A Simple Method for Designing Infiltration Low Impact Development Techniques Considering Effects of Urbanization and Climate Change*

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Abstract

Infiltration-focused low-impact development (LID) solutions aim to gradually absorb surface runoff, playing a critical role in reducing both peak flow rates and runoff volumes. These systems are commonly designed with specific surface areas and controlled ponding depths to provide sufficient storage capacity that counteracts the increased runoff resulting from urban expansion. Nevertheless, arbitrarily selecting the ponding depth—without systematic optimization—may lead to designs that either lack sufficient storage or utilize unnecessary space. This work presents MoDOBR, an optimized sizing framework specifically developed for retention systems. The methodology is demonstrated through four practical examples. A key aspect of the proposed approach is ensuring that the design ponding depth closely aligns with the required storage volume to manage excess runoff, particularly under conditions where infiltration pathways and underdrainage may become clogged or ineffective. The model dynamically solves the mass balance at both the system's surface and at the interface with the saturated soil zone. The application of MoDOBR across various case studies, including a soil sensitivity assessment, reveals that sandy soils typically require LID installations covering about 2.9% of the drainage area, while clayey soils may demand up to 50% coverage when accounting for runoff from a storm with a 5-year return period.

Keywords: Retention Basin Design, Baseline Hydrologic Conditions, Optimized Sizing of Retention Structures, Sustainable Urban Drainage Systems

1. Introduction

Retention basins are an essential component of low-impact development (LID) strategies aimed at controlling stormwater by temporarily storing runoff and gradually releasing it to nearby drainage systems, with the added benefit of enhancing groundwater recharge (Baptista et al., 2011; Fletcher et al., 2015). Infiltration-based solutions, such as retention reservoirs, play a significant role in reducing peak discharges and delaying flow peaks (Winston et al., 2016). When constructed in areas with soils that allow for rapid infiltration, these basins are considered both cost-efficient and technically viable (Mano et al., 2008), and they contribute to the partial restoration of pre-urban hydrological conditions.

A variety of hydrological models are available for simulating the behavior of retention basins. Water balance models are commonly used to capture the storage and release dynamics of these systems. The PULS method (Zoppou, 1999), in particular, is widely applied for determining storage volumes. This method solves the mass balance equation by considering precipitation inputs, inflows, and infiltration losses, and its straightforward computational structure makes it especially suitable for spreadsheet-based calculations (Júnior, 2019; Gomes Jr et al., 2023; Ferreira et al., 2019). The PULS technique allows designers to estimate the minimum storage requirement based on a specified maximum outflow. For more advanced simulations,

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the Stormwater Management Model (SWMM) (Rossman and Huber, 2016) is a powerful tool that incorporates detailed LID modules capable of simulating bioretention cells and rain gardens, using comprehensive mass and energy balance formulations to develop flow hydrographs. Another relevant tool is DRAINMOD (Skaggs et al., 2012), initially developed for daily-scale simulations, but which has recently been enhanced to handle sub-daily runoff dynamics (Braswell et al., 2024).

Retention basin design typically requires the definition of key parameters such as surface area, maximum allowable water depth, and, when applicable, the sizing of overflow and safety structures. In some projects, particularly those aiming for enhanced safety margins, designs may consider extreme rainfall events associated with return periods of up to 200 years (Fletcher et al., 2015).

A commonly used sizing principle for LIDs is the zero-impact design approach (PG County, 2007), which compares the hydrological behavior of the catchment before and after urbanization. In its natural state, the catchment is usually covered by permeable surfaces that facilitate infiltration, while urban development increases imperviousness and consequently amplifies runoff volumes. The difference in runoff between these two conditions is used to calculate the minimum storage needed, and pre-development peak flows are often used as benchmarks for sizing discharge structures such as spillways (Rosa, 2016).

Another widely referenced technique is the envelope curve method (Santos et al., 2021; Silveira and Goldenfum, 2007), which assumes a constant outflow rate and relies on inflow hydrographs derived from rainfall intensities obtained through Intensity-Duration-Frequency (IDF) relationships. This method combines cumulative rainfall volumes with assumed constant outflows to determine the necessary storage volume, similar to the concept behind Rippl diagrams (Tomaz, 2003).

Both the zero-impact and envelope curve methods are useful in preliminary design because they offer simplified ways to mitigate the hydrological effects of urbanization by providing adequate storage for runoff. However, they do not capture the time-dependent and non-linear infiltration processes that are intrinsic to infiltration-based LIDs. According to Duke et al. (2024), the absence of dynamic water level tracking in LID systems across consecutive rainfall events can compromise their effectiveness, emphasizing the importance of time-based hydrological modeling that accounts for changing initial conditions.

Simplified pre-design models such as the envelope curve continue to be practical due to their ease of application (Silveira and Goldenfum, 2007), but they neglect variations in infiltration rates over time. For more precise modeling of unsaturated soil flow, the Richards equation (Richards, 1931) is often considered the most robust option. However, its complexity and the numerical stability required to solve it generally make it unsuitable for hand calculations or spreadsheet applications. Explicit solution methods for the Richards equation are typically too demanding for routine engineering use. As an alternative, the Green-Ampt model (Green and Ampt, 1911) provides a simpler, yet effective, approach to simulate the non-linear infiltration process. This model is a reasonable approximation of soil infiltration dynamics and can be readily implemented in hydrological tools, including SWMM (Rossman and Huber, 2016).

The main goal of this research is to develop a fast and practical method for designing retention basins and other infiltration-driven systems, explicitly considering the time-varying nature of infiltration at the system's surface. The approach focuses on determining the minimum ponding depth required to provide sufficient storage for managing the additional runoff generated by urbanization, even under conservative scenarios where the infiltration layer and drainage components are assumed to be completely blocked. It is critical to understand that this minimum storage volume does not always align with the flood volumes estimated by non-linear hydrological simulations. Therefore, solving this design problem demands either an iterative process or an optimization routine to properly match the storage and infiltration conditions.

Given the growing uncertainties linked to climate change and the evolving behavior of rainfall intensity-duration-frequency (IDF) patterns (Paiva et al., 2024), this study also proposes a flexible sizing strategy that includes an allowance for freeboard. This additional storage margin accounts for the potential increase in runoff associated with future rainfall intensification. To demonstrate the model's applicability, four distinct examples are explored. The first example presents the design of a retention system for a 5,000 m^2 catchment with predefined soil characteristics. The second example investigates the effect of varying soil types on the required design heights. In the third scenario, a bottom drainage orifice is introduced as a control feature to achieve design compatibility for a target ponding height of 80 cm. The final example involves a detailed sensitivity assessment that evaluates the influence of each model parameter under different soil conditions.

2. Methods

2.1. MoDOBR: Model for Optimized Design of Retention Basins

This section presents the mathematical foundation for estimating the inflow hydrograph and the infiltration-driven reduction of this flow at the base of a retention basin. The approach models a small catchment using the rational method, where runoff is routed to a retention facility that allows gradual percolation into the subsurface.

The peak discharge is calculated using the rational method as originally proposed by Mulvaney (1851) and later expanded by Kuichling (1889):

$$Q^p = C \cdot i \cdot A_c \tag{1}$$

where Q^p is the peak flow rate associated with the triangular hydrograph $[\mathrm{L} \cdot \mathrm{T}^{-3}]$, C is the runoff coefficient, i is the design rainfall intensity $[\mathrm{L} \cdot \mathrm{T}^{-1}]$, and A_c is the contributing catchment area $[\mathrm{L}^2]$. In this study, A_c strictly represents the upstream contributing area and does not include the surface area of the retention basin itself.

This expression is applied separately for pre-urbanization and post-urbanization conditions, with distinct runoff coefficients and times of concentration to reflect each scenario. The rainfall intensity i is a function of the time of concentration and is estimated using established empirical formulas. One frequently used equation is the SCS-Lag method from NRCS (2010), which estimates the time of concentration as:

$$t_\mathrm{c} = 23.19 \left[\frac{l^{0.8} \left(S_\mathrm{SCS} + 1\right)^{0.7}}{1140\, Y^{0.5}} \right] \tag{2}$$

where t_c is the time of concentration [min], l is the flow path length [m], Y is the average slope of the catchment [%], and S_SCS is the maximum potential retention [mm], computed using the relation $S_\mathrm{SCS} = 25400/\mathrm{CN} - 254$, with CN being the curve number used in NRCS hydrological models.

There are various ways to estimate rainfall intensity from IDF relationships. In this study, the Sherman-type IDF formula is adopted, following the format described by Gomes Jr et al. (2021):

$$i = \frac{K \cdot \mathrm{TR}^a}{(b + t_c)^c} \tag{3}$$

where K, a, b, and c are empirical coefficients specific to the IDF curve, and TR is the design return period.

Once both the peak flow and the time of concentration are known, a triangular hydrograph can be generated, which represents the time evolution of runoff. The inflow hydrograph is constructed with a rising limb up to the peak and a subsequent linear recession, and is defined as:

$$\begin{aligned} Q(t) &= Q^p \cdot \left(\frac{t}{t_c}\right), \quad \text{for} \quad t \le t_c \\ Q(t) &= Q^p \cdot \left[\max\left(1 - \frac{t - t_c}{t_c}, 0\right)\right], \quad \text{for} \quad t > t_c \end{aligned} \tag{4}$$

where $Q(t)$ is the instantaneous flow rate at time t $[\mathrm{L}^3 \cdot \mathrm{T}^{-1}]$.

This time-dependent inflow representation is key to the MoDOBR model as it integrates both the urban hydrological response and the infiltration processes that control surface water storage in retention basins.

The minimum storage volume required for the retention basin is determined by the difference in runoff volumes between the pre-urbanization and post-urbanization scenarios. This critical volume can be calculated by integrating the difference between the inflow hydrographs over time and identifying the point of maximum storage requirement, as shown in the following expression:

$$V_\mathrm{min} = \underset{t \in \tau}{\mathrm{argmax}} \left[\int_0^t \left(Q_\mathrm{post}(t) - Q_\mathrm{pre}(t)\right) \mathrm{d}t \right] \tag{5}$$

where argmax identifies the specific time when the largest difference in cumulative volumes occurs, and τ denotes the total simulation period.

This minimum volume, $V_{\min}$, represents the essential storage that the retention basin must provide to safely manage the additional runoff introduced by urbanization. The model adopts a conservative approach, assuming a worst-case scenario in which the infiltration layer is completely obstructed and the basin's drainage components, such as underdrains and outlet controls, are inoperative. This safety margin is particularly important for aging systems that may experience reduced efficiency due to inadequate maintenance or clogging over time.

Retention systems can be designed with various outflow control features, including orifices, weirs, and pumps, which help regulate discharge rates (Gomes Jr et al., 2023; Gomes Júnior et al., 2022). In the specific case of infiltration basins, however, the primary mechanism for outflow is the infiltration through the base of the structure. Since these basins typically have a wide surface area relative to their depth, it is common practice to disregard lateral infiltration for design simplification. Nevertheless, in smaller LID facilities such as rain gardens or permeable pavements, lateral infiltration can have a noticeable influence on system performance and should be explicitly considered (Gomes Jr et al., 2023; Lee et al., 2015).

For the MoDOBR model, the design assumes that lateral infiltration is negligible, which is a reasonable simplification given the geometric proportions of conventional retention basins where the surface area is substantially larger than the water depth.

The infiltration capability of the soil beneath the retention basin can be approximated using the Green-Ampt method (Green and Ampt, 1911), which incorporates key soil characteristics to estimate the infiltration capacity at any given time:

$$C(t) = k_{\text{sat}} \frac{\Delta\theta\,(\psi + h_p(t))}{F(t)} \tag{6}$$

where k_{sat} is the saturated hydraulic conductivity of the soil, ψ represents the soil's suction potential, $h_p(t)$ is the ponding depth at the basin surface at time t, $F(t)$ is the cumulative infiltration depth, and $\Delta\theta$ is the difference in moisture content, often referred to as the effective porosity (Green and Ampt, 1911).

When discretizing the process with small time intervals, the actual infiltration rate $f(t)$ can be approximated as the lower value between the soil's infiltration capacity and the maximum infiltration possible based on available surface water:

$$f(t) = \min\left(C(t),\ \frac{h_p(t)}{\Delta t}\right) \tag{7}$$

This approach provides an explicit solution that avoids the complexity of solving the full non-linear form of the Green-Ampt equation, which typically requires iterative numerical techniques. The simplified explicit method remains stable if sufficiently small time steps are adopted.

The progression of cumulative infiltration over time is calculated as:

$$F(t + \Delta t) = F(t) + \Delta t \cdot f(t) \tag{8}$$

The evolution of the ponding depth at the basin surface follows the principle of mass conservation, leading to the following balance equation:

$$h_p(t + \Delta t) = h_p(t) + \Delta t \cdot \left(\frac{Q(t)}{A} - f(t) - S(t)\right) \tag{9}$$

where $S(t)$ represents other potential inflow or outflow components such as discharges through engineered structures (e.g., orifices or weirs), irrigation contributions, or surface water losses due to processes like evaporation.

The maximum ponding depth observed during the simulation is extracted as:

$$h_p^{\max} = \max_{t \in \tau} (h_p(t)) \tag{10}$$

2.2. Detention Time Requirement

When designing LIDs that temporarily retain water on the surface, it is crucial to consider the detention time to prevent the creation of conditions favorable to mosquito breeding and waterborne diseases. Common design recommendations suggest that all ponded water should drain within 24 to 48 hours. In this work, a conservative maximum allowable drainage time of 24 hours (t_d) is established. Computationally, the objective is to determine the time t_v when the ponding depth falls below a small threshold δ (typically 1 mm), indicating that the basin has effectively drained.

2.3. Determination of Compatible Design Height

As previously explained, the design of retention or detention basins must include two critical elements: the storage surface area and the design ponding depth. For a given required storage volume $V_{\min}$ and a selected design height h, the basin area (assuming a rectangular plan view) is computed by:

$$A = \frac{V_{\min}}{h} \tag{11}$$

This calculated surface area ensures that, even in a conservative scenario where infiltration is fully blocked (e.g., due to sediment accumulation or clogging), the basin has enough volume to store the excess runoff generated in the post-development condition. In real-world applications, a portion of this volume will typically infiltrate through the basin base, potentially allowing adjustments to the design height.

The central design task addressed in this study is to find a combination of storage area (A) and ponding depth (h) that:

- Provides sufficient volume to capture the required excess runoff ($V_{\min}$).
- Ensures the maximum simulated ponding depth ($h_p^{\max}$) does not exceed the selected design height.
- Meets the drainage time constraint, ensuring that the basin fully empties within the allowable duration ($t_v \leq t_d$).

The required storage volume is directly tied to the runoff volume difference between pre- and post-urbanization scenarios, while the basin's footprint is a function of both this volume and the chosen ponding depth. The design procedure therefore seeks an optimal height h that satisfies all constraints while minimizing land use and preventing prolonged ponding durations, which could pose environmental and public health risks.

By aiming to minimize the difference between the design ponding height and the maximum simulated ponding depth, the problem can be structured as an optimization process, described in the following formulation:

Compatible Design Height

$$\begin{aligned} \underset{h}{\text{minimize}} \quad & \left| h - h_p^{\max} \right| \\ \text{subject to:} \quad & h_{\min} \leq h \leq h_{\max} \\ & \text{Eq. (5) to Eq. (11)} \\ & t_v \leq t_d \end{aligned} \tag{12}$$

In this optimization scheme, $h_{\min}$ and $h_{\max}$ define the permissible range of design ponding heights, commonly between 0.1 m and 2 to 3 m, depending on local geotechnical and safety considerations. If a solution cannot be identified that meets all the constraints set forth in Eq. (12), it is recommended that a different type of low-impact development (LID) system be explored.

The advantage of this optimization approach is that it ensures the selected ponding height is not only hydraulically feasible but also spatially efficient, minimizing the footprint of the retention basin. The proposed method aligns the storage area with the dynamic ponding behavior, enhancing design accuracy and potential cost savings. The MoDOBR framework is structured for application in spreadsheet software,

	Detention Ponds				Retention Ponds											
	(Eq. 1)	(Eq. 1)	$(Q_{pos} - Q_{pre})\Delta t$	(Eq. 5)	(Eq. 8)	(Eq. 9)	h(t) x A / 1000	(Eq. 6)	(Q_{pos} / A)	Source Term	$h_p / \Delta t$	(Eq. 7)	f x A	(Eq. 8)	(Eq. 9)
t(min)	$Q_{pré}(t)$ [m^3/s]	$Q_{pós}(t)$ [m^3/s]	ΔV (m3)	V_{acum} (m^3)	F(t) [mm]	hp(t) [mm]	Vol [m^3]	C [mm/h]	Qin [mm/h]	S(t) [mm/h]	Pin [mm/h]	f [mm/h]	Qinf [m^3/s]	F(t + Δt) [mm]	h(t+Δt) [mm]
0	0.00	0.00	0.00	0.00	5.00	0.00	0.00	176.88	0.00	0.00	0.00	0.00	0.00	5.00	0.00
1	0.00	0.01	0.58	0.58	5.00	0.00	0.00	176.88	613.40	0.00	0.00	0.00	0.000	5.00	10.22
2	0.00	0.02	1.17	1.75	5.00	10.22	0.67	201.42	1226.80	52.65	613.40	201.42	0.004	8.36	26.44
3	0.00	0.03	1.75	3.51	8.36	26.44	1.72	155.84	1840.20	84.66	1586.14	155.84	0.003	10.95	53.10
4	0.01	0.04	2.34	5.85	10.95	53.10	3.46	155.21	2453.60	119.98	3185.84	155.21	0.003	13.54	89.40

Figure 1: Illustration of the spreadsheet tool used for basin sizing. The diagram shows the step-by-step workflow and the key equations integrated into the MoDOBR model.

using Excel's Generalized Reduced Gradient (GRG) nonlinear solver to efficiently find the optimal solution (Smith and Lasdon, 1992).

Figure 1 illustrates the spreadsheet configuration, detailing the sequential steps and formulas applied throughout the design process.

2.4. Freeboard Design and Climate Change Considerations

Recent studies suggest that projected rainfall intensities across South America may rise by approximately 20 to 40% for specific return periods due to climate change effects (Paiva et al., 2024). To incorporate this expected increase, a correction factor γ is applied to the design rainfall intensity, as shown below:

$$i_d(\mathrm{TR}, t_c) = \gamma \cdot i(\mathrm{TR}, t_c) \tag{13}$$

where i_d is the climate-adjusted design rainfall intensity, γ represents the climate change adjustment factor, and i is the original rainfall intensity obtained from the regional IDF curve. For simplicity, the notation is hereafter shortened to $i_d = i_d(\mathrm{TR}, t_c)$ and $i = i(\mathrm{TR}, t_c)$.

To provide an additional safety margin against increased runoff volumes, a freeboard height h_b is included in the design. The freeboard accounts for the extra volume generated by higher rainfall intensities while assuming that the basin's time of concentration does not change. The freeboard height can be determined by comparing runoff volumes for the adjusted and original rainfall intensities, leading to the following expression:

$$h_b = \frac{C_{\mathrm{pos}} \cdot [(\gamma - 1) \cdot i(\mathrm{TR})] \cdot A_c \cdot t_c}{A} \tag{14}$$

where $(\gamma - 1)$ quantifies only the additional rainfall intensity attributed to climate change, excluding the baseline design condition.

Besides providing extra storage, it is crucial to include an overflow structure to safeguard the system against potential overtopping during extreme rainfall events. For the spillway design, it is recommended to adopt a return period greater than that used for the retention basin sizing, typically ranging from 10 to 25 years.

The surface spillway discharge can be estimated using hydraulic principles based on the Francis formula for weirs (Porto, 2004), expressed as:

$$Q^{p*} = C_d \cdot L_{\mathrm{ef}} \cdot (h + h_b - h_s)^{3/2} \tag{15}$$

where C_d is the discharge coefficient (commonly taken as 1.8), L_{ef} is the effective length of the weir, h is the retention basin's design ponding height, h_b is the calculated freeboard, and h_s is the elevation of the spillway crest relative to the basin surface (often set equal to h). The peak discharge over the weir, Q^{p*}, is calculated using the post-urbanization runoff coefficient and the climate-adjusted design rainfall intensity for the spillway's return period (TR_v), given by:

$$Q^{p*} = C_{\mathrm{pos}} \cdot i_d(\mathrm{TR}_v) \cdot A$$

This procedure ensures that the system remains resilient even under increased rainfall intensities, providing both freeboard and emergency overflow capacity.

By selecting the elevation of the spillway crest h_s, Eq. (15) can be rearranged to directly estimate the minimum effective spillway length required to safely convey excess flow without causing overtopping. The equation is given by:

$$L_{\text{ef}} = \frac{C_{\text{pos}} \cdot i_d \cdot A}{C_d \cdot (h + h_b - h_s)^{3/2}} \tag{16}$$

In principle, the dynamic influence of the weir could also be integrated into the surface water balance equation (Eq. (9)). This would involve replacing the static design height h in Eq. (15) with the instantaneous ponding depth $h_p(t)$, allowing the model to account for time-dependent spillway discharge by incorporating it into the source term $S(t)$. Under this more detailed configuration, the system could potentially operate with a smaller design ponding depth, since whenever $h_p(t)$ exceeds the spillway crest elevation h_s, the spillway would begin releasing water, effectively limiting further increases in ponding depth.

However, this study deliberately excludes the active spillway discharge from the simulation, adopting a conservative design assumption. This choice accounts for the realistic possibility that the spillway could become blocked by debris or sediment, rendering it temporarily ineffective. As a result, the additional volume associated with the freeboard is fully allocated to surface storage, ensuring the system can manage the increased runoff volume due to climate change and potential losses in system efficiency, without depending on spillway performance.

The essential input parameters required to apply the proposed design methodology and conduct the simulations are as follows:

- Saturated hydraulic conductivity of the soil (k_{sat})
- Soil matric suction potential (ψ)
- Initial soil moisture (cumulative infiltration) at the start of the simulation ($F(0)$), as referenced in Eq. (8)
- Initial surface ponding depth ($h_p(0)$)
- Selected time step for numerical integration (Δt)
- Total duration of the simulation (t_f)
- Parameters of the IDF curve: K, a, b, and c
- Runoff coefficients representing both pre-urbanization and post-urbanization conditions
- Time of concentration values for pre- and post-urbanization scenarios

These parameters form the basis for setting up the simulations and calculating the optimal sizing of infiltration-based retention basins using the proposed MoDOBR framework.

3. Typical Design Scenario

Consider a commercial property situated in São Carlos, São Paulo, encompassing a total drainage area of 5,000 m^2. The site layout includes 40% impervious surfaces (runoff coefficient $C = 1$) and 60% permeable areas (runoff coefficient $C = 0.6$). The goal is to design an infiltration basin capable of managing runoff generated by a rainfall event with a return period of 5 years. For the post-urbanization condition, the time of concentration is 10 minutes. Before development, the area functioned primarily as pasture, characterized by a lower runoff coefficient of $C = 0.35$ and a longer time of concentration of 25 minutes.

The regional rainfall characteristics are described by an intensity-duration-frequency (IDF) relationship with the following parameters: $K = 819.67$, $a = 0.138$, $b = 10.77$, and $c = 0.75$ (Gomes Jr et al., 2021). The infiltration basin is to be constructed in sandy soil, with the following properties: saturated hydraulic conductivity $k = 120.4$ mm $\cdot$ h^{-1}, effective porosity $\Delta\theta = 0.42$, and suction potential $\psi = 49.5$ mm. Initial conditions for the system include a starting infiltration depth of $F(0) = 5$ mm and an initial surface ponding depth of $h(0) = 0$ mm.

The primary task is to calculate the required surface area and design ponding depth for the retention basin using the optimization procedure outlined in Eq. (12), ensuring that the maximum simulated ponding depth does not exceed the design height. The optimized results will then be compared with an alternative scenario where the ponding depth is arbitrarily fixed at 0.8 m to assess the risks of over- or under-design in the absence of proper optimization.

3.1. Solution

The runoff coefficient for the post-development scenario is determined using a weighted average based on the proportional coverage of the site:

$$C_{\text{pos}} = (0.4 \times 1) + (0.6 \times 0.6) = 0.76$$

Next, the design rainfall intensity is calculated by applying the time of concentration for the post-urbanized condition into the IDF formula:

$$i_{\text{pos}} = \frac{819.67 \cdot 5^{0.138}}{(10.77 + 10)^{0.75}} = 69.68 \text{ mm} \cdot \text{h}^{-1}$$

Using this intensity, the peak flow rate for the post-development case is computed as:

$$Q^p_{\text{pos}} = 0.76 \times \left(\frac{69.68}{1000 \times 3600}\right) \times 5000 = 0.111 \text{ m}^3 \cdot \text{s}^{-1}$$

For the pre-development (pasture) scenario, the peak discharge is:

$$Q^p_{\text{pre}} = 0.034 \text{ m}^3 \cdot \text{s}^{-1}$$

By simulating both pre- and post-urbanization hydrographs according to Eq. (4), the results can be visualized in Fig. 2(b). The minimum volume required for the basin to offset the additional runoff, as calculated from Eq. (5), is $V_{\text{min}} = 52.14$ m^3.

If we initially assume a design ponding depth of $h = 0.8$ m, the basin would need to cover an area of 65.18 m^2, which is approximately 1.3% of the total contributing catchment. However, the simulation with this preset height reveals that the maximum ponding depth surpasses the design height, indicating that the system is inadequately sized when using this arbitrary assumption.

The simulation shows that the ponding depth in the arbitrarily designed system peaks at 88 cm, which exceeds the assumed design height and suggests that overflow could occur.

When the optimization routine defined in Eq. (12) is applied, the model finds that a design ponding height of 0.36 m and a required surface area of 143 m^2 (which corresponds to roughly 2.9% of the drainage area) provide a viable solution. This optimized configuration ensures that the minimum storage requirement is met, even under the conservative condition where infiltration is fully impaired. The optimized larger surface area improves the infiltration response, aligning the system performance with the design parameters and avoiding overtopping risks. The time-dependent behavior of infiltration, ponding depth, and stored volume for both cases is depicted in Fig. 2.

4. Discussions

The analysis conducted in Example 1 demonstrates that arbitrarily selecting the ponding depth is not a reliable strategy when designing retention basins. Such a decision may result in either an undersized or an oversized system. For detention basins that do not rely on infiltration, adjusting the surface area can usually compensate for design height variations, since the primary concern is storage volume alone. However, this flexibility does not apply to retention basins where infiltration is a key process. In these systems, an arbitrary ponding height significantly impacts infiltration dynamics and may cause ponding depths to exceed design limits, especially due to the non-linear relationship between infiltration capacity and ponding depth.

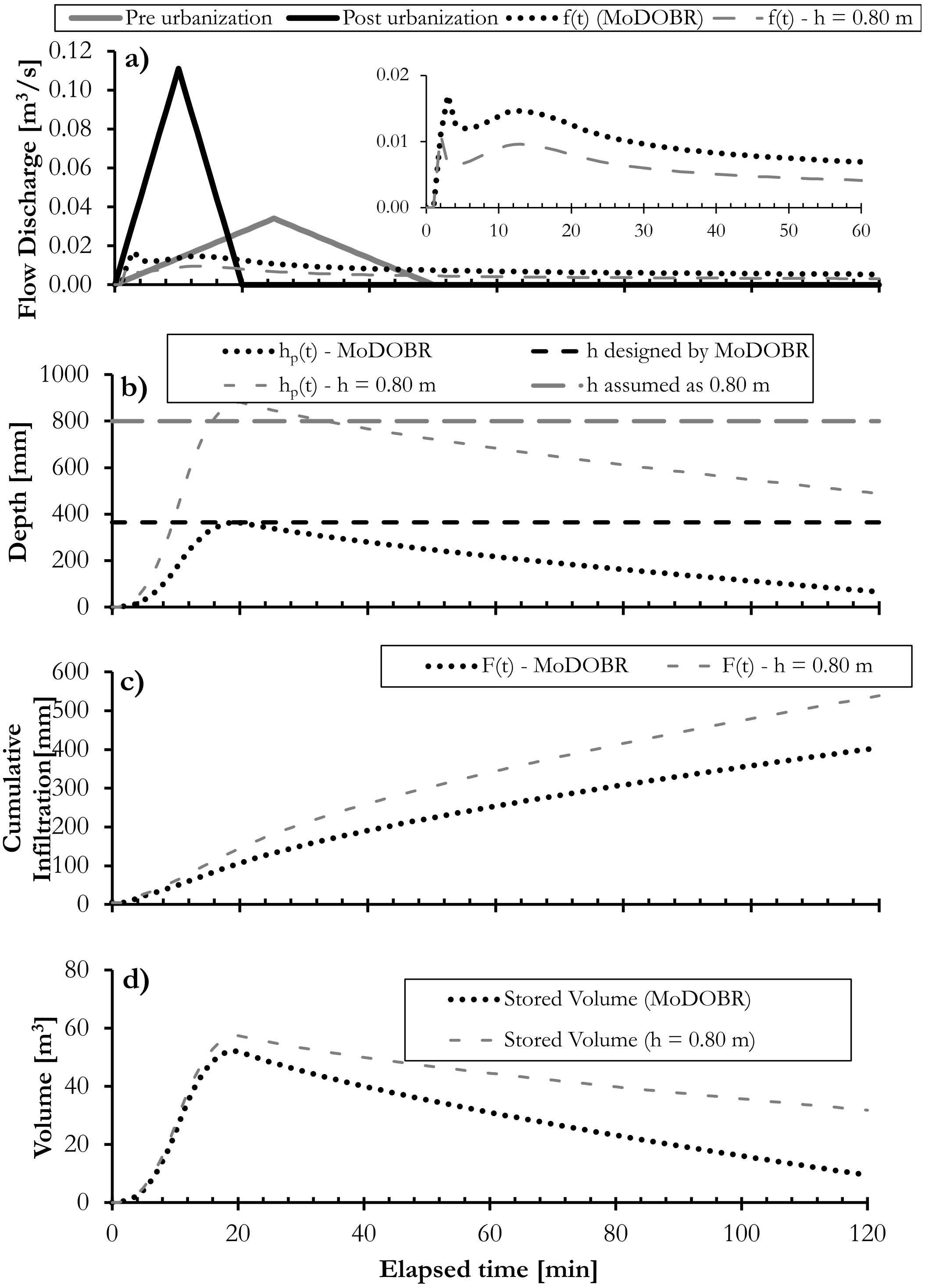

Figure 2: Hydrological simulation comparing an arbitrarily selected ponding height of 0.80 m with the optimized solution derived from Eq. (12). (a) Inflow and infiltration curves over time. (b) Evolution of ponding depth for both scenarios. (c) Cumulative infiltration during the simulation period. (d) Volume storage variation over time.

The scenarios explored in this study consistently apply a conservative design perspective, where it is assumed that the filtration layer may become entirely clogged over time and that auxiliary drainage infrastructure might fail. This cautious approach ensures that the system is robust enough to fully store the surplus runoff generated by urbanization without exceeding the designed storage capacity, even under degraded long-term conditions.

5. Conclusions

This study presented the development and application of the Model for Optimized Design of Retention Basins (MoDOBR), tested through four numerical examples. The proposed methodology is accessible and can be executed using spreadsheet software, offering a simple yet robust tool for practical design applications. An open-source spreadsheet version of the model has been made available to facilitate replication and to support the design of various infiltration systems.

MoDOBR requires fundamental input data about the catchment characteristics and soil properties specific to the base of the retention basin. Although the primary focus is on retention systems, the model can be readily adapted to other infiltration-based techniques such as rain gardens, infiltration basins, detention basins (assuming zero infiltration), and permeable pavements.

Based on the case study analyses, the following key conclusions were drawn:

- Choosing a design ponding height without verifying its compatibility with the maximum simulated ponding depth can result in systems that are either undersized or oversized. When spatial or height limitations exist at the project site, incorporating perforated underdrains can help align the system's performance with the design constraints.
- For each design condition, there is a single ponding height that satisfies both volume and drainage time requirements, provided that the defined lower and upper bounds on the design height are sufficiently broad. This guarantees that the optimization problem is solvable using gradient-based algorithms. In this study, the Excel GRG Nonlinear solver proved to be an effective and practical solution method.
- The proposed method ensures that, even if the soil becomes fully clogged and infiltration is completely obstructed, the surface volume of the retention basin remains adequate to manage the excess runoff produced by urban development.

Although this study employed a triangular hydrograph based on the rational method, the proposed modeling framework is flexible and can be adapted to incorporate alternative hydrograph shapes. In the present analysis, the rainfall duration was assumed to be equal to the catchment's time of concentration. Future investigations should consider the impact of longer rainfall durations to assess how this assumption influences design outcomes. Additionally, incorporating economic factors such as land value, excavation expenses, and construction costs in future developments of the model could support the creation of an optimized cost-based design procedure. In such cases, the compatibility of ponding heights would become an explicit constraint within the cost minimization process. The spreadsheet tool developed for this study is openly available for public use in the online repository referenced in (Gomes Jr., 2024).

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