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Compensatory alternatives for flooding control in urban areas with tidal influence in Recife - PE

Alternativas compensatórias para controle de alagamentos em área urbana com influência das marés no Recife - PE

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ABSTRACT

This paper presents a study of compensatory alternatives in urban drainage, using SWMM model (Storm Water Management Model), for the critical point of flooding in an urban area and vulnerable to tide fluctuations, located in Recife. For this, we used the registered information of the micro-drainage network and defined the parameters and variables required for modeling, such as: the subareas of contribution to the drainage system, indicating the percentage of soil waterproofing, equivalent width, slope, and infiltration rate; project rain; and tide curve. Two alternatives were simulated after the model has been calibrated. The first, which is an adaptation of the drainage network, presented maximum reductions in the volume of flooding of 37% for the events with recurrence period of two years and of 58% for five years of recurrence. The second, based on the deployment of a detention tank in the existing network, presented satisfactory results for the event of two years and reduced approximately 38% for events of five years. The results showed that there was a reduction in the area of flooding for the conditions simulated. However, the first alternative would not solve the local flooding problems, it would only attenuate and would increase the overload of the drainage pipes downstream of the modified system, while the second alternative could solve the problem of flooding, with the occurrence of an event of two years.

Keywords: Urbanization; Hydrological model (SWMM); Flooding control; Compensatory alternatives.

RESUMO

Este trabalho apresenta um estudo de alternativas compensatórias em drenagem urbana, utilizando o modelo SWMM (Storm Water Management Model), para o ponto crítico de alagamento numa área urbana e vulnerável as oscilações de maré, situada no Recife. Para isto, utilizaram-se informações cadastrais da rede de microdrenagem e definiram-se os parâmetros e variáveis necessários para a modelagem, tais como: subáreas de contribuição ao sistema de drenagem, indicando o percentual de impermeabilização do solo, largura equivalente, declividade e taxa de infiltração; chuva de projeto; e curva de maré. Com o modelo calibrado, foram simuladas duas condições alternativas. A primeira, que trata de uma adequação da rede de drenagem, apresentou reduções máximas no volume de alagamento de 37% para os eventos com período de retorno de dois anos e de 58% para cinco anos de recorrência. A segunda, baseada na implantação de um reservatório de detenção na rede existente, apresentou resultado satisfatório para o evento de dois anos e reduziu cerca de 38% para eventos de cinco anos. Os resultados mostraram que houve uma redução na área de alagamento para as condições simuladas. No entanto, a primeira alternativa não resolveria os problemas de alagamentos locais, apenas atenuaria e aumentaria a sobrecarga dos condutos a jusante do sistema modificado, enquanto que a segunda alternativa poderia resolver o problema dos alagamentos, com a ocorrência de um evento de dois anos.

Palavras-chave: Urbanização; Modelo hidrológico (SWMM); Controle de alagamentos; Alternativas compensatórias.

INTRODUCTION

The accelerated, unplanned urbanization process of Brazilian municipalities has brought major changes to the urban environment reflected as significant impacts on urban drainage.

Yannopoulos et al. (2013) emphasizes that this process generates increased and accelerated surface run-off of rainfall waters in addition to decreasing the infiltration capacity of water in the soil causing flooding.

This situation is more serious especially on coastal lowlands, as in the case of the city of Recife, where the urban occupation process was unorganized having resulted in a drainage system highly vulnerable to tidal oscillations, which may cause serious flooding scenarios along intensive rainfall periods combined with high tide (SILVA JÚNIOR, 2015).

In the context of such geographical and urban peculiarities in which the city was conceived, Recife currently has 159 flooding points, classified as the most critical ones (EMLURB, 2013).

The inclusion of the sustainability concept as a urban drainage theme has revealed compensatory alternatives aiming at systematically compensating the effects of urbanization on urban waters by retrieving natural hydrologic functions (MIGUÉZ et al., 2014).

The objective of applying such measures is to adjust hydrologic cycle processes through infiltration and storage of the run-off generated in order to increase the concentration time of the basin and decrease the outflow peaks generated by a rainfall event (SILVA; CABRAL, 2014).

Among the measures of run-off control, Canholi (2005) highlights the use of detention pond to mitigate flow peaks. The author emphasizes the efficiency of these reservation works in relation to conventional drainage solutions, especially regarding not transferring flooding downstream, which certainly would occur in case the solution had been only replacing the network of galleries by another of a larger dimension.

A way to estimate the performance of these mitigating solutions before their actual implementation is by using hydrologic and hydraulic models, from which it is possible to simulate hydrologic events to assess the consequences of applying these control measures (DECINA; BRANDÃO, 2016). In this context, this paper presents compensatory alternatives to a strongly urbanized area with flooding issues locate in the neighborhood of Soledade, Recife.

For this purpose, our study on these alternatives was based on the results pointed out using the SWMM model (Storm Water Management Model), which verified two possibilities for intervention in the local drainage network: adjustment of the drainage network and implementation of a detention reservoir.

MATERIAL AND METHODS

Case study

The critical flooding point, located at the crossing of João de Barros Avenue with Joaquim Felipe Street, has been chosen in a joint effort with EMLURB (public company in charge of Recife drainage system).

In order to consolidade the study, it was necessary to analyze the increase in the run-off volume caused by urbanization, upstream to the critical point in question. Therefore, other surrounding neighborhoods (Boa Vista and Santo Amaro) were considered in the analysis.

Figure 1 illustrates an area of contribution to the flooding point and the coverage area of the study, defined from the drainage system belonging to the main system of the studied area up to its discharge at Capibaribe River.

The area of contribution to the flooding point has approximately 4.68 ha with a perimeter of nearly 1,086 m; the coverage area of the study has 38.36 ha with a perimeter of 4,893 meters and a compactness index (Kc) of 2.23.

Along the rainy periods, the flooding occurrences in the Soledade neighborhood concentrate in parts of João de Barros Avenue, more specifically next to Joaquim Felipe Street (Figure 2).

Hydrologic model

In order to simulate the hydrologic responses of the area of study, we used the SWMM v.5 model, developed by U.S EPA (United States Environmental Protection Agency) in 1971 (PROCEL-SANEAR, 2012). The SWMM is a rainfall-flow dynamics model used for urban drainage management that simulates both the amount and the quality of the surface run-off, especially in urban areas. It can be used to simulate a single rainfall event continuously and on short-term as well as for wastewaters drainage (JIANG et al., 2015).

The surface run-off simulation considers that the subareas behave as non-linear reservoirs from the combination of Manning and continuity equations. The run-off propagation in the drainage network, in turn, is calculated by complete Saint Venant equations through modified Euler method (GARCIA; PAIVA, 2006).

The model is divided in many different computational blocks that can be simulated separately. Among these, for the

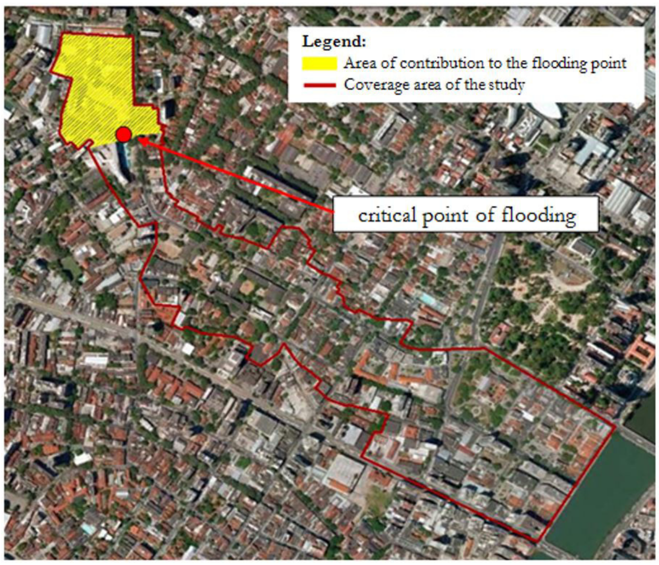

Figure 1. Area of contribution to the flooding point and area of study coverage.

modelling process of the area of study, we simulate the following blocks: "Runoff" in the transformation of the rainfall into flow; "Transport", which simulated the transport in the network of galleries according to the concept of kinematic wave; "Extran", for the hydrodynamic modelling of the conduits (network of galleries), and "Statistics", which separated the record in hydrogram for independent rainfall events in addition to conducting statistical calculation and frequency analyses.

Parameters and variables for model input

The parameters required for the hydrologic rainfall-flow simulation of the study area, in addition to infrastructure register information of the existing drainage, are the physical characteristics of each contributing subarea to this drainage system (area, percentage of the permeable and the impermeable area, representative width, declivity, manning's roughness coefficient, and depression storage level).

Furthermore, variables such as precipitation, infiltration and tide were also considered in this process, which are detailed below.

- Parameters:
  - Register of the microdrainage: The register of the drainage system of the rainfall waters of the area of study was granted in digital media by EMLURB.

The abovementioned register had to be complemented with the information of the cadaster plan of the microdrainage system of Recife, carried out by the *Companhia Pernambucana de Saneamento* – Sanitation Company of Pernambuco (Compesa) in the 1980s, also granted by the institution in printed media on a scale of 1:1.000. In addition, the technical team of Emlurb conducted complementary topographic surveys at some manholes in order to improve the data granted.

- Physical characteristics of the subareas: We carried out the subdivisions of each contribution area to each manhole considered in the model, numbering 34 subareas.

After this process and once we obtained the area, equivalent width, declivity and impermeabilization rate of each subarea, other aspects were defined for inclusion in the model, such as the manning's roughness coefficient (permeable areas: 0.15 $s/m^{1/3}$/ impermeable areas: 0.024 $s/m^{1/3}$) and storage height in depressions (permeable areas: 5 mm/ impermeable areas: 2.54 mm).

- Variables:
  - Rainfall: We used hourly precipitation data of the automatic station RECIFE-A301 operated by Inmet, located at radius of nearly 6 km of distance from the area studied.

For the hydrologic simulations, we considered the rainfall events from June 25 and 26, 2014 and May 17, 2013. The objective of including the latter event was to adjust the model to the local in study providing support to its calibration. Table 1 presents the characteristics of the simulated events, while Figure 3 illustrates the histograms of the respective rainfall events.

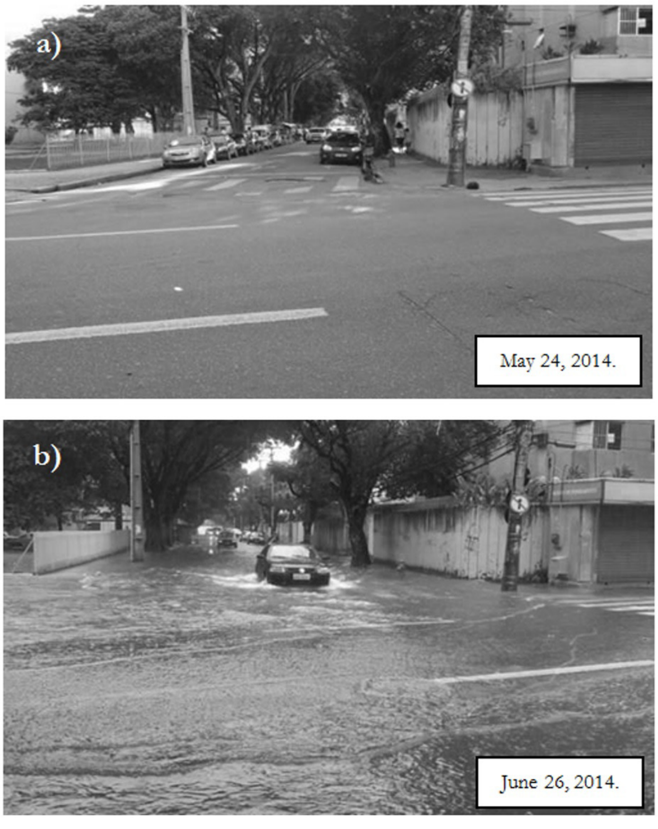

Figure 2. Joaquim Felipe Street in the dry season (a) and in the rainy season (b). Source: Silva Júnior (2015).

Table 1. Synthesis of simulated events.

Date of event	Duration (h)	Rainfall (mm)	TR calculated
June 25-26, 2014.	20	107.60	2 years
May 17, 2013.	13	150.80	5 years

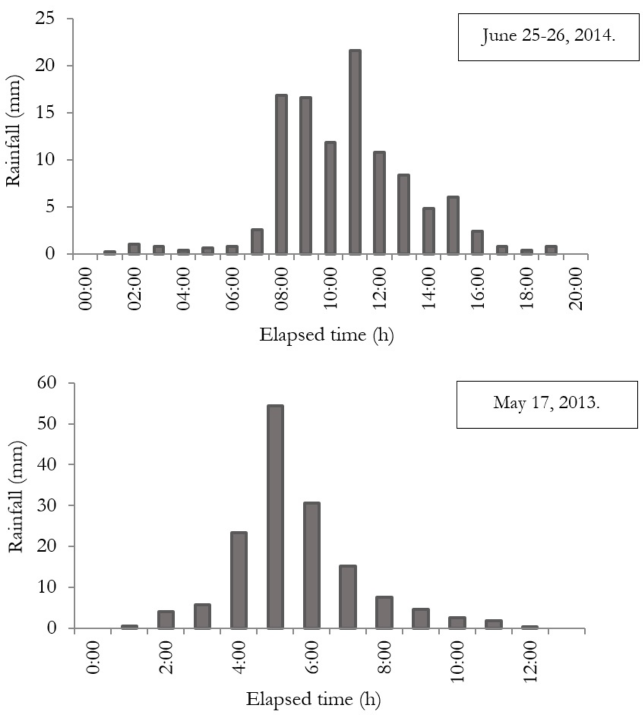

Figure 3. Rainfall charts for the simulated events.

- Infiltration: The SWMM considered the Horton infiltration model for infiltration determination in permeable areas. The parameters inherent to this infiltration model were obtained from infiltration assays using infiltrometer with a single ring. The assays were carried out in a parcel of the soil without vegetation at an area close to the flooding point in study from five points distributed in the shape of a trapezium (Figure 4).

Despite the proximity among the five points tested, the resulting curves of the infiltration assays presented different behaviors, according to the chart in Figure 5. This can be associated with the compaction of the soil in the points investigated as well as with the conditions of initial moisture of the assay.

Facing this conclusion, we decided to use the curve of infiltration capacity with the best adjustment based on the correlation index (R^2). The value of the determination coefficient here described states only how much the curve was able to represent the points. Thus, according to this criterion, we verified that the infiltration capacity curve of Point 1 had a correlation index closer to one (R^2=0.9808), as illustrated in Figure 5.

Based on the infiltration capacity curve equation for Point 1, adjusted to the Horton equation, it was possible to obtain the parameters required to feed the model: the maximum and minimum infiltration rates (467.56 mm/h and 62.79 mm/h, respectively) and the decay coefficient (3.282 h^{-1}).

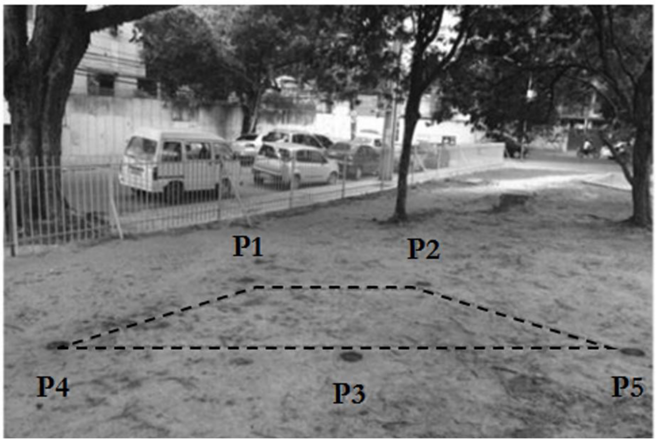

Figure 4. Distribution of the points tested.

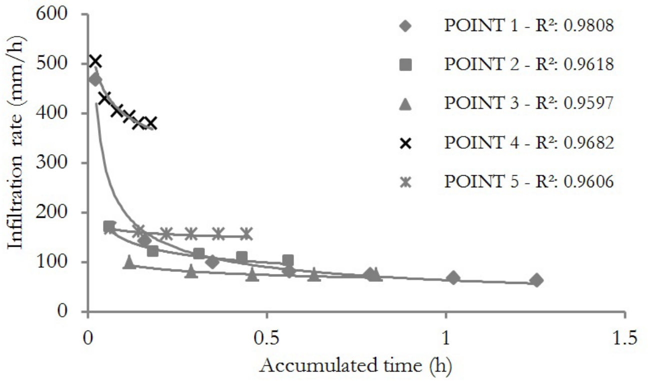

Figure 5. Curves of infiltration capacity in the five points tested and correlation indices.

- Tide: Applied as return condition in the "Discharge" junction, the tidal levels were obtained using time interpolation of the tidal records for the Recife Harbor, made available on the website of the *Diretoria de Hidrografia e Navegação* – DHN – Board of Hydrography and Navigation (Figure 6).

Assembly of the network and calibration

After obtaining the register of the rainfall waters system, the drainage network was included in the SWMM with the identification of the objects: Joints/Nodes and Conduits attributed with topographic quotes and extensions of the existing elements of drainage.

The focus of the mesh of the drainage system in the implemented model was mainly the interventions in the main pipeline of the system (trunk network), responsible for the drainage of the rainfall waters in the area of study.

Figure 7 illustrates the model layout based on the aforementioned information, representing the scheme of the drainage network implemented in the SWMM considering the Emlurb register.

After including the input parameters and variables in the model, we carried out the model calibration based on the extreme precipitation events occurred in June 25-26, 2014 and May 17, 2013, thus constituting Scenario 01 (Current Condition).

In the absence of level sensors and measurers for flow in the area studied, necessary to process a sensitivity study, calibration and validation of the parameters, we decided to conduct a simplified calibration of the model, based exclusively on the adjustment of the simulated flooding volume to that observed during the occurrence of the considered events, only at the control point.

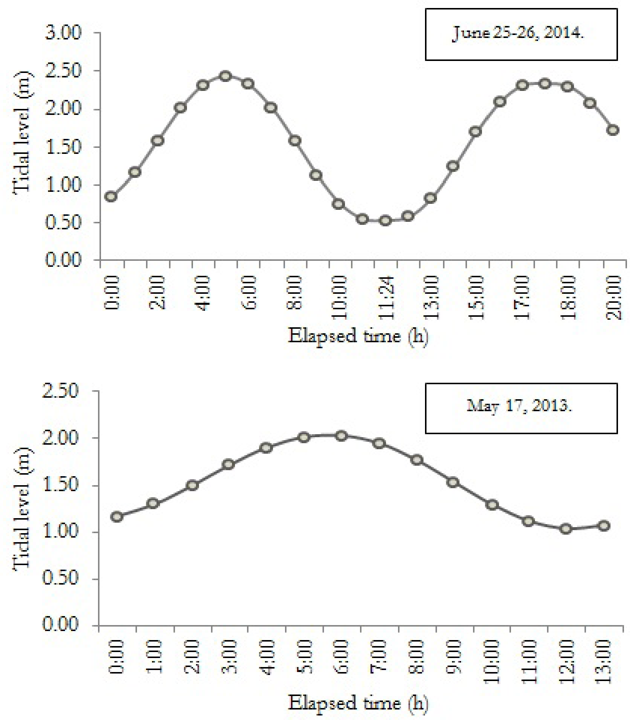

Figure 6. Tidal level for the simulated events.

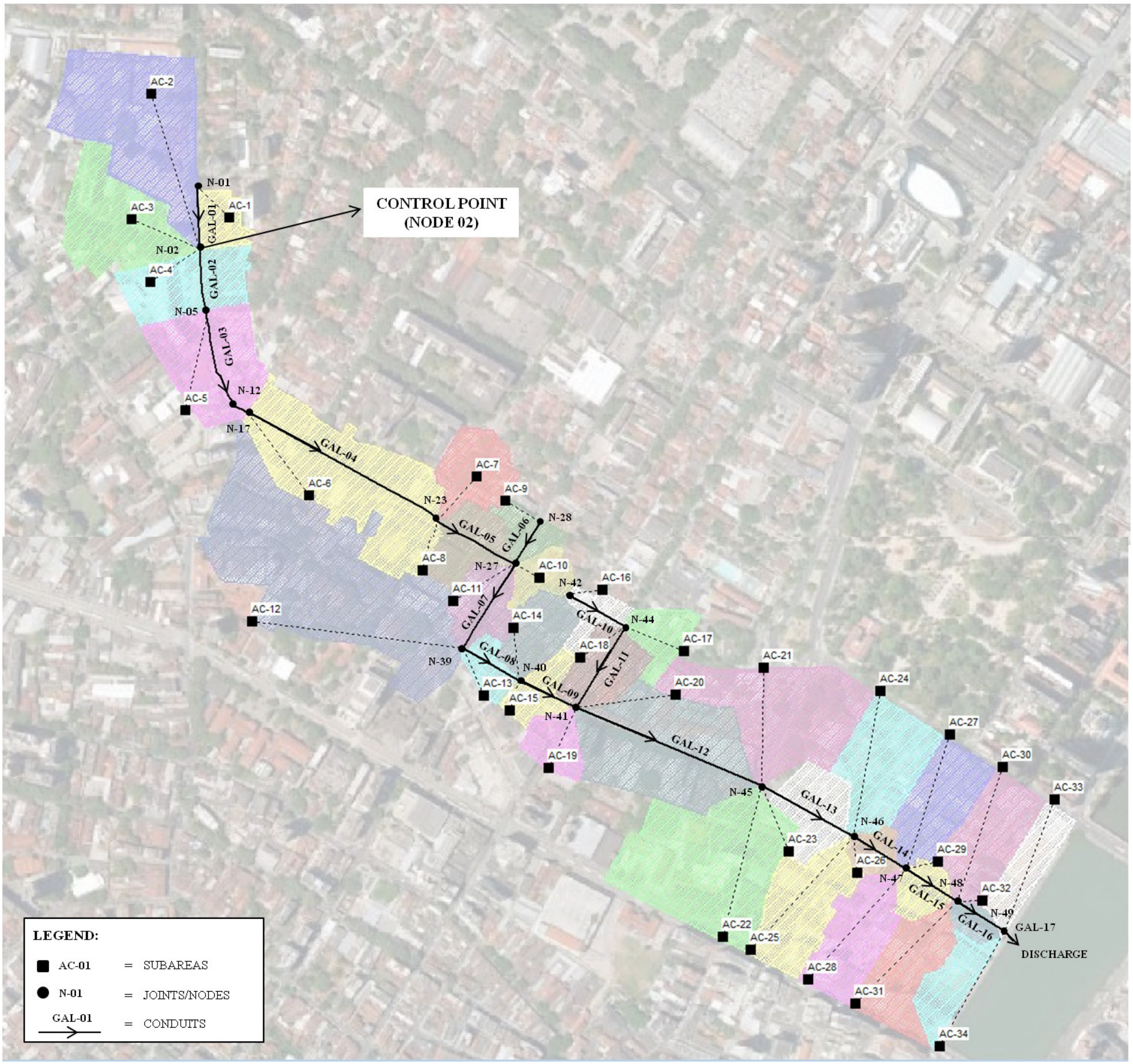

Figure 7. Drainage network implemented in the SWMM.

With this procedure, the calibration obtained can be defined as a summary procedure since it was carried out manually by manipulating both parameters: manning's roughness coefficient for the conduits and height of the sediment layer deposited in the conduits.

In general, the objective of this process was to represent the current functioning of the system considering the flooding of Junction "Node 2" (Figure 7) defined as control point by manipulating the parameters mentioned for the maximum volume of simulated flooding in the aforementioned junction would be compatible with the inspections proceeded at the location during the rainfall events considered in the study. This same procedure had also been adopted by other authors, such as Silva and Cabral (2014).

The calibration enabled the simulation of the alternatives for flooding control in the area based on the proposition of interventions in the existing drainage network, either due to the readjustment of the network itself or by implementing an on-line reservoir of the system (Scenario 02 - Alternative Condition).

RESULTS AND DISCUSSION

Scenario 01: current condition

For the calibration of the model, we used the heights observed in the water line of the respective rainfall events as well as the flooding area (Table 2 and Figure 8), delimited from the local topography and field observation.

This process revealed that the event from June 25-26, 2014 had a mean correlation coefficient between the estimated and the simulated volumes at the control point of 0.97. For the event from May 17, 2013, the mean correlation coefficient was of 0.96 (Table 3). The results reveal that despite the obtained calibration is defined as a summary procedure, the mean correlation between the simulated and the observed conditions proved satisfactory, with indices very close to 1.

The continuity errors found during the simulation process (Table 3) are within the acceptance limit having presented values below 10%, according to recommendation in the model user's manual (PROCEL-SANEAR, 2012). This same acceptance limit had been adopted by other authors, such as Silva and Cabral (2014). In these conditions, Figures 9 and 10 illustrate the hydrographs simulated at the control point (Node 2) in the occurrence of the simulated precipitation events with peak flows.

Figure 8. Flooding area in the event from June 25-26, 2014.

Table 2. Heights and volumes of flooding observed and simulated in the control point (event from June 25-26, 2014).

Event	Hour *SWMM*	Height of water line (m)		Flooding volume	Simulated volume
		in loco	*SWMM*	(m^3)*	(m^3)**
June 25-26, 2014.	10:00	0.09	0.09	117.0	105.7
	11:00	0.08	0.07	104.0	91.6
	12:00	0.14	0.12	182.0	155.1
	13:00	0.08	0.06	104.0	81.9

*Volume resulting from the product between the mean flooding area delimited for this event (1,300 m^2) and the height of the corresponding flooding. **Flooding volumes simulated through the SWMM for the event in question based on the height of the corresponding flooding.

Table 3. Main results obtained from the simulation.

Aspects		June 25-26, 2014.		May 17, 2013.	
		Observed	Simulated	Observed	Simulated
Maxium flooding blade (m)		0.14	0.12	0.27	0.27
Maximum flooding rate (m)		5.24	5.22	5.37	5.37
Maximum flooding volume (m^3)		182.0	175.0	679.9	681.0
Continuity errors	**Surface run-off**	−0.03%		−0.04%	
	Outflow propagation	−8.57%		−2.18%	
Mean correlation coefficient		0.97		0.96	

Figures 9 and 10 illustrate that the event from June 25-26, 2014 (107.6 mm and Tr=2 years) presents an affluent peak flow at Node 2 of 0.21 m^3/s, while for the event from May 17, 2013 (150.8 mm and Tr=5 years) the maximum simulated flow was 0.53 m^3/s.

Aiming at verifying the consistency of the flows simulated through the SWMM, with the hydrographs presented the previous figures, we applied the rational method to determine the flows of the contribution area to the control point considering a run-off surface coefficient of 0.86, characteristic to an area of consolidated urbanization, according to Emlurb (2014).

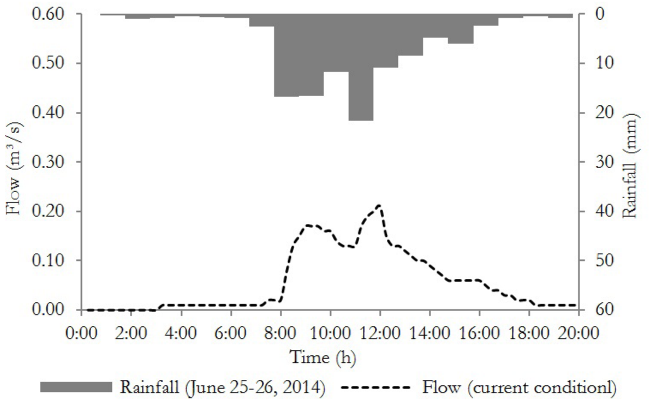

Figure 9. Hydrograph from the model calibration for the event from June 25-26, 2014.

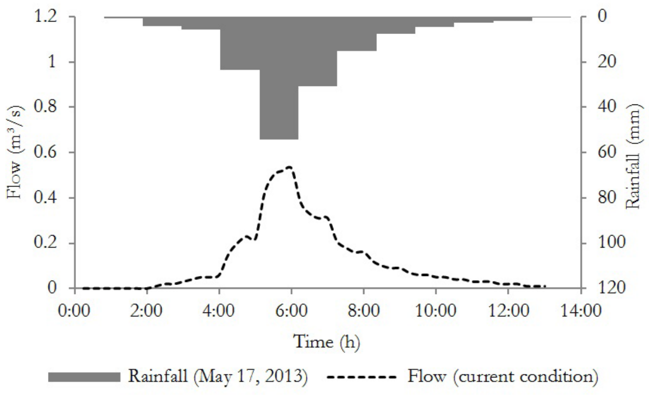

Figure 10. Hydrograph from the model calibration for the event from May 17, 2013.

Table 4 indicates the results of the calculations carried out based on the rational method application considering the following aspects:

- For the event from June 25-26, 2014 (Tr=2 years), when many precipitation peaks were recorded distributed and concentrated within four-hour interval (240 min), the precipitation intensity was calculated based on this duration.
- For the event from May 17, 2013 (Tr=5 years), the precipitation intensity was calculated for the rainfall duration of 1 hour (60 min).

For the event from June 25-26, 2014, the maximum flow obtained from the simulation was of 0.21 m^3/s, while with the application of the rational method, it reached a flow of 0.21 m^3/s. The simulations of the event from May 17, 2013, in turn, obtained a flow of 0.53 m^3/s, while the rational model rational reached 0.56 m^3/s. As expected, we conclude that the rational method has a tendency to escalate the flows. From this relationship, we may conclude that the criteria adopted for the calibration model, carried out in a simplified form, represent satisfactorily the conditions verified in the field for the occurrence of the events considered in the study.

Scenario 02: alternative conditions

Suitability of the drainage network

The drainage system with direct influence on the critical point studied, located at João de Barros Avenue up to beginning of Príncipe Street, has irregularities in its network, presenting parts with declivity problems, which interferes on the run-off flow displacement, which can be one of the causes to the flooding occurrences in the area.

The drainage network in João de Barros Avenue begins in the manhole represented in the SWMM model by Node 1 and is constituted of a gallery network with a diameter of d=0.40m along its whole extension, with an increase in diameter (d=0.60m) in the surroundings of Príncipe Street (starting at Node 12).

Figure 11 illustrates the longitudinal profile of the drainage network and the waterline located at João de Barros Avenue up to Príncipe Street, highlighting the parts with declivity opposite to the flow interfering in the run-off displacement, which can be one of the causes to the flooding occurrences in the area.

Table 4. Outflows calculated using the rational method for the two- and five-year recurrence.

t (min)	Tr = 2 years		Tr = 5 years		$Q_{2\ years}$ (m^3/s)	$Q_{5\ years}$ (m^3/s)
	i (mm/h)	Rainfall (mm)	i (mm/h)	Rainfall (mm)		
5	139.50	11.62	162.58	13.55	1.42	1.65
15	95.63	23.91	111.45	27.86	0.97	1.13
30	68.99	34.50	80.41	40.21	0.70	0.82
60	47.44	47.44	55.29**	55.29	0.48	0.56**
120	31.65	63.31	36.89	73.78	0.32	0.37
240	20.77*	83.07	24.20	96.81	0.21*	0.25

*Tr: 2 years (4h duration); ** Tr: 5 years (1h duration).

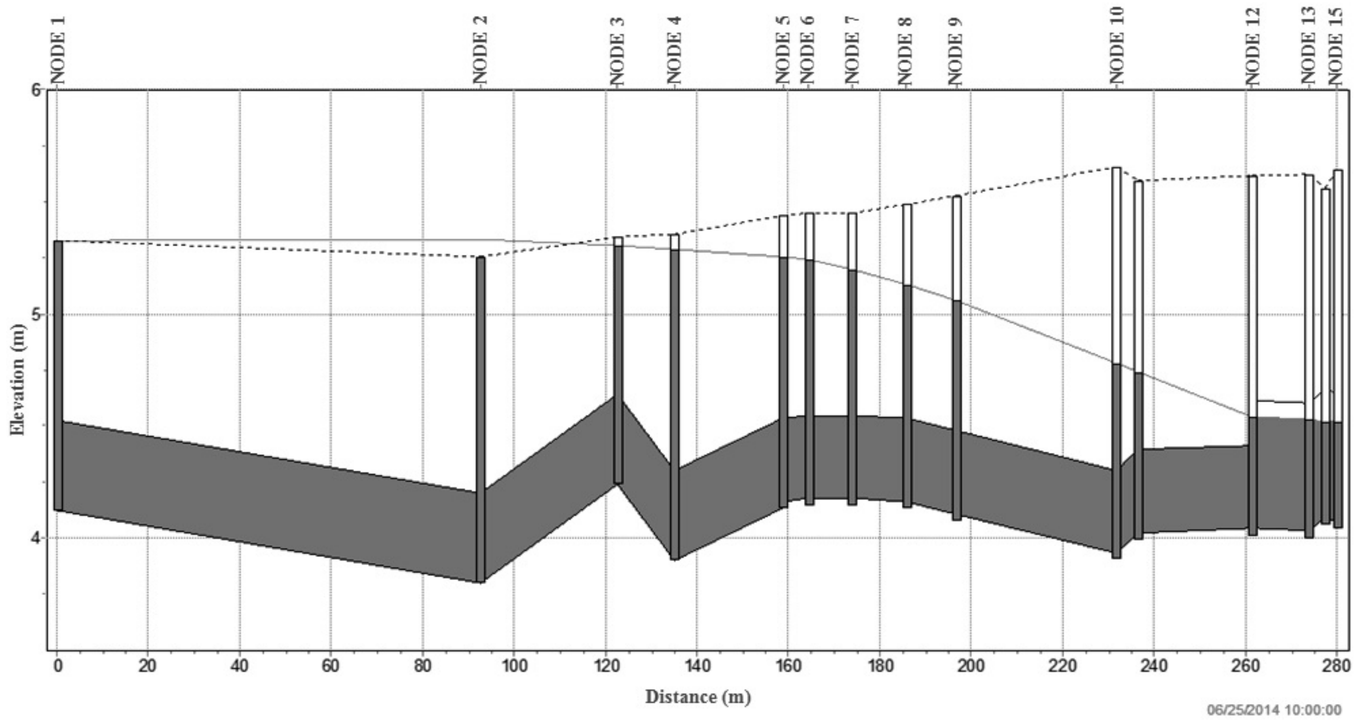

Figure 11. Profile to the gallery of João de Barros Avenue with Príncipe Street.

Primarily, we sought to readjust the depth rates of the manholes by consequently adjusting the declivities of the conduits. Next, we proceeded with the alterations in the diameter of the conduits, maintaining the criteria established in the model calibration (Manning's rugosity coefficient and the height of the sediment layer deposited in the conduits). According to this criterion, we carried out four readjustment alternatives for the drainage network in João de Barros Avenue, namely:

- Alternative 1 - Consisted of the readjustment of the declivities of the drainage conduits, more specifically at the part concentrated between the manholes represented from Node 1 to Node 6;
- Alternative 2 - Based on the readjustment of the declivities of the conduits proposed in alternative 1, with an increase in the diameter of the gallery (d=0.60m) from the manhole represented by Node 4;
- Alternative 3 - Defined by the rearrangement of the declivities of the conduits between Nodes 1 and 6, in addition to the increase in the pipe diameter (d=0.60m) downstream to the manhole represented by Node 2 (control point); and
- Alternative 4 - The declivities of the conduits between Nodes 1 and 6 were altered and the pipe diameter was increased to d=0.60m to substitute the gallery network with a diameter of d=0.40m existing at João de Barros Avenue.

Table 5 indicates the new declivities of the conduits from the adjustment of the depth rates and presents the diameters considered as well as the parts corresponding to each simulated alternative.

The results of the simulations of the four verifications considered in this study are shown in Table 6, which presents the maximum heights of the water blades of the flooding, the maximum volumes and flooding time for Node 2 (control point), in addition to continuity errors found during the simulation process of the verifications proposed for the events from June 25-26, 2014 and May 17, 2013.

With the adjustment of the drainage network in the area of influence on the flooding point, simulated for the four verifications, we observed that alternative 1 showed no variation in relation to the current scenario for the simulated precipitation events. Only the readjustment of the rates proposed for this first verification guaranteed favorable conditions exclusively for the run-off of the rainfall waters through the conduits, which before was hampered by the irregularities present in the network. We can conclude that by observing the continuity errors generated through the simulation reports, defining the mass balance equation for the run-off and propagation of the flows in the conduits (PROCEL-SANEAR, 2012). In the current condition, the outflow propagation error was of −8.57% and decreased to −2.32% for the event from June 25-26, 2014. For the event from 17 May, 2013, the error, which in the current condition was −2.18%, decreased to −1.25%.

In alternatives 2, 3 and 4 for the event from June 25-26, 2014, the flooding volumes were reduced in 28.57%, 31% and 37%, respectively. For the event from May 17, 2013, the decrease values were 1%, 51% and 58%, respectively. The adoption of such measures does not guarantee a definite solution for the flooding issues occurred in the area studied, it is just a mitigation. Furthermore, in cases in which changes are proposed for the diameter of the existing pipe section, we verified an increase in the overload of

Table 5. Synthesis of the declivities of the conduits and diameters corresponding to the current and alternative conditions.

Condition	Readjustment of the declivities of the galleries				
	Span Node 1 - Node2	Span Node 2 - Node 3	Span Node 3 - Node 4	Span Node 4 - Node 5	Span Node 5 - Node 6
Current	0.35%	−1.47%	2.67%	−0.99%	−0.21%
Alternative 1	0.17%	0.18%	0.17%	0.17%	0.16%
	Permanence of the diameter of the implemented pipe (d=0.40m)				
Alternative 2	0.17%	0.18%	0.17%	0.17%	0.16%
	Permanence of the diameter of the implemented pipe (d=0.40m)			Increase in the pipe diameter (d=0.60m)	
Alternative 3	0.29%	0.01%	0.00%	0.01%	0.00%
	d=0.40m	Increase in the pipe diameter (d=0.60m)			
Alternative 4	0.08%	0.01%	0.00%	0.01%	0.00%
	Increase in the pipe diameter (d=0.60m)				

Table 6. Synthesis of the results from the simulations.

Aspects			Rainfall events			
			June 25-26, 2014.		May 17, 2013.	
			Simulated value	% Decrease	Simulated value	% Decrease
Alternative 1	Maximum flooding height (m)		0.12	0%	0.27	0%
	Maximum flooding volume (m^3)		175.00	0%	681.00	0%
	Flooding time (h)		5.16	0%	5.19	0%
	Continuity errors	Surface run-off	−0.03%		−0.04%	
		Outflow propagation	−2.32%		−1.25%	
Alternative 2	Maximum flooding height (m)		0.10	17%	0.27	0%
	Maximum flooding volume (m^3)		125.00	28.57%	675.00	1%
	Flooding time (h)		1.38	73.25%	4.07	21.6%
	Continuity errors	Surface run-off	−0.03		−0.04%	
		Outflow propagation	−5.12%		−2.18%	
Alternative 3	Maximum flooding height (m)		0.09	25%	0.13	52%
	Maximum flooding volume (m^3)		120.00	31%	332.00	51%
	Flooding time (h)		1.36	74%	4.00	23%
	Continuity errors	Surface run-off	−0.03		−0.04%	
		Outflow propagation	−5.13%		−2.17%	
Alternative 4	Maximum flooding height (m)		0.08	33%	0.11	59%
	Maximum flooding volume (m^3)		110.00	37%	289.00	58%
	Flooding time (h)		1.31	75%	4.00	23%
	Continuity errors	Surface run-off	−0.03		−0.04%	
		Outflow propagation	−5.20%		−2.17%	

the conduits in the region downstream to the modified system, with mean increase values in the simulated flow in relation to the current scenario of 23% for the event from June 25-26, 2014 and 35% for the event from May 17, 2013.

Figure 12 illustrates the comparison between the hydrograms of the drainage network immediately downstream to the stretch with proposed interventions (represented by the conduit located between Nodes 12 and 13) considering the event from June 25-26, 2014. Table 7 presents the results indicated in the simulation report for the different simulated conditions for the same precipitation event.

Table 7. Simulation report (maximum outflow in the conduit between Nodes 12 and 13).

Scenario	Maximum outflow (m^3/s)	Inst. of the max. occurrence (h:min)	Maximum velocity (m/s)
Current condition	0.173	12:16	0.69
Alternative 2	0.254	12:40	0.92
Alternative 3	0.261	12:38	0.94
Alternative 4	0.261	12:38	0.94

Detention reservoir

We considered the implementation of a detention reservoir (on-line) along the main drainage network, more specifically in the area of influence on the flooding point. There are different methods to estimate the detention volume, many of these are known as simplified. Experience has revealed that the methods based on the use of hydrodynamic models (SWMM, HECRAS, etc.) compensate the additional effort at the dimensioning stage for allowing lower costs and the verification of hydraulic dimensioning failures in the project, which would be impossible to visualize using more simplified methodologies (NEVES et al., 2005).

With the use of the SWMM to simulate the efficiency of the solution proposed for the preliminary dimensioning of the detention reservoir, we applied methods that define the storage volume as well as the flow through the rational method and continuity equation. Among the existing methods to obtain the storage volume, we decided to apply the rainfall method defined by Baptista et al. (2005) for its lower amount of intput parameters requiring basically only the IDF/PDF curves regarding long periods. The method can be expressed as:

$$DH_{max} = Max.[P(D,T) - q_s.D] \quad (1)$$

where DH_{max} is the maximum height to store (m), P(D,T) is the maximum precipitation (mm), D is the precipitation duration (min or h), and q_s is the specific flow (mm/min), given by equation:

$$q_s = Q_s / A_a \quad (2)$$

where Qs is the output flow (restriction) (m^3/s) and A_a is the effective drainage area (m^2). This area is given by A_a = AC, where A is the basin area (m^2), and C is the post-urbanization run-off coefficient. The result must be converted to mm/min to be employed in Equation 1.

The maximum volume of the reservoir is given by the product between the maximum storage height (DH_{max}) and the effective drainage area (Aa).

For the input parameters to obtain the affluent flow (Qs) using the rational method regarding the rainfall intensity of the project, we adopted a recurrence time interval of 2 and 5 years (coinciding with the periods of return of the simulated precipitation events), generally used for the conception of microdrainage projects (DNIT, 2006), and the rainfall duration equal to the mean concentration time of the upstream area to the critical point studied. The rainfall intensity was obtained using the IDF equation of Recife (EMLURB, 2014); the coefficient of the run-off adopted (C) was 0.86, peculiar to a consolidated urbanization area, according to Emlurb (2014). Table 8 shows an application

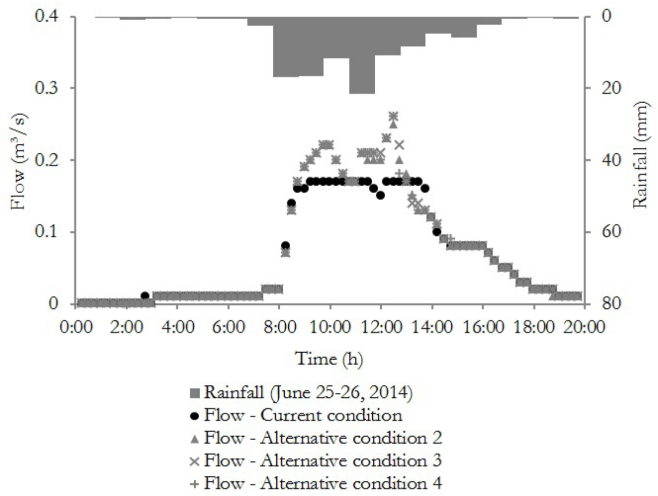

Figure 12. Hydrograph of the drainage network downstream to the modified system (conduit located between Nodes 12 and 13).

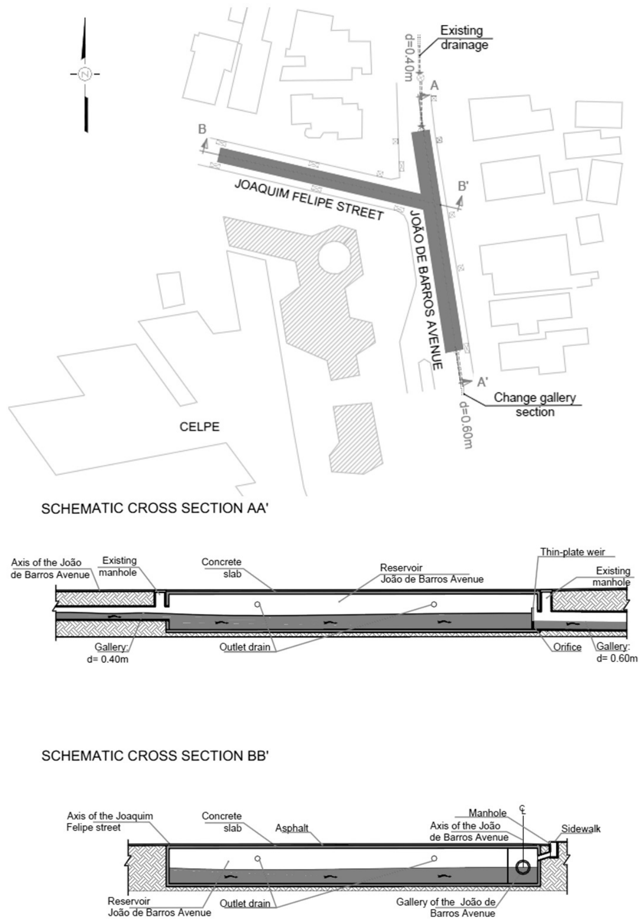

Figure 13. Detention reservoir on-line in the drainage network.

Table 8. Rainfall method: output flow of the upstream area (Qs) using the rational method.

TR (years)	i (mm/h)	P(D,T) (mm)	Q_s (m^3/s)	*qs=Qs/Aa* (mm/min)	Height of the discharged water *qs.t* (mm)	Storage height *P-qst* (mm)	Storage volume (m^3)
2	95.63	23.91	1.07	1.60	11.87	12.04	483.08
5	111.45	27.86	1.24	1.86	13.83	14.03	563.00

of the rainfall method to obtain the reservoir storage volume for the pre-stablished recurrences.

The dimensions required for the storage are restricted to local conditions based on the width of the spaces and the depth of the existing manholes. The small rate variations limit the volume to 560 m^3 and the useful inside height to 1.00 m (Figure 13).

For the conception of this solution, we adopted the readjustment of the drainage network proposed in Alternative 2, with the drainage network in the entrance of the reservoir with a diameter of d=0.40m and the exit of the reservoir through a 0.70m high sharp crested weir and orifice of 40 × 40 cm connected to an outbound gallery with a diameter of d=0.60m (from the manhole represented by Node 4).

The determination of the weir height conceived in thin wall without lateral contractions was defined so that the square conduit (40 × 40 cm) would work as an orifice. In addition, the definition of the weir height was also limited to the manhole rate located upstream to the reservoir (Node 1=5.325), assuring the discharge of the waters whenever its level inside the reservoir reaches the quote of the weir threshold.

Figure 14 reveals that for the event from June 25-26, 2014 (Tr of 2 years), the dimensioned reservoir becomes satisfactory once the volume produced by the event is within its total storage capacity with the water heights in accordance with the depth established for the reservoir. Still according to the mentioned figure, it is possible to observe the occurrence of an overflow of the excess water inside the reservoir through the weir considering a height of 0.70m. The discharge time was 1 hour and 15 minutes.

Figure 15, obtained for the event from May 17, 2013 (Tr of 5 years) reveals a flattening in the chart of the volumes stored between 5h15 and 8h along two hours and 45 minutes, providing these flooding conditions of lower range in relation to the effectively occurred. The heights of the excess waters reaches a maximum of 17cm, with a flooding volume of 425m^3.

In brief, Table 9 presents a synthesis of the results obtained through the simulation of the reservoir and the current condition for the considered events indicating the decrease percentages caused by the effect of this solution. The results shown are some of the data in the simulation report regarding the flooding point studied (Node 2) for the flooded time and respective flooding volume.

It is possible to observe that in the control scenario a decrease in the flooding time (30.25%) occurs, the flooding volume is lower, with a reduction of nearly 38%, which corroborates with the decreased peak flow for events with a five-year recurrence. For events with a two-year recurrence, the detention reservoir proposed is satisfactory.

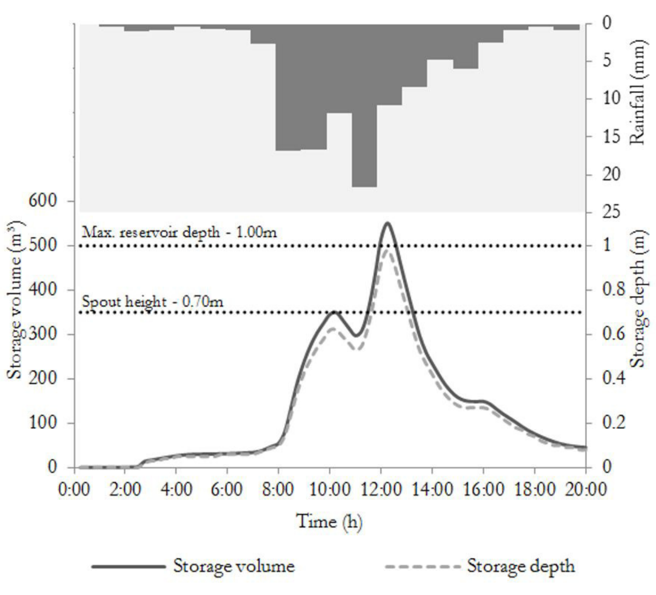

Figure 14. Relationship between water height and volume in the reservoir (Event: June 25-26, 2014).

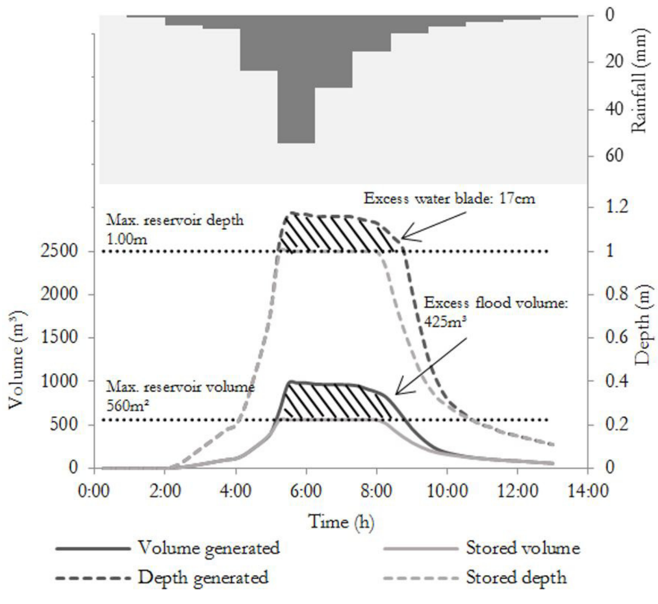

Figure 15. Relationship between water height and volume in the reservoir (Event: May 17, 2013).

Table 9. Synthesis of the results from the simulations.

Event	Condition	Time of flooding (h)	Water line (m)	Volume of flooding (m^3)
June 25-26, 2014.	S/ Control	5.16	0.12	175
	C/ Control	ZERO	ZERO	ZERO
Decrease (%)		-	-	-
May 17, 2013.	S/ Control	5.19	0.27	681
	C/ Control	3.62	0.17	425
Decrease (%)		30.25	37.03	37.59

CONCLUSIONS

During the modelling, we verified that the location presents an unfavorable run-off condition due to the underground conduits with parts of the galleries with negative declivity. This shows that in addition to the gap in the drainage system conceived for the implementation of the urban pattern, the irregularity of the network may be one of the causes of the recurrent flooding in the studied area.

The adjustment of the drainage network simulated in four alternatives does not guarantee a definite solution to the flooding issues occurred in the studied area, it is just a mitigation. This occurs in cases in which an alteration in the diameter is proposed for the existing pipe section, we verified an increase in the overload of the conduits in the region downstream to the modified system, with mean increases in the simulated flow in relation to the current scenario of 23% for the event from June 25-26, 2014 and 35% for the event form May 17, 2013.

The implementation of a detention reservoir in the occurrence of the event from June 25-26, 2014 proved satisfactory. For the event from May 17, 2013, in turn, the flooding volume had a decrease of nearly 38%, presenting a flooding time also shorter compared with the current condition (30.25% shorter).

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Authors contributions

Marcos Antonio Barbosa da Silva Junior: structuring and writing of the article; processing of data; hydraulic-hydrological modeling; and analysis of results.

Simone Rosa da Silva: orientation of the study; structuring and revision of the article; and discussion of the results.

Jaime Joaquim da Silva Pereira Cabral: co-orientation of the study; structuring and revision of the article; and discussion of the results.