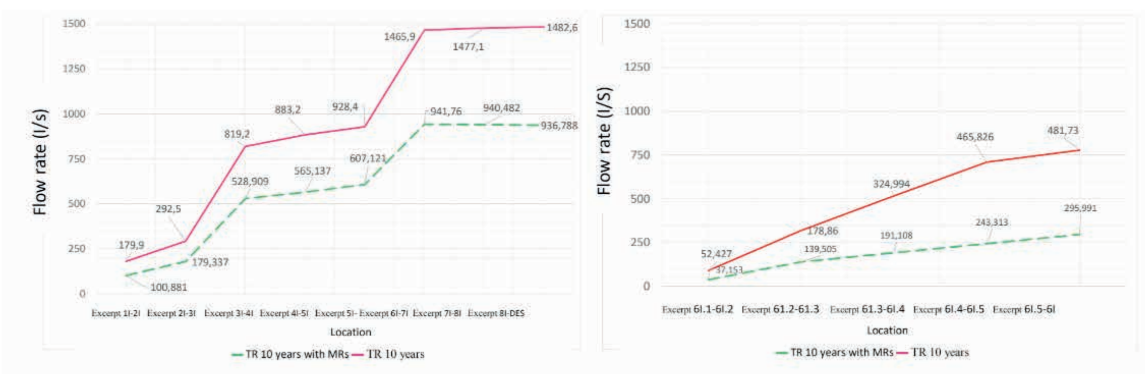

Figure 11 – Maximum flows with (dashed line) and without (solid line) MRs from PV 6I.1 to PV-6I (left) and PV 1I to PV-Outflow (right). Source: Authors

referring to rainwater for use, was 346,319 liters or 346,319 m³ of water.

The tracing of the curve representing the area versus volume ratio is configured in a simplified methodology for estimating the volumes captured by the roofs in relation to those of the RMs.

Roofs with areas greater than 230 m², the reservoirs had a volume of 5,000 liters. Thus, for lots with roof areas between 230 m² and 610 m² (Figure 12), the adoption of reservoirs with a capacity of 5,000 liters or more is suggested, as the affluent volumes exceed 4413 liters of accumulation, with excess flow poured into the drainage network.

The relationship between the roof area and the volume of the reservoir considers the local rainfall equation and the constraints of the implemented drainage network.

The rainwater harvesting system implemented in a single-family home in the Sulacap neighborhood, Rio de Janeiro, demonstrated a water supply of approximately one-third (32.6%) of all household water demand (OLIVEIRA, FLÁVIO GIRO DE., 2020).

It is estimated that rainwater harvesting can provide from 12% to 100% of the volume of water needed to maintain a family (MUSAYEV; BURGESS; MELLOR, 2018).

These factors show studies carried out with rainwater harvesting systems for various purposes, and that the implementation of RMs in urban lots can mitigate flood peaks, in addition to proportional the use of rainwater used for non-potable purposes.

CONCLUSION

Based on the results of the hydraulic and hydrological modeling with the SWMM model, the technical viability of the implantation of MRs intralots (MR) for the damping and use of rainwater in an urban sub-basin in the region of Tijuca, Rio de Janeiro was evaluated. – RJ.

The implantation of intralots RM can help in the attenuation of short-term floods, which normally cause damage in microdrainage networks and the hydrological and hydraulic modeling obtained satisfactory results with the propagation of dynamic wave flow, which is not normally used in microdrainage networks. This form of calculation considers the hydraulic backwater throughout the network and identifies hydraulic controls that are difficult to perceive with uniform flow.

The simulated RMs proved to be efficient and attenuated the 10-year flood peak of recurrence satisfactorily, with a significant reduction in the average peak flow of 37% and in depth and with a 31% reduction in the entire drainage system;

The analysis of the volumes for commercial reservoirs highlights the ease of implantation of RM in function of the roof area.

Among the work restrictions, the following stand out: a) all basins were adopted with 1% slope; b) uncertainty of guaranteeing that all the reservoirs will be empty when the flood needs to be attenuated; c) lack of emptying rate of the RMs; d) Given the difficulty of measuring flow in underground drainage networks, a calibration was adopted that requires greater adjustments based on more compatible methods such as algorithms using multiobjective evolutionary algorithms (FORMIGA et al., 2016).

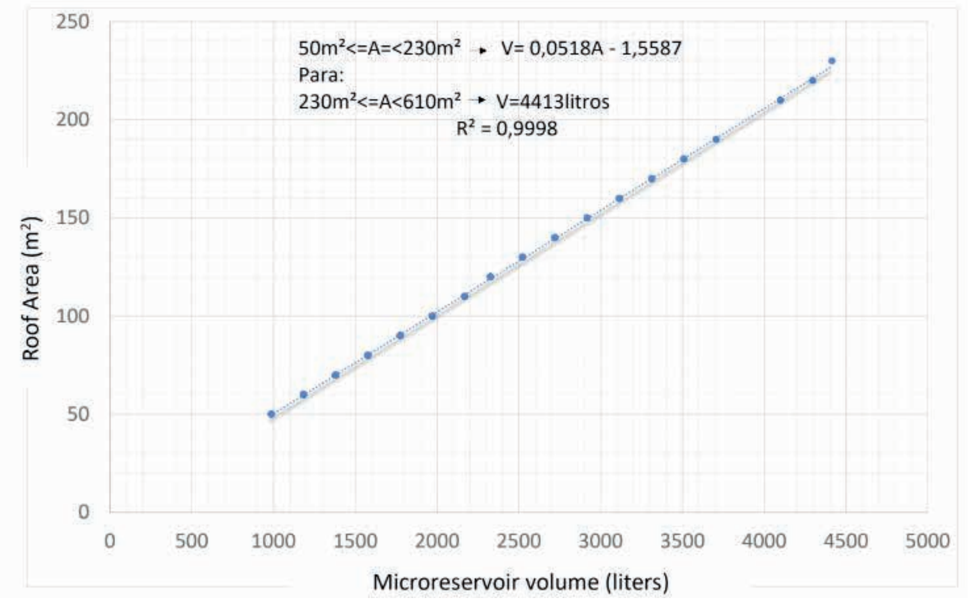

Figure 12 – Volumes (V) of MRs (liters) as a function of roof area (A) (m²).

Source: Authors

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