Universidade Federal da Paraíba

Centro de Tecnologia

PROGRAMA DE PÓS-GRADUAÇÃO EM ENGENHARIA CIVIL E AMBIENTAL

– MESTRADO–

PERFORMANCE OF DIFFERENT CONFIGURATIONS OF ROOFTOP RAINWATER HARVESTING USED FOR MANAGED AQUIFER RECHARGE: A STORMWATER MANAGEMENT APPROACH IN AN URBANIZED AREA

Por

Victor Santos Galvão Baptista

Dissertação de Mestrado apresentada à Universidade Federal da Paraíba para obtenção do grau de Mestre

João Pessoa – Paraíba **Novembro de 2020**

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Dissertação submetida ao Programa de Pós-Graduação em Engenharia Civil e Ambiental da Universidade Federal da Paraíba, como parte dos requisitos para a obtenção do título de Mestre.

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Mestrado e Doutorado

Ata da reunião da banca julgadora da 324ª Dissertação de Mestrado do PPGECAM apresentada pelo mestrando **VICTOR SANTOS GALVÃO BAPTISTA,** no dia 17 de setembro de 2020.

Aos dezessete dias do mês de setembro do ano de dois mil e vinte, às nove horas, ocorreu, através da plataforma Google Meet, no endereço **https://meet.google.com/xap-ryrz-kae**, a apresentação da dissertação de mestrado intitulada ***"PERFORMANCE OF DIFFERENT CONFIGURATIONS OF ROOFTOP RAINWATER HARVESTING USED FOR MANAGED AQUIFER RECHARGE: A STORMWATER MANAGEMENT APPROACH IN AN URBANIZED AREA"***, pelo mestrando **VICTOR SANTOS GALVÃO BAPTISTA**, matrícula 20181014283, do corpo discente deste Programa, com vistas à obtenção do título de **Mestre em Engenharia Civil e Ambiental**. A Banca Examinadora esteve composta pelos Professores Doutores: Cristiano das Neves Almeida - UFPB (Orientador), Victor Hugo Rabelo Coelho – UFPB (Coorientador), Gustavo Barbosa Lima Silva - UFPB (Examinador Interno), Suzana Maria Gico Lima Montenegro – UFPE (Examinadora Externa), e pelo Doutor Catalin Stefan – TU Dresden (Examinador Externo), sendo presidida, por indicação dos seus membros, pelo Prof. Dr. Cristiano das Neves Almeida. Instalada a Banca Examinadora, o senhor Presidente passou a palavra ao mestrando para apresentar a defesa do seu trabalho de dissertação. A exposição oral iniciou às _9:05_, sendo concluída às _9:52_, iniciando-se, logo a seguir, a arguição pelos examinadores, finalizando às _11:30_. Em seguida, o senhor Presidente convidou a Banca Examinadora a reunir-se reservadamente para deliberação. Concluída a reunião, o senhor Presidente convocou o mestrando e demais presentes para proclamar o resultado, sendo atribuído por unanimidade, ao mestrando, o conceito **_APROVADO_**, nos termos do Regulamento Geral dos Cursos e Programas de Pós-Graduação *Stricto Sensu* da Universidade Federal da Paraíba (Resolução nº 12/00-CONSEPE) e do Regulamento do Curso de Pós-Graduação em Engenharia Civil e Ambiental do Centro de Tecnologia da Universidade Federal da Paraíba (Resolução nº 05/2018-PPGECAM). Ao final da sessão, após os agradecimentos por parte do mestrando, o senhor Presidente, por recomendação dos membros da banca, estabeleceu um prazo máximo de _30_ dias para o mestrando providenciar as correções recomendadas, ficando a emissão do diploma do título de Mestre condicionada a essas retificações que serão verificadas pelo Orientador. Às_11:45_hs, o senhor Presidente encerrou os trabalhos, determinando a leitura para fins de aprovação e a lavratura da presente ata que vai assinada por todos os membros da Comissão Examinadora.
João Pessoa, 17 de setembro de 2020.

(Orientador):	PROF. DR. CRISTIANO DAS NEVES ALMEIDA
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The urbanization process in urban coastal areas has led to intense groundwater consumption whereas reducing permeable areas and increasing the frequency and magnitude of floods. To compensate these adverse effects, this research investigated the performance of different configurations of rooftop rainwater harvesting systems as tools for at-source managed aquifer recharge and sustainable stormwater management in a Brazilian coastal city located in a sedimentary aquifer system. Several combinations of rooftop area and water tank capacity were tested. The systems are connected to the unconfined Barreiras Formation through a six-inch diameter injection well. Rainfall-runoff processes and water balances were simulated from monitored, high-temporal resolution, rainfall data and insights acquired after experimental tests results (pumping tests and injection tests). Results show that these systems can provide mean annual rainwater retention rates higher than 50% for most studied configurations (tanks capacity higher than 3 m³ and catchment areas ranging from 10 to 5,000 m²). When managed aquifer recharge is the priority of a rainwater harvesting system, results indicate that rainwater retention rates of 75% or higher must be pursued. Lower rates would only produce substantial increases in runoff overflowing to the downstream drainage network. When flood control and mitigation is pursued, the efficiency of the system must be close to 100%, a fact that produces rainwater retention close to 100% as well. This study shows the importance of coordination of sustainable urban drainage solutions and managed aquifer recharge schemes, an approach that can contribute to raising the groundwater supply in urban areas while reducing the risk and severity of floods.

KEYWORDS: Flood control, rooftop rainwater harvesting, managed aquifer recharge.

RESUMO

O processo de urbanização em áreas urbanas costeiras tem levado ao consumo intenso de água subterrânea enquanto tem reduzido as áreas permeáveis e aumentado a frequência e a magnitude das inundações. Para compensar esses efeitos adversos, essa pesquisa investigou o rendimento de diferentes configurações de sistemas de captação de água de chuva em telhados como uma ferramenta para recarga gerenciada de aquíferos na fonte e gestão sustentável do escoamento superficial, em uma cidade brasileira costeira, localizada em um sistema-aquífero sedimentar. Variadas combinações de área de captação e capacidade de caixa d'água foram testadas. Os sistemas se encontravam conectados à Formação Barreiras, não-confinada, através de um poço de injeção de seis polegadas de diâmetro. Processos de chuva-vazão e de balanço hídrico foram simulados a partir de dados de chuva monitorados, com alta resolução espacial, e de percepções adquiridas após resultados de testes experimentais (testes de bombeamento e testes de injeção). Os resultados indicam que esses sistemas podem prover taxas médias anuais de retenção de água de chuva maiores que 50% para a maioria das configurações estudadas (tanques com capacidade superior a 3 m³ e áreas de captação variando de 10 até 5.000 m²). Quando a recarga gerenciada do aquífero é a prioridade de um sistema de coleta de água da chuva, os resultados indicam que taxas de retenção de água da chuva de 75% ou mais devem ser buscadas. Taxas mais baixas produziriam apenas aumentos substanciais no transbordamento do escoamento para a rede de drenagem a jusante. Quando se busca o controle e mitigação de enchentes, a eficiência do sistema deve ser próxima a 100%, fato que também produz taxas de retenção de água da chuva próximas a 100%. Este estudo mostra a importância da coordenação de soluções de drenagem urbana sustentável e esquemas de recarga gerenciada de aquíferos, uma abordagem que pode contribuir para aumentar o abastecimento de água subterrânea em áreas urbanas, ao mesmo tempo que reduz o risco e a gravidade das inundações.

PALAVRAS-CHAVE: Controle de enchentes, coleta de água de chuva de telhado, recarga gerenciada de aquíferos

SUMMARY

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1 INTRODUCTION

Sustainable social and economic development relies on water resources available to meet current and future domestic, agricultural, and industrial water demands. However, urbanization (among population growth) leads to rising water demand (Gale and Dillon 2005), which poses a threat to groundwater resources (Jacobson 2011; Gleeson et al. 2012; Coelho et al. 2018); produces changes in the natural environment that increases the frequency and severity of floods (Trigo et al. 2016; Eckart et al. 2017); and causes ecological degradation in urban areas (Silva et al. 2006; Oleson et al. 2015; Bertrand et al. 2016). These issues are especially important for coastal urban cities since half of the World's population and human activities are concentrated on surface coastal zones (Chatton et al. 2016). Many coastal water resources are limited and dependant upon groundwater, which is critical for cities in developing countries, where rapid demographic growth is noticed, and basic services are not evenly distributed among inhabitants (Dillon et al. 2018). Thence, provision of sufficient storage capacity under growing water demand and increasing climate variability is among the concerns of water managers (Tuinhof and Heederik 2002).

Although the current global annual groundwater exploitation is estimated at around 8% of global mean annual recharge, it is responsible for major groundwater depletion locally, particularly in arid and semi-arid areas (Dillon et al. 2018). Arguments gathered from Dillon et al. (2018) leads to the conclusion that the resource cannot be seen as renewable in practice since 1) the minimum estimation of the global groundwater mean residence time is higher than 250 years (Dillon et al. 2018) and 2) more than 94% of the estimated total volume of groundwater is aged more than 100 years (Gleeson et al. 2016).This means that the alarming rate of groundwater depletion (145 km^3/year, Konikow 2011) is not likely to reduce due to natural recharge but to keep increasing in the coming years, representing a bigger share on the global groundwater extraction (currently, 980 km^3/year, Margat and van der Gun 2013). Among specific consequences of groundwater depletion in urban coastal areas there are saline water intrusion (Bertrand et al. 2016; Coelho et al. 2018), which can disable existing extraction wells; and land surface subsidence, phenomena that have the potential to prevent future aquifer recharge due to the decrease in porous spaces, in addition to causing irremediable damage to structures (Silva et al. 2006; Valverde et al. 2018).

While urbanization and population growth contribute to groundwater depletion by a poorly managed water demand, they also put more stress on conventional stormwater management systems (Chen et al. 2016) by increasing the risk and magnitude of urban flooding

(Eckart et al. 2017). Therefore, they contribute to an undesired reduction of groundwater resources and also to an undesired augmentation of surface runoff. Currently, the former is widely viewed as a valuable resource (Dillon et al. 2018) while the latter is commonly seen only as a problem (Gimenez-Maranges et al. 2020). Flooding is defined as the overflow of watercourses and/or the generation of surface runoff in areas usually not submerged during precipitation events (Gimenez-Maranges et al. 2020). The prevalence of impervious areas in the urban catchments can reduce the concentration time of watersheds to scales much smaller than the duration of extreme rainfall events thus increasing the probability of a flood event to cause loss of life, injury or other health impacts, property damage, social and economic disruption, and environmental damage (Dethier et al. 2016; Trigo et al. 2016). Climate change puts further stress on groundwater resources and stormwater management systems since it leads to more frequent climatic extremes (floods and droughts). IPCC (2007) stated that heavy precipitation events are likely to occur in the future (with a >90% probability of occurrence), which will consequently lead to intensified flood events, with alarming consequences particularly for developing countries where investments in water infrastructure are already below needs (Tuinhof and Heederik 2002).

There is an urgent need for new urban water management strategies to deal with climate change, urbanization, and social and ecologic concerns (Tuinhof and Heederik 2002; Fletcher et al. 2015; Eckart et al. 2017; Sohn et al. 2019; Gimenez-Maranges et al. 2020). Sustainable strategies for stormwater management have been developed in many countries (Fletcher et al. 2015) and are currently being applied worldwide (Eckart et al. 2017; Kaykhosravi et al. 2018; Sohn et al. 2019; Gimenez-Maranges et al. 2020). Called as sustainable urban drainage systems (SUDS) or sustainable drainage systems (SuDS) in Europe (Fletcher et al. 2015; Eckart et al. 2017), they are claimed to restore hydrology in the catchment scale (Fletcher et al. 2015) by mimicking natural processes (Perales-Momparler et al. 2017) through surface runoff/peak flow reduction (Burns et al. 2012; Sohn et al. 2019) and infiltration/baseflow improvement (Burns et al. 2012; Eckart et al. 2017). Furthermore, SUDS are expected to play a role in meeting the United Nations sustainable goals (Gimenez-Maranges et al. 2020) through dispersing application within catchments, matching natural hydrological features (Sohn et al. 2019) and providing multiple benefits rather than hydrologic restoration, such as regulation of biochemical cycles (Eckart et al. 2017), protection of groundwater and its dependant aquatic ecosystems against non-point pollutant sources (Fletcher et al. 2015; Sohn et al. 2019), improvement of urban biodiversity, carbon dioxide sequestration, and reduction of urban heat effect (Charlesworth 2010; Woods Ballard et al. 2015).

Rooftop rainwater harvesting (RWH) systems are among initiatives to provide an additional source of water to meet domestic water demand, widespread particularly in arid and semi-arid areas (Li and Gong 2002; Kumar et al. 2016; Taffere et al. 2016; Almazroui et al. 2017; Santos and de Farias 2017; Shubo et al. 2020), which contributes to reducing the usage of groundwater and mains water to meet less noble, non-potable water uses (Adham et al. 2016; Palla et al. 2017). RHW systems are adequate especially in consolidated urban areas with few or almost no available space for retrofit (Brown et al. 2016). RHW systems are usually connected to the rooftop of domestic buildings and houses, designed for cleaning of houses and clothing, toilet flushing (Palla et al. 2017; Teston et al. 2018; Freni and Liuzzo 2019), garden irrigation (Petrucci et al. 2012), car washing (Burns et al. 2012), etc. Besides, many initiatives have been carried out recently to study the potential of urban RWH systems for stormwater management besides water conservation objectives, hence enlisting it in the technologies comprising SUDS (Burns et al. 2012; Petrucci et al. 2012; Teston et al. 2018; Freni and Liuzzo 2019). Palla et al. (2017) claimed that RWH systems implemented at the urban catchment scale operate as source control solutions which contribute to limit overflow discharges thus reducing the amount needed for pre-treatment before discharge in the receiving water bodies. Other expected benefit of RWH systems is their contribution towards resilience to climate change (Charlesworth 2010; Woods Ballard et al. 2015; Eckart et al. 2017).

RWH systems act by intercepting surface water runoff from rooftops of buildings and conveying it to storage structures designed for water conservation and/or stormwater management (Burns et al. 2015; Palla et al. 2017). Both objectives are reconciled since the prescribed water demand gives a destination to surface runoff which would otherwise be directed to downstream drainage network, however, studies have found that conventional demands are not sufficient to empty storage structures right during successive and/or before heavy rainfall events thus leading to poor stormwater management (Petrucci et al. 2012; Burns et al. 2015; Palla et al. 2017; Teston et al. 2018; Freni and Liuzzo 2019). A solution mentioned by Burns et al. (2015) is to increase the effective demand by diverting overflow to infiltration systems, as an empirical study by Burns et al. (2012) have shown the retention strategy comprising a tank overflowing to a green infiltration system was able to restore a typical urban land parcel close to the pre-development state. Another way to go, least explored, is to increase the effective water demand by conveying overflow from urban RWH systems via at-source direct injection into aquifers. This is claimed to compensate for impacts of large impervious areas and the high groundwater abstraction under many cities, a solution that can reduce wastewater infrastructure costs and improve public health in general (Tuinhof and Heederik

2002).

RWH systems which collect and store rainwater temporarily in storage structures, designed for intentional and controlled aquifer recharge, are also part of managed aquifer recharge (MAR) technology (Zhang et al. 2020). MAR is a viable and sustainable alternative for the augmentation of water availability and to engage against numerous environmental and socioeconomic issues (Dillon et al. 2018). MAR consists of broad and diverse strategies aimed at maintaining, enhancing, and securing groundwater systems (Stefan and Ansems 2018). In the last decades, MAR has undergone remarkable growth, with the expectancy of about ten times increase of current reported global water capacity in the future (Dillon et al. 2018). Besides, MAR is expected to increase in the coming years to overcome global challenges in Latin America and the Caribbean - LAC (Valverde et al. 2018), in a proportion at least similar to the current level of India and the USA, which have similar groundwater use and where MAR is much more established (Dillon et al. 2018). In general, MAR methods are used to recharge aquifers purposefully, leading to increase in groundwater supply in wet seasons with an expectance of use shortly or at dryer seasons (Gale and Dillon 2005; Kretschmer 2017; Dillon et al. 2018).

MAR schemes have a wide range of applications, including water resources adjustment, ecological and environmental protection, water quality improvement, and utilization of geothermal resources (Zhang et al. 2020). One objective of the environmental protection context is flood control and mitigation. The most explicit MAR method which manages floods is the method that takes the same name – controlled flooding, a technique used to divert excess river water or floods over a large available land area, enough to promote spreading as a thin sheet that enhances infiltration (Tuinhof and Heederik 2002; IGRAC 2007). Other MAR methods also contribute to stormwater management by detaining stormwater and enhancing groundwater supply: recharge dams, mentioned by Dillon et al. (2018) in an arid area in Namibia, where an alluvial aquifer is recharged using floodwater, and in Oman, where 43 dams are used to support irrigation and protect against devastating flash floods (Tuinhof and Heederik 2002; Dillon et al. 2018); aquifer storage and recovery wells (ASR; Maliva et al. 2020), e.g. in sites in Saudi Arabia (Missimer et al. 2015; Alataway and El Alfy 2019) and South Australia (Kretschmer 2017); boreholes (or ‘dry wells’), in Mexico (Valverde et al. 2018); bank filtration schemes, e.g. in India (Sandhu et al. 2018); and RWH systems in Australia (Page et al. 2010), Brazil (Silva et al. 2006), India, and China (IGRAC 2020). In any of these examples, groundwater recharge is pursued whereas flood control and mitigation are promoted through underground storage.

Underground storage, that is, storage of water within confined and/or unconfined aquifers has many advantages over surface-water storage (in both large dam reservoirs, small dams, and surface reservoirs; (Keller et al. 2000). Although not being as wide-spread as the storage of water in surface reservoirs (Zhang et al. 2020), underground storage does not have evaporation losses nor the possibility of structural failure intrinsic to surface reservoirs following disasters, e.g. earthquakes and floods (Bouwer 2002; Tuinhof and Heederik 2002; Minsley et al. 2011). Some methods of aquifer storage might also require smaller land area than surface-water storage and are less prone to sediment accumulation (Tuinhof and Heederik 2002), algae blooms and atmospheric fallout of pollutants (Hartog and Stuyfzand 2017). Besides, global current surface-water storage in dams and lakes, albeit uncertain, ranging from 12.900 km^3 to 120.000 km^3 (Tuinhof and Heederik 2002; Dillon et al. 2018) is much times smaller than the estimated global storage volume of groundwater (13.3 – 31.7 million km^3, (Gleeson et al. 2016), which is even higher when including the potential of void space of aquifers to receive recharge (Tuinhof and Heederik 2002). This shows the opportunity of using managed aquifer recharge schemes over surface-water reservoirs for temporary storage, especially in a context where the construction of new large dams is being discouraged due to its ecological impacts and unequal distribution of benefits over affected communities (Dillon et al. 2018). Underground storage also facilitates logistic of supply and demand in place and time, since it can enable the capture of water at-source and its on-demand reuse (Keller et al. 2000; Tuinhof and Heederik 2002). Conjunctive use of subsurface storage and surface dams is often better than solely the former since such an implementation promotes the optimal deployment of the latter (Tuinhof and Heederik 2002), however, subsurface water storage has been through no significant increase in contrast to the large increase in dam capacity over the 1950s onwards (Tuinhof and Heederik 2002).

Storing runoff from urban rooftop catchments directly into aquifers through injection wells seems to be a feasible solution since they simultaneously contribute to controlled flooding and to increase groundwater supply. This is particularly true in arid and semi-arid areas where recharge enhancement has the potential to store excess runoff into fractured rock aquifers (Dillon 2005), and also in urban areas, where recharge wells have greatly enhanced recharge in the last sixty years (Dillon et al. 2018). Such practice of water resources management could allow a gradual recovery of the water table from depleted aquifers beyond diminishing flooding in the rainy season (Silva et al. 2006). Recently, (Alataway and El Alfy 2019) estimated a 44% increase of groundwater recharge in a shallow unconfined aquifer by the usage of injection wells to receive rainwater intercepted by dams, with a concomitant reduction of 86% in the evaporative losses. (Yang and Scanlon 2019) investigated the quantity of water that could be stored into the Texas Gulf Coastal Aquifer system to reduce high magnitude flows. They concluded that 37 km³ of water could be intercepted in 2017-2018 to recharge the depleted aquifer system. These two above-mentioned studies exemplify that the storage of rainwater from floods into aquifers results in many benefits such as the use of water during drought events (Dillon et al. 2018) and the enhancement of the water system resilience (Gale and Dillon 2005; Kretschmer 2017). Moreover, depleted aquifers can also exhibit a much larger and cheaper storage capacity when compared to surface reservoirs (Perrone and Rohde 2016; Scanlon et al. 2016; Yang and Scanlon 2019). Dillon (2005) also stated that dams need to be several orders of magnitude larger than underground storage to be more economic. Examples of such hypothesis are confirmed by experimental investigations assessing aquifer recharge rates by small-diameter injections wells (e.g., Händel et al. 2016; Liu et al. 2016), which concluded that small-diameter structures represent a much more cost-effective technology when compared to surface basins.

MAR and SUDS research fields are multidisciplinary in nature, and many of their aspects intersect. Hence, integration of SUDS and MAR research and implementation could benefit both fields. SUDS implementation and evaluation of hydrological function and pollutant removal capabilities require monitoring of groundwater flow, which is generally neglected in the practice of SUDS (Jacobson 2011) whereas it is a crucial aspect of MAR schemes (Zhang et al. 2020). Moreover, flood control, one of the least reported objectives of MAR projects (mentioned in a generalist manner, among other ecological benefits; Dillon et al. 2018; Zhang et al. 2020), but the main goal of urban drainage systems, could give one more argument towards its implementation in urban catchments, making it more interesting for stakeholders and the public. As stated by Dillon et al. (2009), it is often a combination of multiple benefits that makes a MAR project financially competitive compared to the traditional water supply.

Important review papers concerning MAR and SUDS show evidence that more effort should be put into monitoring activities (Eckart et al. 2017; Dillon et al. 2018; Sohn et al. 2019; Gimenez-Maranges et al. 2020; Zhang et al. 2020). For instance, the lack of adequate monitoring prevents a) predicting and managing clogging adequately, as well as deeply understanding the subsurface water purification mechanism (Zhang et al. 2020), which is a concern for both infiltration-based SUDS technologies and MAR; b) existing MAR operations to assist reliably in the planning and implementation of future projects globally (Dillon et al. 2018; Zhang et al. 2020); c) comparing multiple MAR methods across multiple sites (Dillon et al. 2018), which is also true for SUDS methods (Gimenez-Maranges et al. 2020); d) establishing climate impacts on SUDS effectiveness (Sohn et al. 2019); and e) drawing meaningful conclusion on SUDS performance in the long-term basis (Clary et al. 2011), under varying spatial scales (Sohn et al. 2019), temporal scales (Campisano and Modica 2015; Sohn et al. 2019) and climate conditions (Sohn et al. 2019), which is also true for some MAR methods such as streambed modifications (Dillon et al. 2018). Adequate timescale is required in hypothetical and empirical research comprising RWH systems focused on stormwater management and aquifer recharge. For reliable assessment of the efficiency of RWH systems, Campisano and Modica (2015) concluded that a) at least hourly time steps are required for evaluation of tank volumetric retention, and that b) sub-hourly time steps are mandatory for the evaluation of stormwater peak reduction. Thence, MAR and SUDS monitoring must focus on both water quantity and water quality aspects, working with small temporal units (e.g. 1-minute to 5-minute resolution), during long-term periods (around 10-30 years or more). MAR could benefit from national monitoring and research programs, as suggested by Dillon et al. (2018), which is also true concerning the SUDS field. The evidence founded on real-time monitoring could be useful to supplement experimental studies controlling interacting factors other than climate conditions and hypothetical studies focused on testing diverse hypotheses such as those related to a changing climate environment (Sohn et al. 2019).

This work presents hypothetical research focused on the technical feasibility of RWH system as a tool for at-source managed aquifer recharge via a direct injection well and sustainable stormwater management in an urban catchment, using long-term data with a high temporal resolution, in an approach integrating empirical and hypothetical research. The usage of injection well is expected to contribute to less space needed for storage, in a context where SUDS sites are still minimally implemented in urban catchments (Sohn et al. 2019). Simulation with the high-temporal resolution is investigated due to its adequate timescale to provide a reliable assessment in terms of stormwater peak reduction (Campisano and Modica 2015). This study proposes a methodology integrating urban stormwater management with groundwater recharge thus introducing an integration among MAR and SUDS paradigms in Brazil, where such coordination is not present (Shubo et al. 2020), besides contributing to insights destined to support effective sustainable urban drainage policy and cope with climate variability (Sohn et al. 2019). Results from this study are also part of steps towards the implementation of a MAR site in João Pessoa (JPA), Brazil, aiming at systematic water quality and volume monitoring, at a high temporal resolution, to allow a scientific approach to support the uptake of MAR in the country (Dillon et al. 2018; Shubo et al. 2020) and contribute towards reducing its associated risks, in the context of the international project SMART-Control INOWAS (2020).

2 OBJECTIVES

The main objective of this work is to investigate the performance of an RWH system as a tool for managed aquifer recharge and sustainable stormwater management in a coastal urban city located in a sedimentary aquifer system in the Northeast Region of Brazil. The specific goals of this study are to:

- Integrate a hypothetical water balance method with empirical data to enable reliable long-term simulations of the RWH system, with high temporal resolution
- Determine water tank volumes which optimize the performance of the RWH system in the function of the rooftop catchment areas, when it is designed for managed aquifer recharge or when it is designed for urban stormwater management, focusing in restoring the hydrology of the site back to its assumed pre-development state

3 THEORETICAL REFERENCE

This section comprises the concept of managed aquifer recharge (MAR), its historical background, its technology, distinguished in broad categories, and the key elements for its successful implementation. Besides, it also comprises the concept of conventional and sustainable urban drainage systems (SUDS), its historical background, its technology and the factors affecting its implementation. Up-to-date challenges and constraints for MAR and SUDS are also detailed.

3.1. Managed aquifer recharge (MAR)

Managed aquifer recharge (MAR) stands for processes that use engineering principles to administer water into aquifers in a controlled way. It refers to methods used to maintain, enhance and secure stressed groundwater systems (Dillon et al. 2018; Stefan and Ansems 2018) by purposefully recharging aquifers aiming at a future recovery of the recharged water or for environmental benefits (NRMMC et al. 2009). MAR also represents an important strategy for sustainable water resources management (Dillon et al. 2009; Ringleb et al. 2016), whose current applications include water resources adjustment, ecological and environmental protection, water quality improvement and utilization of geothermal resources (Zhang et al. 2020). Including the benefits of aquifer storage mentioned in the introduction (over surface-water storage), MAR can also solve the mismatch between water supply and demand, which are often present in surface-water-based management schemes (Hartog and Stuyfzand 2017).

The term MAR cut across three basic concepts. The first concept is *recharge*, a natural process of water input into aquifers, a component of the water cycle where precipitation, surface and/or subsurface flows infiltrate into saturated zones (Tuinhof and Heederik 2002). *Aquifers* are all geological strata that receive this recharged water, whose borders are geological controls considered impermeable or semi-impermeable, favouring the storage and/or distribution of water. And the word *managed* is related to the artificial process, engineered, where water is intentionally conducted to aquifers via injection and/or infiltration structures, aiming at enhancing the natural recharge (Dillon et al. 2018). The term managed aquifer recharge was first coined by the hydrogeologist Ian Gale (Gale and Dillon 2005) and it became prevalent among other terms (e.g. artificial recharge) since it implies that risks are managed in a quantitative way (Pervin 2015).

Groundwater renewal, under natural conditions, usually occurs over geological timescales (Coelho et al. 2018) but its value as a resource has been raised only in recent years. Zhang et al. (2020) state that, until the period comprising the mid-19th century and mid-20th century, water demand was solely sustained by surface-water schemes, which were compromised by the growing water demand by industries and also by its poor management (the inadequate sanitation and industrial pollution at the time). Besides, since the 1960s, most anthropogenic actions to raise the groundwater supply were unmanaged or incidental. According to Dillon et al. (2018), until then, drainage wells for flood relief, disposal of sewage water via septic tanks or seepage beneath surface-water irrigated crops all contributed to aquifer recharge but were not responsibly managed (e.g. risks of groundwater pollution were not considered) nor intended (e.g. aquifer recharge not thought as an objective of such schemes). The actions promoted undesired consequences such as waterlogging, land salinization, or groundwater pollution (Dillon et al. 2018). Although valued as a renewable resource, always available to humankind, groundwater extraction was limited due to technical constraints back then (Dillon et al. 2018).

After the mid-1960s, when the oil industry’s electric submersible pump was adapted to be used in water pumping from deep wells, alongside with the already existent widespread electric distribution and rotary drilling technology, unprecedented groundwater extraction and exploitation has been observed, especially in arid and semi-arid regions where groundwater extraction is critical (Dillon et al. 2018). Since then the concept of groundwater as a renewable resource has been threatened and its value as an essential but limited resource has been raised. Groundwater became prevalent to solve population growth challenges such as the increasing water demand of concentrated urban areas and increasingly agriculture demand for the raised food production that accompanied it, including other important socio-economic factors (Dillon et al. 2018). Besides, accelerating global groundwater exploitation rates (15% of global groundwater extraction; Konikow 2011; Margat and van der Gun 2013) poses an increasing risk of water shortage due to the slow residence time of groundwater systems, generally higher than 250 years (Dillon et al. 2018).

Managed aquifer recharge development followed the gradual rise of groundwater in importance (Dillon et al. 2018; Zhang et al. 2020). Zhang et al. (2020) divided its historical development into four stages. Initially (221 BC to 1850 AD) MAR was mainly applied to provide a raise in groundwater level for agriculture irrigation, although ancient schemes for water supply augmentation and runoff control are registered. MAR raised in development in theory, technology, and application during the 1850-1950 period, after MAR schemes were successfully implemented to improve surface-water quality and increase its availability in the context of the Second Industrial Revolution (Zhang et al. 2020); and also in the 1950-1990 period, where an urgent need of water availability in terms of quality and quantity arose from the post-war (2nd world war) reconstruction and construction-related activities (Zhang et al. 2020). From 1990 onwards, MAR further widened in scope and application in various countries to meet water demand problems caused by climate change, urbanization, and population expansion, as well as for environmental reasons (Dillon et al. 2018). It is being applied in both developed and developing countries in the assessment, design, and operation of MAR schemes, totalizing around 10 km^3/year of capacity nowadays (Dillon et al. 2018) – about 1% of estimated global groundwater extraction (Margat and van der Gun 2013). MAR is seen as an important water management strategy to mitigate future impacts of climate change (Dillon et al. 2018) and to alleviate widespread, current, and future water scarcity problems, in the context of a growing global population (Zhang et al. 2020). According to Dillon et al. (2018), global MAR capacity is expected to increase in 10 times in the future, and to be applied to a wider range of conditions and settings since their techniques provide robust, effective, sustainable, and cost-effective freshwater management solutions (Zhang et al. 2020). MAR is expected to play a major role to meet the UN Millennium Goal for Water Supply, especially for communities in semi-arid and arid areas (Dillon 2005).

3.1.1. Categories of MAR technology

MAR technology is currently recognized into five sub-categories (Zhang et al. 2020), as a result of a changing terminology process on the literature (e.g. Tuinhof and Heederik 2002; Gale and Dillon 2005; IGRAC 2007; Escalante et al. 2016; Stefan and Ansems 2018). These five sub-categories are distributed within two broad categories (Stefan and Ansems 2018):

techniques a) referring primarily to getting water infiltrated (spreading methods or water spreading, induced bank filtration, and recharge wells or well, shaft and borehole recharge) and techniques b) referring primarily to intercepting the water (streambed or in-channel modifications and runoff/rainwater harvesting). Dillon et al. (2018) addressed MAR technology into four sub-categories, stating that the runoff/rainwater harvesting sub-category refers to any of the other methods addressed in the IGRAC MAR Portal (IGRAC 2020). However, since three out of five sub-categories are clearly defined (i.e. spreading methods, induced bank filtration, and in-channel modifications, as stated in the following paragraphs), the confusion is probably within the remaining technologies: recharge wells and runoff/rainwater harvesting, whose definitions are more generalist. It is worth noting that it is common to find some MAR sites with combined technology, e.g. recharge wells schemes combined with spreading methods in Argentina (Valverde et al. 2018), and with in-channel modifications in Colombia (IGRAC 2020), what may raise confusion when classifying these systems into one category or the other. Zhang et al. (2020) sourcing data from the IGRAC MAR Portal (IGRAC 2020), found that MAR technology is distributed globally in the following proportion: 31% are well, shaft and borehole recharge schemes; 29% are spreading methods; 20% are in-channel modifications; 15% are induced bank filtration schemes; and 5% are rainwater/runoff harvesting. According to Valverde et al. (2018), MAR technology is distributed within LAC in the following proportion: slightly less than 50% falls under in-channel modifications; a bit more than 20% are among spreading methods; slightly more than 10% each are well, shaft & borehole recharge schemes and rainwater/runoff harvesting; and less than 10% of the reported MAR projects fall are categorized as induced bank filtration.

Spreading methods are more applied in regions with shallow unconfined aquifers that are natural zones of recharge through permeable material, hence being the most common and cheap MAR techniques (Zhang et al. 2020). Some specific types of MAR spreading methods are infiltration ponds and basins, soil aquifer treatment (SAT), controlled flooding, excess irrigation, ditches, and furrows, etc. (Zhang et al. 2020). One of the oldest MAR technologies (much older than the term MAR itself), its application has been widely used to meet agricultural demand, causing unintentional groundwater recharge, but that has been usually converted into a MAR technology (hence with intentional groundwater recharge to meet proposed uses) after pioneering research was conducted in Arizona (USA) and the Netherlands, in the 1960s and 1970s (Dillon et al. 2018). Infiltration ponds are structures, excavated or enclosed by levees, that retain water to enhance infiltration (IGRAC 2007). SAT is a technique that aims at improving the quality of the source water through a controlled soil percolation using intermittent surface spreading, which requires unsaturated conditions below the infiltration basin (Nadav et al. 2012; Hannappel et al. 2014). IGRAC (2007) provides a short description on the remaining techniques: controlled flooding is a MAR technique that uses excess river water or water from flooding, which is spread as a thin sheet over a large available land area; excess irrigation occurs when water is spread by irrigating cropland with excess water during non-irrigating seasons, that is capable to counteract against intensive agriculture water-related impacts; and ditches, furrows or drains stand for point or linear, interconnected, flat bottomed and closely spaced structures. These structures are either excavated or made of perforated drainage conduits.

Spreading methods represent a substantial share of MAR projects within LAC (All schemes in Paraguay; 50% of all schemes in Argentina; around 45% of Chilean MAR projects; almost 30% of all MAR sites in Mexico; and almost 20% of all schemes in Brazil; Valverde et al. 2018). In LAC, infiltration ponds and basins are found in (1) Argentina, implemented by the Instituto Nacional de Tecnología Agropecuaria (National Agricultural Technology Institute, INTA), in some cases combined with ASR wells, to improve groundwater quality of a shallow unconfined aquifer using rainwater (Valverde et al. 2018); (2) Brazil, well-known as 'barraginhas', widespread in the territory (small dams; Shubo et al. 2020), used in rural areas to prevent soil erosion and to produce food, especially in the semi-arid, and in urban areas however as a component of the drainage system, especially in Natal (Valverde et al. 2018). In fact, Shubo et al. (2020) stated that more than 500,000 infiltration ponds were constructed up to 2013 on the behalf of the Barraginhas Project alone, hence this kind of technology is underreported in estimates by Valverde et al. (2018) and IGRAC (2020); (3) Chile, with projects using river water to maximize natural storage to meet domestic and agricultural demand (Valverde et al. 2018; IGRAC 2020); (4) Mexico, to improve water supply resilience, using stormwater and also reclaimed wastewater (Humberto et al. 2018; IGRAC 2020); and (5) Paraguay, to improve the water quality of the unconfined aquifer, which contains water with high salinity (Valverde et al. 2018). Valverde et al. (2018) reported other spreading methods, least used in LAC, with isolated cases (ditches and furrows used to give a final destination to effluent wastewater in Brazil, following the SAT methodology; a channel spreading used to enhance water supply for agricultural purposes in Chile).

In-channel modifications are MAR techniques where modifications are made in rivers, streams, or channels to divert part of their flow into structures that infiltrate the stored water (Zhang et al. 2020). Among these structures, there are recharge dams, subsurface dams, sand storage dams, channel spreading, etc. (Zhang et al. 2020). In general, this technology allows infiltration rates one or two orders of magnitude less than infiltration basins (Dashora et al. 2018). Recharge dams are designed to convey runoff in a surface reservoir upstream, thus enhancing groundwater recharge by percolation ponds or recharge releases into the downstream riverbed, favouring infiltration (IGRAC 2007). Subsurface dams are almost impervious underground barriers aimed at slowing or stopping underground flow thus creating a zone of enhanced groundwater storage upstream of the dam (IGRAC 2007). Technical requirements must be met by subsurface dams: low salinity rates of source waters and location concerns, such as sandy alluviums, almost levelled thalwegs, impermeable layer depth greater than 1.5 metres (Shubo et al. 2020); and streambed scoured naturally by high flows (Dillon et al. 2018), which might be achieved by placing the structures at the narrowest part of the riverbed, far from the river head (Shubo et al. 2020). Sand storage dams are constructions located above ground in intermittent streams, that creates an artificial aquifer upstream, made of large diameter sediments transported during high flows, which can store runoff water (IGRAC 2007). Channel spreading is a technique that artificially increases the wetted area and infiltration rate of the streambed by widening, levelling, scarifying, dredging, or installing L-shaped levees on it (IGRAC 2007).

In-channel modification is the most reported MAR technology in LAC, mainly due to Brazil, where it represents almost 65% of the reported MAR Brazilian projects (Valverde et al. 2018). This kind of technology is also reported in Mexico (around 40% of the schemes), Colombia (20%), Cuba (slightly more than 15%), Argentina (slightly more than 10%) and Chile (around 10%; Valverde et al. 2018). Subsurface dams are used especially in Brazilian semi-arid regions to store water for food production for poor families (Valverde et al. 2018). A widespread social technique in Brazil, the Brazilian subsurface dams were further categorized by Shubo et al. (2020) into submersible dams (that uses a totally buried impermeable septum that constrains the groundwater flow and another made of rocks, bricks, or clay over the riverbed that constrains the surface runoff thus creating a water pond) and submerged dams (that uses only the buried septum thus constraining only the groundwater flow). Recharge dams are reported in Mexico, mainly for maximization of natural storage to meet domestic and agricultural demand, using river water and stormwater; in Argentina, to capture stormwater to meet agricultural needs and provide other benefits; and in Colombia, to meet domestic demand by enhancing physical aquifer management using stormwater (IGRAC 2020). Mexican and Chilean channel-spreading projects are reported as well, aiming at providing physical aquifer management for domestic use (IGRAC 2020) and to enhance groundwater resilience to meet agricultural demands, respectively (Arumí et al. 2009).

Induced bank filtration techniques are applied to improve the quality of recovered water by extracting groundwater usually from a well near a river or lake (Zhang et al. 2020). Dug, vertical or horizontal wells, drains or other techniques are used to extract groundwater (Dillon et al. 2018). The groundwater extraction induces a water level gradient that forces the flow from the river or lake into the well, passing through the soil porous media along the way. Dillon et al. (2018) divide the induced bank filtration into three distinct techniques: riverbank filtration (the most commonly applied), lake bank filtration, and canal bank filtration, which are used for numerous purposes such as attenuating water quality variations, removing turbidity, pathogens, and organic compounds as well as to prevent overexploitation of aquifers. The definition of river, lake, or canal bank filtration depends mainly on the source of recharge water. Some authors call induced lake filtration as dune filtration, when the topography is made of dunes, which is the medium in which the groundwater flow passes through (Zhang et al. 2020).

Induced bank filtration is the least reported MAR technology in LAC, with few examples of application in Costa Rica (all projects in the country), Peru (almost 70% of the schemes), Colombia (40%), and Brazil (least than 10% of schemes; Valverde et al. 2018). As stated by Dillon et al. (2018) and Valverde et al. (2018), reports on induced bank filtration are generally underestimated in LAC. Riverbank filtration is present in Colombia, Costa Rica, and Peru for drinking water supply by contributing to the management of water distribution systems (Valverde et al. 2018; IGRAC 2020). In Brazil, riverbank filtration and lake bank filtration have been implemented mainly for research and ecological purposes, aiming at strengthening water quality management (IGRAC 2020).

Well, shaft and borehole recharge, or recharge wells, refer to MAR techniques usually applied in regions with deep unconfined aquifers or with low permeable surface layers (Zhang et al. 2020). This sub-category includes open wells and shafts, vadose zone recharge (Liang et al. 2018), aquifer storage and recovery (ASR; Maliva et al. 2020) and aquifer storage, transfer, and recovery (ASTR) techniques. Open wells, shafts, or pit infiltration stand for structures used to recharge shallow aquifers, especially in locations where surface layers are of low permeability (IGRAC 2007), and vadose zone recharge, also called ‘dry wells’, are structures which are always dry, i.e. the water table is at a distance below the bottom of the well thus providing an unsaturated zone in the soil to the groundwater flow which enhances water quality prior to reaching the saturated zone (Liang et al. 2018). ASR involves water injection through a borehole into a deep aquifer, for further recovery at the same structure, while ASTR, an improvement of ASR, allows injection only, focusing on recovery on another borehole, a distance away, thus enabling enhancement in quality when the water passes through the aquifer matrix (IGRAC 2007). Besides, ASR and ASTR schemes are usually governed by pressure injection and recovery. It is worth noting that induced bank filtration also uses well structures, albeit these wells are designed and used solely for groundwater extraction, not for aquifer recharge.

One of the least applied technologies in LAC, well, shaft and borehole recharge represent a huge share of Cuba's schemes (almost 85%), a substantial share of Argentina's and Mexico's projects (around 38% and 33%, respectively), and a portion of Colombia's and Brazil's managed aquifer recharge schemes (20% and around 3%, respectively; Valverde et al. 2018). ASR wells are found in Cuba, to counteract saline water intrusion (Valverde et al. 2018); in Argentina, to some extent combined with spreading methods and in-channel modifications, to improve groundwater quality and for physical aquifer management (Valverde et al. 2018; IGRAC 2020). Open wells, shafts, and pit injections are reported in Mexico, to maximize natural storage using reclaimed wastewater (IGRAC 2020); in Colombia, combined with a recharge dam, and Brazil, for physical aquifer management to meet domestic demand by using stormwater (IGRAC 2020). In Mexico, 'dry wells' are among the most used structures to improve drainage and flood control (Valverde et al. 2018).

Runoff/rainwater harvesting techniques stand for the process where rainwater and surface runoff are collected and diverted to deep structures that enable percolation and further water reuse (Zhang et al. 2020). Rainwater harvesting is the least applied MAR technology (Zhang et al. 2020). This sub-category includes barriers and trenches, aiming at reducing surface runoff and erosion and enabling agriculture in hilly terrain (Hannappel et al. 2014), which differ slightly (barriers obstruct overland flow and enhance percolation behind them by reducing flow velocity, while trenches catch the overland flow and infiltrate it through the bottom and sides of the structure; IGRAC 2007). Rooftop harvesting is also included in this category of MAR technology, which collects rainwater and stores it temporarily in settling tanks aiming at recharging unconfined aquifers through connected dug wells or boreholes (Hannappel et al. 2014). The aquifer recharge by rooftop harvesting schemes is governed by gravity (a major difference from ASR or ASTR wells, which are usually governed by pumping), although some authors have named these rooftop harvesting related wells as ASR (Page et al. 2010).

Runoff/rainwater harvesting techniques are the least reported technology worldwide (IGRAC 2020), and its application within countries in LAC is no different (Valverde et al. 2018). This MAR technology application only surpasses the induced bank filtration category in LAC, but there is a strong expectance that the latter is underrated (Valverde et al. 2018). Runoff/rainwater harvesting represents all schemes in Bolivia; around 45% of all MAR schemes in Chile; around 32% of Peruvian projects; 20% of Colombian projects and slightly less than 10% of Brazilian schemes; Valverde et al. 2018). Infiltration trenches are reported in Bolivia, to promote agricultural development (Valverde et al. 2018), and in coastal regions of Chile, to assist in the plantation of pines by capturing humidity from the ocean and to retain rainwater (Valverde et al. 2018). Trenches are also found in Bolivia, Chile, and Peru to promote ecological benefits (IGRAC 2020), and in Brazil, to promote ecological benefits as well (particularly flood mitigation; Valverde et al. 2018) and for physical aquifer management aiming at assisting in meeting domestic water demand; barriers and bunds are found in Colombia with this same objective (IGRAC 2020). Within LAC, only Brazilian rooftop rainwater harvesting schemes are reported, almost all corresponding to pilot projects (Valverde et al. 2018); many of these, in semi-arid regions, store water in cisterns aiming at a direct use for drinking water purposes, albeit a project for physical aquifer management, to assist in domestic demand, is found in a coastal urban area (Silva et al. 2006).

3.1.2. Key elements for successful MAR implementation

MAR implementation is a process that must be addressed with a scientific evidence-based approach in order to be successful. Dillon (2005) advised against potential unwanted consequences of MAR, such as waterlogging, foundation damage, soil salinization, slope instability, impacts on other intended groundwater uses, etc. that are prevented by solid implementation that follows what has been achieved by the literature. Zhang et al. (2020), based on a study from Yuan et al. (2016), divided the process of MAR implementation into four stages: a) planning, b) investigation, c) design and construction and d) operation. Zhang et al. (2020) described each stage's key elements that need to be considered for a successful implementation.

The definition of ultimate uses of recovered water from aquifers, sources of recharge water and relevant regulation are crucial in the MAR planning stage. The ultimate uses of recovered water should be clearly stated. In general, MAR projects are designed to assist on meeting residential, agricultural, or industrial water demands or to provide an ecological benefit, such as groundwater-dependent ecosystems protection, land subsidence prevention and seawater intrusion prevention (Zhang et al. 2020). Usually, according to (IGRAC 2007), spreading methods are designed to meet agriculture, domestic and industrial water demands (most of them); strategic water storage are provided by in-channel modifications and rainwater/runoff harvesting; induced bank filtration schemes are used to water quality improvement, along with SAT (a spreading method) and ASTR (a well, shaft and borehole recharge method); and well, shaft and borehole recharge techniques are used to recover groundwater levels and to serve as a barrier for saline water intrusion. In the cases where environmental benefits are aimed it may not be planned any recovery of water from the targeted aquifer, depending on the MAR technology adopted.

Furthermore, among the potentially diverse sources of water available on the site (river water, stormwater, rainwater, desalinated seawater, treat effluent, groundwater flow from other aquifers; Dillon et al. 2018), it must be assured that the chosen source(s) provide(s) enough quantity for sustainably managed aquifer recharge and that its quality parameters meet both ends uses purposes and comply with MAR regulations (Zhang et al. 2020). According to IGRAC (2007), rainwater/runoff harvesting sites have a system capacity adequate to meet a family to village scale (10^2 – 10^3 m³/year); SAT, in-channel modification, induced bank filtration, and well, shaft and borehole recharge methods have system capacities adequate to meet a village to town scale (10^4 – higher than 10^6 m³/year); and spreading methods have a wide range of system capacity (family to town scale). IGRAC (2007) reported that some MAR methods require specific and unique water sources, such controlled flooding, riverbank filtration, canal bank filtration and most in-channel modifications, except sand dams, whose water source is river water only; rainwater/runoff harvesting systems, whose water source is rainwater only; sand dams, fed only by stormwater; lake bank filtration, fed only by lake water; and excess irrigation, whose source is the same of the irrigation water. The remaining methods (SAT, infiltration ponds and basins; ditches, furrows, and drains; and well, shaft and borehole recharge schemes) are adequate using a wide range of water coming from different sources (river water, stormwater, treated wastewater, lake water, etc.; IGRAC 2007).

MAR regulations refer to all relevant policies, regulations, and guidelines such as those related to water quantity and aquifer storage aspects and water quality aimed at preserving the human health and the environment (Zhang et al. 2020), particularly for well, shaft and borehole recharge schemes, which have a higher potential to pollute the targeted aquifers.

MAR investigation is driven by one key element: hydrogeology. The understanding of the hydrogeology of the region where the site is being proposed is considered a decisive factor for selecting the optimum location and suitable MAR structure (Zhang et al. 2020). Zhang et al. (2020) list several important parameters to provide this kind of knowledge: geological and hydraulic boundaries, aquifer distribution, aquifer type and depth, inflow and outflow of waters, storage capacity, porosity, hydraulic conductivity, transmissivity, natural recharge and discharge, water availability for recharge and water balance.

Induced bank filtration, rainwater/runoff harvesting, recharge dams and channel spreading schemes are suitable to recharge unconfined aquifers in general (IGRAC 2007), whereas, according to IGRAC (2007), most techniques are applied to specific geological conditions, such as unconfined aquifers composed of permeable sedimentary rocks (SAT, controlled flooding, excess irrigation, infiltration ponds, ASR, ASTR, and ditches, furrows, and drains); unconfined aquifers composed of fractured crystalline rocks (infiltration ponds, and ditches, furrows, and drains); unconfined aquifers with a shallow impervious layer (subsurface dams); unconfined aquifers, with unconsolidated rocks and a shallow almost impervious layer (open wells, shafts, and pit infiltration); a media containing crystalline rocks with sandy riverbeds (sand dams); and confined aquifers composed of unconsolidated rocks (ASR and ASTR).

Three elements are defined in the design and construction stage: recharge method, recharge site, and water quality control measures. The recharge methods available are described in the section 3.1.1; the suitable methods are based on hydrogeology, land availability, groundwater quality and costs, ultimate uses, and environmental impacts (Zhang et al. 2020). Comparing all methods, induced bank filtration, SAT, and ASTR schemes are the most expensive (medium to high relative costs; IGRAC 2007); rooftop harvesting have a medium relative cost (IGRAC 2007); infiltration ponds, ditches, furrows, and drains, recharge dams, channel spreading, open wells, shafts, and pit infiltration, and ASR have low to medium relative costs (IGRAC 2007); and controlled flooding, excess irrigation, subsurface dams, sand dams, barriers and bunds, and infiltration trenches are the cheapest MAR technologies (low relative costs; IGRAC 2007).

The recharge site selection process must consider all abovementioned aspects for the recharge method selection whereas including other factors such as political, social, and economic factors (Zhang et al. 2020). According to IGRAC (2007), spreading methods require sites with permeable soils, able to guarantee water quality standards of the recovered water, while ditches, furrows and drains are also suitable for soils with upper impermeable layers; induced bank filtration, rainwater/runoff harvesting and in-channel modifications are suitable in sites with sandy soils or with higher particle size soils; and well, shaft and borehole recharge methods are applicable to sites with any kind of soil. Regarding topography (IGRAC 2007), most MAR methods require or are more applicable in flat or gently sloped terrains; induced bank filtration are suitable in natural, gently sloped drainage channels, with recharge and subsurface dams more applicable on intermittent stream conditions; rooftop harvesting schemes are more suitable in urban areas; while ASR and ASTR techniques does not have any kind of topography constraint.

Besides, the water quality control measures stand for procedures taken to pre-treat recharge water or post-treat recovered water aiming at removing specific contaminants, reducing the risk of clogging or accumulation of pollutants in aquifers (Zhang et al. 2020). Pre-treatment is particularly critical for well, shaft and borehole recharge schemes and most spreading methods, depending on the water source quality, mostly to prevent clogging (IGRAC 2007). Some methods do not require any pre-treatment, such as controlled flooding, subsurface dams, sand dams, barriers and bunds, infiltration trenches, and induced bank filtration schemes (IGRAC 2007). Pre-treatment may be required prior to injecting water in SAT systems, and rooftop harvesting schemes may require pre-treatment depending on specific conditions (if green roofs and/or injection wells are part of the system; IGRAC 2007).

The last stage of a successful MAR implementation is the operation procedure, which comprises two key elements. The former is the in-situ verification if the proposed system achieves its intended goals (e.g. if it successfully provides a certain amount of high-quality water or if the recharged water was improved in terms of quality to the desired level). The latter refers to monitoring and maintenance. Monitoring provides a record of relevant information to enable the detection of water quantity and quality changes (Zhang et al. 2020), triggering of clogging mechanisms and information that can assist in the development of future projects in the region (Dillon et al. 2018). Monitoring can also be used to optimize maintenance schedules, which are essential to maintain the system working effectively and sustainably on a long-term basis (Zhang et al. 2020). In general, monitoring practices have been limited to certain small- and medium-term periods and conditions due to their high costs (Eckart et al. 2017). Some MAR methods have been monitored more than others (Dillon et al. 2018) and one reason may be the relative cost of monitoring concerning the overall cost of the MAR projects, which vary in function of the method chosen.

3.1.3. Challenges and constraints for MAR implementation

Zhang et al. (2020) enlisted some concerns and challenges concerning MAR implementation. For example, further research on the infiltration and seepage calculation for MAR is needed. According to Zhang et al. (2020), the seepage calculation under artificial recharge usually adopts the theory of flow in pumping wells (e.g. the Dupuit formula, widely used in steady-state flow condition in an unconfined aquifer), since recharge is considered as the reverse process of pumping, but this approach is not precise in many cases such as under the condition of recharge in the vadose zone or a pressure injection.

One of the major challenges for sustainable operations of MAR projects is the occurrence of clogging (Zhang et al. 2020), which can triggered by numerous mechanisms (e.g. physical, chemical, biological, and mechanical clogging), but in general results in reduction of MAR effectiveness. Dillon et al. (2018) has stated that the literature has made huge progress in understanding clogging mechanisms, albeit the numerous types are usually interrelated (Zhang et al. 2020). Physical clogging is caused by accumulation of aquifer sediments and by inorganic and organic suspended soils in source water (Bouwer 2002; Pavelic et al. 2007; Zhang et al. 2020); chemical clogging is caused by reaction processes triggered by the disturbance of water-rock interaction that may follow the introduction of the recharged water (Bouwer 2002; Zhang et al. 2020); biological clogging is caused by microorganisms that form biofilm (their attachment or accumulation on the medium particles), which may be present on the source water (Zhang et al. 2020); and mechanical clogging is caused by air bubbles that may block pore spaces in aquifer matrix and screened well casing, which can arise from the cascading of water inside the recharge well (Liu et al. 2016; Zhang et al. 2020). Water quality control (pre-treatment) is considered effective to prevent the abovementioned clogging mechanisms (Pavelic et al. 2007; Zhang et al. 2020), except mechanical clogging (Zhang et al. 2020). Although relatively secure preventive measures for clogging and aquifer rehabilitation are prescribed in the literature, Dillon et al. (2018) states that efficient clogging (triggered by individual or multiple mechanisms; Zhang et al. 2020) prediction and management is still a challenge, mainly due to lack of adequate water quality monitoring and other specific evaluations at existing operational MAR sites.

Another important concern enlisted by Zhang et al. (2020) is the purification mechanism of MAR, a complex process that occurs in the subsurface, influenced by many factors such as mechanical filtering, sorption, biodegradation, chemical reaction, etc., which is particularly important for systems that are sourced by poor quality water of that use a direct injection in a potable aquifer (Zhang et al. 2020). Although recently there is evidence reported on water quality improvements by MAR projects (e.g. for organic chemicals; Dillon et al. 2018), further research on purification mechanism of MAR is warranted in a wide range of aquifer systems and water sources that would verify its reliability as a post-treatment water quality control measure and assist in identifying its risks (Zhang et al. 2020). Dillon et al. (2018) also pointed out the need of methods for evaluation of aquifer microbiological ecosystems to verify their capacity in contaminants attenuation in changing geochemical conditions. According to (Zhang et al. 2020), achieving accuracy in the understanding of microscopic purification mechanisms is important because this step may help optimize the MAR process, without underutilizing the aquifer purification potential, which leads to unnecessary cost increase, nor prescribing poor pre-treatment measures that may damage the targeted aquifer, leading to further costs (aquifer remediation is expensive and time consuming).

Further relevance must be given to current efforts of establishing a global inventory of MAR (Stefan and Ansems 2018; IGRAC 2020). Zhang et al. (2020) pointed out that monitoring existing operations and providing information to public repository will enable application of big data analytics and artificial intelligence to reliably assist in the planning and implementation of future projects (Dillon et al. 2018; Zhang et al. 2020). Dillon et al. (2018) suggested the establishment of national monitoring and research programs, initially sized at 2-10% of the planned investment in new recharge infrastructure, since the lack of monitoring of some MAR methods (e.g. ASR systems are generally better monitored than streambed modifications) warrants comparative evaluations with multiple methods across multiple sites (Dillon et al. 2018).

3.2. Sustainable urban drainage systems (SUDS)

The so-called conventional or traditional urban drainage stands for approaches that employ large structures specifically targeted to provide a fast conveyance of stormwater from cities (Gimenez-Maranges et al. 2020). Usually, these structures are centralized drainage systems of pipes and gutters (Porse 2013), designed to handle 1-5 years storm events (Sohn et al. 2019). The first stormwater management systems were created in the mid-1990s when the frequency and magnitude of urban floods increased due to expanded soil imperviousness that followed urban sprawl (Fletcher et al. 2015). In general, they are not environmentally friendly since they promote soil imperviousness and reduce the hydrologic functions of the landscape, such as infiltration (thus also reducing baseflows and compromising habitat of sensitive faunal species; Burns et al. 2012), retention, and evapotranspiration (Gimenez-Maranges et al. 2020); nor sustainable because they typically contribute to high flow peaks, large runoff volumes, and high frequency of floods downstream after small rainfall events (Burns et al. 2012) hence requiring further system reinforcements in regions already densely occupied (Conte et al. 2012). Besides, the conventional approach often provides a false sense of security and promotes development in flood-prone areas (Sohn et al. 2019). Even traditional approaches focused on load-reduction have their shortcomes since they generally do not consider the broader hydrologic changes caused by surface water runoff (for example, Burns et al. (2012) shows that groundwater flow rates kept intercalating from zero to values higher than natural, pre-development baseflow rates). Thence, the traditional urban drainage approach does not contribute to sustainable urban development (Paule-Mercado et al. 2017) and simplify a complex issue that can only be effectively addressed by combining organizational responsibilities of urban issues, such as urban planning and sewer management (Gimenez-Maranges et al. 2020).

Urban drainage has been focused primarily on the conveyance of water away from urban areas, focused solely on flood mitigation until the 1960s (Fletcher et al. 2015). From the 1960s onwards, it has increased to address other subjects as well such as recreation & aesthetics, water quality (pollution), flow regime restoration, the ecology of receiving waters, resilience, microclimate, etc. (Fletcher et al. 2015). Stormwater, once seen only as a problem, has started to be widely recognized as a resource with intrinsic opportunities (e.g. additional water supply, increased biodiversity, improved microclimate; Fletcher et al. 2015), gathering and engaging a broader range of disciplines such as architects, planners, ecologists, and social scientists (Fletcher et al. 2015), rather than just hydro-engineers, stakeholders and other water-related professionals. Hence, to counter the problems identified in traditional systems, new approaches to stormwater management have been pursued, researched, and applied worldwide (Eckart et al. 2017; Kaykhosravi et al. 2018; Sohn et al. 2019; Gimenez-Maranges et al. 2020). These approaches have been developed locally and thus named differently in different regions of the world, including low impact development (LID), mainly in the USA's; urban design and development (LIUDD) in New Zealand; water sensitive urban design (WSUD) in Australia; and sustainable urban drainage systems (SUDS) in Europe (Fletcher et al. 2015; Eckart et al. 2017). In general, these new urban drainage approaches are sustainable since they mimic natural processes (infiltration, evapotranspiration, filtration, retention, and reuse) to handle potential flooding events (Burns et al. 2012; Perales-Momparler et al. 2017). These sustainable stormwater management systems contribute to ecological resiliency to urban floods that could not be achieved solely by traditional conveyance (Sohn et al. 2019) and pollution load-reduction systems (Burns et al. 2012), which are unable to provide catchment-wide hydrologic restoration (Fletcher et al. 2015).

Among the many acronyms for alternative urban drainage approaches, LID might be the most used in research activities (Fletcher et al. 2015). LIDs appear to have been used first in the USA (1977) as an attempt to minimize stormwater management costs by achieving a functionality equivalent to the natural hydrologic landscape's (Fletcher et al. 2015). SUDS were first coined in the UK (in 1997; Fletcher et al. 2015) as an attempt to replicate the natural, pre-development drainage from a site (Fletcher et al. 2015; Sohn et al. 2019). Since the definition of SUDS is consistent with the principles behind LID (Fletcher et al. 2015) and it is applied in a broader range of countries (European Union; Fletcher et al. 2015; Gimenez-Maranges et al.

2020), the former will be used hereon to refer to these sustainable stormwater management systems. SUDS are applied in an attempt to bring the hydrology of urban catchments back to their natural prior conditions. For that matter, they are placed dispersedly within a catchment, matching its green spaces and natural hydrologic features (Eckart et al. 2017) to provide a reduction in runoff volume and peak flow (Sohn et al. 2019), and to improve infiltration and to increase baseflow (Eckart et al. 2017). SUDs are claimed to be able to reduce water pollution, assisting with biochemical cycles regulation (Eckart et al. 2017) and to protect against water quality degradation by non-point pollutant sources (Sohn et al. 2019). There is a growing understanding that SUDS will play a role to help meet the United Nations sustainable development goals (Gimenez-Maranges et al. 2020).

Nowadays there is still a resistance to the adoption of SUDS and conventional flood management practices remain dominant, at least in the UK, where SUDS have been extensively studied (Melville-Shreeve et al. 2018; Gimenez-Maranges et al. 2020). One reason for this may be the ineffectiveness of SUDS in acting solely to control hydrological impacts of larger return period events (Eckart et al. 2017) since their performance on reducing volume and peak flow diminishes with increasing storm intensity (Sohn et al. 2019). SUDS effectiveness also diminishes when submitted to events of large storm sizes, long duration, and wet initial conditions (Jackisch and Weiler 2017; Sohn et al. 2019). After conducting a review on performance and implementation of SUDS, Eckart et al. (2017) concluded that SUDS alone fail to return watersheds to pre-development conditions in most cases. Another reason might be because albeit a wide range of ecologic benefits of SUDS are also claimed by the literature, little effort has been put into evaluating these benefits in the field, except water quantity- and quality-related issues (Fletcher et al. 2015; Lähde et al. 2019). Fletcher et al. (2015) argued that there is a lack of critical reviews which examine if the alternative stormwater management practices have been successful in meeting their objectives, such as the improvement of water quality, the protection of aquatic ecosystems and the mitigation of flooding. SUDS are also claimed to increase urban biodiversity and carbon dioxide sequestration and to reduce the urban heat island effect (Charlesworth 2010; Woods Ballard et al. 2015), although there is a lack of studies evaluating these benefits in the field. Overall, there is a distinct shortage of evidence regarding their sustainability in comparison to traditional urban drainage systems (Fletcher et al. 2015; Eckart et al. 2017; Gimenez-Maranges et al. 2020) and the uncertainties intrinsic to climate change also pose a doubt to whether SUDS will be effective during potential climatic extremes (Sohn et al. 2019). In practice, SUDS schemes have been better applied when combined with traditional stormwater best management practices, such as detention ponds

(Eckart et al. 2017) and this may discourage its adoption over conventional approaches. Besides, Gimenez-Maranges et al. (2020) pointed out that the low interest of the scientific community on social dimensions of SUDS may be a reason for its limited deployment in the European Union. The referred authors concluded that, in general, the society still perceives stormwater as a waste product, not as a valuable element in the urban environment, and this mindset is also responsible for the weak cooperation among stakeholders and persistently limited crossover between science and practice (Gimenez-Maranges et al. 2020).

3.2.1. Categories of SUDS technology

Stormwater management technologies are divided into two categories (Fletcher et al. 2013): infiltration-based and retention-based technologies, similarly to MAR technology (Stefan and Ansems 2018). In general, a combination of retention and infiltration-based techniques are required to successfully restore the hydrology of a watershed (Burns et al. 2012). Woods Ballard et al. (2015) provides technical details of most stormwater management technologies, including general descriptions, selection and sitting decision-making, overall design (hydraulic, water quality treatment, amenity and biodiversity), physical specifications, construction, operation and maintenance requirements, among other important key issues. Infiltration-based technologies are the ones that aim at restoring baseflows through recharge of subsurface flows and groundwater (Fletcher et al. 2013). Examples of infiltration-based techniques include soakaways, infiltration trenches, infiltration basins, filter strips, filter drains, swales, bioretention cells, pervious pavements, etc. (Woods Ballard et al. 2015; Eckart et al. 2017). According to Woods Ballard et al. (2015), 1) soakaways are underground cuboid structures filled with a void-forming material which stores water temporarily thus enhancing infiltration (Butler and Davies 2010); 2) infiltration trenches are linear soakaways that receives runoff distributed from the catchment area, which use storage and filtration to remove suspended soils and other contaminants from stormwater (Butler and Davies 2010; Glendenning and Vervoort 2011; Barkdoll et al. 2016); 3) filter drains, that might be included below trenches, are perforated pipes in the base of the trench that assist drainage (Hatt et al. 2007); 4) infiltration basins are flat-bottomed, shallow landscape depressions that store runoff and enhance infiltration into the subsurface soils, usually constructed in large open spaces (Woods Ballard et al. 2015); 5) swales, similar to infiltration basins, are shallow, flat bottomed, grassed open spaces but with longitudinal dimension prevailing (thus a channel), which convey and infiltrate runoff at the same time besides favouring sedimentation, filtration and evapotranspiration (Barret 2008; Barkdoll et al. 2016; Kaykhosravi et al. 2018); 6) filter strips, also called buffer strips, are vegetated low slope terrains designed to drain runoff from adjacent impermeable areas, which flows as a thin sheet, distributed across the area at low velocities aiming at filtering out suspended solids from stormwater, usually designed as a pre-treatment device prior to conducting runoff to other SuDS techniques (Barkdoll et al. 2016); 7) bioretention systems are shallow planted depressions that provide temporary storage of water surface runoff, filtration through vegetation and the soil matrix, which are effective to encourage infiltration, promote evapotranspiration, recharge groundwater, protect stream channels, reduce peak flow and pollutant loads (Barkdoll et al. 2016); and 8) pervious pavements are structures made of materials suitable for traffic of people and vehicles which also promotes infiltration of water surface runoff into foundation layers, such as modular permeable paving, porous asphalt, porous concrete, sports surfaces, etc. (Barkdoll et al. 2016).

Retention-based techniques focus on temporarily retaining stormwater to reduce outflow (Fletcher et al. 2013). Examples of retention-based techniques are wetlands, detention basins, attenuation storage tanks, green roofs, rainwater harvesting tanks, barrels, and downspout disconnections (Woods Ballard et al. 2015; Eckart et al. 2017; Gimenez-Maranges et al. 2020). Wetlands are ponds designed to store runoff above permanent wet pools thus enhancing the settlement of suspended soils and biological removal of pollutants, with shallow bottomed zones that promote the growth of plants (Woods Ballard et al. 2015); 2) detention basins are depressions in the landscape normally dry which attenuate runoff and provide water quality treatment when vegetated, primarily designed to temporarily store water surface runoff and further release it at a longer period (Woods Ballard et al. 2015); 3) attenuation storage tanks are underground void structures that serve as space for temporary storage of runoff before infiltration, controlled release, or other uses, which are made of a variety of materials: geocells, plastic corrugated arch, oversized concrete or plastic pipes, precast or in situ concrete boxes, etc. which provide a high storage volume in comparison to aggregate-filled structures and are able to be placed beneath roads, car parks, recreational areas, and other open spaces (Woods Ballard et al. 2015); 4) green roofs are vegetated surfaces, installed on the top of buildings, which promote evapotranspiration, attenuate surface water runoff and enhance its quality at some level (Woods Ballard et al. 2015; Barkdoll et al. 2016); 5) rainwater harvesting (RWH) are, as stated by Woods Ballard et al. (2015), "storage systems that collect runoff within the boundary of a property (from roofs and/or surrounding surfaces) for use on site, where the use is sufficiently great to ensure that storage of runoff is achievable for most rainfall events", which use water tanks (Woods Ballard et al. 2015) and/or rain barrels (Kaykhosravi et al. 2018); and 6) downspout disconnections, which are downspouts that discharge surface water runoff, captured from rooftops, directly into infiltration-based SuDS techniques; (Borris et al. 2016). Rainwater harvesting systems combined with green roofs (Damodaram et al. 2010) and/or downspout disconnections (Borris et al. 2016) are also reported in the literature. SuDS technologies which comprise vegetated areas are also called green infrastructure (GI), which also shares concepts and principles with stormwater management systems (Fletcher et al. 2015) but goes beyond, focusing on other benefits as well such as reduced energy consumption, reduced urban heat island effect, improved air quality and reduced dioxide carbon emissions (Eckart et al. 2017). Therefore, some of the described SuDS technologies (bioretention cells, porous pavements, swales, green roofs, infiltration trenches, etc.) also fall under techniques of green infrastructure thus also providing these abovementioned benefits.

3.2.2. Factors affecting SUDS implementation

Sohn et al. (2019) divided factors affecting SUDS' effectiveness into two categories: internal and external factors. The internal factors are described by facility size, layer configuration, vegetation type, and soil type/composition (its permeability, in general, limit the usage of certain stormwater management practices; Eckart et al. 2017). External factors include climate conditions (stormwater characteristics, amount of sunlight, temperature, etc.) and climate change impacts, land use and topography, water table depth and distance to the sea, among others. Important stormwater characteristics are its size, intensity, duration, peak location, and antecedent moisture content (Sohn et al. 2019). Climate change is expected to induce higher temperatures, which may diminish surface runoff by reducing water viscosity and thus the soil's hydraulic conductivity and increase evapotranspiration (Sohn et al. 2019). However, rainfall intensity is also expected to increase as a consequence of climate change (Eckart et al. 2017) and thus put additional stress on urban drainage stormwater management systems (Eckart et al. 2017; Sohn et al. 2019), since even small changes in rainfall intensity and duration at highly impervious urban watershed can cause severe floods (Karamouz and Nazif 2013). Moreover, shallow water tables limit the applicability of several techniques (Eckart et al. 2017; Sohn et al. 2019), which may be especially critical in coastal areas where the effect of sea-level rise on water table is significant (Joyce et al. 2017). In general, SUDS depend on local meteorological and hydrological properties to be successful (Eckart et al. 2017).

There is a clear communication between elements that drive infiltration-based SUDS implementation and those that drive a successful MAR implementation. Hence, SUDS framework implementation might benefit from the insights given by Yuan et al. (2016) and Zhang et al. (2020), concerning the important steps (planning, investigation, design and construction, and operation) and their key elements for MAR implementation, where applicable. Firstly, policies, regulations and guidelines that affect MAR implementation should be considered in the planning stage of infiltration-based SUDS technologies, such as those related to aquifer storage aspects and water quality aimed at preserving the human health and the environment (Zhang et al. 2020). Moreover, SUDS investigation stage must give attention to hydrogeology (as described with more detail in section 3.1.2; Zhang et al. 2020), since research on SUDS have pointed out that their performance are related to hydrogeology aspects such as soil type/configuration (Eckart et al. 2017), and water table depth and variation in the function of sea level (Joyce et al. 2017). Suitable recharge methods and recharge sites in the design and construction stage of infiltration-based SUDS technologies are also based on hydrogeology, among other aspects such as land availability, groundwater quality, environmental impacts, and political and socio-economic factors (Zhang et al. 2020). The design and construction stage should also give special care to procedures taken to pre-treat recharge water to reduce risk of clogging or accumulation of pollutants in the targeted infiltration zone (Zhang et al. 2020). The last but not least important stage of MAR implementation is operation. With this particular stage, all recommendations by Zhang et al. (2020) are applicable to the deployment of effective and sustainable SUDS schemes (as described in section 3.1.2).

3.2.3. Challenges and constraints to SUDS implementation

(Eckart et al. 2017) pointed out some constraints to SUDS implementation. Firstly, each SUDS technology is limited by its affecting internal and external factors (Eckart et al. 2017; Sohn et al. 2019) and by the tolerable level of risks associated to their implementation such as potential groundwater contamination (Eckart et al. 2017). Another important issue is the lack of community engagement, especially when using decentralized stormwater management practices (Eckart et al. 2017); education and financial incentives are seen by the literature as viable approaches to raise widespread public participation and awareness of SUDS benefits (Eckart et al. 2017). Besides, albeit being extremely important for the evaluation of SUDS performance (Eckart et al. 2017), there is a lack of monitoring or insufficient monitoring to support meaningful conclusions, particularly in the long-term basis (Clary et al. 2011). From a systematic review of SUDS empirical articles, Sohn et al. (2019) found out that they ranged from 10 months to 4 years of monitoring. Most reviewed studies examined less than 100 storm events. To overcome this and provide successful SUDS implementation, Eckart et al. (2017) argued that a multidisciplinary approach between different government agencies, community groups and the private sector is required.

Future research is needed in sustainable or alternative urban drainage systems. How SUDS perform under different spatial scales, from the building/neighbourhood scale to the city, regional and national scale (Eckart et al. 2017; Gimenez-Maranges et al. 2020) incorporating their structures within the wider hydrology of the watershed (Kaykhosravi et al. 2018), is still an unaddressed question. Also, the literature has not yet assessed SUDS performance under different temporal scales; studies focusing on meteorological measures averaged by or aggregated into smaller temporal units (e.g. daily and sub-hourly resolutions) are needed since working with larger temporal units (e.g. annual resolution) is difficult to establish climate impacts on SUDS effectiveness (Sohn et al. 2019). Besides, Eckart et al. (2017) observed that long-term monitoring studies are required, which can enable exploration of both extreme and non-extreme storm events, as addressed by Sohn et al. (2019). Another unanswered question is how SUDS perform under different climate conditions (Eckart et al. 2017), except the temperature maritime climate zone, extensively studied in the EU (Gimenez-Maranges et al. 2020), especially in their ability to treat stormwater pollutant (Sohn et al. 2019) and to provide data to calibrate models at a scale of module or sub-watershed (Sohn et al. 2019). Furthermore, there is a lack of studies assessing how SUDS perform in removing emerging and difficult to measure contaminants (Eckart et al. 2017) and in assessing how these stormwater management techniques, rather than green roofs (Sohn et al. 2019; Gimenez-Maranges et al. 2020), permeable pavements (Gimenez-Maranges et al. 2020), and bioretention systems (Sohn et al. 2019) perform in terms of restoring water balance (Eckart et al. 2017). Concerning these abovementioned different spatial scales, temporal scales, and climatic conditions, both empirical experimental investigations and integrated empirical-hypothetical research are required (Sohn et al. 2019).

3.3. Rooftop rainwater harvesting (RWH) systems

There are many definitions for rainwater harvesting (RWH) in the literature, which vary in accordance to the perspective of the field of study, the majority focusing on water conservation to meet non-potable uses, as listed by Adham et al. (2016). Independent on the definition adopted, an RWH system is made of at least three main components: a catchment, a storage facility, and a target (Adham et al. 2016). In literature focusing on stormwater management systems, RWH is usually viewed as storage systems designed to collect rainwater, intercepted by rooftops of buildings (thus yielding to surface water runoff), and temporarily store it for use on-site, with a concern that water demand will be high enough to ensure that storage of runoff is achievable for most rainfall events (Woods Ballard et al. 2015; Teston et al. 2018). The referred use of the harvested water is usually non-potable domestic, commercial, industrial, and/or institutional demand, such as for house cleaning, washing floors, toilet flushing (Palla et al. 2017; Teston et al. 2018; Freni and Liuzzo 2019), garden watering (Petrucci et al. 2012), and car washing (Burns et al. 2012). On the other hand, literature comprising MAR studies treat RWH as a set of techniques where surface runoff is collected and diverted to deep structures, enhancing infiltration, and enabling further water reuse (Zhang et al. 2020). The definition given to RWH by MAR researchers is more likely to refer to low impact developments as a whole (since the referred MAR method includes alongside with rooftop harvesting, barriers, bunds, and infiltration trenches as well), while rooftop harvesting, a category of the broader rainwater/runoff harvesting technique, is more in line with the RWH concept as used by sustainable urban drainage specialists. However, MAR rooftop harvesting systems collect and store rainwater temporarily in settling tanks aiming at recharging unconfined aquifers through connected dug wells or boreholes, filled with sand or gravel (Dillon 2005; Hannappel et al. 2014), i.e. their primary goal is to use the water harvested for controlled aquifer recharge, aiming at enhancing groundwater supply and further use of the recovered water. Some risks of water quality degradation, albeit low, are present in RWH systems (Gale and Dillon 2005; Page et al. 2010). The main sources of contamination are air pollution, animal droppings and insects, and from reactions of the contact of rainwater with the materials constituting the system (Gale and Dillon 2005). In general, using chemically inert materials in drainpipes, rooftops, and storage tanks, discarding first flushes (Gale and Dillon 2005), cleaning gutters regularly and taking pre-treatment measures before withdrawing water from tanks are sufficient measures to ensure compliance with water quality requirements (Page et al. 2010).

MAR rooftop harvesting systems (MAR-RWH) and SUDS rainwater harvesting systems (SUDS-RWH) share some similarities in their definitions, but also some differences. MAR-RWH is designed focusing primarily to enhance groundwater supply or improve its quality, for example, in brackish aquifers whose native groundwater is not suitable for irrigation of gardens (Page et al. 2010), while in SUDS-RWH, these systems focus primarily on water conservation and stormwater management. In SUDS-RWH, non-potable uses are met simultaneously or before providing stormwater management goals, whereas in MAR-RWH non-potable uses are met only after managed aquifer recharge and further recovery of the recharged water. Although there are differences in definitions and objectives, MAR-RWH and SUDS-RWH may not conflict and even help to achieve each other's goals. The design of SUDS-

RWH focus on water conservation (supply of on-site water demand) and/or stormwater management (including further storage capacity rather than just the designed for water conservation; Teston et al. 2018; Freni and Liuzzo 2019). In general, there is a concern that the water conservation demand will be sufficient to keep the storage structure relatively empty at most times, with enough void space to temporarily store the inflow and enable a controlled, slowed outflow downstream. However, systems with high reliability in providing water for non-potable demand are hardly emptied, with a smaller capacity of reducing peak runoff flows at daily steps, at least (Teston et al. 2018). There is, therefore, an untapped opportunity to store the unused surplus in aquifers and hence enable better functioning of the storage tanks in retaining storm from rainfall events (Burns et al. 2012, 2015). If a SUDS-RWH system has aquifer recharge among its prescribed water uses, and preventive measures are taken to ensure the aquifer is safe from groundwater contamination, then the referred project can also be termed as a MAR-RWH scheme (Dillon et al. 2018), thus enabling both groundwater recharge and aiding at flood control at the same time (besides providing non-potable water for other demand). The inclusion of this new water demand for the rooftop rainwater harvesting project (i.e. the borehole's maximum injection rate) provides the benefit of giving further assurance that storage of runoff will be achievable for most rainfall events. If the system is not involved in aquifer recharge, then it should not be termed as a tool for MAR (Shubo et al. 2020), which is the case of many social technologies that store rainwater in arid and semi-arid areas, which uses buried or semi-buried reservoirs to assist in food production although without infiltration of any proportion of the harvested rainwater.

SUDS-RWH systems are claimed to have numerous benefits such as the attendance of local water demand, which contributes to sustainability and resilience to climate change (Woods Ballard et al. 2015; Eckart et al. 2017). SUDS-RWH are designed as gravity-based systems, pumped systems, or composite systems. Gravity-based systems are those where rainwater is collected and stored at elevation to supply to non-potable end-users by gravity (Freni and Liuzzo 2019), which are constrained by *in-situ* properties: the structural capacity of the location to provide storage at a given height, limiting operating pressure, among others (Woods Ballard et al. 2015); pumped systems are those where rainwater is collected and diverted to underground storage (or tanks in the ground) through gutters and downspouts, and then pumped to header tanks (which are at a more elevated spot than the gutters) for further distribution to non-potable end-uses by gravity or pumped directly to non-potable end-uses (Woods Ballard et al. 2015); and composite systems are a mixture from both systems, in which runoff goes to header tanks by gravity, while the excess runoff by-pass it being directed to the underground storage, or to a water tank in the ground. Woods Ballard et al. (2015) argued that composite systems pick up the best properties from both systems: they allow smaller amounts of pumping in comparison to pumped systems because the pump is activated only when the header tank, normally filled by gravity, is emptied. Composite and pumped systems allow passive or active operations, while gravity-based allow only passive operations. Passive operations are those in which the water level within the storage tank relies only on the balance between inflow and outflow (Woods Ballard et al. 2015), prone to failure (overflow) if there is not sufficient storage at the start of a given rainfall event, thus usually leading to larger tanks than the necessary (Gee and Hunt 2016); while active operations, or active systems, are those in which real-time monitoring assists in the stormwater management by enabling a controlled release of water sufficient enough to prepare the system for a coming storm up to the designed rainfall depth, achieved in practice by using data from forecasted precipitation and water level monitored within the tank (Gee and Hunt 2016) or by pumping out the stored water down to a set level whenever a threshold is exceeded (Woods Ballard et al. 2015).

RWH systems seem to be more studied in SUDS context than in MAR's, albeit a limited number of articles were cited in recent review studies. Gimenez-Maranges et al. (2020) reviewed 80 SUDS articles in the European Union context, an effort which resulted in only around 8-9% of articles comprising SUDS-RWH systems (and using rain barrels only). Sohn et al. (2019), reviewing worldwide sustainable stormwater management techniques' efficiency in response to climate variability, found that 10 out of the 43 reviewed articles comprised SUDS-RWH systems (ranging from 2003 to 2017), the majority of these combined with other techniques such as bioretention systems, porous pavements, green roofs, and downspout disconnections. Experimental and numerical research on the effectiveness of SUDS-RWH systems is still scarce (Palla et al. 2017). No empirical (experimental and non-experimental) studies have been found in the review by Sohn et al. (2019). Hence, further research is still required to enable SUDS-RWH to become a significant component of urban stormwater management (Fletcher et al. 2013).

Some studies evaluated the SUDS-RWH potential in mitigating floods, employing evaluation of peak runoff flow reductions, which varied in function of factors such as tank size, precipitation patterns and water use: Teston et al. (2018) conducted hypothetical research evaluating the impact of RWH on the drainage system of a condominium of houses in Curitiba, Brazil, simulating for different scenarios of non-potable water demand (the 1st considering only cleaning; the 2nd, cleaning and floor washing; and the 3rd, cleaning, floor washing and toilet flushing). They used 17 years of rainfall dataset (daily records). They concluded that their system was able to provide a 24.3% reduction in runoff volume, compared to total surface runoff produced from rainfall in the period, concerning the 3rd demand scenario (the highest, which comprised a 4.5-m³ tank). The smaller the demand, the smaller the reduction in surface runoff volume, they concluded. However, the 4.5-m³ tank promoted only a 4.4% reduction in peak flow for an extreme event simulated in accordance to monitored data (113 mm raining for three hours). Freni and Liuzzo (2019) carried out a study to evaluate the performance of RWH tanks, installed at roughly 400 single-family houses in a residential area in Palermo, Italy, for toilet flushing supply and flood mitigation. Their study used daily rainfall data recorded during a 7-year period. They found that using 5 m³ tanks in each house can reduce 35 and 100% of the flooded area for rainfall events up to 50 mm and smaller, respectively. However, the system used in their study presented an inefficiency for heavy rainfall events, which makes the combination of other drainage techniques or the adoption of a larger RWH tank necessary. It is worth noting that the evaluations of stormwater peak reduction in studies by Teston et al. (2018) and Freni and Liuzzo (2019) might be compromised due to the large temporal unit used (Campisano and Modica 2015).

Petrucci et al. (2012) reported two rainfall-runoff measurement campaigns (around 6 months each, with 5-minute resolution), which have been conducted on a district of Champigny-sur-Marne, France, before and after the RWH tank's installation, enabling the assessment of the effect of the tanks on runoff. The so-called RWH tanks are more adequately called water butts (Woods Ballard et al. 2015) or rain barrels (Gimenez-Maranges et al. 2020) because of their small sizes (0.6 and 0.8 m³). 117 out of around 450 parcels, 400 m² rooftop area on average, were equipped with RWH tanks, used for garden watering. Results indicate that they were not able to prevent any stormwater overflow, following Woods Ballard et al. (2015), even in the simulation considering RWH tanks in every household. The authors concluded that larger tanks are more prone to prevent overflows for rain events with return periods between 5-10 years.

Palla et al. (2017) conducted hypothetical research using a pumped RWH system to meet toilet flushing demand and assist in stormwater management at three residential buildings located in Genoa, Italy (ranging from 420 to 680 m² rooftop area; from 233.6 to 367.2 m³/year toilet flushing demand; and from 14 to 28 m³ tank capacity). Simulations occurred based on 26 years rainfall dataset with high temporal resolution (1-minute), resulting in peak and volume reduction rates of 33% and 26%, respectively. Results from extreme storm events indicated the system performed best at supplying water demand on the rainy season but with ineffective peak reductions due to limited storage in the tanks and performed best at reducing volume and peak runoff in the summer, where tanks were usually empty at the beginning of storms.

Burns et al. (2015) assessed the ability of 12 pumped SUDS-RWH systems in reducing potable mains water usage and in retaining runoff from rainfall events, which are connected to rooftops in a peri-urban catchment in South-Eastern Australia. Data loggers with pressure sensors operated for two months, recording water levels within the systems at a 6-min resolution. The storage tanks ranged from 3 m^3 up to 28 m^3, with corresponding catchment varying from 35 m^2 to 466 m^2, connected to demands such as toilet flushing, clothes flushing, and hot water usage. Some were connected to other demands such as garden watering, car washing, and even drinking. Results indicated that most tanks were not able to retain runoff from rainfall events since they were not emptied often enough, even in dry seasons. One suggestion the authors raised is to increase water consumption from the water tanks by resealing water to irrigation systems since the non-potable water demands normally considered are not enough to enable more efficient retention of runoff from rainfall events. This approach had been adopted by (Burns et al. 2012), in a study where the authors modelled the hydrology of a 500 m^2 impervious area in the suburbs of Melbourne, Australia. Three approaches were considered: drainage-efficiency (no retention or treatment), load-reduction (using a standard lined biofiltration system with an underdrain), and flow-regime (using a combined RWH system and vegetated infiltration system), all scenarios compared to the pre-development state. The scenarios were assessed using a 1-year rainfall data (6-min resolution) and the flow-regime scenario was coupled with a 7 m^3 rainwater tank designed to provide water for laundry, toilet flushing, and irrigation of a 350 m^2 garden. The tank was connected to a 250 m^2 rooftop, while its overflow, and the surface runoff flow from the 250 m^2 pavement, was diverted to a rain garden, designed to achieve a surface runoff frequency similar to the forested state (~6 days/year). Compared to the forested, pre-developed state, both the drainage-efficiency and load-reduction approaches led to 5 times more discharge, with no filtered flow in the former, and 68% of total discharge being filtered in the latter, however with peak values frequently surpassing the estimated catchment baseflow rate; and the load-reduction approach was able to promote reduced frequency, magnitude, and volume of stormwater runoff, leading to 1.1 up to 3 times more discharge (depending on the percentage of transmission losses assumed between the system and the stream), but with a contribution to baseflow restoration regardless of the of uncertainty over the extent of transmission losses. The hypothetical RWH system proposed by (Burns et al. 2012) is an example of an unaware MAR-RWH since infiltration was prescribed, along with groundwater pollution prevented by the rain garden. It is possible that some SUDS-RWH sites located worldwide have gone unnoticed by MAR researchers, or that some SUDS-

RWH sites can go through adaption to turn into real MAR-RWH projects, with intentional and monitored aquifer recharge included.

MAR-RWH systems have a typical devolution/scale of a family (10^2 to 10^3 m^3/year) and a typical unit cost of 10 US$/m³ (Dillon 2005; IGRAC 2007). Comparing to other water supply methods (surface dam and its treatment plant, desalination, and other MAR methods), they have low investigation costs, low technical knowledge requirements, and low regulation difficulty (Dillon 2005). The recharge usually occurs by gravity-based injection, or by percolation through the unsaturated zone to the water table, where the water is collected by pumping from a well (not necessarily the same used for recharge; Dillon (2005). Even though rainwater /runoff harvesting systems are considered beneficial to restore water in the hydrological cycle in urban areas (Teston et al. 2018), this MAR method is the least reported (and applied) technology worldwide, accounting for 55 out of the 1,104 MAR sites registered at the IGRAC MAR Portal (IGRAC 2020). From these 55 projects, 34 stands for MAR-RWH (twenty in India, ten in China, two in Brazil, one in Australia and the UK each; data gathered from IGRAC 2020). MAR-RWH systems are also the least studied concerning hypothetical modelling studies – no paper concerning such technique was found in a review analysing 233 case studies which apply flow and transport models to evaluate MAR, from 1985 to 2015 (Ringleb et al. 2016). Page et al. (2010) reported a MAR-RWH site in Kingswood, Australia, that used rainwater collected from a 285 m^2 residential development to recharge an unconfined aquifer for 39 months, using a 3 m^3 water tank as interim storage, aiming at enhancing the quality of native groundwater to meet irrigation demand. Two injection wells (4-inch and 5-inch diameter, respectively; filter lengths of 12 m) were used to recharge the aquifer via gravity. The system was able to recharge 487 m^3 in the whole period (at a mean recharge rate of 0.018 m^3/h) but was decommissioned since the proposed system was unable to improve the groundwater quality for irrigation use, mainly due to the hydrogeology of the region and the relatively small amount of injected water, when compared to lateral flows. For preventive water quality measures, a tank strainer and a 100-micron filter were prescribed by local authorities, after the site went through a simple risk assessment. Pumped MAR-RWH schemes, combined with ASR wells, might be reported in the MAR Portal (IGRAC 2020), such as the case reported by (Kretschmer 2017) in the City of Salisbury, South Australia, in which a 80-ha urban residential catchment conveys stormwater (60,000 m^3/year) to mitigate flooding (among other objectives) via a 164-metre deep well with open-hole completion. This may also be the case of some MAR sites in the MAR Portal whose main objectives falls under the ecological benefits' or other benefits' categories.

Several artificial gravity-based recharge tests are reported in the literature, some of them in the context of the implementation of MAR-RWH: Diniz et al. (2008) were able to recharge around 27 m³/h in 30 minutes, during gravity-based injection tests using a 14-inch tubular well, concluding that operation of a MAR-RWH scheme is feasible in the region; (Silva et al. 2006) reported the preliminary studies of a MAR-RWH in Recife, Brazil, that resulted in the injection of 6.4 m³ of water into a confined aquifer using a 4-inch diameter well (16 m screen length). The injection test reported took 150 minutes, with a mean injection rate of 2.07 m³/h. Based on the experimental results and local hydrogeology, the study was able to implement numeric modelling which showed that a small-term simulation (three months) can significantly raise the water table in the site's surroundings; Liu et al. (2016) was able to recharge around 92 m³ in a shallow unconfined aquifer for 15 hours, with a mean rate of 6.2 m³/h, using a 2-inch direct-push well; Conrad (2019) performed injection tests on a 2-inch diameter well (screen length of 8 meters) connected to a confined aquifer in Recife, Brazil, obtaining a mean recharge rate of 0.072 m³/h after the initial 100 L injection. The small recharge rate might have occurred due to air entrapment, friction losses, and presence of fine sandstone with clayey intercalations surrounding the screen length; Händel et al. (2016) used a 1-inch diameter injection well (0.3 m screen length) to recharge almost 910 m³ during a 14-day injection test at a shallow aquifer (mean recharge rate of 2.708 m³/h); and Barbassa et al. (2014) investigated the retention of suspended solids after injecting water from 11 simulated rainfall events (2 years return period; water mixed with suspended solids coming from the site), using a 5 m³ water tank. The infiltration well for rainwater harvesting, studied by Barbassa et al. (2014), consisted of a 135 m diameter well (i.e. around 5,300 inches). The mean recharge rates reported in the abovementioned studies are dependent on many site-specific factors, particularly driven by local hydrogeology and structure of the injection well. However, the reported gravity-based recharge rates are promising to promote an increase in water demand connected to the settling tanks, thus contributing to retention-based stormwater management strategies (Burns et al. 2012, 2015).

4 STUDY AREA DESCRIPTION

4.1. João Pessoa city

This study was carried out in João Pessoa (JPA), a coastal urban city located in the Paraíba State, Northeast of Brazil, between the coordinates 7° 6’ 55’’ S – 34° 51’ 40’’ W (Figure 1a). JPA has a tropical climate, with a mean annual temperature above 25.9 ºC and mean relative humidity of 75% (hourly measurements from 2007-07-21 to 2019-12-31, station A320, INMET

2020). The mean annual rainfall in JPA is approximately 1,511 mm, 78% concentrated from March to August (hourly measurements from 2008-01-01 to 2019-12-31, station A320, INMET 2020). JPA is the twenty-fourth largest city in Brazil, with ~1 M inhabitants in their metropolitan region.

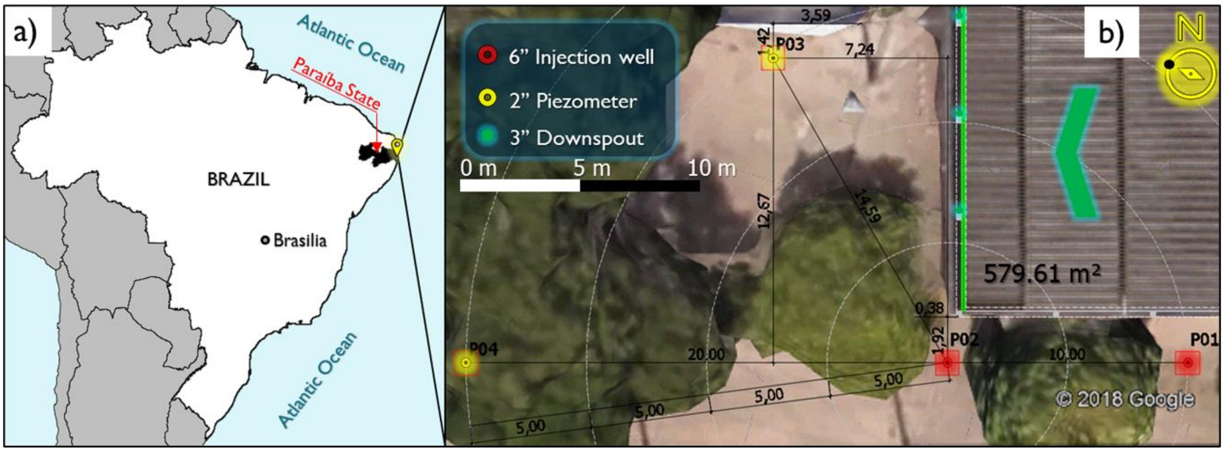

Figure 1 - Study area a) location (adapted from Microsoft PowerPoint), b) sketch (adapted from Google Earth)

JPA is located at the Paraíba sedimentary basin, mostly inside the central Alhandra sub-basin (Rossetti et al. 2012; Walter 2018), predominantly made of sedimentary soils (sandy and clay deposits). Four distinct lithologic layers are found in JPA: the Barreiras, Farinha, Gramame, and Beberibe Formation (Walter 2018). Rossetti et al. (2012) have identified a fifth layer, the Itamaracá Formation. Beberibe Formation is a confined aquifer, a 360 metres thick medium- to course-grained sandstones (Rossetti et al. 2012), directly connected to the Barreiras Formation (unconfined aquifer) in the west of JPA. In the east, the Gramame Formation, a richly fossiliferous unit (Rossetti et al. 2012), is located between them (Walter 2018). The Barreiras Formation's morphoestructure is mainly made of poorly consolidated clayey sands dated from the Miocene (Furrier et al. 2006; Rossetti et al. 2012), alluvial sediments and sandstone, with thickness up to 80 metres in the east of JPA, its mean thickness around 20 to 40 metres (Bertrand et al. 2017; Walter 2018). The hydraulic conductivity of the Barreiras Formation is estimated as below 3.47×10^{-5} m/s (Walter 2018), although values around 1×10^{-4} m/s have been reported (Fernandes 2017).

According to Walter (2018), the annual rainfall in JPA strongly influences its water availability – the water table is mainly dependant upon vertical local recharge since inland groundwater recharge is practically nonexistent. The vertical groundwater recharge is substantially restricted in JPA because of soil impermeabilization as a consequence of the urbanization process in the city (Walter 2018). The urbanization in JPA reduced the water infiltration over the region, intensifying the erosion and runoff processes (Furrier and Barbosa 2016). According to Furrier and Barbosa (2016), the city faces an urban development that does not consider geomorphology aspects, thus becoming more prone to floods and its intrinsic damages.

On the other hand, the open availability of roofs shows an opportunity for RWH systems implementation, similar to remarks made by Coelho et al. (2018) concerning the Recife Metropolitan Region (RMR), suited 117 km distance from João Pessoa. These RWH systems can both increase the groundwater recharge in the urban area as well as reduce the risk of flooding in the region. This ancillary benefit might draw the attention of stakeholders on the potential of MAR in mitigating water-related issues and promote the implementation of other MAR schemes in the region, especially in the context of the need of more detailed regulatory criteria at national-scale to consolidate MAR implementation (Silva et al. 2019; Shubo et al. 2020).

4.2. Site of the RWH system

Specifically, this study was carried out in a site inside the João Pessoa Campus of the Federal University of Paraíba (7°8'32.47" S and 34°50'58.99" W; Figure 1b), where two injection wells (P1 and P2) with a 6-inch diameter and two monitoring wells (P3 and P4) with 2-inch diameter are available. The wells are 42 meters deep, drilled in the Barreiras unconfined aquifer with a screen length measuring 12 meters (from depth 28 to 40 meters). Annexe B shows the report provided by the drilling company, concerning all referred wells. The static water table in the site when the construction of the wells was concluded (2019-05-23) was about 28 meters. Several buildings in the surroundings of the experiment and a large amount of rainfall over the region are virtually available for roofing-water collection to implement managed direct injection of rainwater into the aquifer. The Hydraulics Laboratory presents the largest rooftop area in the surroundings of the wells, with ~ 580 m^2.

Figure 2 shows the lithological profile of the injection wells, which are composed of fine sand and clay layers that vary in colour from yellow to reddish-yellow. This lithological profile was drawn based on results of liquidity and plasticity limit tests carried out with soil material collected during the drilling of all wells and piezometers. The soil material was collected at every two meters depth and analysed in cooperation with the Geotechnics and Paving Laboratory (LAPAV) of the Federal University of Paraíba (UFPB). The lithological profile drawn from material in the site is consistent with descriptions from Walter (2018) and (Rossetti et al. 2012), in which the Barreiras Formation can be characterized by its poor selection, irregular stratification and the occurrence of varying colours.

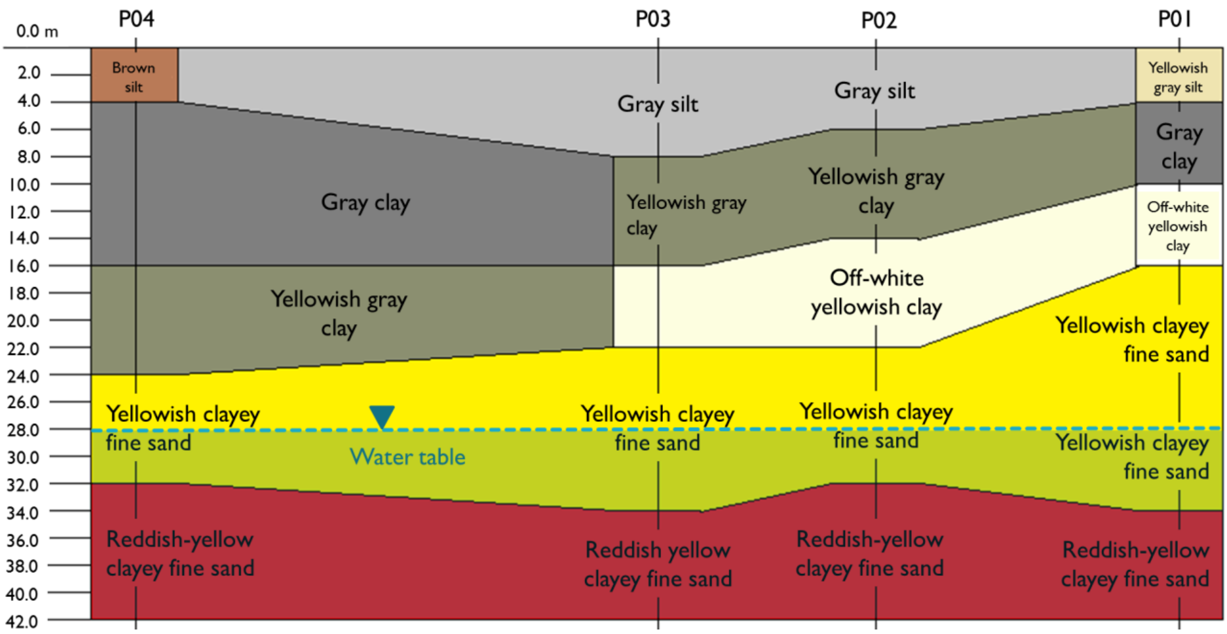

Figure 2 - Lithological profile of the RWH system's site

5 MATERIALS AND METHODS

This work presents a hypothetical study since hydrologic processes were simulated using a computer model. Both empirical non-experimental (monitored real-time rainfall) and experimental (based on *in-situ* tests) data were gathered and used in the study. The rainfall data used will be briefly presented in the next section. Then, pre- and post-development scenarios will be discussed in section 5.2. These are the key inputs to the hydraulic simulations, to follow in section 5.3.

5.1. Rainfall data

This study used rainfall data with a high temporal resolution to enable the execution of refined water balance simulations. The rainfall dataset for the period 2004-01-01 to 2019-12-31 was acquired from automatic tipping bucket rain gauges with a 1-min temporal resolution when it rains and 5-min over no-rain periods. This rainfall dataset is located within the Guaraíra Experimental Basin (GEB; Coutinho et al. 2014), which is an experimental river basin monitored by the water research group from the Laboratory of Water Resources and Environmental Engineering (LARHENA) of the UFPB, near to the pilot experiment. GEB rainfall data comprises data not only from one single pluviograph but is made of the best data acquired at each rain gauges, pre-processed by the research group. Figure 3 shows the mean monthly rainfall in the period recorded, which lies around 1,571 mm/year on average. Data from INMET (2020), disclosed in section 4.1, was also plotted in Figure 3, to enable a visual comparison between data from both sources.

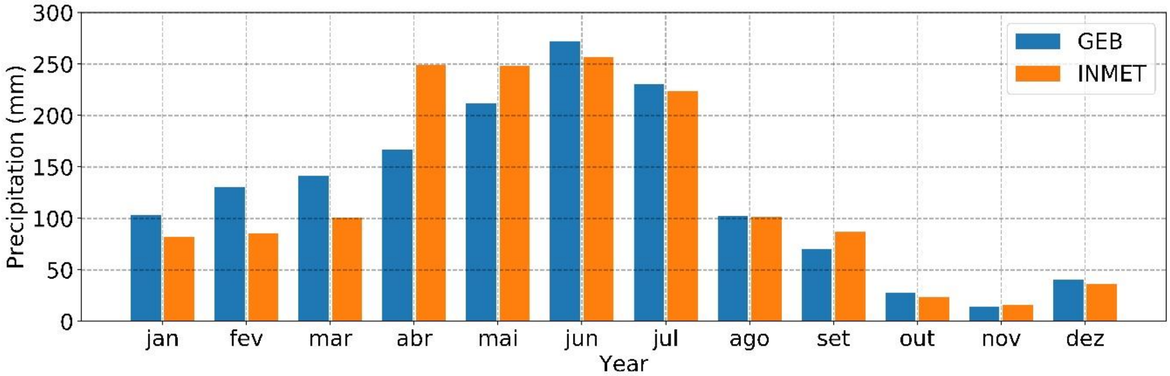

Figure 3 – Mean monthly rainfall recorded in GEB (2004 – 2019) and INMET (2008 – 2019)

5.2. Pre- and post-development scenarios

This study comprised two development scenarios with different inflow rates to evaluate the feasibility of the rainwater harvesting system. The former scenario, herein called a post-development scenario, in which the catchment area is made by an impervious roof, with roughness coefficient of 0.027, from a metallic built-up channel, unpainted smooth steel surface (Chow 1959); and the latter scenario, called a pre-development scenario, in which the catchment area is constituted by a permeable surface soil with the characteristics found *in situ* from an infiltration test carried out with double-ring 60-cm diameter infiltrometers (see section 5.6) and roughness coefficient of 0.012, assumed to be from an excavated channel, earth, straight and uniform, with short grass and few weeds (Chow 1959). The post-development scenario represents the current conditions of the site whereas the pre-development scenario represents the site's conditions before development had been established.

The rainfall-runoff process was computed using the U.S. Environmental Protection Agency (EPA) Storm Water Management Model (SWMM; Rossman 2015), using the whole one-minute step rainfall data from GEB (16-year period), rain format in volume. SWMM is an open source software developed in 1971. It is free, broad (it allows for runoff generation, flow routing, and stormwater collection networks modelling) and diverse (multiple hydrologic and hydraulic computation methods are available; Kaykhosravi et al. 2018). It also can be viewed as a physically based LID toolbox (Eckart et al. 2017), better suited for more advanced modelling phases (preliminary and detailed design/analysis), where information such as peak flow, runoff amount and volume within conduits are required (Kaykhosravi et al. 2018). LID simulation for water quality and GIS integration is not offered in the open source version however more advanced proprietary software based on SWMM (e.g. Mike-Urban, PCSWMM), in which these tools are available, do exists. SWMM is popular among stormwater management researchers and is used worldwide for planning, analysis and design related to drainage systems (Petrucci et al. 2012; Wang and Altunkaynak 2012; Karamouz and Nazif 2013; Cipolla et al. 2016; Zhang et al. 2016; Avellaneda et al. 2017; Palla et al. 2017; Paule-Mercado et al. 2017; Zanandrea and Silveira 2018). Among existing models, a recent review of hypothetical studies has found that the SWMM model was the most utilized, representing 38% of selected studies (Sohn et al. 2019). The model is commonly used in studies based on current and historic climate data, and on short-term event-based analysis (Sohn et al. 2019), and has been used to evaluate RWH hydrologic performance on stormwater management (e.g. Petrucci et al. 2012; Palla et al. 2017).

The pre-development scenario considered no percentage of impervious area and was drawn using the Horton infiltration method, while the post-development scenario considered a 100% impervious area hence no infiltration method was applicable. The kinematic wave routing model was used with a 1-minute time step. The catchment area's outlet was considered as the tank inlet, that is, the flow of water through gutters and downspouts were neglected.

Before running the simulations, the GEB rainfall dataset had to be converted into text files (.dat format, Figure 2), one file per year. The main difference between the raw rainfall dataset and the converted ones is that in the former the data is following the Julian calendar whereas in the latter data follows the Gregorian calendar. A script in Python was written to perform the referred conversion.

```
id,array,year,jday,hour,rain
13965,20,2019,1,2251,0.254
13967,20,2019,1,2309,0.254
13976,20,2019,2,642,0.254
14121,20,2019,7,956,0.254
14123,20,2019,7,1003,0.254
14124,20,2019,7,1100,0.254
14126,20,2019,7,1101,0.254
14127,20,2019,7,1102,0.254
14128,20,2019,7,1103,0.762
14129,20,2019,7,1104,1.016
14130,20,2019,7,1105,0.508
14131,20,2019,7,1106,0.508
```

ID	Year	Month	Day	Hour	Minute	Value
GEB	2019	1	1	22	51	0.254
GEB	2019	1	1	23	9	0.254
GEB	2019	1	2	6	42	0.254
GEB	2019	1	7	9	56	0.254
GEB	2019	1	7	10	3	0.254
GEB	2019	1	7	11	0	0.254
GEB	2019	1	7	11	1	0.254
GEB	2019	1	7	11	2	0.254
GEB	2019	1	7	11	3	0.762
GEB	2019	1	7	11	4	1.016
GEB	2019	1	7	11	5	0.508
GEB	2019	1	7	11	6	0.508

Figure 4 – Conversion of rainfall raw data into readable SWMM rainfall data

Pre- and post-development hydrographs were calculated considering a collection of 13 rooftop catchment squared areas. The reason why the rooftop catchment collection (shown in Table 1) spanned from 100 up to 5,000 m² is that it covered a variety of residential developments, from standard single-family residences up to multi-family residential condominiums (ABNT 2006).

Table 1 - Collection of squared rooftop catchment areas used in the hydraulic simulations

Rooftop area (m²)	Width of the overland flow path (m)	Rooftop area (m²)	Width of the overland flow path (m)
10	3.16	800	28.28
20	4.47	900	30.00
30	5.48	1000	31.62
40	6.32	1100	33.17
50	7.07	1200	34.64
100	10.00	1300	36.06
150	12.25	1400	37.42
200	14.14	1500	38.73
250	15.81	2000	44.72
300	17.32	2500	50.00
400	20.00	3000	54.77
500	22.36	3500	59.16
600	24.49	4000	63.25
700	26.46	4500	67.08
750	27.39	5000	70.71

Each combination of year, catchment area and scenario required a unique water balance simulation. Hence, the SWMM software was manipulated through the usage of a batch file (Figure 3) which allowed serial execution of the software. Each row in the batch file has the name of the software (swmm5.exe) and a pair of files (project file and report file). The project file (.inp format, Figure 4) contains the inputs for the simulation, such as the information of the rain gauge, its format (volume) and its rain unit (millimetres); and the information of the subcatchment rooftop, its area (ha), width (m), percentage of slope, percentage of imperviousness and coefficient of roughness. Besides, other information is also stored in the project file, such as the start and end analysis and reporting dates, and information regarding the routing and infiltration model to be used. The report file (.rpt format, Figure 5) is initially empty and, after the simulation finishes, stores data regarding the simulation in the subcatchment, in terms of precipitation (mm/h) and runoff values (in m³/s) for each time step. Before running the batch file, every project and report files were created using a routine written in Python.

```
swmm5.exe GEB_2004_1min_10m2_postDev.inp GEB_2004_1min_10m2_postDev.rpt
swmm5.exe GEB_2004_1min_10m2_preDev.inp GEB_2004_1min_10m2_preDev.rpt
swmm5.exe GEB_2005_1min_10m2_postDev.inp GEB_2005_1min_10m2_postDev.rpt
swmm5.exe GEB_2005_1min_10m2_preDev.inp GEB_2005_1min_10m2_preDev.rpt
swmm5.exe GEB_2006_1min_10m2_postDev.inp GEB_2006_1min_10m2_postDev.rpt
swmm5.exe GEB_2006_1min_10m2_preDev.inp GEB_2006_1min_10m2_preDev.rpt
swmm5.exe GEB_2007_1min_10m2_postDev.inp GEB_2007_1min_10m2_postDev.rpt
swmm5.exe GEB_2007_1min_10m2_preDev.inp GEB_2007_1min_10m2_preDev.rpt
swmm5.exe GEB_2008_1min_10m2_postDev.inp GEB_2008_1min_10m2_postDev.rpt
swmm5.exe GEB_2008_1min_10m2_preDev.inp GEB_2008_1min_10m2_preDev.rpt
swmm5.exe GEB_2009_1min_10m2_postDev.inp GEB_2009_1min_10m2_postDev.rpt
swmm5.exe GEB_2009_1min_10m2_preDev.inp GEB_2009_1min_10m2_preDev.rpt
swmm5.exe GEB_2010_1min_10m2_postDev.inp GEB_2010_1min_10m2_postDev.rpt
```

Figure 5 – Example of a batch file content

```
[TITLE]
;;Project Title/Notes
Tutorial Example

[OPTIONS]
;;Option             Value
FLOW_UNITS           CMS
INFILTRATION         HORTON
FLOW_ROUTING         KINWAVE
LINK_OFFSETS         DEPTH
MIN_SLOPE            0
ALLOW_PONDING        NO
SKIP_STEADY_STATE    NO

START_DATE           01/01/2019
START_TIME           00:00:00
REPORT_START_DATE    01/01/2019
REPORT_START_TIME    00:00:00
END_DATE             01/01/2020
END_TIME             00:00:00
SWEEP_START          01/01
SWEEP_END            12/31
DRY_DAYS             0
REPORT_STEP          00:01:00
WET_STEP             00:01:00
DRY_STEP             00:01:00
ROUTING_STEP         0:00:30
RULE_STEP            00:00:00

INERTIAL_DAMPING     PARTIAL
NORMAL_FLOW_LIMITED  BOTH
FORCE_MAIN_EQUATION  H-W
VARIABLE_STEP        0.75
LENGTHENING_STEP     0
MIN_SURFAREA         12.557
MAX_TRIALS           8
HEAD_TOLERANCE       0.005
SYS_FLOW_TOL         5
LAT_FLOW_TOL         5
MINIMUM_STEP         0.5
THREADS              1
```

post-dev project file

```
[RAINGAGES]
;;Name           Format    Interval SCF      Source
;;-------------- --------- ------ ------ ----------
Gage1            VOLUME    0:01     1.0      FILE       "C:\path\GEB_year.dat" GEB       MM

[SUBCATCHMENTS]
;;Name           Rain Gage        Outlet           Area     %Imperv  Width    %Slope   CurbLen  SnowPack
;;-------------- ---------------- ---------------- -------- -------- -------- -------- -------- --------
S1               Gage1            Out1             0.001    100      3.16     30       0

[SUBAREAS]
;;Subcatchment   N-Imperv   N-Perv     S-Imperv   S-Perv     PctZero    RouteTo    PctRouted
;;-------------- ---------- ---------- ---------- ---------- ---------- ---------- ----------
S1               0.012      0.027      0          0          100        OUTLET

[INFILTRATION]
;;Subcatchment   MaxRate    MinRate    Decay      DryTime    MaxInfil
;;-------------- ---------- ---------- ---------- ---------- ----------
S1               419.212    30.48      4.8        2          0
```

pre-dev project file

```
[RAINGAGES]
;;Name           Format    Interval SCF      Source
;;-------------- --------- ------ ------ ----------
Gage1            VOLUME    0:01     1.0      FILE       "C:\path\GEB_year.dat" GEB       MM

[SUBCATCHMENTS]
;;Name           Rain Gage        Outlet           Area     %Imperv  Width    %Slope   CurbLen  SnowPack
;;-------------- ---------------- ---------------- -------- -------- -------- -------- -------- --------
S1               Gage1            Out1             0.001    0        3.16     1        0

[SUBAREAS]
;;Subcatchment   N-Imperv   N-Perv     S-Imperv   S-Perv     PctZero    RouteTo    PctRouted
;;-------------- ---------- ---------- ---------- ---------- ---------- ---------- ----------
S1               0.012      0.027      0          0          100        OUTLET

[INFILTRATION]
;;Subcatchment   MaxRate    MinRate    Decay      DryTime    MaxInfil
;;-------------- ---------- ---------- ---------- ---------- ----------
S1               419.212    30.48      4.8        2          0
```

Figure 6 – Example of post-dev and pre-dev project files. Information within the left box is shared by both kinds of files and highlighted information (orange dashed small boxes) shows data that is unique for each kind of file (%Imperv and %Slope stands for the percentage of imperviousness and the percentage of slope, respectively)

```
Subcatchment Runoff Summary
***************************

------------------------------------------------------------------------------------------------------------------------------
                         Total      Total      Total      Total      Imperv       Perv      Total       Total     Peak   Runoff
                        Precip      Runon       Evap      Infil      Runoff     Runoff     Runoff      Runoff   Runoff    Coeff
Subcatchment                mm         mm         mm         mm          mm         mm         mm    10^6 ltr      CMS
------------------------------------------------------------------------------------------------------------------------------
S1                     1645.16       0.00       0.00       0.00     1668.73       0.00    1668.73        2.34     0.06    1.014


********************
Subcatchment Results
********************

<<< Subcatchment S1 >>>
--------------------------------------------------------
Date        Time         Precip.     Losses     Runoff
                           mm/hr      mm/hr        CMS
--------------------------------------------------------
 01/01/2019 00:01:00       0.000      0.000     0.0000
 01/01/2019 00:02:00       0.000      0.000     0.0000
           ...               ...        ...        ...
 01/01/2019 22:50:00       0.000      0.000     0.0000
 01/01/2019 22:51:00      15.240      0.000     0.0000
 01/01/2019 22:52:00       0.000      0.000     0.0015
 01/01/2019 22:53:00       0.000      0.000     0.0010
 01/01/2019 22:54:00       0.000      0.000     0.0007
 01/01/2019 22:55:00       0.000      0.000     0.0005
 01/01/2019 22:56:00       0.000      0.000     0.0004
 01/01/2019 22:57:00       0.000      0.000     0.0003
 01/01/2019 22:58:00       0.000      0.000     0.0002
 01/01/2019 22:59:00       0.000      0.000     0.0002
 01/01/2019 23:00:00       0.000      0.000     0.0002
 01/01/2019 23:01:00       0.000      0.000     0.0001
 01/01/2019 23:02:00       0.000      0.000     0.0001
```

Figure 7 – Example of a post-dev report file, showing part of the precipitation (mm/h) and runoff (m³/s) temporal series (2019-01-01 to 2019-12-31) for a 5,000 m² subcatchment area

The last step towards the hydraulic simulations was to convert every group of pre-dev and post-dev report files, for each catchment area and year, into singles files (.csv format, herein called input runoff files), with runoff units changed to L/min (Figure 6). This last measure was taken for speed processing purposes (opening an input runoff file with pre-dev and post-dev temporal series was found to be faster than opening two report files simultaneously, one for each scenario).

	Date	Time	Pre Runoff (L/min)	Post (Runoff (L/min)
0	01/01/2019	00:00:00	0	0
	...	...	...	...
1372	01/01/2019	22:52:00	0	180
1373	01/01/2019	22:53:00	0	144
1374	01/01/2019	22:54:00	0	114
1375	01/01/2019	22:55:00	0	96
1376	01/01/2019	22:56:00	0	78
1377	01/01/2019	22:57:00	0	66
1378	01/01/2019	22:58:00	0	54
1379	01/01/2019	22:59:00	0	48
1380	01/01/2019	23:00:00	0	42
1381	01/01/2019	23:01:00	0	36
1382	01/01/2019	23:02:00	0	30
1383	01/01/2019	23:03:00	0	30
1384	01/01/2019	23:04:00	0	24
1385	01/01/2019	23:05:00	0	24
1386	01/01/2019	23:06:00	0	18
1387	01/01/2019	23:07:00	0	18
1388	01/01/2019	23:08:00	0	18
1389	01/01/2019	23:09:00	0	18
1390	01/01/2019	23:10:00	0	240

Figure 8 – Example of an input runoff file, showing part of the pre-dev and post-dev runoff (m^3/s) temporal series (2019-01-01 to 2019-12-31) for a 5,000 m^2 subcatchment area

5.3. Hydraulic simulations

Water balance calculations were carried out considering the RWH system, which comprises three elements: a) the rooftop catchment area, b) the interim storage made of a water tank and c) the injection well P02. The input runoff goes into the interim storage before being introduced into the injection well for further aquifer recharge. Figure 7 exhibits a sketch of the RWH system. After processing the input runoff files (runoff temporal series for each rooftop catchment area, year, and scenario), these data were used as input for the water balance simulations. The hydraulic simulations were carried out through a routine written in Python.

For each input runoff file (that is, for each catchment area and year), a collection of different water tanks was tested, aiming at determining the proportion of runoff that effectively recharges the aquifer and the one that is spilt over the tank. Table 2 exhibits the collection of water tanks, whose dimensions were taken from commercial catalogues provided by Brazilian water tank suppliers. Dimensions of water tanks from 500 to 5,000 L were taken from Tigre S/A (2016); from 7,500 to 15,000 L were taken from Fortlev (2019); from 20,000 and 25,000

L were taken from Caixa Forte (2020); and dimensions of the 30,000 L water tank were taken from Bakof Tec (2020). The average price of water tanks, in the function of their volume, were calculated after a brief survey was carried out on the internet, in June 2020 (Annexe C). Prices of water tanks with a capacity of 30,000 L were not found in the survey.

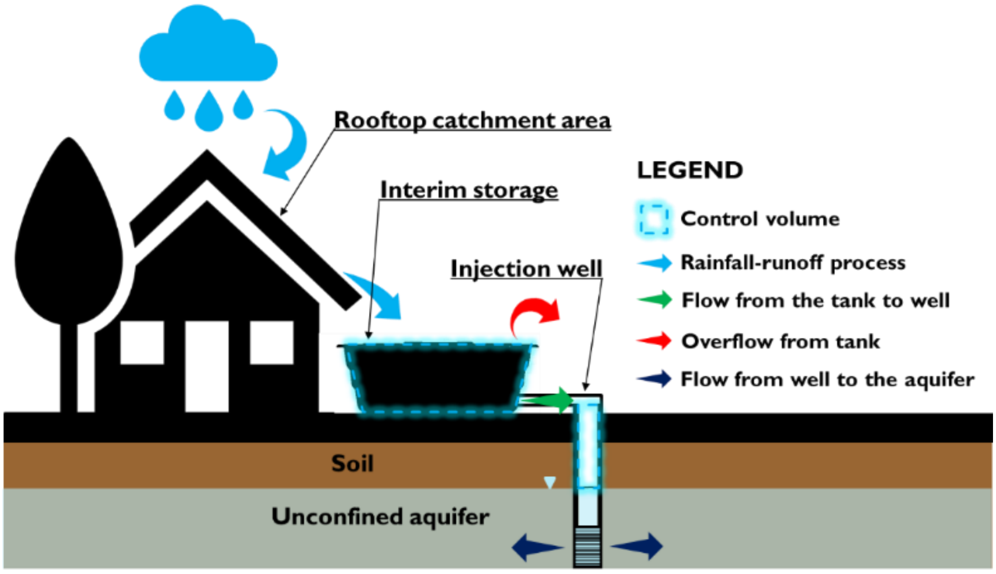

Figure 9 – Sketch of the main components of the rainwater harvesting system

Table 2 - Collection of tank volumes used in the hydraulic simulations

Tank volume *S* (L)	Height *H* (cm)	Lower diameter D_2 (cm)	Upper diameter D_1 (cm)	Mean price
500	53.34	97.83	121.20	R$ 167,72
1,000	77.50	114.57	144.00	R$ 300,93
3,000	112.41	172.17	215.50	R$ 1,341.89
5,000	162.00	182.36	233.40	R$ 2,361.27
7,500	181.00	224.00	270.00	R$ 3,151.12
10,000	203.00	241.00	292.00	R$ 3,127.77
15,000	262.00	267.00	315.00	R$ 5,965.45
20,000	323.00	244.00	335.00	R$ 8,470.00
25,000	387.00	235.00	338.00	R$ 7.740,00
30,000	473.00	250.00	320.00	---

Source: (Tigre S/A 2016; Fortlev 2019; Bakof Tec 2020; Caixa Forte 2020)

The water balance simulation considers two control volumes (illustrated in Figure 7): the first one for the inner volume of the water tank (called ST_{max}) and the second for the inner space of the injection well (called SW_{max}; measured from the top of the casing to the water table).

Hydraulic simulations were executed in both control volumes using a lumped flow routing method (modified Puls method; David R. Maidment 1992), which is based on the mass conservation equation. The modified Puls method, a slight change from the original method proposed by L. Goodrich Puls in 1928 (Chow 1959; Maidment 1992; Ghasemzadeh et al. 2020), has been extensively used in research comprising reservoir routing (Singh and Snorrason 1984; Madadi et al. 2015; Baptista and Paz 2018; Ferreira et al. 2018, 2019; Kamis et al. 2018; Ghasemzadeh et al. 2020). When the mass conservation equation is approximated by finite differences and subsequently rearranged it may assume the forms given in Equation 2 and Equation 3:

$$\frac{2\mathrm{ST}_{t+\Delta t}}{\Delta t} + \mathrm{QT}_{t+\Delta t} = \mathrm{IT}_{t+\Delta t} + \mathrm{IT}_{t} - \mathrm{QT}_{t} - 2\mathrm{OT} + \frac{2\mathrm{ST}_{t}}{\Delta t} \tag{2}$$

$$\frac{2\mathrm{SW}_{t+\Delta t}}{\Delta t} + \mathrm{QW}_{t+\Delta t} = \mathrm{IW}_{t+\Delta t} + \mathrm{IW}_{t} - \mathrm{QW}_{t} + \frac{2\mathrm{SW}_{t}}{\Delta t} \tag{3}$$

where ST, IT, QT and OT are the storage (L), inflow rate (L/min), outflow rate (L/min), and overflow rate (L/min) in the tank control volume ST_{max}, respectively; SW, IW, and QW are the storage (L), inflow rate (L/min), and outflow rate (L/min) in the injection well control volume SW_{max}, respectively. Equation 2 differs from the finite differences continuity equation since the OT term was included to cover the situation where the water in the tank is overflowing (when that is not the case, OT is null). The subscripts t and $t+\Delta t$ indicate the instant of time t (min) and the next instant, according to the time step Δt (min). Two serial hydraulic simulations were performed: the former in the water tank control volume and the latter in the well control volume. The first simulation followed Equation 2 and the second followed Equation 3.

The initial conditions were that IT_t, QT_t, ST_t, OT_t, QW_t, and SW_t were setup as nil. Hence, at a given time t, the terms on the left in Equation 2 and Equation 3 are unknown while terms on the right are known. To solve this issue, the modified Puls method (Chow 1959; Ghasemzadeh et al. 2020) prescribes the use of auxiliary curves for each control volume, which are built through known relations between the water height h inside the control volumes and its storage S and outflow rate Q at given equal time steps Δt. The construction of these auxiliary curves is meant to provide values of outflow rate as a function of the sum of values of the terms on the left in Equation 2 and Equation 3, for each control volume. These curves are based on the premise that each control volume is fixed, and its geometry is known. In this study, the tank control volume ST_{max} was approximated as a circular truncated cone, whose storage S_t is computed as a function of its height h (m), the lower diameter D_2 (m), and diameter at the water level within the tank $D(h)$, as shown in Equation 4 and Equation 5:

$$S_t(h) = 1{,}000 \times \frac{\pi h}{3}\left[\left(\frac{D(h)}{2}\right)^2 + \left(\frac{D(h)}{2}\right) \times \left(\frac{D_2}{2}\right) + \left(\frac{D_2}{2}\right)^2\right] \tag{4}$$

$$D(h) = \frac{h}{H}(D_1 - D_2) + D_2 \tag{5}$$

where D_1 is the tank's upper diameter (m) and H is the tank's height (m). Equation 5 describes a linear interpolation equation, using the height H, lower and upper diameters D_1 and D_2 as interpolation limits. The outflow rate Q_t (L/min) at a given water level within the tank was calculated as a function of the hydraulic head h (m) and the cross-section of the discharge outlet $A_Ø$ (m²), considering a 60-millimetre diameter tube (Porto 2006), as shown in Equation 6:

$$Q_t = 60{,}000 \times C_d A_{\emptyset} \sqrt{2gh} \tag{6}$$

where C_d is the discharge coefficient, whose value is roughly 0.6076 for the given outlet tube diameter (Porto 2006). On the other hand, the injection well control volume SW_{max} was approximated as a cylinder with a fixed diameter, where its storage S_w varies in the function of its height h (m) ranging from the top of the casing to the water table, as shown in Equation 7:

$$S_w = 1{,}000 \times A_w h \tag{7}$$

where A_w is the inner cross-section of the 6-inch diameter injection well (m²) and h is the dynamic level of water above the static water level (m). Since the premise of the Puls method implies this control volume can be assumed unchangeable (ΔSW_{max} and ΔST_{max} are null), pumping tests and injection tests were performed *in situ* to test the validity of this hypothesis (see section 5.6). The injection tests also provided knowledge on the relationship between the outflow rate Q_w and the water level h within the injection well, demanded to construct the auxiliary curve for the well control volume.

$IT_{t+\Delta t}$ is the post-development hydrograph in each input runoff file while IW_t and $IW_{t+\Delta t}$ are respectively equivalent as QT_t and $QT_{t+\Delta t}$ since the outflow from the tank is the inflow in the well. The outflow from the well is the amount of water that recharges the unconfined aquifer. When the left terms of Equation 2 surpass a maximum known value associated with the fullness of the tank control volume, $QT_{t+\Delta t}$ is calculated using this maximum known value and then the overflow rate (OT) can be computed from Equation 2. On the other hand, when the left terms of Equation 3 surpass a maximum known value associated with the fullness of the injection well control volume, $QW_{t+\Delta t}$ is calculated using this maximum known value and $IW_{t+\Delta t}$ becomes equal to $QW_{t+\Delta t}$ and $QT_{t+\Delta t}$ equal to $IW_{t+\Delta t}$, demanding an update in the tank control volume simulation since this condition forces less water to flow from the tank into the well and therefore more water may overflow.

5.4. Metrics used for evaluation of the simulations

Two metrics were used to assess the RWH system performance, given by Equation 8 (Freni and Liuzzo 2019) and Equation 9 (Baptista and Paz 2018):

$$R = \frac{\sum_{t=1}^{T} \mathrm{IT}_t - \sum_{t=1}^{T} \mathrm{OT}_t}{\sum_{t=1}^{T} \mathrm{IT}_t} \times 100 \tag{8}$$

$$E = \frac{\mathrm{OT}_{\mathrm{max}} - \mathrm{IT}_{\mathrm{max,post}}}{\mathrm{IT}_{\mathrm{max,pre}} - \mathrm{IT}_{\mathrm{max,post}}} \times 100 \tag{9}$$

where R is the rainwater harvesting system retention, given in percentage, being $\Sigma \mathrm{IT}_t$ the runoff inflow volume (L) and $\Sigma \mathrm{OT}_t$ the runoff overflow volume (L) at time interval spanning from t to T (min); and E is the rainwater harvesting system efficiency, representing how much close the runoff peak overflow ($\mathrm{OT}_{\mathrm{max}}$) is from the runoff peak pre-development flow ($\mathrm{I}_{\mathrm{Tmax,pre}}$), the latter considered as the condition where the system effectively contributes towards no flooding downstream; $\mathrm{I}_{\mathrm{Tmax,post}}$ is the runoff peak post-development flow.

R, which ranges from 0% to 100%, represents the proportion of runoff that is effectively drained to the well for further aquifer recharge, hence $1 - R$ represents the part of the input runoff that overflows from the water tank, potentially leading to flooding.

E, which ranges from 0% to $+\infty$, gives a numerical evaluation of how well the system mitigates flooding. When its value is higher than 100% then it means the RWH system is capable to dampen the post-dev runoff peak flow to a level smaller than the pre-dev runoff peak flow, which means the system is effective towards flood mitigation. If E is 100% then it means either that R is 100% (hence no proportion of input runoff overflowed from the tank) or that the runoff peak overflow is equal to the pre-dev runoff peak overflow. E smaller than 100% means that the RWH system's runoff peak overflow is higher than the runoff peak pre-development flow. The smaller the value of E the most inefficient towards flood mitigation the RWH system is and when its value is zero it means the system is completely useless.

5.5. Cost considerations

The costs of the RWH systems (injection well cost in addition to water tank cost) was compared to the cost of the building that provides its catchment area. The cost of the injection well is the cost of the well P02 (around R$ 16,000,00) while each water tank's mean cost is displayed in Table 2).

The collection of rooftop catchment areas from Table 1 can be associated with representative basic unit cost in the Paraíba State. Table 3 exhibits typically constructed areas of standard residential developments in the city and it shows the basic unit cost per square meter, discounting taxes, of these developments in June 2020 (these reference values are updated monthly by the Union of the Construction Industry of João Pessoa, SINDUSCON/JP (2020). Annexe D shows the details of all standard designs, as found in (ABNT 2006). Hence the cost of the RWH system (water tank plus injection well) can be compared to the cost of the development (estimated using values from Table 3 according to the rooftop catchment area).

Table 3 – Typical areas (m²) and basic unit costs of standard designs at João Pessoa in June 2020. Unencumbered prices.

Code	Description	Typical area (m²)	Construction cost (R$/m²)
R1-N	Normal standard single-family residence	106.44	1,260.77
R1-A	High standard single-family residence	224.82	1,543.62
PIS	Projects of social interest	991.45	694.35
PP-B	Low standard popular building	1,415.07	956.35
PP-N	Normal standard popular building	2,590.35	1,164.93
R8-B	R8 low standard multi-family residence	2,801.64	906.93
R8-N	R8 normal standard multi-family residence	5,998.73	1,021.35

Source: ABNT (2006) and SINDUSCON/JP (2020)

5.6. Experimental investigations

5.6.1. Infiltration test

The infiltration test was executed following Bouwer (2002), using double-ring diameter infiltrometers (60-cm outer diameter), to provide knowledge on the saturated infiltration rate in the soil surface. The infiltrometers are pieces of metal in a cylinder shape, driven straight down about 5 cm into the ground with a hammer. The test consists of systematically filling both rings with water and monitoring the time spent to provide a specific level drawdown, that is, of 30 millimetres, measured with a ruler. A flat rock was placed on the soil within the outer cylinder for erosion prevention when adding the water. After a 60 millimetres drawdown, the cylinder was refilled to the top and the clock was reset. Bouwer (2002) recommends that the test must cease after 50 cm of accumulated infiltration is observed or after six hours of each test has passed, whichever comes first. The data gathered from the tests (water level drawdown and its duration) were used to derive the saturated infiltration rate f_c (mm/h) and decay constant k (1/h), following the Horton equation:

$$f = f_c + (f_0 - f_c)e^{-\beta t} \tag{10}$$

$$F = f_c t + \frac{f_0 - f_c}{k}(1 - e^{-kt}) \tag{11}$$

where f_0 is the initial observed infiltration rate (mm/h), β is a local parameter and t stands for the instant of time of the test (h) and F is the accumulated infiltration (mm) until the instant of time t. Observed infiltration rates for each 30-millimetre drawdown were calculated and each value has discounted the value of f_c, initially estimated as null, to adjust into the following Equation 12 (an exponential trendline):

$$y' = a' e^{-\beta t} \tag{12}$$

where y' is $f - f_c$ and a' is $f - f_0$. The value of f_c that best fits the observed data into the exponential trendline was determined by trial and error and then f_0 was calculated based on the parameter a'. Then, the values of Equation 11 were calculated at every 3 minutes and the area below the curve was compared to Equation 12, using a k initially null – the value of k that best matched the observed and modelled F was also obtained by trial and error.

5.6.2. Pumping test

The pumping test consists of pumping water from a well at a constant rate enough to produce a drawdown on the well being tested and on the wells in the vicinity. Pumping tests were executed in the injection wells P01 (24-May-2019) and P02 (23-May-2019) by the drilling company. Aquifer response was assessed through water level monitoring in the well being tested (using a manual level meter, readings done by a worker from the company; Figure 8a) and the other three wells in the surroundings, using pressure sensors. Readings using the manual level meter were carried out at specific time steps, starting with at every minute on the first five minutes, at every two minutes until reaching ten minutes of the test and finishing with hourly time steps. On the other hand, the pressure sensors used were programmed to record data every 30 seconds. Both tests lasted 12 hours. A water pump with 5 HP of power was used. The pumping rate was intended to be the maximum possible, which is strongly restrained by the capacity of the used filter (2 to 3 m^3 per meter length) and by the aquifer transmissivity. A gate valve was used to enable a manual adjustment of the water flow. To ensure the water could flow without a significant head loss, a metal structure was used to support the hose and keep it straight (Figure 8b). Besides, water flow was measured roughly every hour by using a

Woltmann hydrometer (Figure 8c) and eventually the values found were checked through the volumetric method, both measures were taken to ensure the pumping rate applied was kept constant during the tests. The objectives of the pumping test were to determine the unconfined aquifer parameters in the study area (its hydraulic conductivity and specific yield) and to monitor the aquifer behaviour due to the pumping rate applied recording the length of time taken by the aquifer to reestablish equilibrium in the water level after being submitted to this different condition and after returning to its normal condition.

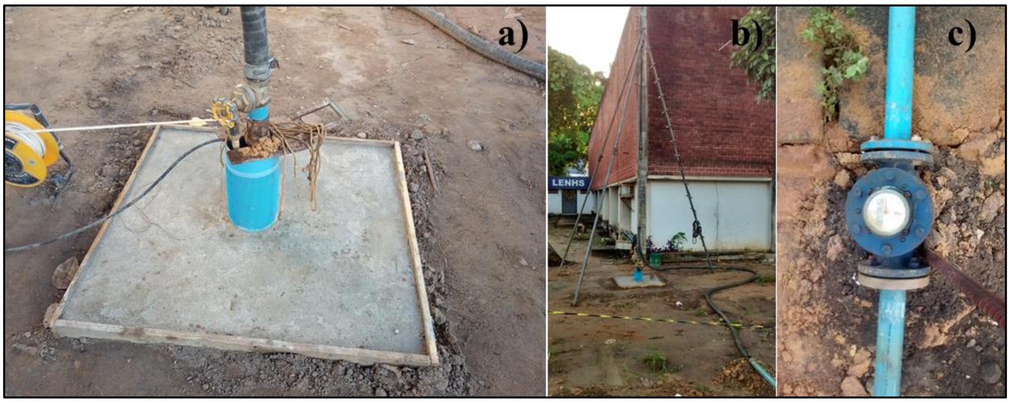

Figure 10 – Pumping test on well P02: a) level meter inside a small tube for level measurement; b) metallic structure to support the hose and prevent the hose from closing and c) Woltmann hydrometer

The pumping test results usually enable the calculation of defining aquifer parameters: its transmissivity T (m²/h) and its specific yield S_y, as shown in Equation 13 and 14 (Cooper-Jacob method for unconfined aquifer; Cooper and Jacob 1946):

$$T = \frac{\Delta t Q}{4\pi \Delta S} \tag{13}$$

$$S_y = \frac{4\mathrm{e}^{-0.5772} T t_0}{r^2} \tag{14}$$

where Q is the pumping rate (m³/h); r is the radius of the well (m), and the ratio $\Delta S/\Delta t$ is known by computing the angular coefficient α from the best fit linear model describing the relationship of the logarithm of time t (min) and corresponding water displacement s (m) in the well, as seen in Equation 15 (where displacement values are computed as the difference of dynamic and static water levels):

$$s = \alpha \ln(\mathrm{t}) + \beta \tag{15}$$

where β is the linear coefficient of the linear model and t_0 is the time where the linear model intersects the t axis (s equal to zero), given by Equation 16:

$$t_0 = e^{-\frac{\beta}{\alpha}} \tag{16}$$

The hydraulic conductivity K (m/dia) in the vicinity of the filter from the well can be computed following Equation 17:

$$K = {}^{T}/_{b} \tag{17}$$

where b is the thickness of the unconfined aquifer (around 45 m in the study area).

5.6.3. Injection test

Furthermore, several injection tests were carried out in the injection well P02 on 9-October-2019. These tests consist of monitoring the water level in the well while a certain volume of water is injected into the aquifer. The idea is to inject as much water as needed until the water level reaches the top of the well casing. After that, the water level drawdown is monitored until the original static water level is reached. Figure 9 displays a diagram representing four stages of a single injection test. When the water level reaches close to equilibrium, the next test can start, by reintroducing volumes of water just as explained. Pressure sensors were installed strategically at 9 and 14 meters depth (since the available equipment does not register pressure higher than 10 meters and should not be submitted to overload pressure of 15 meters or more) and water level drawdown below 14 meters was recorded from level meter manual measurements. A barometer was also installed to monitor the atmospheric pressure, to enable further compensation. A 15 m³ capacity water truck was used to recharge the aquifer during the tests (Figure 10), with water flowing pressurized into the well through a 2-inch hose that had been dipped below the groundwater level to avoid air entrapment (Liu et al. 2016).

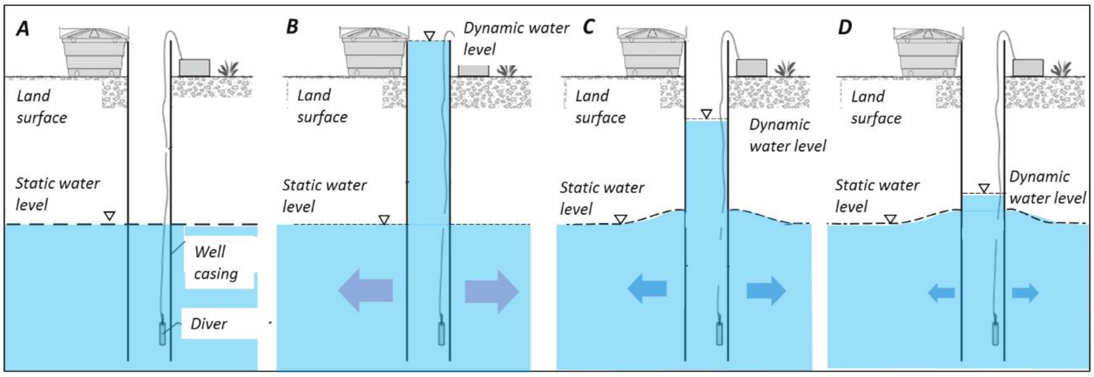

Figure 11 - Well diagram representing four stages of the injection test: a) before the test; b)

right at the beginning of the test; c) during the test and d) moments before it finishes (adapted from Conrad 2019)

Figure 12 – Water truck pumping water into the injection well P02 through a 2-inch hose.

Many phenomena and parameters are being aggregated in an injection test, since the relationship between hydraulic load and recharge rate is complex, being influenced by factors such as hydrogeologic parameters (aquifer transmissivity, specific yield, and water table depth), well structure dimensions (cross-section diameter, screen length, and material, inner roughness of tube, which may raise friction losses to critical values; Händel et al. 2016), seasonal effects (infiltration rate reductions in winter, due to higher viscosity of water and slower drying, and on summer, where the well is prone to more intense biological activity; Bouwer 2002) not to mention quality issues related to physical, chemical and biological clogging which may develop in the long-term run, depending on the quality of source water and if no control measure such as membrane filtration or periodic backwashing is prescribed (Bouwer 2002). Yet successive injection tests were performed to check if a mathematically modellable relationship between the recharge rate and water level could be built to enable the construction of the well control volume auxiliary curve since the sensibility of such a curve, on a large-term basis, regarded only by the well hydraulic behaviour during recharge of high-quality water (where quality issues can be neglected) is not exactly known. The recharge rate (or outflow rate from the well) can be calculated using Equation 18:

$$Q_w = \frac{A_w}{1{,}000} \frac{(h_t - h_{t+\Delta t})}{\Delta t} \tag{18}$$

where Q_w is the recharge rate (L/min) at the time step t (min); h_t and $h_{t+\Delta t}$ are the dynamic water levels (m) in the well at the time step t and on its following $t+\Delta t$, being Δt equal to 1 minute; A_w is the well inner cross-section (m²).

6 RESULTS

This section is divided into three parts. The experimental results that backed the hydraulic simulations are disclaimed first, then it comes to the rainfall-runoff process. Finally, the main results of the hydraulic simulations are explored.

6.1. Field investigation results

6.1.1. Infiltration test results

The double-ring infiltrometer test was carried out in the surroundings of the site. A ruler was used to observe the water level at three specific heights: 14, 11 and 8 cm, that spans three centimetres. The test initiated when the water level was at 14 cm and the time spent on each 3-centimetre water level drawdown was recorded. Then, when the water level reached 8 cm, more water was added into both infiltrometers, as fast as possible, until the water level was re-established to 14 cm, to restart the process. After an accumulation of roughly 150 cm of infiltrated water, about three times more than recommended by Bouwer (2002), the infiltration test ceased. The whole process lasted for about one hour. Table 4 shows the data gathered at the field, that is, the duration of each 3-centimetre water level drawdown.

Table 4 – Infiltration test field notes

Time (mm:ss)	Water level (cm)	Time (h)
---	14	---
03:48	11	0.06
04:48	8	0.08
---	14	---
04:52	11	0.08
05:43	8	0.10
---	14	---
05:30	11	0.09
06:45	8	0.11
---	14	---

06:08	11	0.10
07:46	8	0.13
---	14	---
06:50	11	0.11
07:50	8	0.13

As explained in the methodology, data in Table 4 was used to estimate *f0*, *fc* and k. The value of f_c that best approximated the observed infiltration curve to the Horton modelled curve is 27.35 mm/h. It can be seen in Figure 11 that *a'*, in this case, is 391.86 and β is 4.83. Hence f_0 is 419.22 and the value of *k* that best approximated the modelled accumulated infiltration to the observed one is 4.798 1/h. The minimum infiltration rate observed is of about 30.48 mm/h. These values found *in situ* for the infiltration curve were used to feed the software to model the pre-development rainfall-runoff scenario, as described in section 4.2.

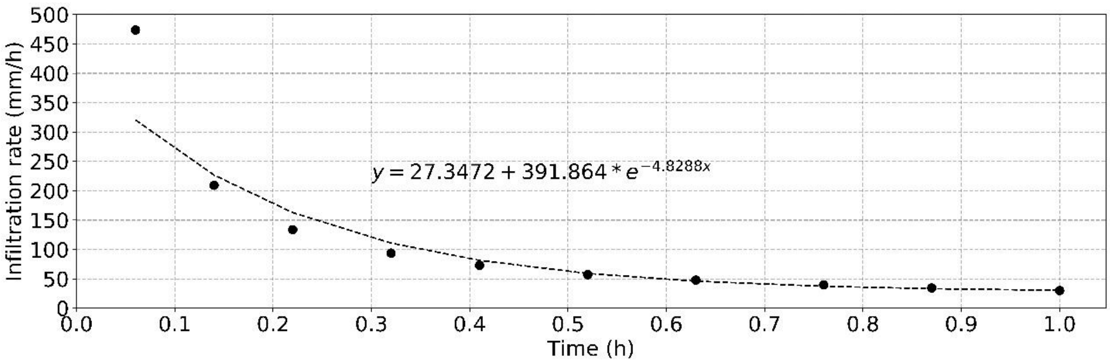

Figure 13 – Observed infiltration curve and curve built based on the Horton method

6.1.2. Pumping tests results

Results from the pumping test at the injection well P02 show a 7-meter drop in its water level while a moderately constant 10.87 m^3/h pumping rate was applied, resulting in a removal of roughly 133 m^3 of water in 12 hours. The drop measured in the water level in the three wells in the vicinity was 30 cm on average (Figure 12b). Results from the pumping test at the injection well P01 show a 9-meter drop in its water level by a 6.76 m^3/h pumping rate and the drop in the other water levels were of 17 cm on average (Figure 12a). Since both wells are screened in the interval 28-40 meters depth, water was flowing from the aquifer through 3 meters length of filter on pumping test at P01, and through 5 meters length of filter on pumping test at P02. Therefore, the observed capacity of the filter was under what was expected from the manufacturer (2.35 and 2.32 m^3/h per meter length from tests on wells P01 and P02, respectively).

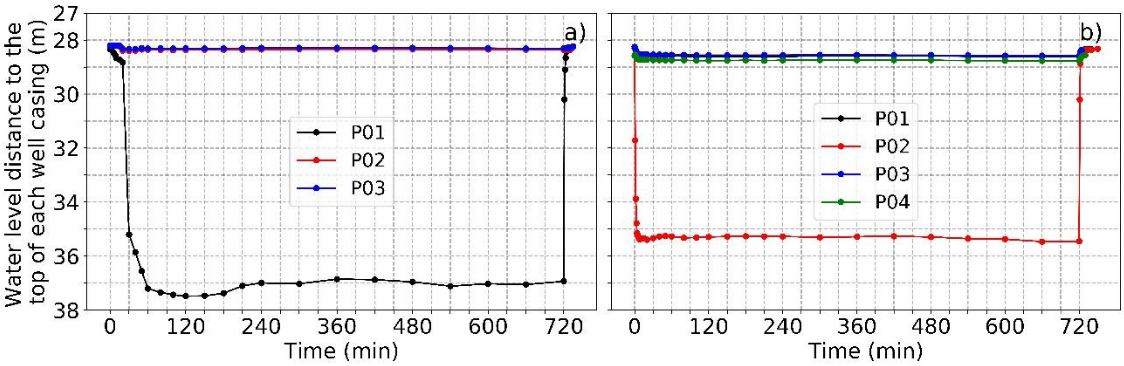

Figure 14 – Water levels monitoring during pumping test executed on well a) P01 and b) P02. Only vicinity wells recorded data gathered at the same time as data from the pumped wells are being exhibited

It lasted less than ten minutes for the water level to reach equilibrium when the pumping test on well P02 started. The same behaviour was not recorded on the pumping test on well P01, however. This happened due to failure to establish a constant flow during the first hour of the test (equipment had to spend the night on-site from one test to another and the drilling company worker argued probably someone had changed the setting of the gate valve which he had previously adjusted). Besides, equilibrium was reached in less than ten minutes after both tests ended (Figure 12). An estimation of the aquifer hydraulic conductivity and specific yield was possible to draw from data obtained in the well P02: 3.52×10^{-6} m/s and 0.057 (using the Cooper and Jacob (1946) method for unconfined aquifer), respectively. These reference values were the most reliable possibility to obtain from pumping test results, drawn from drawdown measured on the pumped well only, since effects in the vicinity were so small that were unable to be used for parameter calculation. Better estimations would have been found, if a higher pumping rate were applied, or if the tests have lasted longer, or if wells in the vicinity were closer, e.g., at least at a 5-meter distance.

6.1.3. Injection tests results

Figure 13a summarizes the whole experiment of several in-series injection tests carried out at well P02. Initially, the pressure given by the truck managed to raise the water level within the well, since the pumping rate applied was higher than the aquifer recharge rate. After less than ten minutes, the water level reached the top of the well casing and therefore the water truck pump was turned off and the water level drawdown due to gravitational force only was monitored until equilibrium was re-established. Later on, water from the truck was pumped again until the water level was raised to the top of the well, enabling the second monitoring of water level drawdown. After the second injection test, the water truck was used to raise the water level a third time and then the operator was instructed to keep the pump at a given pressure that would maintain the pumping rate close to the recharge rate, thus retaining the water level roughly constant and very close to the top of the casing. This measure was kept until emptying the water truck. The experiment provided three distinct curves from three successive injection tests, being the first one with an equilibrium initial condition and the last after 15 m³ were introduced into the aquifer. Despite this fact, no substantial difference between curves was observed, leading to the conclusion that these resulting curves are representative of the well recharge rate in correlation to its water level position, being these curves not dependent upon the total amount of injected water. The third curve, shown in Figure 13b, was used to build the auxiliary curve demanded by the modified Puls method (Chow 1959; Ghasemzadeh et al. 2020).

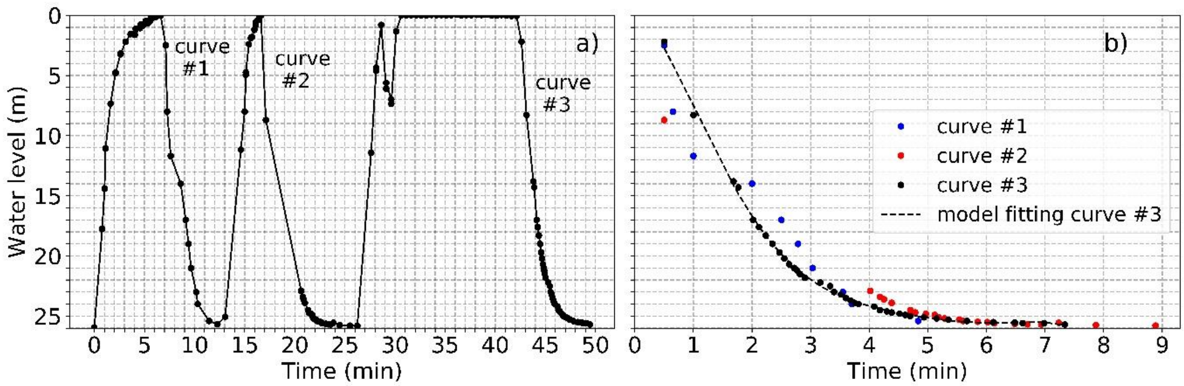

Figure 15 – Water level monitoring during injection tests at well P02: a) Whole data series, showing both water level raising, due to water truck pumping, and lowering by gravitational force only; and b) all water level drawdown with time

The third recharge test curve provided a relation of the water level h and the time t, which is strongly correlated to a sixth-degree polynomial regression (r^2 equal to 0.998), shown in Equation (19):

$$h = 0.0042t^6 - 0.1072t^5 + 1.0672t^4 - 5.030t^3 + 9.926t^2 + 1.694t - 0.071 \quad (19)$$

Then, values of h were computed using this polynomial regression for a sequence of values of t from zero up to 360 minutes, incremented by 1 minute. Equation (18) was used to compute values of Q_w for each pair of values in the t sequence (going through the sequence starting in the second value; picking the current value and the next). After this procedure, a relationship between Q_w (L/min) and h (m) was established (r^2 equal to 0.9999), shown in Equation (20):

$$Q_w = -0.0003h^3 - 0.432h^2 + 18.799h - 5.2008 \quad (20)$$

The maximum value of Q_w, found when h is maximum, at the top of the well casing, equal to 25.9 meters, is equal to 186.5 L/min (0.187 m³/min).

6.2. Pre- and post-development runoff temporal series

Table 5 shows the parameters supplied to SWMM to retrieve both pre- and post-development continuous runoff concerning the GEB rainfall dataset (2004 – 2019). The pre-development scenario relied on results of infiltration modelled by the Horton infiltration method, shown in section 5.2.1. The drying time (time needed to fully dry the soil) was assumed to be of about two days.

Table 5 - Parameters considered on the SWMM rainfall-runoff process for both studied scenarios

Parameter	Pre-development scenario	Post-development scenario
Slope	1%	30%
Manning’s coefficient	0.027	0.012
Maximum infiltration rate	419.21 mm/h	–
Minimum infiltration rate	30.48 mm/h	–
Decay constant	4.8 h^{-1}	–
Drying time	2 days	–

Figure 14 shows the simulated pre- and post-development runoff series for the period comprehending from 2015-03-06 05:00 to 2015-03-06 08:00, for selected rooftop catchment areas ranging from 100 to 5,000 m². The pre-development runoff series is made mostly of nil values, for the whole rainfall dataset (2004 – 2019) and rooftop catchment areas ranging from 10 to 5,000 m². Figure 14 shows four out of the five periods where pre-development runoff was higher than zero, for a 5,000 m² catchment (which were from 2004-02-01 01:06 to 2004-02-01 02:44; at 2005-02-18 11:47; from 2011-05-19 22:52 to 2011-05-20 02:00; from 2015-03-06 05:25 to 2015-03-06 05:56; and from 2016-04-16 13:22 to 2016-04-16 15:21). It can be seen that when the rainfall depth in one minute was higher than 10 mm (which was recorded once in the dataset), the pre-development flow was observed (Figure 14b). The other pre-development flow appearances were after a high amount of water was accumulated in a short time (Figure 14a to Figure 14d). Hence, the efficiency of the RWM system will be compared to a scenario of no flow for almost all years and most catchment areas.

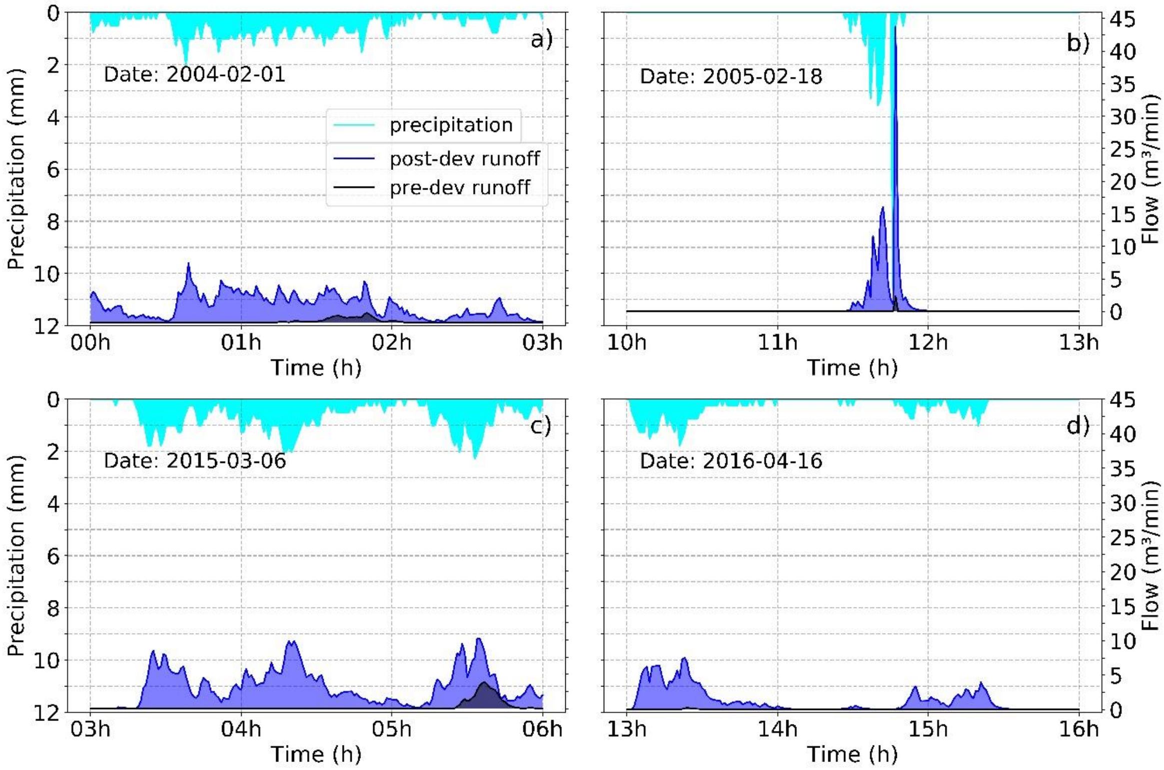

Figure 16 – Pre- and post-development runoff temporal series for selected periods using a 5,000 m² rooftop catchment area

6.3. Hydraulic simulations results

The auxiliary curves from the well and the water tanks are a preliminary step in the solution of the continuity equation, under the PULS method. The well's storage S_w (L) and discharge Q_w (L) were computed for a sequence of values of hydraulic head h from zero up to 26 meters, incremented by 1 centimetre. S_w values were calculated following Equation (7) while Q_w values were calculated from Equation (19). Hence $Q_w+2S_w/\Delta t$ was calculated for each value in the sequence, considering Δt equal to 1 minute.

Before proceeding to the hydraulic simulation of each RWH system setup, the auxiliary curve regarding the given water tank was calculated. Figure 15a shows the curves of each water tank used in this study. These curves were calculated by computing the pair of values of S_t and Q_t for a sequence of values of water level h from zero until reaching the tank capacity, using Equation (4) and Equation (6), respectively. The increment of this sequence was 1 mm. Figure 15b shows the well auxiliary curve, plotted with some curves of the smaller water tanks. It can be seen that the well auxiliary curve is similar to the curve of the 3,000 L water tank.

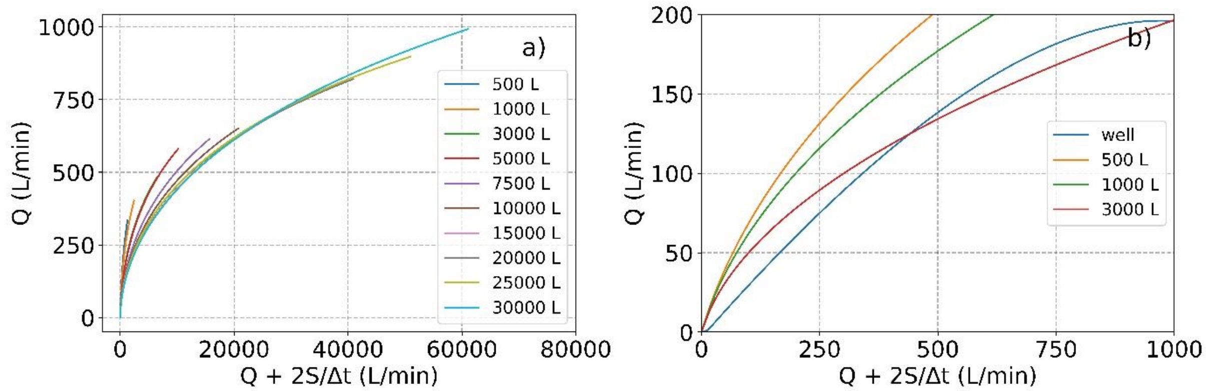

Figure 17 – PULS method auxiliary curves a) for all water tanks and b) for the well and some smaller tanks

Hydraulic simulations were executed for each yearly post-development runoff temporal series (2004 – 2019), considering the collection of catchment areas (Table 1) and water tanks (Table 2) used in this study, what led to 300 different setups of RWH systems (in terms of rooftop catchments, water tank capacities and a single injection well).

Figure 16 to Figure 18 show results from hydraulic simulations on some catchment areas (250, 1,000 and 5,000 m²) using selected water tanks (ranging from 500 to 30,000 L). For visualization purposes, only the period ranging from 2015-03-06 03:00 to 2015-03-06 10:00 has been displayed in the referred Figures, a period where an intense rainfall has been observed (the same event shown in Figure 14c).

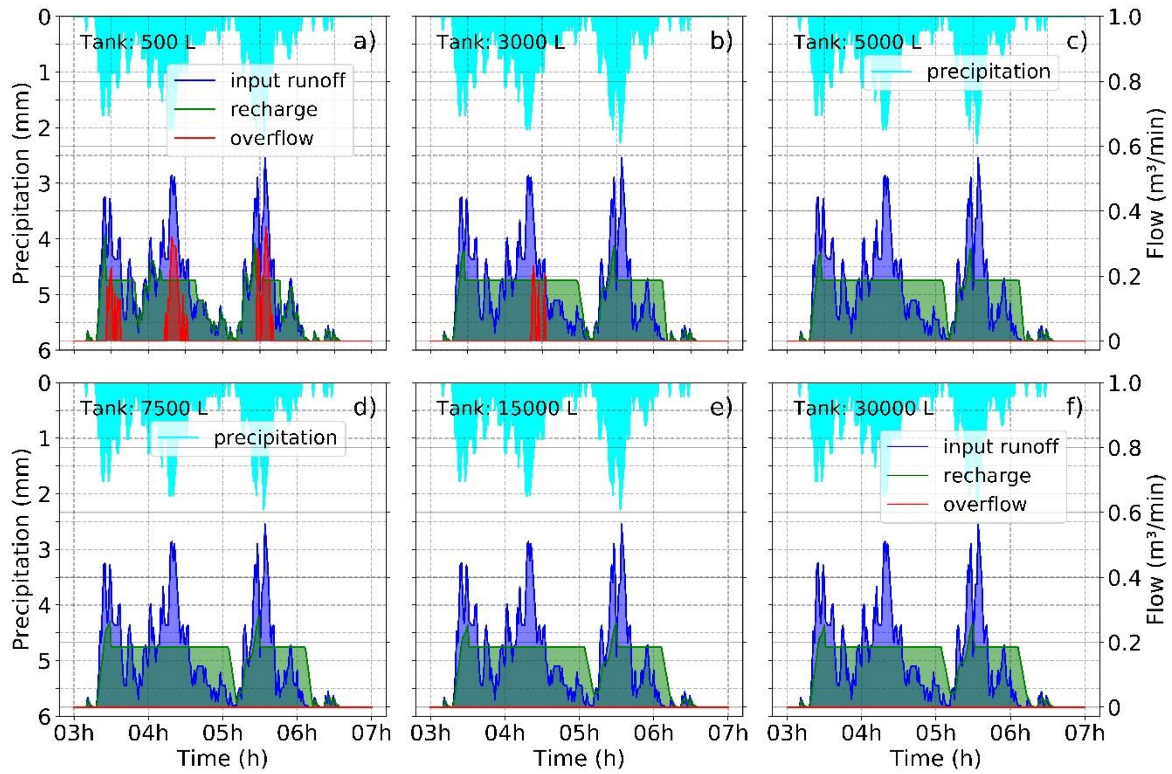

Figure 18 – Hydraulic simulations using selected water tanks (L) and a 250 m² rooftop catchment, for the period comprehending from 2015-03-06 03:00 to 2015-03-06 07:00

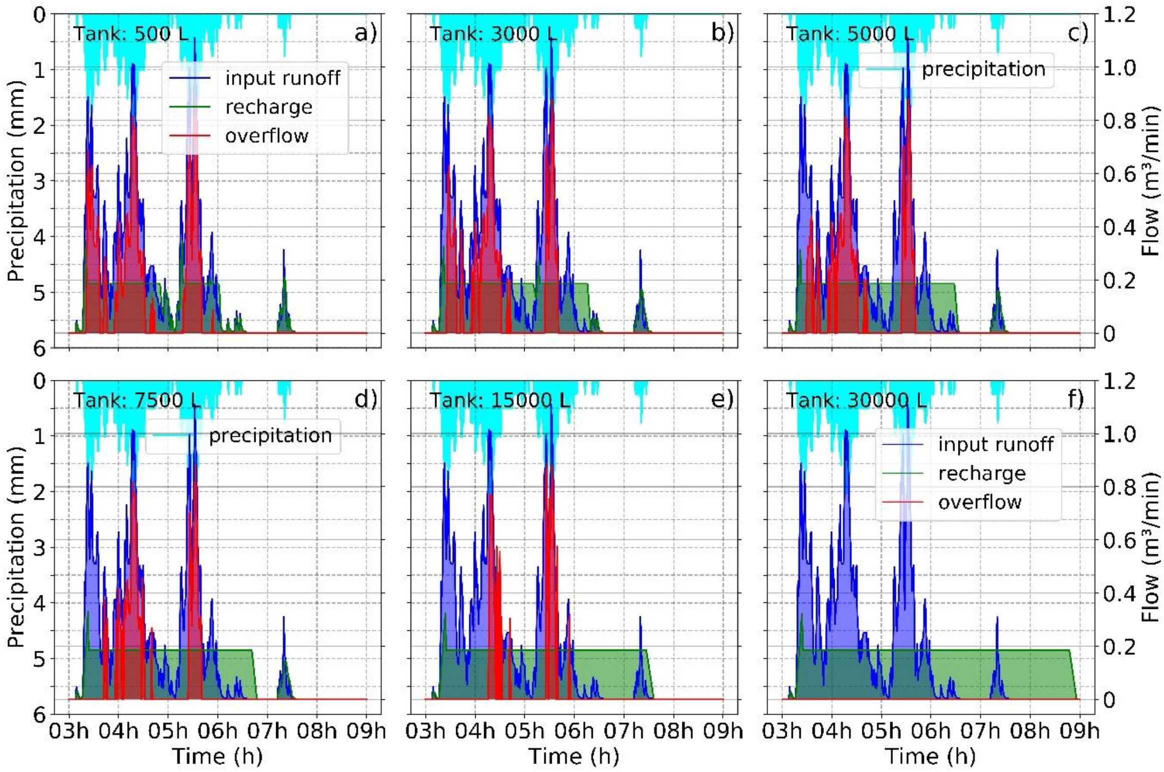

Figure 19 – Hydraulic simulations using selected water tanks (L) and a 1,000 m² rooftop catchment, for the period comprehending from 2015-03-06 03:00 to 2015-03-06 09:00

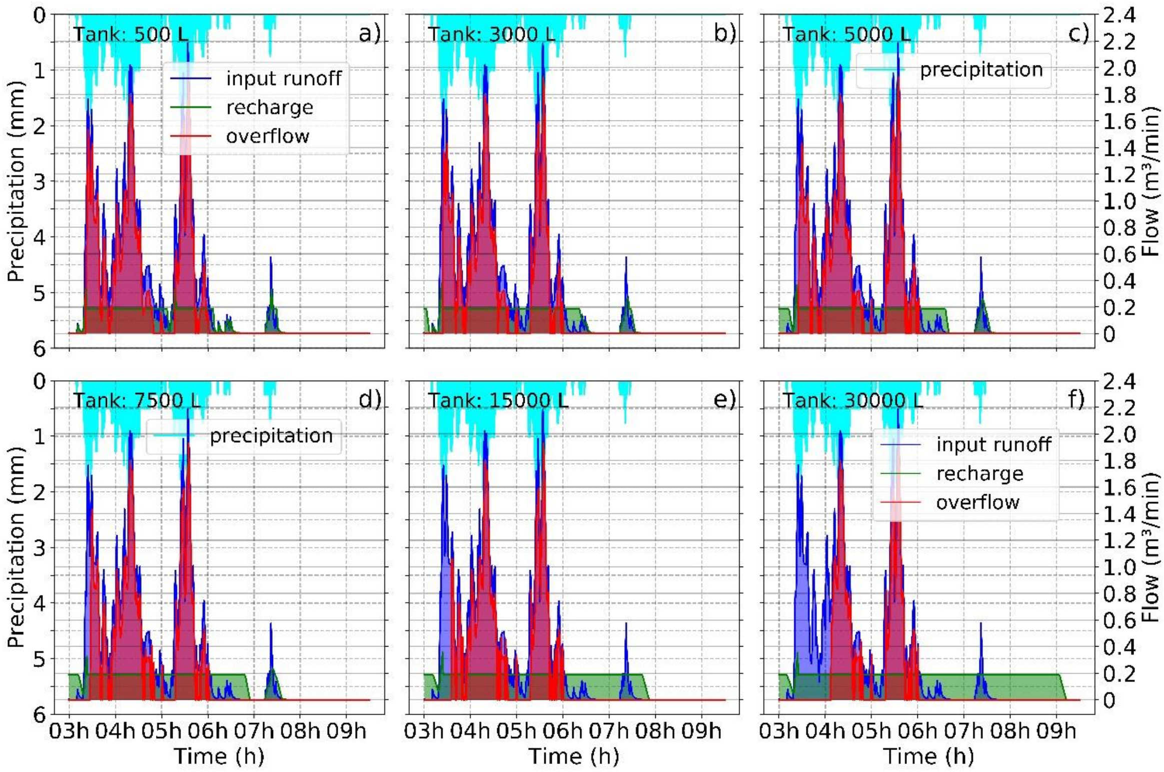

Figure 20 – Hydraulic simulations using selected water tanks (L) and a 1,000 m² rooftop catchment, for the period comprehending from 2015-03-06 03:00 to 2015-03-06 09:30

Moreover, the post-development runoff, recharge and overflow temporal series were resampled to a 5-minute time step and were aggregated to provide annual total volume estimates. For each annual aggregation, peak runoff overflow and peak pre- and post-development runoff flows were recorded, thus creating an annual database whom the following results come from.

Figure 19 to Figure 21 show the distribution of total annual volumes of aquifer recharge, tank overflow and post-development runoff (m^3) for selected water tanks (ranging from 500 L to 30,000 L), considering, respectively, a 250, 1,000 and 5,000 m^2 rooftop catchment area.

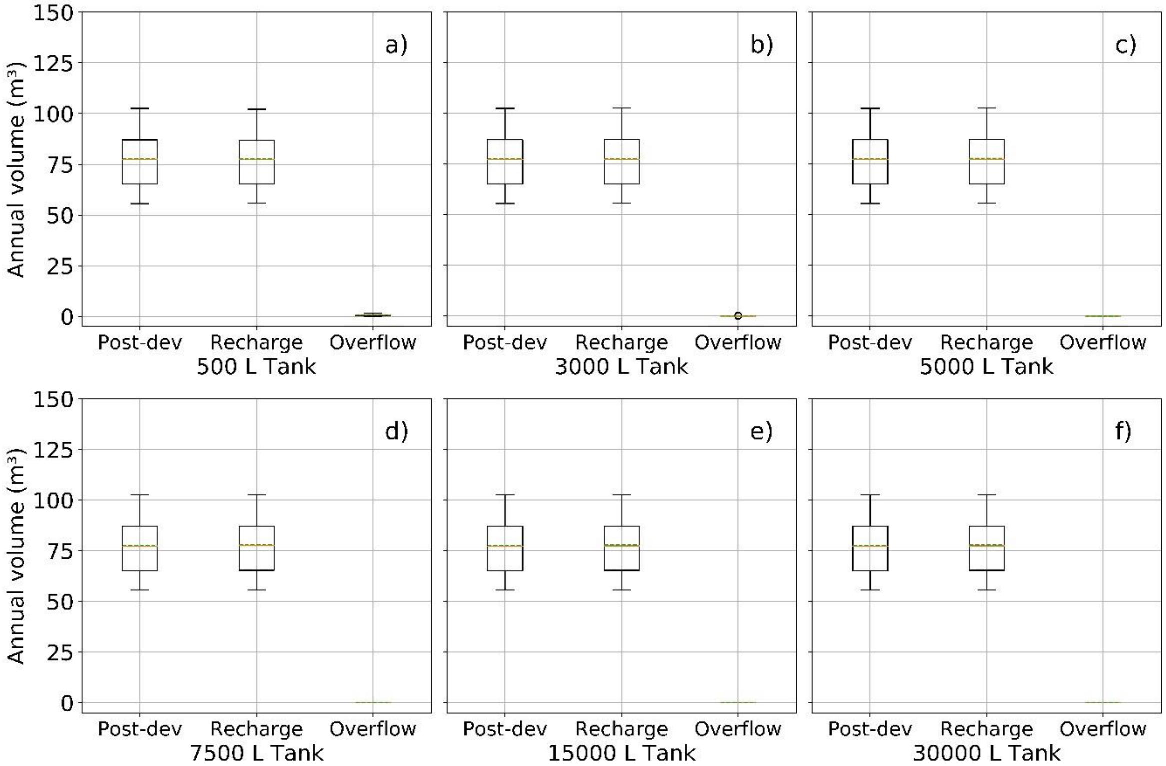

Figure 21 – Distribution of annual volumes of post-dev runoff, aquifer recharge and tank overflow (m^3) for selected water tank capacities (L) considering a 250 m^2 catchment area. Orange lines and green dashed lines stand for median and mean, respectively

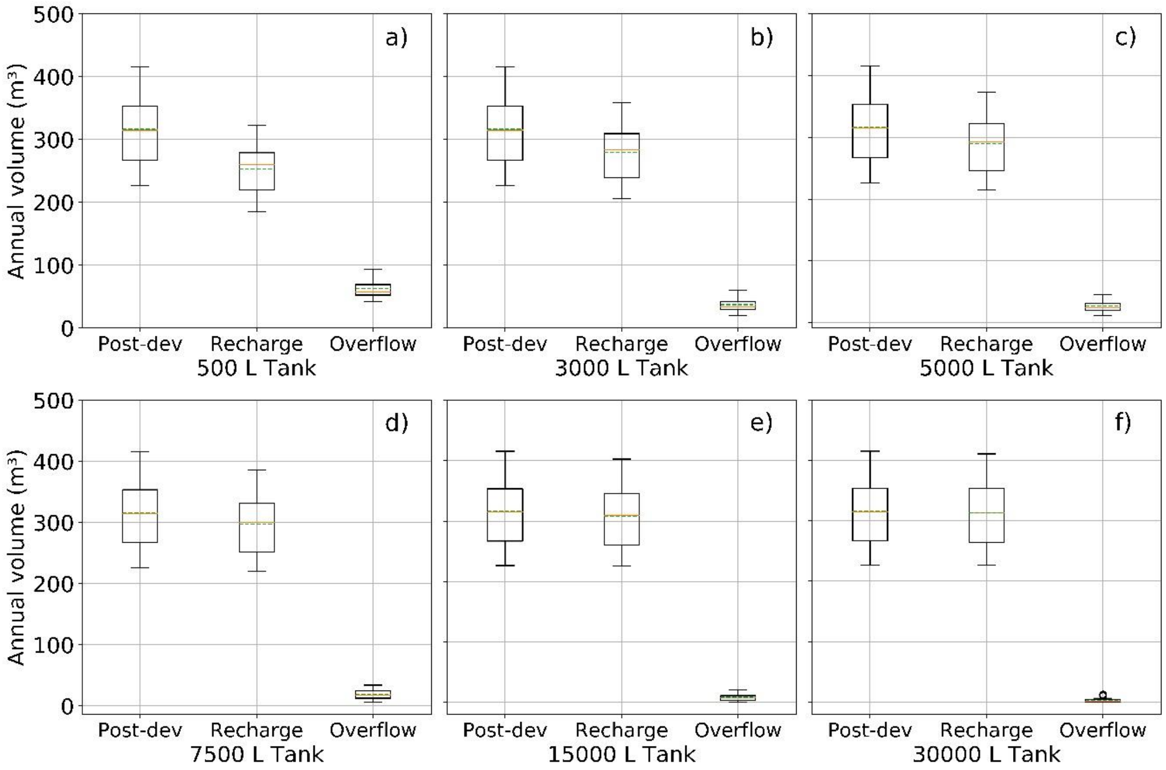

Figure 22 – Distribution of annual volumes of post-dev runoff, aquifer recharge and tank overflow (m^3) for selected water tank capacities (L) considering a 1,000 m^2 catchment area. Orange lines and green dashed lines stand for median and mean, respectively

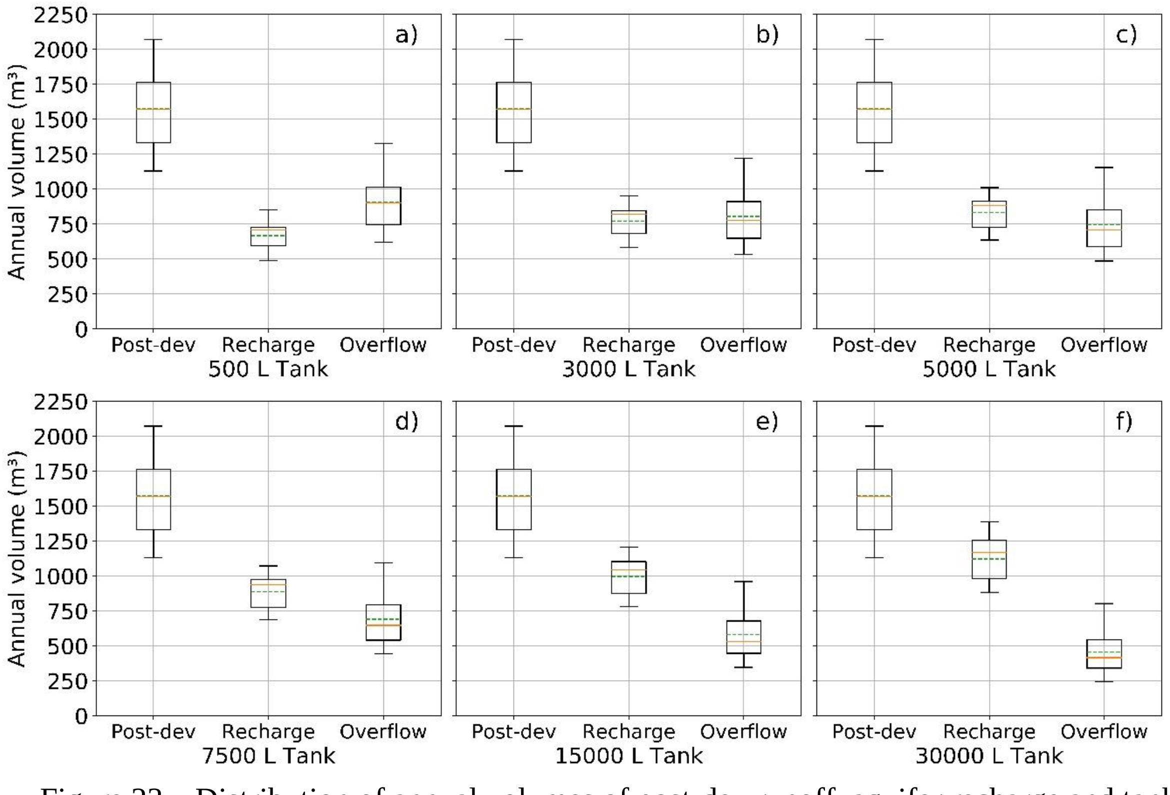

Figure 23 – Distribution of annual volumes of post-dev runoff, aquifer recharge and tank overflow (m^3) for selected water tank capacities (L) considering a 5,000 m^2 catchment area. Orange lines and green dashed lines stand for median and mean, respectively

Figure 22 exhibits the mean annual volumes of aquifer recharge, tank overflow and post-development runoff (thousands of m³/year) in the function of the catchment area (thousands of m³) for selected water tanks whose capacity ranged from 500 L to 30,000 L. The hatched areas surrounding each line represent the standard deviation of each mean value. Mean annual volumes of post-development runoff are equal to the sum of mean annual volumes of aquifer recharge and tank overflow, for any given rooftop catchment area. What changes with the catchment area is the proportion of aquifer recharge and tank overflow – the higher the area, the more volume of post-development runoff that moves from aquifer recharge towards tank overflow. The catchment area corresponding to the maximum difference between mean annual volumes of aquifer recharge and tank overflow have been pointed out for each water tank capacity.

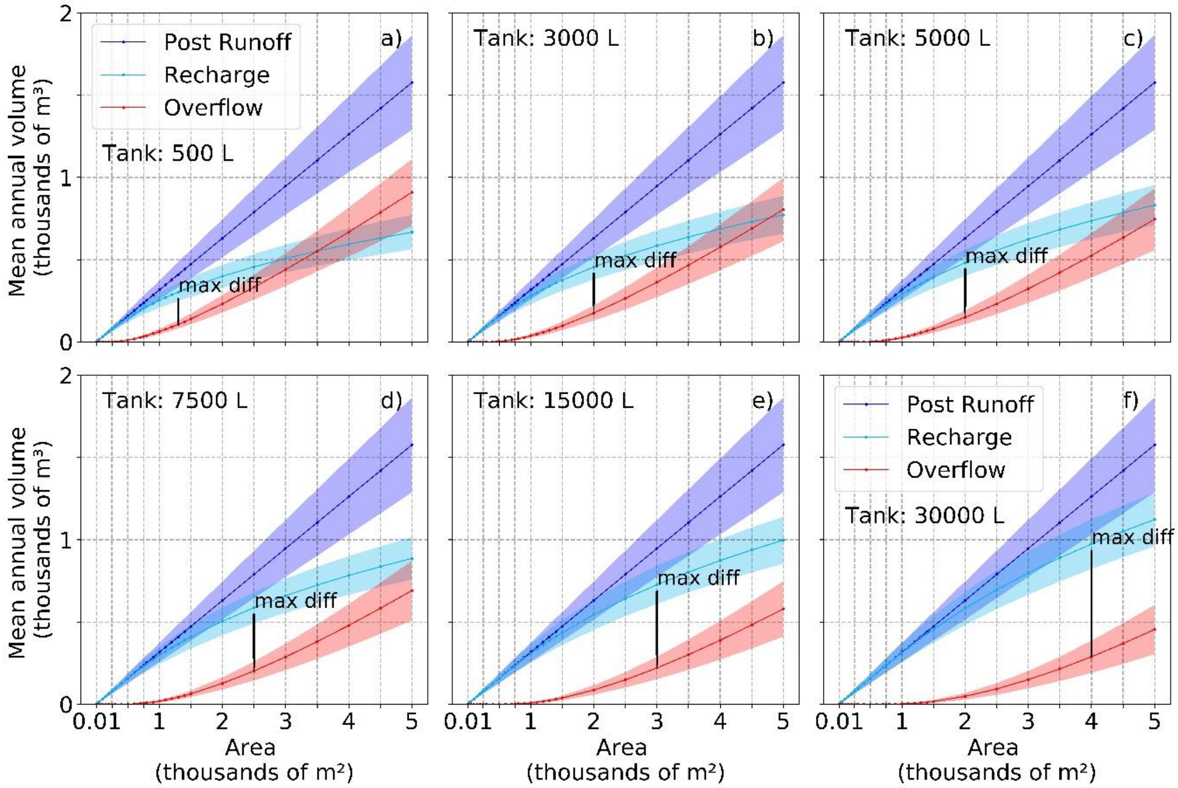

Figure 24 – Mean annual volume of recharge, overflow and post-dev runoff (thousands of m³/year) in the function of the water tank capacity (L) and rooftop catchment area (thousands of m²)

Another way to visualize the mean annual volumes shown in Figure 22 is displayed in Figure 23, using contour plots. Figure 23a shows the mean annual post-development runoff volumes in the function of catchment areas (which are not influenced by water tank capacities), while Figure 23b and Figure 23c show the mean annual recharge and overflow volumes. The difference between interpolated values from Figure 23b and Figure 23c was computed, resulting in Figure 24, in which the maximum difference between mean annual recharge and overflow values can be found for any water tank capacity. The white-coloured hatched area in Figure 23 and Figure 24 represent nil values.

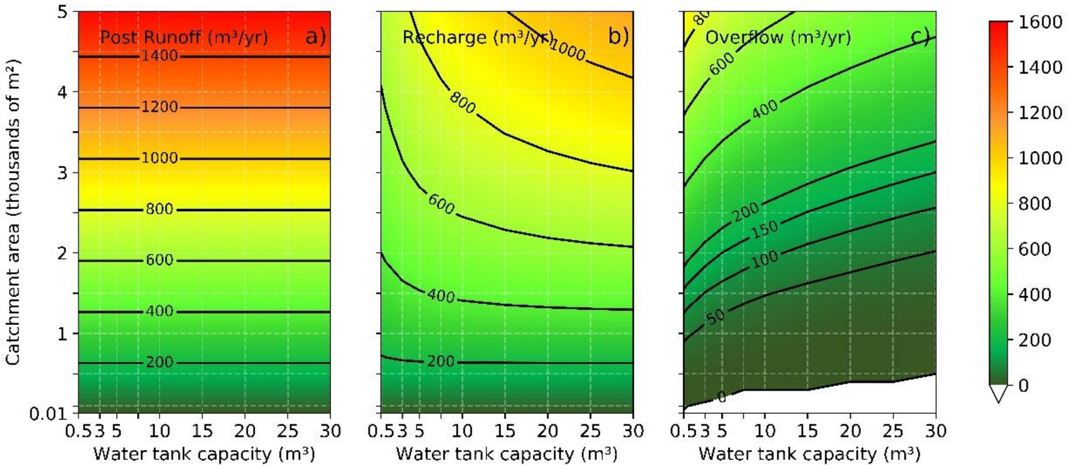

Figure 25 – Mean annual volume of a) post-dev runoff, b) recharge and c) overflow (m^3/year) in the function of the water tank capacity (m^3) and rooftop catchment area (thousands of m^2)

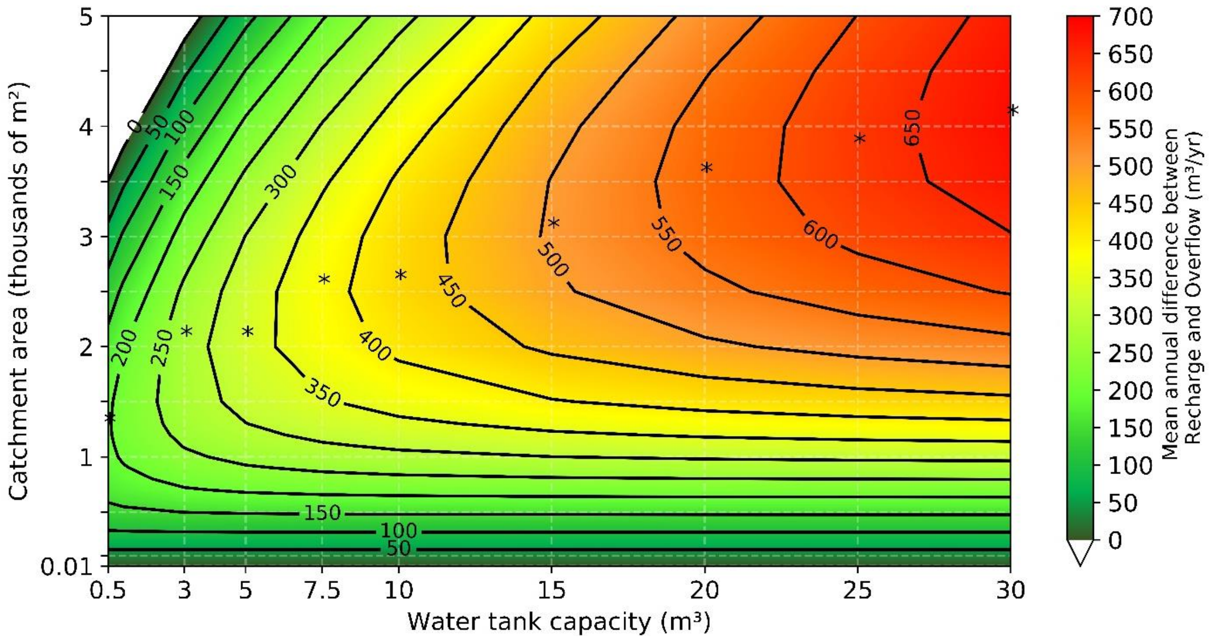

Figure 26 – Mean annual difference between recharge and overflow volumes (m^3/year) in the function of the water tank capacity (m^3) and rooftop catchment area (thousands of m^2). The * symbol stands for the point whose coordinates states water tank capacity and its corresponding catchment area that maximize the observed mean annual difference

So far, results have focused on total amounts only. On the other hand, Figure 25 to Figure 27 show the distribution of annual peak runoff flows in the pre- and post-development scenarios and of annual peak tank overflows (L/s/year) for selected water tanks (ranging from

500 L to 30,000 L), considering, respectively, a 250, 1,000 and 5,000 m² rooftop catchment area.

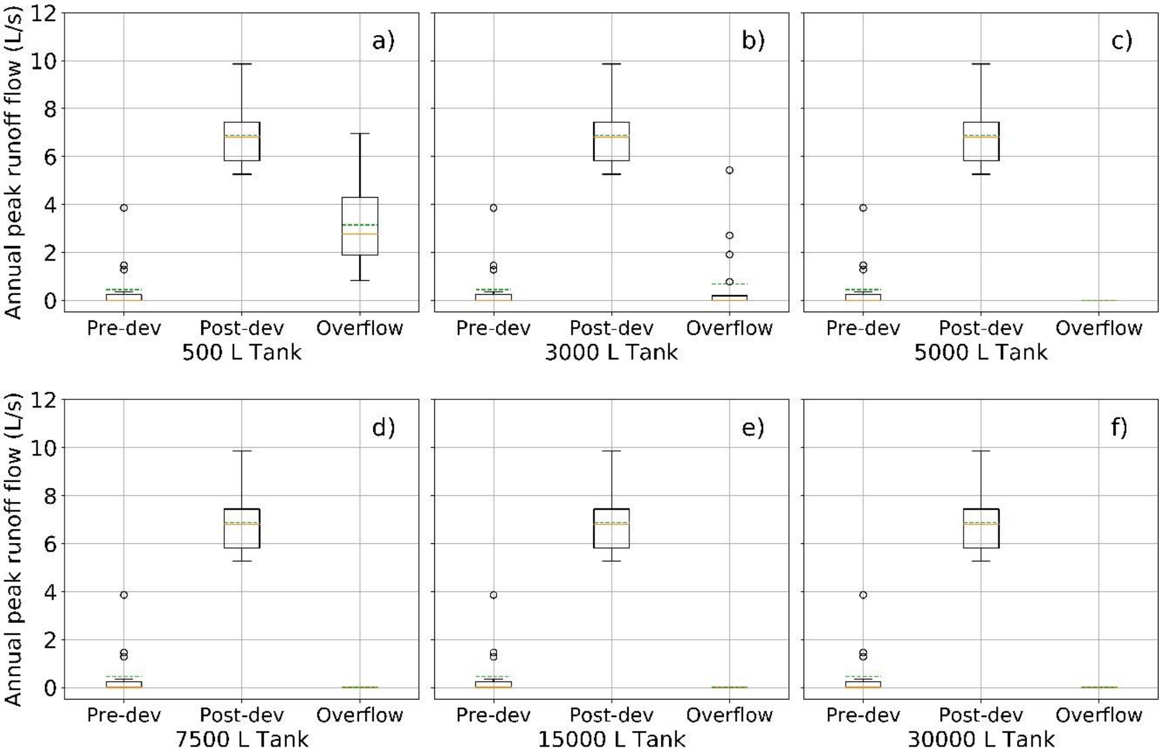

Figure 27 – Distribution of annual peak runoff flows of pre- and post-development scenarios and of annual peak overflows (L/s) for selected water tank capacities (L) considering a 250 m² catchment area. Orange lines and green dashed lines stand for median and mean, respectively

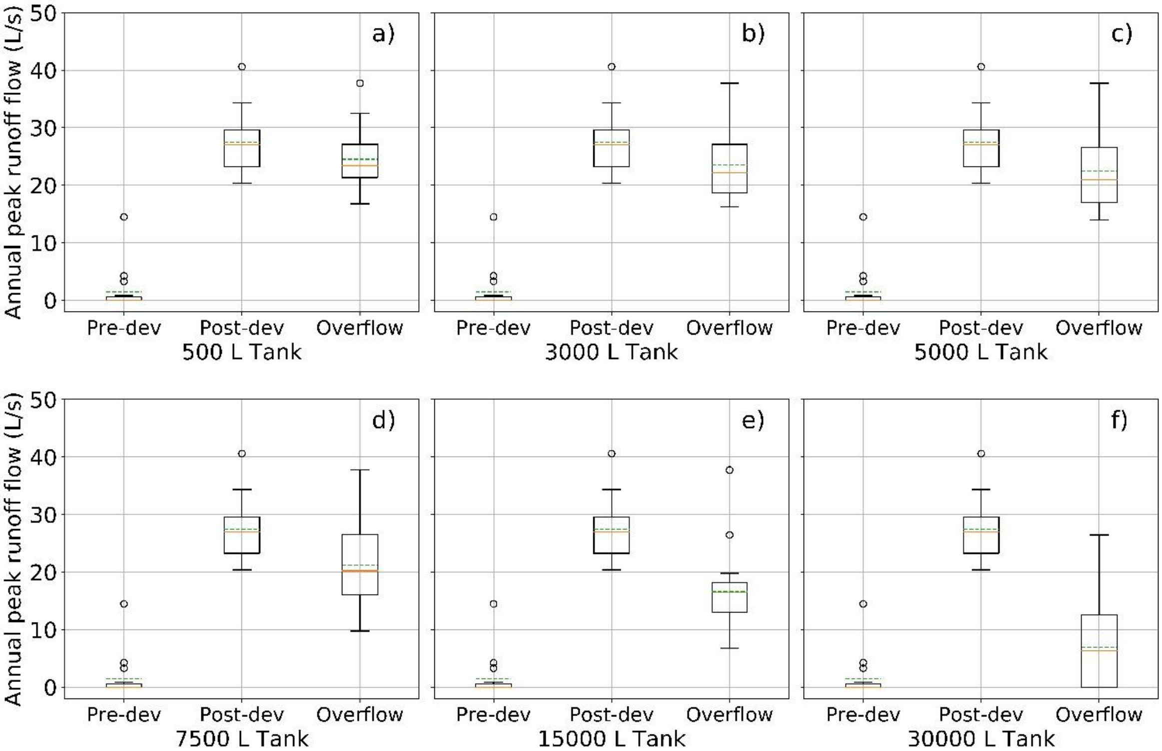

Figure 28 – Distribution of annual peak runoff flows of pre- and post-development scenarios and of annual peak overflows (L/s) for selected water tank capacities (L) considering a 1,000 m² catchment area. Orange lines and green dashed lines stand for median and mean, respectively

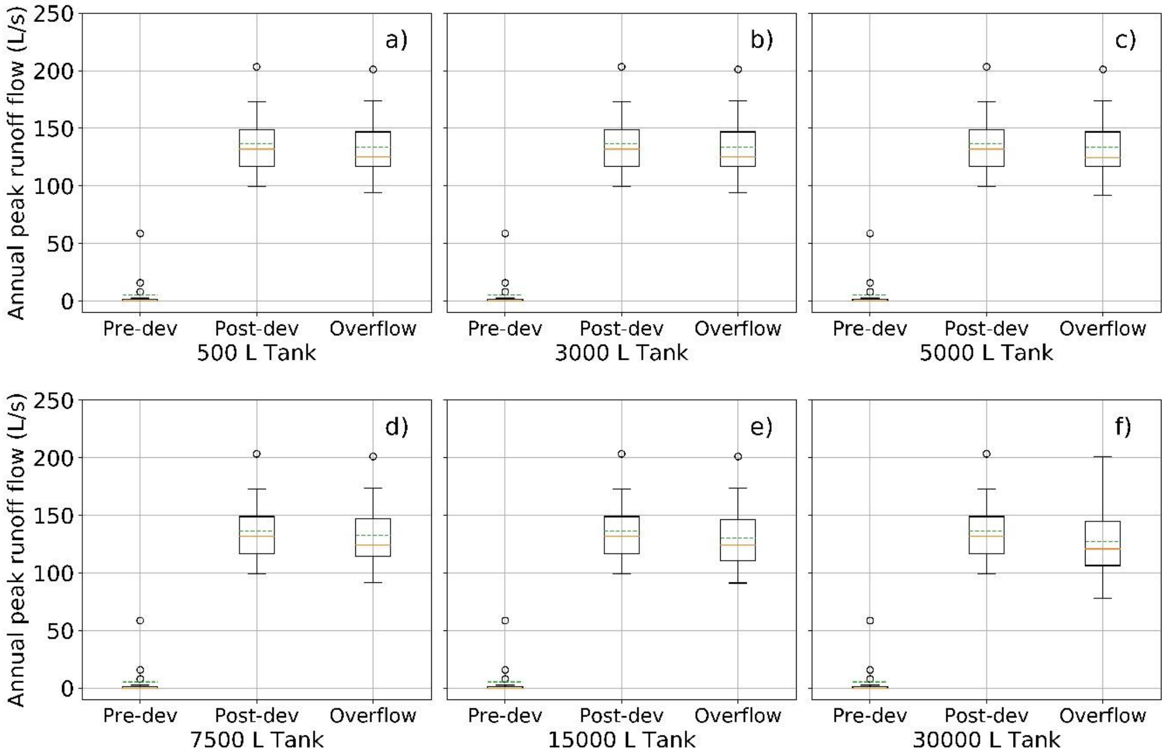

Figure 29 – Distribution of annual peak runoff flows of pre- and post-development scenarios and of annual peak overflows (L/s) for selected water tank capacities (L) considering a 5,000 m² catchment area. Orange lines and green dashed lines stand for median and mean, respectively

Figure 28 exhibits the mean annual peak runoff overflow and the mean annual peak pre- and post-development runoff flows (L/s/year) in the function of the catchment area (m²) for selected water tanks whose capacity ranged from 30,000 L to 500 L. The hatched areas surrounding each line also represent the standard deviation of each mean value.

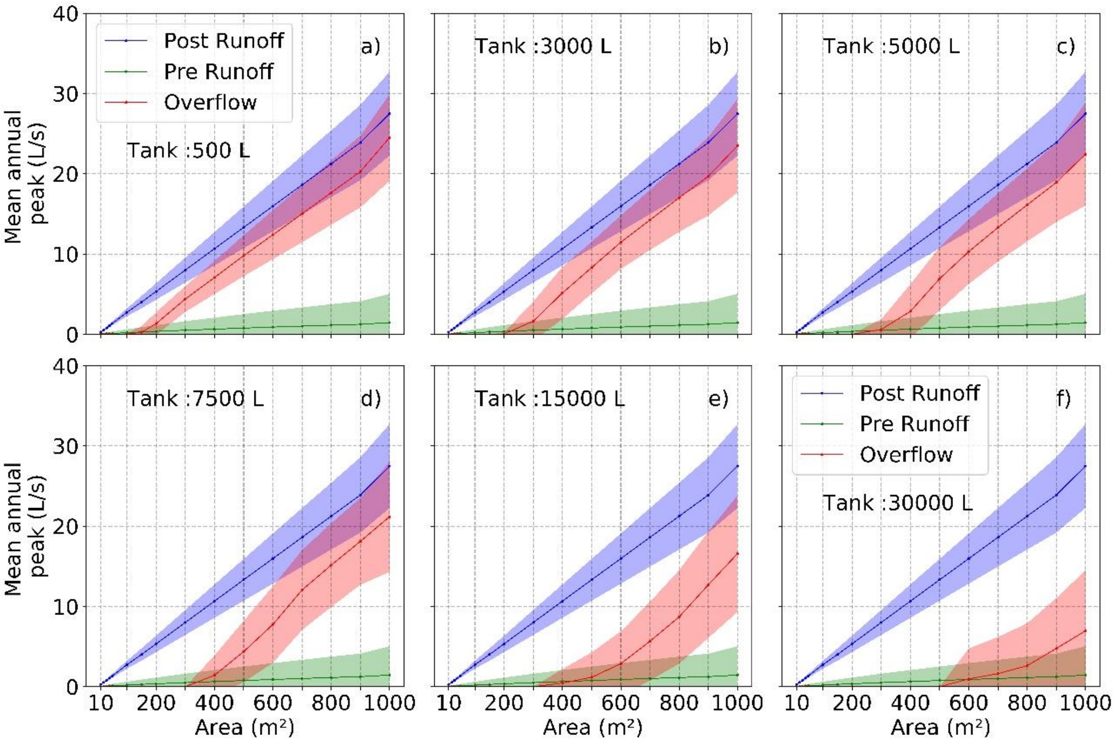

Figure 30 – Mean annual peak pre- and post-development runoff flow and peak overflow (L/s/year) in the function of the water tank capacity (L) and rooftop catchment area (m²)

Figure 29 shows the information displayed in Figure 28, whereas using contour plots. Figure 29a and Figure 29b shows the mean annual peak pre- and post-development runoff flows (L/s/year) in the function of the catchment area (m²). Figure 29c shows the mean annual peak overflow values (L/s/year) in the function of the catchment area (m²) and water tank capacity (m³).

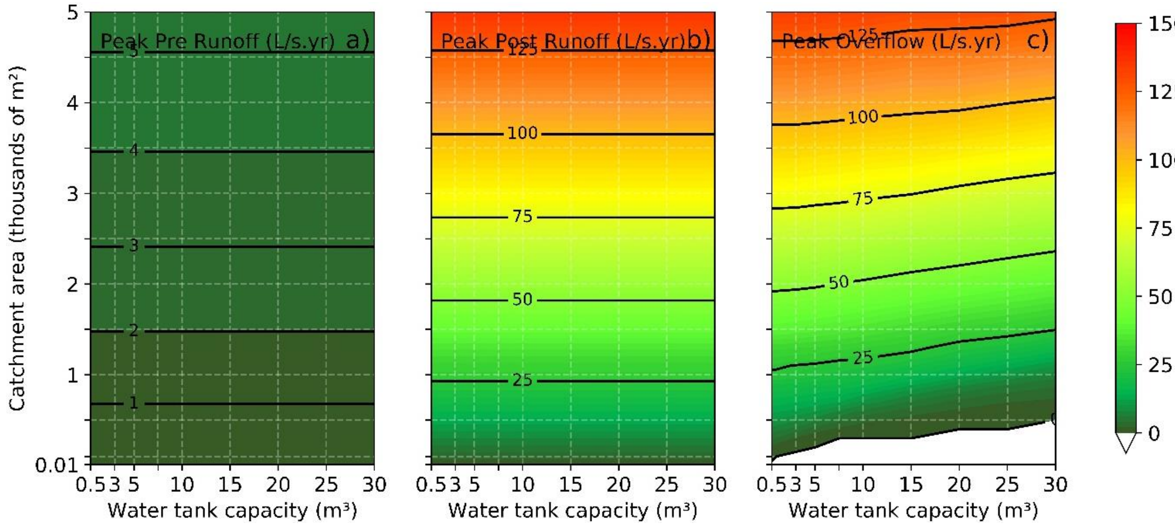

Figure 31 – Contour plots of a) rainwater retention R and b) efficiency E in the function of water tank capacity (m³) and rooftop catchment area (thousands of m²)

Mean annual values of rainwater retention R (Equation 8) and efficiency E (Equation 9) were calculated from the annual database: from mean annual total post-development input and recharge volumes and from peak mean annual post-development runoff flows and peak runoff overflows, respectively, thus resulting in the construction of Figure 30.

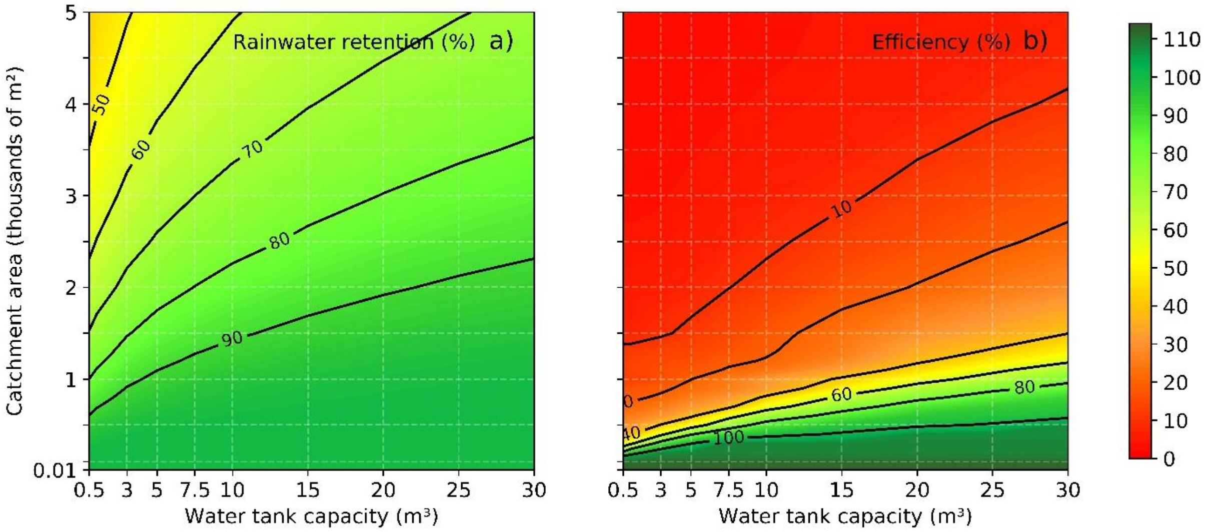

Figure 32 – Contour plots of a) rainwater retention *R* and b) efficiency *E* in the function of water tank capacity (m³) and rooftop catchment area (thousands of m²)

7 DISCUSSION

7.1. Discussion of experimental results

Pumping test results in the study area show the Barreiras unconfined aquifer re-establishes equilibrium in the water level in less than ten minutes (Figure 12). This observation is especially important since it shows that rainfall events spanning more than ten minutes apart from each other may be consistently considered independent since it is assumed that, at least when an event producing a runoff rate of roughly 10 m³/h in 12 hours, the aquifer response due to an event will not be affected by the previous one. This means that negligible residual increases in the water table are expected due to single rainwater inputs (hence the well control volume can be assumed as unchangeable in the whole yearly hydraulic simulations). The well control volume is expected to change only due to the water level natural variation (which is not being considered in this study) and not by the administered volumes of managed aquifer recharge.

Results from Conrad (2019) when performing injection tests on a 2-inch diameter injection well with an 8-meter screen length, indicates that a roughly constant mean recharge rate of 1.2 L/min was obtained after an initial volume of 100 L of water was introduced into a confined aquifer. Successive injection tests were carried out but results are not following the trend observed in Figure 13b: non-negligible differences between curves were found since the water level drawdown took more time with each test conducted, showing a strong dependency of the recharge rate on the total injected volume – reasons are probably the more intense friction loss, occurrence of air entrapment and presence of fine sandstone with clayey intercalations surrounding the screen length (Conrad 2019). The author also pointed out that the well structure (two tubes screening different depths of the confined aquifer, situated at the same borehole and interconnected via the gravel filter surrounding both screens) may also have influenced this since an accumulation of the initial injected water volume occurred from one tube to another.

In this study, it was possible to inject 15 m³ of water using a 6-inch diameter well in less than one hour. The maximum observed recharge rate (the well's outflow rate Q_w) was about 186 L/min, a value much higher than found in other recent artificial recharge experiments. Pumping test results indicate that this rate could be maintained for at least 12 hours without any substantial reduction in this value thus leading to roughly 134 m³ of recharge in 12 hours. If this rate does not change in a two-week recharge period, it is possible to recharge the aquifer with around 3,750 m³ of water in total. Händel et al. (2016) used a 1-inch diameter injection well to recharge a shallow aquifer with a roughly constant rate of 45 L/min for 14 days (almost 910 m³ in total), while Liu et al. (2016) used a 2-inch direct-push well, being able to recharge a near-surface aquifer with a rate of about 102 L/min for 15 hours (almost 92 m³ in total).

7.2. Discussion of hydraulic simulation results

Before proceeding to the hydraulic simulations using the post-development runoffs from the SWMM, a brief discussion is disclaimed regarding the hydraulic behaviour of the system using a generic input. Figure 31 illustrates the hydraulic simulations of several RWH systems (different water tanks connected to the well P02), using hypothetical hydrographs as input. Areas in blue represent the input hydrograph while areas in green and red respectively stand for the recharge and overflow hydrographs. The RWH system (water tank plus injection well) act by getting the blue input area and returning the green and red output areas. The sum of red and green areas is equal or approximately equal to its respective blue areas (the negligible non-exactness is due to error intrinsic to the modified PULS method).

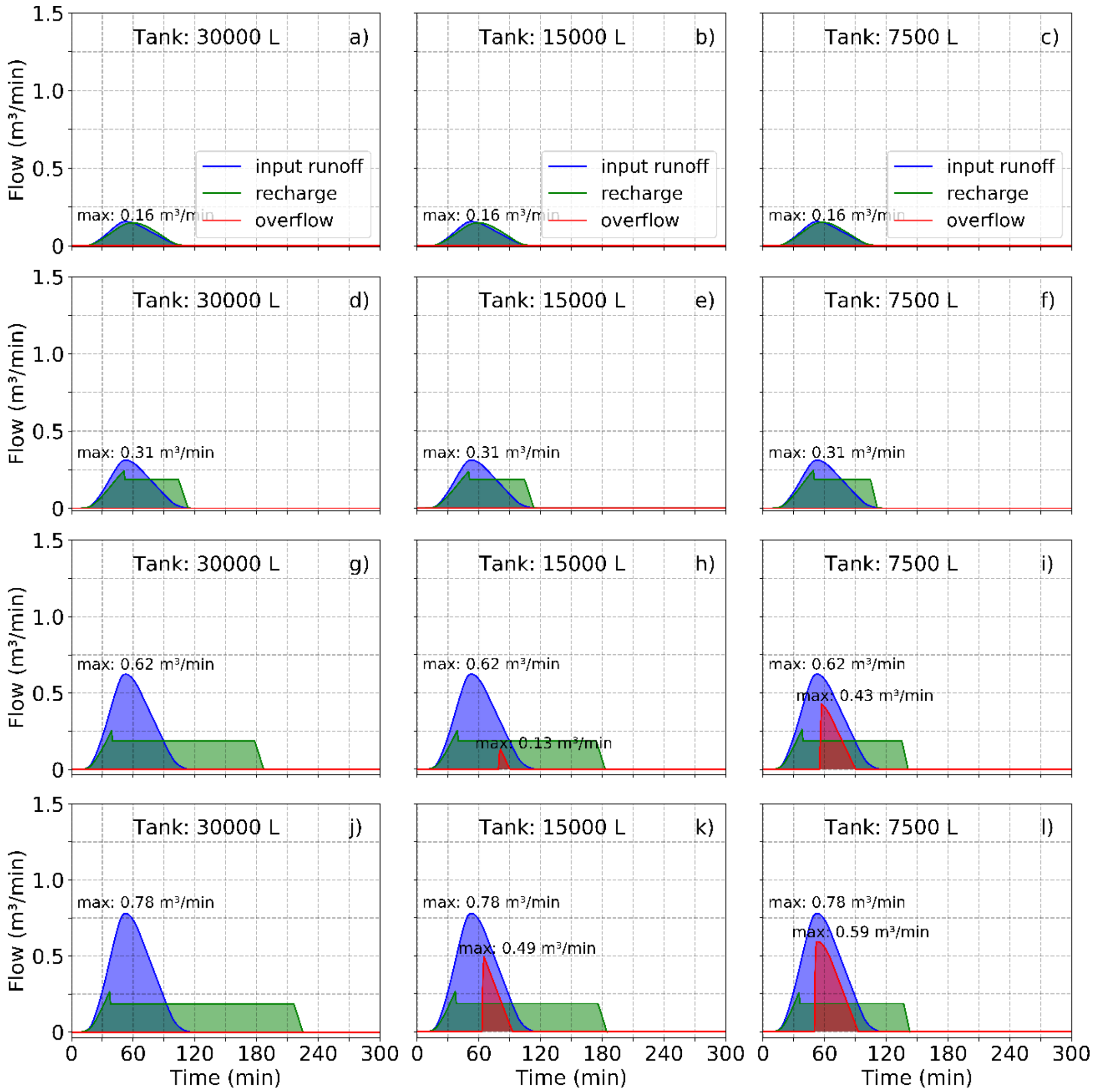

Figure 33 – Illustration of results of hydraulic simulations for different RWH systems with a-c) a hypothetical input; d-f) the input 2x intense; g-i) the input 4x intense and j-l) the input 8x intense

Figure 31a to Figure 31c show results of 30,000 L, 15,000 L and 7,500 L water tanks using a hypothetical hydrograph, while the remaining plots within Figure 31 show results from these same tanks, but with this referred input intensified (two times more intense in Figure 31d to Figure 31f; four times more intense in Figure 31g to Figure 31i; and eight times more intense in Figure 31j to Figure 31l).

When the input peak runoff flow is smaller than the maximum outflow rate Q_w then all runoff goes through the water tank directly into the well thus recharging the unconfined aquifer. In this condition, illustrated in Figure 31a to Figure 31c, the water tank is not necessary since the well can store all input water volume. In theory, input peak runoffs smaller than this threshold (smaller than the maximum Q_w) could be set indefinitely, without storing any volume in the water tank nor raising the unconfined aquifer water level. In reality, the threshold (maximum Q_w) may rise or decrease at some level depending on the natural seasonality that induces a water level variation (which is being assumed as negligible in this study).

Figure 31d to Figure 31f shows a condition where the input runoff flow is intense enough to fill the control volume of the well, but its peak is close to Q_w. The output peak runoff flow (from the water tank) is governed by the discharge tube equation (Equation 6), being slightly higher than Q_w until the well is filled. From that point, the output peak runoff flow is forced to be equal to Q_w and the water tank plays a major role in the water balance. Since its presence avoids the occurrence of runoff overflow, the volume of water that is not immediately drained into the well remains in the water tank to be released at the Q_w rate. In this condition, the capacity of the tank is not particularly important since any water tank (up from 500 L) produces similar hydraulic responses.

At more intense runoff events (as represented in Figure 31g to Figure 31l), the capacity of the water tank raises in importance, when the input runoff is high enough not only to fill the well control volume (thus the discharge at the water tank is governed by Q_w) but to substantially raise the water level within the tank. In this condition, the storage in the water tank can maintain a constant aquifer recharge for some time even after input runoff has ceased. Depending on the capacity of the tank, it may not be able to store all input runoff, thus converting the excess volume into a runoff overflow, as illustrated in Figure 31h to Figure 31l. Besides, each water tank has a maximum total recharge that it enables after the water tank is full. Until the water tank is kept full any excess water will not be used for aquifer recharge. The magnitude of peak runoff overflow will be close to the peak input runoff value when the water tank becomes full before the inflow hydrograph reaches its peak (this is generally the case for intense input hydrographs and relatively small tank capacities).

The abovementioned discussion is also true for the simulated post-development runoff temporal series derived from real measured rainfall data. Figure 16 shows part of a yearly hydraulic simulation in a 250 m² catchment area, in a period where intense precipitation was recorded. It can be observed that, for the given catchment area, the smaller water tanks (500 and 3,000 L) were not enough to fully control the overflow hydrograph. With these tanks, the well worked partially full in the rain recession periods (in these periods, Q_t governed the simulation instead of Q_w). On the other hand, the water tanks equal and higher than 5,000 L were able to maintain the well full for higher periods. The bigger tanks (15,000 L and 30,000 L) resulted in recharge patterns similar than the 5,000 L water tank, suggesting that these bigger tanks were an exaggeration in this case. This is confirmed by analysing the annual volume distribution of post-development runoff, recharge, and overflow (Figure 19), where water tanks equal or higher than 5,000 L were able to use the whole runoff series (2004 – 2019) for managed aquifer recharge since no overflow has been recorded in the simulations. In Figure 19a and Figure 19b it registered a very thin distribution of annual overflow volume, that would be negligible were it not for the fact that it produced annual peak overflow distributions whose mean were much higher than pre-development runoff's, even though a substantial reduction has been observed in comparison to peak post-development runoff flows (Figure 25a and Figure 25b).

For the 500 m^2 catchment area (Figure 17), only the 30,000 L water tank was able to fully convert the input runoff into recharge. On the other hand, for the 1,000 m^2 catchment area no tanks were able to stop the occurrence of overflow, for the period shown in Figure 18. For the whole period, the water tanks were able to keep the recharge annual volume distribution close to the post-development annual volume and much higher than the overflow's (Figure 18). Annual peak overflow distributions were close to the annual peak post-development distributions for almost all water tanks (Figure 26a to Figure 26d) and much higher than pre-development distributions in any case.

For the 5,000 m^2 catchment area the mean annual recharge distribution was smaller or close to the mean annual overflow distribution (Figure 21a and Figure 21b, respectively) and much smaller than the mean annual post-development distribution. For any water tank, annual peak overflow distributions were similar to the annual peak post-development distributions (Figure 27).

It is seen from Figure 22 that the mean annual post-development runoff has a linear relationship with the rooftop catchment area, while mean annual tank overflow and aquifer recharge relationships are asymptotics. The mean annual aquifer recharge volume tends to fit an almost horizontal asymptote when the rooftop catchment area tends to $+\infty$, while the mean annual tank overflow tends to fit a line parallel to the line that corresponds to the mean post-development runoff. In all cases, both curves intersect at some point that represents the combination of the rooftop catchment area and interim storage which produces an equal division of rainwater, on average: half recharges the unconfined aquifer and half overflows to the current drainage system. If managed aquifer recharge solely is the goal of an RWH system, then the optimal value of the catchment area is found below the one from the half-to-half division, when the volume difference of recharge to overflow is maximized, depending on the interim storage capacity. This is best visualized in Figure 24, where the asterisks (*) locate the points whose coordinates are the water tanks (x-axis) and the catchment area that maximize the difference of annual recharge to overflow (y-axis). The values of rainwater retention *R*, for these combinations of water tanks and catchment areas, lies around 75%. Further increasing the catchment area, for any water tank, is not interesting since it will not further significantly increase the annual recharge amount (in fact, this would reduce the rainwater retention *R*, as stated in Figure 30b). Table 6 shows the optimal catchment area for each water tank used in this study when managed aquifer recharge is the priority of the RWH system.

On the other hand, in terms of flood flow reduction, optimal rooftop catchment areas according to specific interim storage volumes are much more restricted than when the overflow amount to existing drainage systems can be neglected. This is true because what matters concerning flood control is not only the total rainfall volume that overflows but its distribution with time in terms of what peak flow it produces. In general, for the RWH system to work properly, when flood flow reduction is important, the peak tank overflow should not surpass the peak pre-development runoff flow, which represents the natural drainage condition before the implementation of the urban development. Figure 30a provides values of optimal catchment areas when efficiency is optimized (*E* of 100%) for the collection of water tanks used in this study. These catchment areas represent the limit of the RWH system when flood control is being prioritized (Table 6).

Table 6 – Optimal catchment areas of the RWH system according to its priority

Water tank (L)	Optimal catchment for flood control (m^2)	Optimal catchment for MAR (m^2)
500	158	1,230
1000	174	1,271
3000	234	2,018
5000	290	2,017
7500	348	2,492
10000	369	2,531
15000	400	3,001
20000	477	3,504
25000	510	3,768
30000	571	4,026

When analysing the behaviour of the metrics used in this study, one can conclude the region where the rainwater retention *R* is found within the 90%-100% interval (Figure 30a) is the same region where the efficiency *E* face a much stronger gradient: it varies from the around 110% to 20% (Figure 30b). Then, only the first 10% decrease in the rainwater retention (thus an increase from nil up to 10% in overflow) is responsible for dramatically reducing the efficiency of the RWH system in mitigating flooding. In other words, any overflow in the system tends to rapidly-produce floods in such a magnitude that the system will not be able to control it. For a given catchment area whose potential runoff is meant to be destined for an RWH system, its interim storage volume will depend on the MAR system objective. If flood control can be neglected, relatively small interim storage tanks will be able to produce satisfactory rainwater retention. If flood control is important, larger tanks will be demanded.

The cost of the RWH systems (injection well cost plus water tank cost) was compared to the cost of a standard building made of its optimal catchment area concerning each water tank capacity, for both objectives contemplated in this study (Table 6). The methodology for computing the costs is described in section 5.5. The optimal catchment areas for flood control, which ranged from 158 to 571 m² in the function of the water tanks, were assumed to be from high standard single-family residences (code R1-A, with a basic unit cost of R$ 1,543.62 per m²). On the other hand, the optimal catchment areas for managed aquifer recharge, which ranged from 1,230 to 4,026 m², were assumed to be from low standard popular buildings (tanks: 500 to 5,000 L; code PP-B, with a basic unit cost of R$ 956.35 per m²), normal standard popular buildings (tanks: 7,500 and 10,000 L; code PP-N, with a basic unit cost of R$ 1,164.93 per m²), and from normal standard multi-family residences (tanks: 15,000 to 30,000 L; code R8-N, with a basic unit cost of R$ 1,021.35 per m²).

Figure 32 shows the relative costs of each RWH system concerning their optimal catchment areas for each water tank and for the scenario where flood control is prioritized and when managed aquifer recharge is the priority.

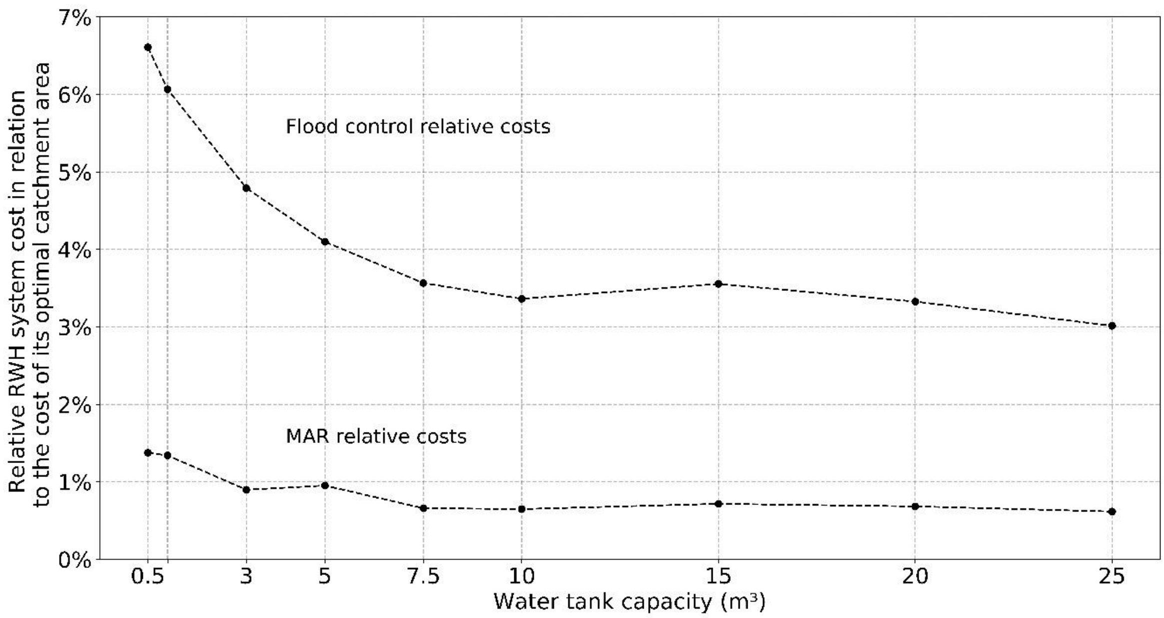

Figure 34 – RWH systems relative costs concerning their optimal catchment areas for each water tank capacity, for both objectives contemplated in this study

One can state that the relative costs of the RWH systems, for managed aquifer recharge, lied around 1%, no matter the water tank being considered, concerning its optimal catchment area. On the other hand, in comparison, flood control costs were quite higher, being further high for the smaller water tanks. For the higher tank capacities, the costs of the RWH systems for flood control measures were around 3.5% of the cost of their optimal catchment. This means that for an RWH system with any given water tank capacity, the optimal catchment area is much more restricted when flood control is being prioritized and therefore the system's cost will share a larger amount of the total cost, including the catchment area's. Other expenses concerning the RWH system are not being considered here (pre-treatment, gutters and downspouts adaption, structure supporting the water tank, pipeline connecting it to the injection well, etc.) hence the percentages shown in Figure 34 are expected to be slightly higher in the case of employment of a RWH system in the study area.

8 CONCLUSIONS

This paper evaluated the technical feasibility of rooftop rainwater harvesting (RWH) systems as a tool for sustainable stormwater management and as a tool for managed aquifer recharge in the João Pessoa city (Paraíba, Brazil). The system is connected to an unconfined aquifer (Barreiras formation) via a 6-inch diameter injection well. By analysing different configurations of RWH systems, it was demonstrated that, for some combinations of rooftop catchment area and water tank volume, the RWH system is efficient to detain surface water runoff, to dampen its peak values, and reduce the volume that would otherwise be directed into downstream drainage network by enhancing aquifer recharge. It has been shown that the RWH system can maintain the mean annual rainwater retention at satisfactory rates for most configurations studied, being the value around 75% the best for managed aquifer recharge schemes in the region, when flood control is not a priority. Further increasing the rooftop catchment area is not interesting since it would not contribute towards a substantial increase in the total recharged volume (it would only substantially increase the amount of precipitated water that overflows from the system). When flood control is important, then the rainwater retention of a given RWH system must be as close as possible to 100%. Stormwater management goals reconcile with managed aquifer recharge since, for a given rooftop catchment area, larger water tanks will be demanded than when it is not being considered and, consequently, the volume of mean annual aquifer recharge will be increased as well.

Considering the largest rooftop catchment available in the study area (~580 m², from the Hydraulics Laboratory), results from this study point out that a 500 L tank would be enough to recharge, on average, 90% of the annual rainfall that is conveyed by the given rooftop. However, the remaining 10% includes all extreme rainfall events, representing the share that most impacts in terms of flooding, leading the RWH system with a 500 L tank to perform poorly, on average, with an efficiency around 30%. For sustainable stormwater management, the RWH system connected to the Hydraulics Laboratory must have interim storage higher than 30,000 L, which would lead, on average, to both rainfall retention and efficiency close to 100%. On the other hand, if only half of the given rooftop is connected to the RWH system (~290 m²), a 5,000 L tank would be enough to recharge, on average, 100% of the annual rainfall and provide an efficiency of 100% in terms of stormwater management. In this case, the relative costs of the RWH system (R$ 18,361.27, from Table 2, in addition to the assumed cost of the injection well) would be around 4% the cost of a typical high standard single-family residence in João Pessoa (Annexe D), costing around R$ 447,649.80 (code R1-A, from Table 3). This example illustrates how the stormwater management goal is much more sensitive towards the combination of rooftop catchment area and water tank volume than the managed aquifer recharge goal. Particularly, there is a strong negative gradient in efficiency values of the RWH system when the rooftop catchment area is raised from 100 m² up to 1,500 m² (Figure 32b), which is not followed by the rainfall retention (Figure 32a). Hence, prioritizing the stormwater management component of a proposed RWH system has the benefit of maximizing its aquifer recharge potential, without leading to a substantial increase in the costs of the RWH system.

Results from this study require validation based on empirical, real-time monitoring of RWH systems like the ones proposed in this hypothetical research (for example, fixing the rooftop catchment area and monitoring their performance with different water tank volumes). Even though not being validated, the level of reliability provided by the results (high-temporal resolutions, based on in-situ injection tests) are expected to provide initial guidelines on the dimension of a proposed RWH pilot system in the study area – as is being currently studied in the scope of the SMART-Control project. There is a need of further studying the relationship between the water level drawdown within the well and its corresponding recharge rate, relating these to local aquifer parameters (transmissivity and specific yield) and the dimensions of the well (diameter, screen length and its flux capacity per metre, depth to the "static" water table). Besides, more experimental injection tests are required using this study's injection wells to provide more statistically significant curves and to cover other climatic conditions, since the tests described in this study were run only on May, in the middle of the wet season (March to

August). More tests, covering both the wet and dry seasons are desired, which could enable the coupling of climate change analysis into further hypothetical works. Although not considered in this study, water quality aspects related to the rainwater and groundwater should not be neglected – these may affect in the long-term performance of the RWH systems if clogging mechanisms are triggered by injection. Albeit the risks of clogging and aquifer contamination in the long-term are expectedly low, preventive measures to minimize them are required. These measures might induce reductions in the optimal catchment areas for flood control and MAR (Table 6), considering each water tank in this study.

Further studies should focus as well in investigating the potential of RWH system for stormwater management and aquifer recharge at the catchment-wide scale, where phenomena such as the spatial distribution of rainfall, heterogeneity in soil and aquifer parameters, water table depth, and land use must all be taken into account. A suggestion of the next step, for example, is to carry on several injection tests using the infrastructure inherited by the BRAMAR project, since several injection wells are widespread in the João Pessoa city. Results from such a study could enable the development of maps concerning the efficiency of the RWH system, depending on location, for each combination of the rooftop catchment area and interim storage. Combining this information with vectorized estimates of rooftop catchments areas in the city, from aerial photographs or satellite data, could enable the construction of optimized maps, showing the efficiency of the RWH system for at-source stormwater management (using the optimal water tank volume corresponding to each building or group of buildings) and providing mean annual estimates of artificial recharge in the city by the set of RWH systems. Similar studies could also be carried out in flood-prone, urban coastal areas where groundwater depletion is an issue. For example, the Recife Metropolitan Region, where many wells were drilled in the last decades and a large number of rooftops present an opportunity for the establishment of this kind of system (Coelho et al. 2018). From a spatial point of view, the combined effect of several RWH systems should not be neglected, from the stormwater management point of view nor the aquifer recharge perspective. For that matter, hydrologic and hydrogeologic models should be considered (calibrated and validated) to provide trustworthy results.

Improvements to the proposed RWH system can be achieved by further research adding water conservation goals to the objectives studied. Water conservation goals, such as toilet flushing (Palla et al. 2017), garden watering (Burns et al. 2012), car washing (Burns et al. 2015), and even drinking (Burns et al. 2015) may contribute to sustainable stormwater management goals, but this contribution is expected to be potentialized due to further increase in the water demand by adding an aquifer recharge demand, as exemplified by Burns et al. (2012). For that matter, both passive and active RWH systems require further study, but passive systems are expected to offer less resistance by householders since they can be designed based solely on gravity. A water-level threshold within the water tank could be defined, in which all water exceeding the given level could be directed to the injection well whereas the water below the threshold would be designed to meet non-potable water demands, estimated using the mean annual total dry days in the region.

Another potential improvement is to carry on hypothetical and empirical research combining the proposed RWH systems coupled with green roofs. Besides, this may even reduce or satisfy the need of pre-treatment systems, in a context where limited space is still a reason why facilities are still minimally implemented in urban watersheds (Sohn et al. 2019). Moreover, the usage of green roofs can provide other benefits, such as reduced energy consumption, reduced heat island effect, reduced dioxide carbon emissions, improved air quality and landscape, etc. (Eckart et al. 2017) hence making the RWH system more attractive to households and the public. Another reason is that green roofs have been recently more studied than RWH systems in terms of empirical monitoring, lab experiments, and computer simulations (Sohn et al. 2019; Gimenez-Maranges et al. 2020). Both abovementioned suggestions for improvement (adding water conservation goals and green roofs, considered solely or together) may provide increases in the optimal areas for flood control and MAR (Table 6) considering each water tank in this study.

Overall, proper monitoring and adaption of existing infiltration-based SUDS systems may turn these unmanaged aquifer recharge schemes into MAR sites, which may be a useful approach to promote the uptake of MAR particularly in already densified urban areas (Dillon 2005). Concerning RWH systems (in the SUDS context solely) is no different – there might be some RWH systems currently in operation that could be adapted to actual MAR sites, contributing to the still required long-term monitoring under varying spatial and temporal scales, and climate conditions (Campisano and Modica 2015; Dillon et al. 2018; Sohn et al. 2019). Analogously to (Fletcher et al. 2015)’s conclusion regarding the jointed benefits of low impact developments and green infrastructures, the widespread adoption of RWH systems for both MAR and SUDS goals is likely to drive managed aquifer recharge towards a more distributed and at-source application in urban areas, especially if green infrastructures are also driving the infiltration-based SUDS implementation. This approach can contribute to raising the groundwater supply in urban areas while reducing the frequency and magnitude of floods.

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ANNEXE A

Paper to be submitted to the Journal of Environmental Management (JEMA)

Performance of different configurations of rooftop rainwater harvesting used for managed aquifer recharge: a stormwater management approach in a coastal urban city

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ABSTRACT: The urbanization process in urban coastal areas has led to intense groundwater consumption whereas reducing permeable areas and increasing the frequency and magnitude of floods. To compensate these adverse effects, this research investigated the performance of different configurations of rooftop rainwater harvesting systems as tools of at-source managed aquifer recharge for sustainable stormwater management in a Brazilian coastal city located in a sedimentary aquifer system. Several configurations of rooftop area and water tank capacity were tested. The systems are connected to the unconfined Barreiras Formation through a six-inch diameter injection well. Rainfall-runoff processes and water balances were simulated from monitored, high-temporal resolution, rainfall data and insights acquired after experimental tests results (pumping tests and injection tests). Results show a strong negative gradient in the efficiency of the system when the catchment area is raised, at a fixed water tank volume, which is not followed by the rainfall retention values. It was concluded that prioritizing managed aquifer recharge with flood control included among its objectives maximizes its aquifer recharge potential without leading to a substantial increase in the initial investment costs. This study shows the importance of managed aquifer recharge schemes for stormwater management, an approach that can contribute to raising the groundwater supply in urban areas while reducing the risk and severity of floods.

Keywords: Flood control, rooftop rainwater harvesting, managed aquifer recharge.

1. Introduction

Half of the World's population and human activities are concentrated on surface coastal zones (Chatton et al. 2016). The urbanization in these areas affects water governance (Romano and Akhmouch 2019), e.g. by raising water demand, threatening groundwater resources (Jacobson 2011; Gleeson et al. 2012; Bertrand et al. 2016) and by increasing the risk of floods by stormwater augmentation (Eckart et al. 2017). Currently, the groundwater is viewed as a valuable resource (Dillon et al. 2018), its depletion related to salt-water intrusion and land-surface subsidence (Silva et al. 2006; Coelho et al. 2018), while the stormwater is seen only as a problem in urban areas (Gimenez-Maranges et al. 2020), related to many socio-economic issues (Oleson et al. 2015; Dethier et al. 2016; Trigo et al. 2016). Climate change puts more pressure on groundwater and stormwater management systems by leading to more frequent climatic extremes (IPCC 2007; Kløve et al. 2014), which tend to exacerbate the competition between water users (Romano and Akhmouch 2019).

Rooftop rainwater harvesting (RWH) systems are among initiatives to provide an additional source of water to meet domestic water demand, widespread in semi-arid areas (Li and Gong 2002; Kumar et al. 2016; Taffere et al. 2016; Almazroui et al. 2017; Santos and de Farias 2017; Shubo et al. 2020). RWH systems are also used to reduce groundwater consumption in urban areas (Adham et al. 2016), being enlisted in the sustainable urban drainage systems (SUDS) technology due to studies that have assessed their potential for water conservation and stormwater management (e.g. Freni and Liuzzo, 2019; Teston et al., 2018). SUDS are strategies claimed to restore hydrology in the catchment scale by mimicking natural processes through surface runoff/peak flow reduction and infiltration/baseflow improvement ((Perales-Momparler et al. 2017).

Studies have revealed that conventional demands of RWH systems for stormwater management (hereon called as SUDS-RWH) are not sufficient to empty storage structures right during heavy rainfall events, thus leading to poor peak runoff reduction (Petrucci et al. 2012; Palla et al. 2017). A step forward better stormwater management is to increase the system's demand by diverting overflow via infiltration (Burns et al. 2015). For example, Burns et al. (2012) have shown that an RWH tank overflowing to a rain garden was able to promote reduced frequency and volume of stormwater runoff whereas contributing to sustainable baseflow restoration. Another solution is to convey overflow via at-source direct injection into aquifers. This is claimed to compensate for impacts of urbanization and groundwater abstraction hence reducing wastewater infrastructure costs and improving public health in general (Tuinhof and Heederik 2002).

RWH systems are also part of managed aquifer recharge (MAR) technology when they store rainwater aiming at controlled aquifer recharge (NRMMC et al. 2009). In general, MAR methods lead to an increase in groundwater supply in wet seasons to help face shortcomes at dry seasons (Gale and Dillon 2005; Kretschmer 2017). MAR will be used to overcome global challenges in Latin America in a proportion at least similar to the current level of countries with similar groundwater use but more established MAR implementation such as India and the USA (Dillon et al. 2018). It consists of strategies for groundwater system resilience (Stefan and Ansems 2018) with many applications such as environmental protection (Zhang et al. 2020). Flood reduction is often not explicitly pursued but is promoted when many MAR methods are applied for groundwater supply augmentation via underground storage (e.g. Missimer et al. 2015; Sandhu et al. 2018; Maliva et al. 2020).

Underground storage within aquifers has many advantages over dams and surface reservoirs. Depleted aquifers can exhibit a much larger and cheaper storage capacity (Dillon 2005; Perrone and Rohde 2016; Scanlon et al. 2016; Yang and Scanlon 2019). It does not have evaporation losses nor the possibility of structural failure intrinsic to surface reservoirs following disasters (Bouwer 2002; Minsley et al. 2011; Alataway and El Alfy 2019). Also, it is less prone to sediment accumulation (Tuinhof and Heederik 2002), algae blooms (Tang et al. 2018) and atmospheric fallout of pollutants (Hartog and Stuyfzand 2017). Furthermore, the estimated global storage of groundwater (1.9 million km^3) (Gleeson et al. 2016) is much higher than current estimates of global storage in dams and lakes (12.900 km^3) (Dillon et al. 2018). Underground storage also facilitates logistic of supply and demand in place and time since it enables the capture of water at-source and its on-demand reuse (Keller et al. 2000).

Storing runoff captured by RWH systems directly into confined and/or unconfined aquifers through injection wells (hereon called MAR-RWH) seems to be feasible since they simultaneously contribute to controlled flooding and increased groundwater supply, particularly in urban areas. This practice can gradually replenish depleted aquifers whereas reducing flood risks in the rainy season by giving a proper destination to rainwater that would otherwise be directed to downstream drainage networks (Silva et al. 2006). Several experimental injection tests (Diniz et al. 2008; Barbassa et al. 2014; Händel et al. 2016; Liu et al. 2016) have reported promising gravity-based recharge rates, suitable to contribute to strategies of stormwater management (Burns et al. 2012, 2015). These systems can be also termed as MAR-RWH schemes if groundwater recharge is monitored and measures are taken to prevent aquifer contamination (Dillon et al. 2018).

A limited number of studies concerning SUDS-RWH systems are found in the literature (Gimenez-Maranges et al. 2020). No empirical RWH studies have been found in a SUDS review by Sohn et al. (2019). Research on the effectiveness of SUDS-RWH systems is still scarce (Palla et al. 2017), including studies with groundwater flow monitoring (Jacobson 2011). In MAR’s context is no different. No paper concerning such technique was found in a review analysing 233 modelling studies evaluating MAR (Ringleb et al. 2016). MAR-RWH projects represent only 3% (34 sites) of the worldwide sites registered at the IGRAC MAR Portal, 59% of these concentrated in New Delhi, India (IGRAC 2020). Further research on SUDS- and MAR-RWH systems are required to enlighten their benefits (Clary et al. 2011). For that matter, long-term simulations with sub-hourly time resolution are required (Campisano and Modica 2015). The evidence founded on real-time monitoring could be useful to supplement both experimental and hypothetical studies (Sohn et al. 2019).

MAR-RWH systems contribute towards stormwater management, however, few studies have evaluated their potential towards flood mitigation. A pumped MAR-RWH scheme combined with ASR wells conveying stormwater from an 80-ha urban residential catchment (Kretschmer 2017) is among the few exceptions found in the literature. This may also be the case of some MAR sites in the MAR Portal (IGRAC 2020) whose main objectives fall under the categories of ecological benefits or other benefits. Many reported gravity-based MAR-RWH schemes did not investigate its potential towards peak runoff flow reduction, such as Page et al. (2010)'s. Furthermore, the influence of input variables (e.g. rooftop catchment area and water tank volume) in the performance of the RWH systems was not evaluated in the above-mentioned examples.

This study presents hypothetical and empirical research focused on the technical feasibility of RWH systems as a tool of at-source managed aquifer recharge via direct injection for sustainable stormwater management in a coastal urban city located in a sedimentary aquifer system in North-east Brazil. This study focuses on three specific objectives, summarized in the following statements: it aims 1) to integrate a hypothetical water balance method with empirical data to enable reliable long-term simulations of the RWH system, with high temporal resolution; 2) to determine water tank volumes which optimize the performance of the RWH system in the function of the rooftop catchment area, with a focus in restoring the hydrology of the site to its assumed pre-development state; and 3) to calculate and compare the initial investment cost of the RWH systems with the costs of the catchment areas being subjected to at-source surface runoff control for a simple initial economic evaluation.

2. Study area description

This study was carried out in João Pessoa (JPA), a coastal urban city located in the Paraíba State, North-east Brazil, between the coordinates 7°03'53.6"S 34°58'16.8"W and 7°14'6.6" S 34°47'39.8" W (Fig. 1). JPA is the twenty-fourth largest city in Brazil, with ~1 M inhabitants in their metropolitan region. According to the study performed by Alvares et al. (2013), the climate in JPA is characterised as tropical with dry summer (As), with mean annual temperature and relative humidity of about 25 ºC and 75%, respectively. The mean annual rainfall in

JPA is approximately 1,800 mm, of which 73% concentrated from March to August (Alvares et al. 2013).

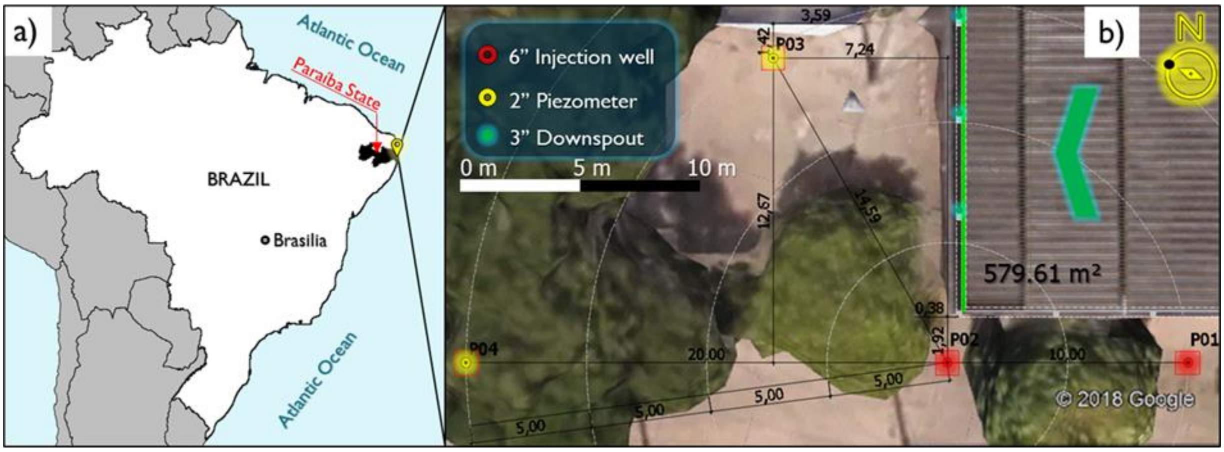

Fig. 1. Study area (a) location and (b) sketch.

JPA is located in the Paraíba sedimentary basin, predominantly composed of sandy and clay deposits. Five distinct lithologic layers are found in JPA, namely: Barreiras, Maria Farinha, Gramame, Beberibe, and Itamaracá formations (Rossetti et al. 2012). The morpho-structure of the Barreiras Formation is mainly composed of poorly consolidated clayey sands dated from the Miocene, alluvial sediments and sandstones (Furrier et al. 2006). The Barreiras Formation has a mean thickness of 20 metres, forming a shallow unconfined aquifer (Walter 2018). The hydraulic conductivity of the Barreiras Formation is estimated as below 3.47×10^{-5} m/s (Walter 2018), although values of approximately 1×10^{-4} m/s have been reported (Fernandes 2017). The Barreiras Formation overlays the Beberibe Formation, a confined aquifer with a mean thickness of approximately 360 metres and comprising medium- to coarse-grained sandstones (Rossetti et al. 2012). The Beberibe Formation is directly connected to the Barreiras unconfined aquifer in the west portion of JPA. In some parts of the east of JPA, the Gramame Formation, a richly fossiliferous unit with a 55-metre thickness, is located between the Beberibe and Barreiras formations (Walter 2018). The Itamaracá Formation comprises a 70-metre thick unit of richly fossiliferous and calciferous sandstone, located in the south-eastern part of the study area (Rossetti et al. 2012). This multi-layered sedimentary aquifer of the Paraíba basin lies over a crystalline regional basement that was affected by the rifting process due to the Atlantic Ocean aperture. The vertical infiltration constitutes the main source of groundwater recharge over the study area, characterised by hydraulic connections between the different hydrological layers and the interaction between surface water and groundwater close to the rivers (Bertrand et al. 2017).

The annual rainfall in JPA strongly influences water availability (Walter 2018). The water table of the unconfined aquifer is mainly dependent upon vertical local recharge, which is sensitive towards soil impermeabilization. According to Furrier and Barbosa (2016), the city faces an urban development that does not consider the geomorphological aspects, becoming more prone to floods and its intrinsic damages. This is a major issue, since JPA currently produces on average 681 million m³ of surface runoff annually, 61% of these concentrated from May to July, showing a clear seasonality with local rainfall (Walter 2018). Most of the annual runoff leaves the system into the sea without being utilized. This value is about five times the average urban water demand in the city (domestic and industrial), of 138 million m³ (Walter 2018). The availability of roofs in the urbanised area opens up opportunities for RWH systems implementation, as suggested by Coelho et al. (2018) for another metropolitan region in North-east Brazil.

More specifically, this study was carried out in a site located inside the João Pessoa Campus of the Federal University of Paraíba (7°08'31.8"S 34°50'59.4"W and 7°08'32.8"S 34°50'59.6"W; Fig.1bb), where two injection wells (P1 and P2) with a 6-inch diameter and two monitoring wells (P3 and P4) with 2-inch diameter are available. All wells are 42 metres deep, drilled in the Barreiras unconfined aquifer with a screen length measuring 12 metres (from 28 to 40 metres). The static water table in the site after the construction of the wells (23-May-2019) was about 28 metres below the surface. Several buildings in the surroundings of the experiment and a large amount of rainfall over the region are virtually available for roofing-water collection to implement managed direct injection of rainwater into the aquifer. The largest rooftop area in the surroundings of the wells is the Hydraulics Laboratory, with ~ 580 m^2.

Fig. 2 shows the lithological profile of the injection wells, which is composed of fine sand and clay layers that vary from yellow to reddish-yellow in colour. This lithological profile was drawn based on the results of granulometry, real density, and plasticity limit tests carried out with soil material collected during the drilling of all wells. The lithological profile drawn from material in the site (collected and analysed at every two metres depth) is consistent with the descriptions performed by Rossetti et al. (2012), in which the Barreiras Formation can be characterised by its irregular stratification and the occurrence of varying colours

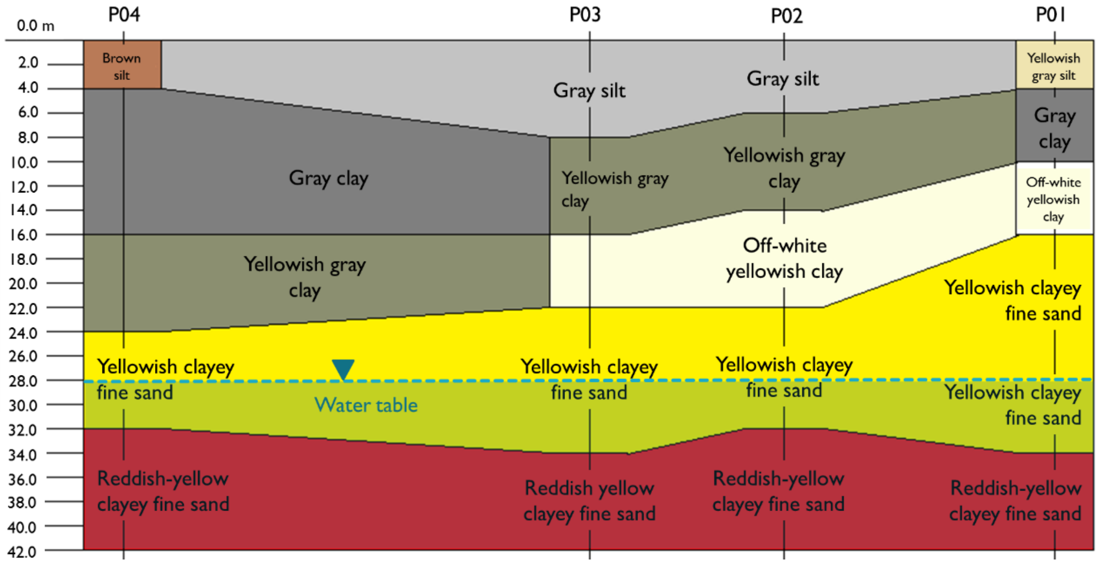

Fig. 2. Lithological profile of the RWH system's site.

3. Materials and methods

This study presents a simulation of the hydraulic behaviour of RWH systems connected to an unconfined aquifer via an injection well, using computational modelling (Sohn et al. 2019). Both empirical non-experimental (monitored real-time rainfall) and experimental data (*in situ* hydro(geo)logical tests) were gathered and used in the study. The methodological approach is described in detail in the next sections, as follows: 1) the rainfall data used in the study will be briefly presented in section 3.1, 2) the pre- and post-development scenarios will be shown in section 3.2, and 3) the key inputs used to the hydraulic simulations will be depicted in section 3.3.

It is important to address the methodological assumptions and limitations in which the hydraulic simulations are based. They must be at least reasonable to provide confidence in the results of this study. These hypotheses are that 1) the aquifer heterogeneity in the screened interval can be approximated to a homogeneous layer; 2) seepage or lateral flows are inexistent or do not play a major influence on the unconfined aquifer water balance; 3) the performance of the RWH system is not significantly affected, in the long-term basis, by external factors such as the natural water level variation, temperature variation nor the quality of the input rainwater; 4) the static water level is not too much sensitive towards pumping nor recharge on the site; 5) effects from pumping or injection on other wells in the vicinity are negligible. Hypotheses (1) and (2) are assumed as true; hypothesis (3) is assumed as true concerning water level and temperature variation, and as a reasonable assumption considering a recharge of high-quality rainwater, where quality issues can be neglected; and hypotheses (4) and (5) were tested in the following methodology, more specifically in section 3.6.

3.1 Rainfall data

This study used rainfall data with a high temporal resolution to enable the execution of refined water balance simulations. The rainfall dataset for the period 01-Jan-2004 to 31-Dec-2019 was acquired from automatic tipping bucket rain gauges with a 1-min temporal resolution when it rains and 360-min over no-rain periods. These rain gauges are located within the Guaraíra Experimental Basin (GEB; Coutinho et al. 2014), which is a small watershed monitored by the water research group from the Laboratory of Water Resources and Environmental Engineering (LARHENA) of the UFPB, near to the pilot experiment. GEB rainfall data comprises data not only from one single station but is made of the available data acquired and pre-processed by the research group at four near rain gauges. Fig. 3 shows the mean monthly rainfall records for the studied period obtained from the monitored rain gauges in GEB.

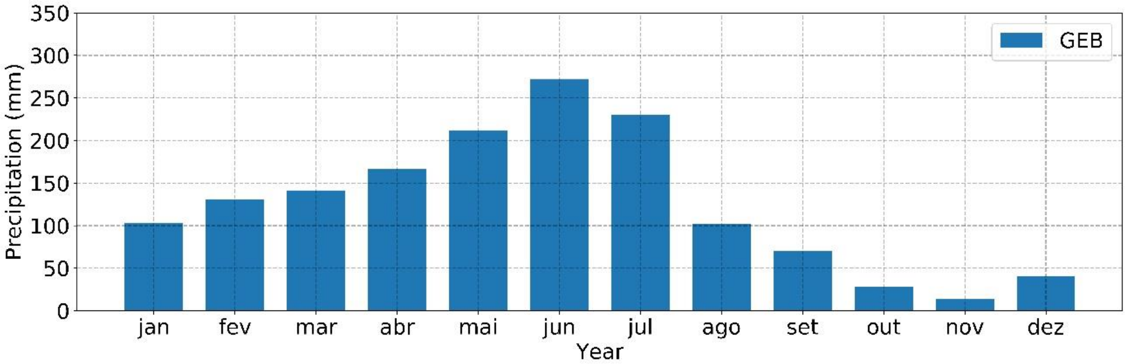

Fig. 3. Mean monthly rainfall recorded in the Guaraíra Experimental Basin (GEB).

3.2 Pre- and post- development scenarios

The simulations performed in this study comprised two development scenarios with different inflow rates to evaluate the feasibility of the rainwater harvesting system. The former scenario, herein called a post-development scenario, refers to a catchment area made by an impervious roof, with roughness coefficient of 0.027 and slope of 30%, from a metallic built-up channel, unpainted smooth steel surface (Chow 1959). The latter scenario, herein called a pre-development scenario, corresponds to a catchment area constituted by a permeable surface soil with the characteristics found *in situ* from double-ring infiltrometer tests (Bouwer 2002), roughness coefficient of 0.012 and slope of 1%, assumed to be from an excavated channel, earth, straight and uniform, with short grass and few weeds (Chow 1959). The post-development scenario represents the current impervious conditions of the site, whereas the pre-development scenario represents the site's conditions before development had been established.

The rainfall-runoff process was computed using the open-source Storm Water Management Model

(SWMM), which was developed by the U.S. Environmental Protection Agency (EPA) (Rossman 2015; Sohn et al. 2019). For this application, the whole one-minute step rainfall data from GEB (16-year period) was used in the study, rain format in volume. SWMM allows for runoff generation, flow routing, and stormwater collection networks modelling by using multiple hydrologic and hydraulic computation methods (Kaykhosravi et al. 2018). It also can be viewed as a physically-based SUDS toolbox (Eckart et al. 2017), better suited for more advanced modelling phases (preliminary and detailed design/analysis) where information such as peak flow, runoff amount and volume within conduits are required (Kaykhosravi et al. 2018). SWMM is popular among stormwater management researchers and widely used worldwide for planning, analysis and design related to drainage systems (e.g. Petrucci et al. 2012; Wang and Altunkaynak 2012; Karamouz and Nazif 2013; Cipolla et al. 2016; Zhang et al. 2016; Avellaneda et al. 2017; Palla et al. 2017; Paule-Mercado et al. 2017; Zanandrea and Silveira 2018). Among existing models, a recent review of hypothetical studies on Sohn et al. (2019) found that the SWMM model was the most utilized, representing 38% of selected studies. The SWMM model is commonly used in studies based on current and historic climate data as well as short-term event-based analysis (Sohn et al. 2019). SWMM has been used to evaluate RWH hydrologic performance on stormwater management (e.g. Petrucci et al. 2012; Palla et al. 2017).

The pre-development scenario considered no percentage of impervious area and was drawn using the Horton infiltration method, while the post-development scenario considered a 100% impervious area and, hence, no infiltration method was applicable. The kinematic wave routing model was used with a 1-minute time-step. The outlet of the catchment area was considered as the tank inlet, i.e., the flow of water through gutters and downspouts were neglected. Table 1 summarizes the parameters considered in the simulations of both scenarios.

Parameter	Pre-development scenario	Post-development scenario
Slope	1%	30%
Manning’s coefficient	0.027	0.012
Maximum infiltration rate	419.21 mm/h	–
Minimum infiltration rate	30.48 mm/h	–
Decay constant	4.8 h^{-1}	–
Drying time	2 days	–

Table 1. Parameters considered on the SWMM rainfall-runoff process for both studied scenarios.

Pre- and post-development hydrographs were calculated considering a collection of 15 different catchment squared areas (between 10 and 5,000 m², as shown in Table 1) to cover a variety of typical rooftop areas of residential developments, i.e., from standard single-family residences to multi-family residential condominiums (ABNT 2006).

Rooftop areas (m²)

10	400	1,300
20	500	1,400
30	600	1,500
40	700	2,000
50	750	2,500
100	800	3,000
150	900	3,500
200	1,000	4,000
250	1,100	4,500
300	1,200	5,000

Table 2. Collection of squared rooftop catchment areas used in the hydraulic simulations.

3.3 Hydraulic simulations

Water balance calculations were carried out considering the RWH system, which comprises three elements (Fig. 4): 1) the rooftop catchment area, 2) the interim storage made of a water tank and 3) the injection well P02. The input runoff goes into the interim storage before being introduced into the injection well for further aquifer recharge. After processing the input runoff files (i.e., runoff temporal series for each rooftop catchment area, year, and scenario), these data were used as input for the water balance simulations.

For each input runoff file, a collection of water tanks with 10 different volumes (from 500 to 5,000 litre, as shown in Table 3) was tested for determining the proportion of runoff that effectively recharges the aquifer and the one that is split over the tank. Table 3 exhibits the collection of different water tank dimensions obtained from commercial catalogues provided by some Brazilian water tank suppliers.

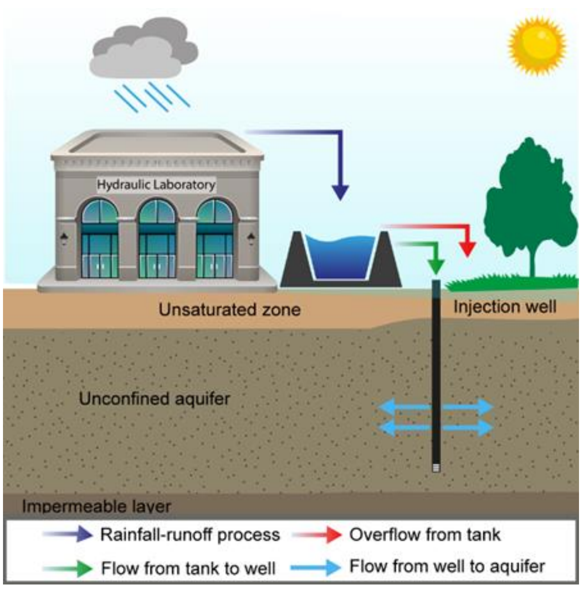

Fig. 4. Sketch of the main components of the rainwater harvesting system.

Tank volume	Height	Lower diameter	Upper diameter	Mean price

S (m³)	H (cm)	D_2 (cm)	D_1 (cm)	(USD)
0.5	53.34	97.83	121.20	32.71
1	77.50	114.57	144.00	58.69
3	112.41	172.17	215.50	261.68
5	162.00	182.36	233.40	460.48
7.5	181.00	224.00	270.00	614.50
10	203.00	241.00	292.00	609.95
15	262.00	267.00	315.00	1,163.33
20	323.00	244.00	335.00	1,651.75
25	387.00	235.00	338.00	1,509.39
30	473.00	250.00	320.00	---

Table 3. Collection of tank volumes used in the hydraulic simulations.

The water balance simulation considers two control volumes, as illustrated in Fig. 4: 1) the inner volume of the water tank, called ST_{max} and 2) the inner space of the injection well, called SW_{max}. SW_{max} is measured from the top of the casing to the water table.

Hydraulic simulations were executed in both control volumes using the hydrologic flow routing method named modified Puls method (Maidment 1992). A hydrologic flow routing method is a procedure to determine outflow hydrographs from known inflow hydrographs, based on the continuity equation in its simplest form. The modified Puls method, a slight change from the original method proposed by L. Goodrich Puls in 1928 (Chow 1959; Maidment 1992; Ghasemzadeh et al. 2020), has been extensively used in research comprising reservoir routing (e.g., Singh and Snorrason 1984; Madadi et al. 2015; Baptista and Paz 2018; Ferreira et al. 2018, 2019; Kamis et al. 2018; Ghasemzadeh et al. 2020). When the continuity equation is approximated by finite differences and, subsequently, rearranged, it may assume the following forms given in Eq. (1) and Eq. (2):

$$\frac{2 \times \mathrm{ST}_{t+\Delta t}}{\Delta t} + \mathrm{QT}_{t+\Delta t} = \mathrm{IT}_{t+\Delta t} + \mathrm{IT}_t - \mathrm{QT}_t - 2 \times \mathrm{OT} + \frac{2 \times \mathrm{ST}_t}{\Delta t} \tag{1}$$

$$\frac{2 \times \mathrm{SW}_{t+\Delta t}}{\Delta t} + \mathrm{QW}_{t+\Delta t} = \mathrm{IW}_{t+\Delta t} + \mathrm{IW}_t - \mathrm{QW}_t + \frac{2 \times \mathrm{SW}_t}{\Delta t} \tag{2}$$

where ST, IT, QT and OT are the storage (L), inflow rate (L/min), outflow rate (L/min), and overflow rate (L/min) in the tank control volume ST_{max}, respectively; SW, IW, and QW are the storage (L), inflow rate (L/min), and outflow rate (L/min) in the injection well control volume SW_{max}, respectively. Eq. (1) differs from the finite differences continuity equation since the OT term was included to cover the situation where the water in the tank is overflowing (when that is not the case, OT is null). The subscripts t and $t+\Delta t$ indicate the instant of time t (min) and the next instant, according to the time step Δt (min). Two serial hydraulic simulations were performed: the former in the water tank control volume and the latter in the well control volume. The first simulation followed Eq. (1) and the second followed Eq. (2).

The initial conditions were that IT_t, QT_t, ST_t, OT_t, QW_t, and SW_t were setup as null. Hence, at a given time t, the terms on the left in Eq. (1) and Eq. (2) are unknown, while terms on the right are known. To solve this issue, the modified Puls method (Chow 1959; Ghasemzadeh et al. 2020) prescribes the use of auxiliary curves for each control volume, which are built through known relations between the water height h inside the control volumes and its storage S and outflow rate Q at given equal time steps Δt. The construction of these auxiliary curves is meant to provide values of outflow rate as a function of the sum of values of the terms on the left in Eq. (1) and Eq. (2), for each control volume. These curves are based on the premise that each control volume is fixed, and its geometry is known. In this study, the tank control volume ST_{max} was approximated as a circular truncated cone, whose storage S_t is computed as a function of its height h_t (m), the lower diameter D_2 (m), and diameter at the water level within the tank $D(h_t)$, as shown in Eq. (3) and Eq. (4):

$$S_t(h_t) = 1{,}000 \times \frac{\pi h_t}{3}\left[\left(\frac{D(h_t)}{2}\right)^2 + \left(\frac{D(h_t)}{2}\right) \times \left(\frac{D_2}{2}\right) + \left(\frac{D_2}{2}\right)^2\right] \tag{3}$$

$$D(h_t) = \frac{h_t}{H}(D_1 - D_2) + D_2 \tag{4}$$

where D_1 is the tank's upper diameter (m) and H is the tank's height (m). Eq. (4) describes a linear interpolation equation, using the height H, lower and upper diameters D_1 and D_2 as interpolation limits. The outflow rate Q_t (L/min) at a given water level within the tank was calculated as a function of h_t and the cross-section of the discharge outlet $A_{\emptyset}$ (m²), considering a 60-millimetre diameter tube (Porto 2006), as shown in Eq. (5):

$$Q_t = 60{,}000 \times C_d A_{\emptyset}\sqrt{2gh_t} \tag{5}$$

where C_d is the discharge coefficient, whose value is roughly 0.6076 for the given outlet tube diameter (Porto 2006). On the other hand, the injection well control volume SW_{max} was approximated as a cylinder with a fixed diameter, where its storage S_w varies in the function of its height h_w (m) ranging from the top of the casing to the water table, as shown in Eq. (6):

$$S_w = 1{,}000 \times A_w h_w \tag{6}$$

where A_w is the inner cross-section of the 6-inch diameter injection well (m²) and h_w (m) is the dynamic level of water above the static water level. Since the premise of the Puls method implies this control volume can be assumed unchangeable (ΔSW_{max} and ΔST_{max} are null), pumping tests and injection tests were performed *in situ* to test the validity of this hypothesis (see section 3.6). The injection tests also provided knowledge on the relationship between the well's outflow rate Q_w and the water level h_w, demanded to construct its control volume's auxiliary curve.

$IT_{t+\Delta t}$ is the post-development hydrograph in each input runoff file while IW_t and $IW_{t+\Delta t}$ are respectively equivalent as QT_t and $QT_{t+\Delta t}$ since the outflow from the tank is the inflow in the well. The outflow from the well is the amount of water that recharges the unconfined aquifer. When the left terms of Eq. (1) surpass a maximum known value associated with the fullness of the tank control volume, $QT_{t+\Delta t}$ is calculated using this maximum known value and then the overflow rate (OT) can be computed from Eq. (1). On the other hand, when the left terms of Eq. (2) surpass a maximum known value associated with the fullness of the injection well control volume, $QW_{t+\Delta t}$ is calculated using this maximum known value and $IW_{t+\Delta t}$ becomes equal to $QW_{t+\Delta t}$ and $QT_{t+\Delta t}$ equal to $IW_{t+\Delta t}$, demanding an update in the tank control volume simulation since this condition forces less water to flow from the tank into the well and therefore more water may overflow.

3.4 Metrics used for evaluation of the simulations

Two metrics were used to assess the RWH system performance, given by Eq. (7) (Freni and Liuzzo 2019) and Eq. (8) (Baptista and Paz 2018):

$$R = \frac{\sum \mathrm{IT}_t - \sum \mathrm{OT}_t}{\sum \mathrm{IT}_t} \times 100 \tag{7}$$

$$E = \frac{\mathrm{OT}_{\max} - \mathrm{IT}_{\max,\mathrm{post}}}{\mathrm{IT}_{\max,\mathrm{pre}} - \mathrm{IT}_{\max,\mathrm{post}}} \times 100 \tag{8}$$

where R is the rainwater harvesting retention, given in percentage; ΣIT_t is the runoff inflow volume (L); ΣOT_t is the runoff overflow volume (L) at a time interval spanning from t to T (min); E is the rainwater harvesting efficiency, representing how much close the runoff peak overflow (OT_{max}, L/s) is from the runoff peak pre-development flow ($I_{Tmax,pre}$, L/s); and $I_{Tmax,post}$ is the runoff peak post-development flow (L/s). The values of R ranges from 0% to 100% and represents the proportion of runoff effectively drained to the well for further aquifer recharge. Therefore, $1 - R$ represents the part of the input runoff that overflows from the water tank, potentially leading to flooding.

The value of E ranges from 0% to $+\infty$ and gives a numerical evaluation of how well the system mitigates flooding. Values of E higher than 100% means that the RWH system is capable to dampen the post-dev runoff peak flow to a level smaller than the pre-dev runoff peak flow and, therefore, is effective towards flood mitigation. Value of E equals to 100% means either that R is 100% (hence no proportion of input runoff overflowed from the tank) and/or that the runoff peak overflow is equal to the pre-dev runoff peak overflow. Value of E smaller than 100% means that the RWH system's runoff peak overflow is higher than the runoff peak pre-development flow. The smaller the value of E the most inefficient towards flood mitigation the RWH system is.

The auxiliary curves from the well and the water tanks are a preliminary step in the solution of the continuity equation following the Puls method. The well auxiliary curve is a function $Q_w(Q_w+2S_w/\Delta t)$, where S_w (Eq. (6)), and Q_w (Eq. (11), explained in section 4.2.1) were computed for a sequence of values of hydraulic head h from zero to 26 metres, incremented by 1 cm. The auxiliary curves of the water tanks assume the form of

$Q_t(Q_t+2S_t/\Delta t)$ functions, where S_t (Eq. (3)) and Q_t (Eq. (5)) were computed for a sequence of values of h from zero until the corresponding height that reached each tank capacity, incremented by 1 mm.

Hydraulic simulations were executed for each yearly post-development runoff temporal series (2004 – 2019), considering a combination of catchment areas (Table 2) and water tanks (Table 3) used in this study, what led to 300 different setups of RWH systems (in terms of rooftop catchments, water tank capacities and the injection well P02).

Moreover, pre- and post-development runoff, recharge and overflow temporal series were resampled to a 5-minute time step. Then, annual totals and peak values were recorded to enable the assessment of the RWH systems using the above-mentioned metrics.

3.5 Cost considerations

The initial investment costs of the proposed RWH systems were compared to the costs of the target buildings whose surface runoffs are being subjected to at-source control. The costs of the RWH systems were computed as the injection well cost in addition to water tank cost. The cost of the injection well is the cost of the well P02 (approximately 3,120 USD), while each water tank's mean cost is displayed in Table 2.

The costs of the target buildings were estimated from the size of the catchment area, following a common methodology applied in Brazil. Each Brazilian state has an agency responsible to provide monthly estimates of the basic unit construction cost per square metre, following different specifications. The collection of rooftop catchment areas from Table 1 can be associated with representative basic unit cost in the Paraíba State. Table 4 exhibits typically constructed areas of standard residential developments in the city (from ABNT (2006)) and it shows the basic unit cost per square metre, discounting taxes, of these developments in June 2020 (SINDUSCON/JP 2020). It is noteworthy that operational and management costs of the RWH systems were not included in this simple calculation, aimed at providing solely the RWH system's initial investment costs in proportion to the costs of the buildings being served.

Code	Description	Typical area (m²)	Construction cost (USD/m²)
R1-N	Normal standard single-family residence	106.44	245.86
R1-A	High standard single-family residence	224.82	301.02
PIS	Projects of social interest	991.45	135.40
PP-B	Low standard popular building	1,415.07	186.50
PP-N	Normal standard popular building	2,590.35	227.18
R8-B	R8 low standard multi-family residence	2,801.64	176.87
R8-N	R8 normal standard multi-family residence	5,998.73	199.18

Table 4. Typical areas (m²) and basic unit costs of standard designs at João Pessoa in June 2020. Unencumbered prices.

3.6 Field investigations

The experimental investigations consisted of pumping and injection tests. Pumping tests were executed in the wells P01 and P02 using a water pump with 5 HP of power. The water level was monitored during the tests in all wells in the study area. The objectives of the pumping test were to determine the unconfined aquifer parameters (transmissivity and specific yield) in the study area and to monitor the aquifer behaviour due to the pumping rate applied (and thence test the validity of hypotheses (4) and (5), mentioned in section 3). The aquifer behaviour was obtained by recording the length of time taken by the aquifer to re-establish equilibrium in the water level after being submitted to this different condition and after returning to its normal condition.

Several injection tests were also carried out in the injection well P02. These tests consist of monitoring the water level in the well while a certain volume of water is injected into the aquifer. The idea was to inject as much water as needed until the water level reached the top of the well casing. After that, the water level drawdown was monitored until the original static water level was reached. A 15 m³ capacity water truck was used to recharge the aquifer during the tests, with water flowing under pressure into the well through a 2-inch hose that had been dipped below the water table to avoid air entrapment (Liu et al. 2016). Many phenomena and parameters are being aggregated in an injection test, since the relationship between hydraulic load and recharge rate is complex, being influenced by factors like hydrogeologic parameters, well structure dimensions (Händel et al. 2016), and seasonal effects (Bouwer 2002). The objective of the tests was to obtain a mathematically modellable relationship between the recharge rate and the water level to enable the construction of the well control volume auxiliary curve. The recharge rate (or outflow rate from the well) can be calculated using Eq. (9):

$$Q_w = \frac{A_w}{1{,}000}\frac{(h_{w,t} - h_{w,t+\Delta t})}{\Delta t} \tag{9}$$

where Q_w is the recharge rate (L/min) at the time step t (min); $h_{w,t}$ and $h_{w,t+\Delta t}$ are the dynamic water levels (m) in the well at the time step t and on its following $t+\Delta t$, being Δt equal to 1 minute; A_w is the well inner cross-section (m²).

4. Results and discussions

This section is divided into three parts, in which results are presented and discussed in the following sequence: 1) experimental investigations that backed the hydraulic simulations, in section 4.1, 2) the rainfall-runoff process, in section 4.2, and 3) the main results of the hydraulic simulations, in section 4.3.

4.1 Field investigation results

4.1.1 Pumping tests results

Results from the pumping test at the injection well P02 show a 7-metre drop in the water level while a moderately constant 10.87 m³/h pumping rate was applied, resulting in a removal of roughly 133 m³ of water in 12 hours. The drop measured in the water level in the three wells in the vicinity was 30 cm on average (Fig. 5b). Results from the pumping test at the injection well P01 show a 9-metre drop in its water level by a 6.76 m³/h pumping rate and the drop in the other water levels were of 17 cm on average (Fig. 5a). Since both wells are screened in the interval 28-40 metres depth, water was flowing from the aquifer through 3 metres length of filter on pumping test at P01, and through 5 metres length of filter on pumping test at P02. Therefore, the observed capacity of the filter was under what was expected from the manufacturer (2.35 and 2.32 m³/h per metre length from tests on wells P01 and P02, respectively).

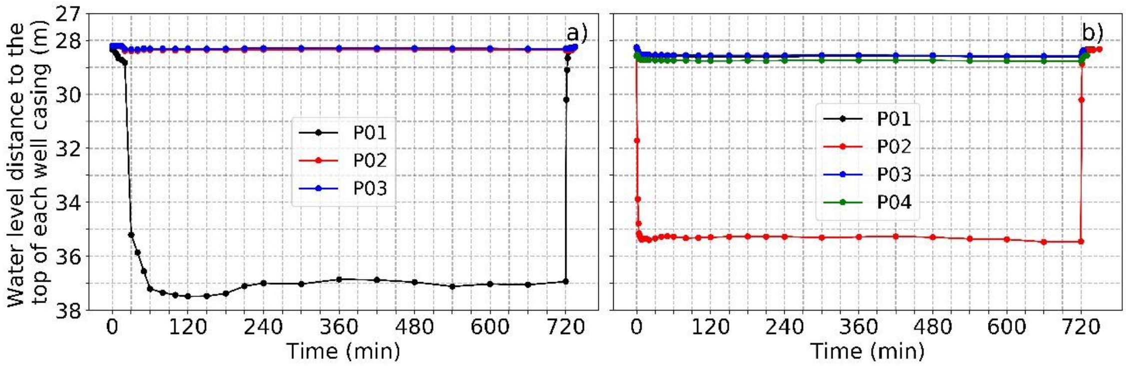

Fig. 5. Water levels monitoring during pumping test executed on well a) P01 and b) P02. Only vicinity wells recorded data gathered at the same time as data from the pumped wells are being exhibited.

The values of transmissivity and specific yield of the aquifer, obtained from the well P02 by the Cooper and Jacob method for the unconfined aquifer, were equal to 1.58×10^{-4} m²/s and 0.057, respectively. These reference values were the most reliable results obtained from the pumping test, calculated only from the drawdown measured in the pumped well, since effects in the other wells were so small that were unable to be used for parameter calculation. Better estimations would have been found if 1) a higher pumping rate were applied, or 2) the tests have lasted longer, or 3) the wells in the vicinity were closer, e.g., at least at a 1 to 5-metre distance.

Pumping tests results in the study area show that the Barreiras unconfined aquifer re-establishes equilibrium in the water level in less than ten minutes (Fig. 5). This observation is especially important since it shows that rainfall events spanning more than ten minutes apart from each other may be consistently considered independent since it is assumed that, at least when an event producing a runoff rate of roughly 10 m³/h in a dozen hours, the aquifer response due to an event will not be affected by the previous one. This also means that negligible residual increases in the water table are expected due to single rainwater inputs (hence the well control volume can be assumed as unchangeable in the whole yearly hydraulic simulations). In other words, the static water level is indeed not influenced by the recharge on the site, confirming the hypothesis (4). The well control volume is expected to change only due to the natural water level variation, which is not being considered in this study, following hypothesis (3), and not by the administered volumes of managed aquifer recharge. It is noteworthy that a sub-hourly timescale is necessary to adequately grasp this phenomenon, which would not have been observed if smaller temporal resolutions (e.g. hourly or daily) time steps were adopted.

4.1.2 Injection tests results

Fig. 6a summarizes the whole experiment of several in-series injection tests carried out at well P02. Three successive injection tests were performed, leading to three distinct curves. Each raise in the water level was induced by the pressure given by the water truck, while each drawdown was due to gravity. Between the second and the third curves, a constant pumping rate was maintained to empty the water truck. Thence, the first curve represented the aquifer behaviour before injecting any quantity of water and the third curve represented its behaviour after all water available was injected (15 m³). Despite this fact, no substantial difference between curves was observed, leading to the conclusion that these resulting curves are representative of the well recharge rate in correlation to its water level position, being these curves not dependent upon the total amount of injected water. The same behaviour was reported by Silva (2004) after successive injection tests in a consolidated aquifer. The third curve, shown in Fig. 6b, was used to build the auxiliary curve demanded by the modified Puls method (Chow 1959; Ghasemzadeh et al. 2020).

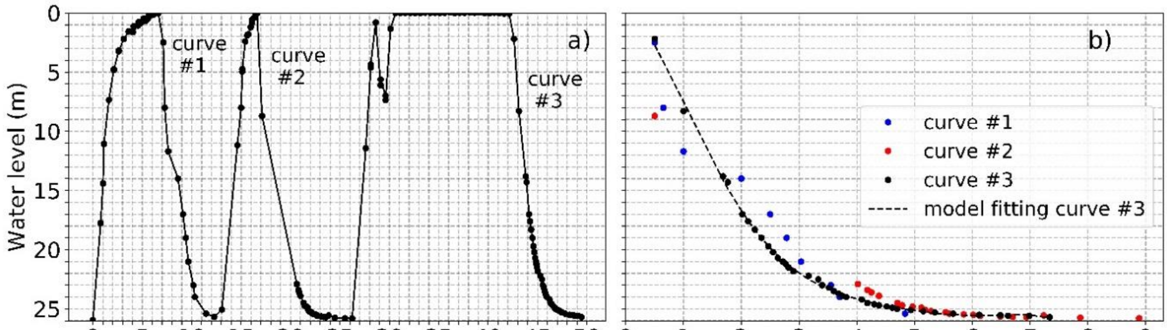

Fig. 6. Water level monitoring during injection tests at well P02: a) Whole data series, showing both water level

raise and drawdown; and b) all water level drawdown with time.

The third recharge test curve provided a relation of the water level h_w and the time t, which is strongly correlated to a sixth-degree polynomial regression (r^2 equal to 0.998), shown in Eq. (10):

$$h_w = 0.00419t^6 - 0.10724t^5 + 1.06724t^4 - 5.02966t^3 + 9.92625t^2 + 1.69354t - 0.07105 \quad (10)$$

Then, values of h_w were computed using this polynomial regression for a sequence of values of t from zero to 360 minutes, incremented by 1 minute. Eq. (9) was used to compute the values of Q_w for each pair of values in the t sequence (going through the sequence starting in the second value; picking the current value and the next). After this procedure, a relationship between Q_w (L/min) and h_w (m) was established (r^2 equal to 0.9999), shown in Eq. (11):

$$Q_w = -0.0003{h_w}^3 - 0.432{h_w}^2 + 18.799h_w - 5.2008 \quad (11)$$

In few occasions, Eq. (10) and Eq. (11) resulted in negative values. In these cases, h_w and Q_w were set as null. The maximum value of Q_w, found when h_w is maximum, at the top of the well casing, equal to 25.9 metres, is equal to 186.5 L/min (11.2 m^3/h). This recharge rate is much higher than values from other artificial recharge experiments, in both consolidated and unconsolidated aquifers (Silva et al. 2006; Händel et al. 2016; Liu et al. 2016; Conrad 2019). Albeit dependent on many factors (e.g. hydrogeology, static water level, screen length), one reason may be the higher well diameter adopted in this study (6 inches), where friction loss did not play a major role in restricting the recharge rate. Pumping test results indicate that this rate could be maintained for at least 12 hours without any substantial increase in the water level beyond at least a 10-metre distance, confirming the validity of hypothesis (5). If greater capacity is needed, the injection well P01 could also be used for recharge. As could be seen, the recharge using the injection well P02 produced an extremely low or almost null impact on the water table within the injection well P01. This means that injecting water using the injection well P02 would not produce any significant impact on the performance of the well P01, for at least 12 hours of recharge, and vice-versa.

4.2 Pre- and post-development runoff temporal series

The pre-development runoff series, whose calculation relied on results of infiltration modelled by the Horton infiltration method, resulted in null values for almost all studied period (2004 – 2019) and all rooftop catchment areas considered in this study (from 10 to 5,000 m^2). This behaviour happened due to the high values of infiltration rates (both maximum and minimum values) used on the SWMM rainfall-runoff process. In other words, the values of parameters representing the pre-development scenario were quite restrictive, hence leading to a very picky scenario in terms of surface runoff generation. For the largest catchment studied, surface runoff flow occurred only on very few occasions. Hence, the efficiency of the RWM systems was compared to a scenario of no flow for almost all the period studied, catchment areas and simulated years. In practice, this means that the rainwater efficiency *E* of the RWH systems, in these cases, also corresponds to the post-development peak runoff flow reduction.

4.3 Hydraulic simulations results

Distributions of return periods were calculated using long-term intensity-duration-frequency curve values for JPA. The rainfall dataset used in this study (2004 – 2019) rarely surpassed return periods higher than 2 years, no matter the duration considered (from 5 to 1440 minutes). In few moments, the return period calculated was around 6 – 7 years (159 – 162 mm precipitated in 24 hours); once around 9 years (20 mm precipitated in 10 minutes) and once around 13 years (11 mm precipitated in 5 minutes).

Fig. 7 shows the results of hydraulic simulations from rainwater captured by a 250 m² rooftop catchment, for selected water tanks, in a period of intense rainfall (total precipitation of 122 mm in 198 minutes, an intensity of 37 mm/h, with a return period of 5 years) started on 6-March-2015 3:00 A.M. The water balance converts the input hydrograph (post-development runoff, the area in dark blue) into two output hydrographs (aquifer recharge and tank overflow, areas in green and red, respectively). It can be seen that for intense runoff events, the capacity of the water tank plays a major role in the simulation when the input runoff is high enough not only to fill the well control volume but to substantially raise the water level within the tank. In this condition, the storage in the water tank can maintain a constant outflow rate for some time even after the input runoff has ceased. Depending on the capacity of the tank, it may not be able to store all input runoff and then overflow for some period (Fig. 7a and Fig. 7b).

Considering runoff conveyed by a 250 m² catchment (Fig. 7), overflow was observed only for the smaller tanks (0.5 and 3 m³), while it was not recorded for the other tanks studied, suggesting that these configurations were enough to restore the hydrology to the pre-development state. This fact is confirmed by analysing the distributions of peak annual overflow compared to peak runoff flow in the scenarios considered in this study (pre- and post-development, Fig. 8). Although the RWH system using a 0.5 m³ tank was able to reduce the mean peak runoff flow in the post-development scenario from 7 to 3 L/s/year (Fig. 8a), it was not enough to restore the surface runoff to values in the pre-development state (mean values less than 1 L/s/year). The whole peak annual overflow distribution was higher than this value, indicating that the peak overflow series frequently surpassed the limit desired, represented by the pre-development peak runoff flow series, particularly in cases of intense rainfall, as illustrated in Fig. 7a. Using a 3 m³ tank led to a peak annual overflow distribution more similar to the pre-development's, however with more frequent and intense outliers (Fig. 8b), as in the case displayed in Fig. 7b. Using a 5 m³ was enough to produce a null peak overflow series in the whole period studied (2004 – 2019) when considering a 250 m² rooftop. The bigger tanks (> 5 m³) resulted in recharge patterns similar than 5 m³ water tank (Fig. 7c to Fig. 7f), suggesting that these bigger tanks were an exaggeration in this case, what is confirmed in Fig. 8c to Fig. 8f.

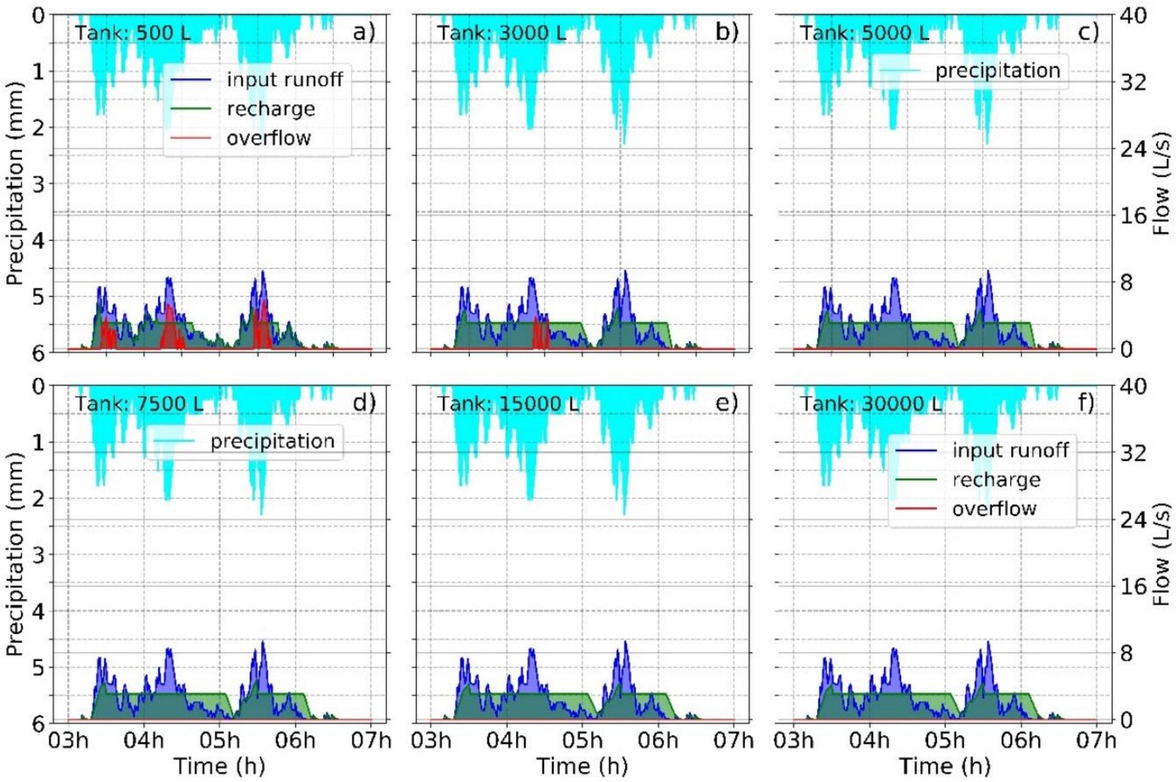

Fig. 7. Simulations of RWH systems for runoff conveyed by a 250 m² catchment and a rainfall event with intensity of 37 mm/h and a 5-year return period.

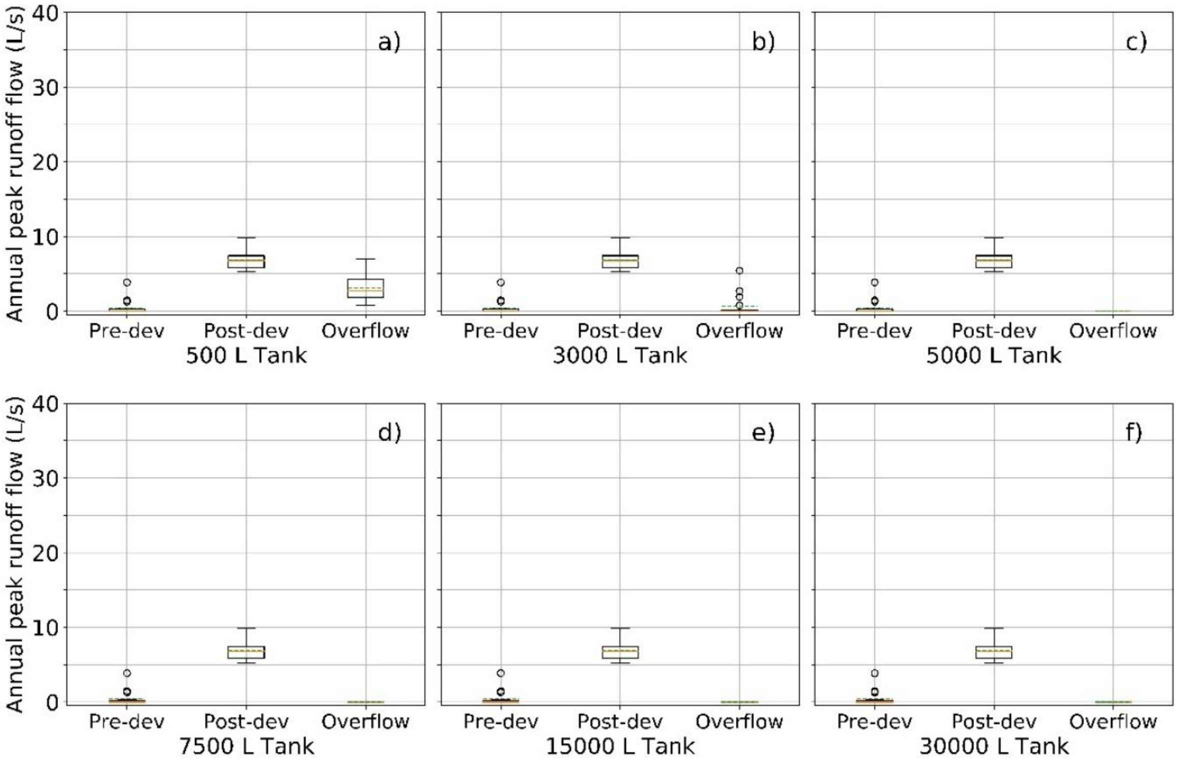

Fig. 8. Annual peak runoff flow of pre- and post-development scenarios and of annual peak overflow (L/s) considering a 250 m² catchment. Orange lines and green dashed lines stand for median and mean, respectively.

Fig. 9 and Fig. 10 are analogous to Fig. 7 and Fig. 8, respectively, but they present results of RWH systems conveying surface water runoff from a 500 m² rooftop. In this case, it can be seen that only the 30-m³ tank did not overflow in the simulation of the intense rainfall event highlighted. This is confirmed in Fig. 10f, that only the largest tank (30 m³) resulted in a null distribution of annual peak runoff flow. In all other cases (tanks smaller than 30 m³), the overflow runoff series resulted in frequent peak values much higher than the ones from the pre-development peak runoff flow series, even though a major difference is observed in the distribution of overflow with increasing water tanks. On the other hand, even the RWH system using a 0.5 m³ water tank was able to result in mean annual aquifer recharge rates close to volume conveyed by the 500-m² rooftop annualy (150 m³/year, on average).

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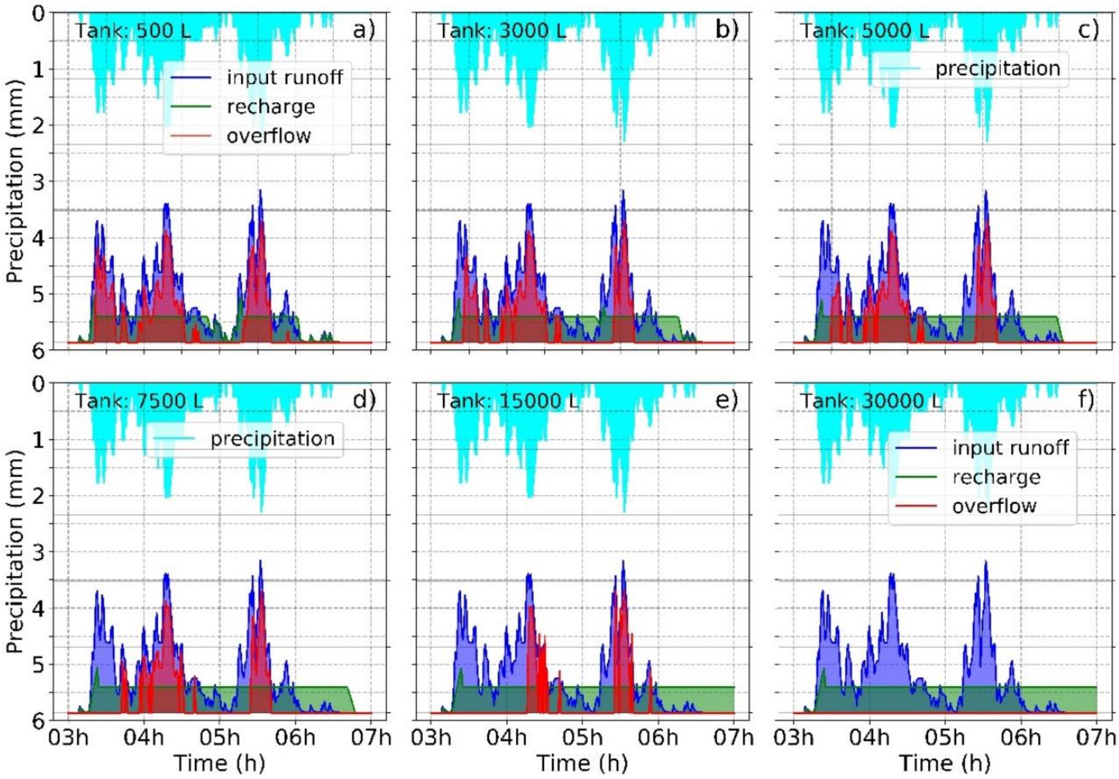

Fig. 9. Simulations of RWH systems for runoff conveyed by a 500 m² catchment and a rainfall event with intensity of 37 mm/h and a 5-year return period.

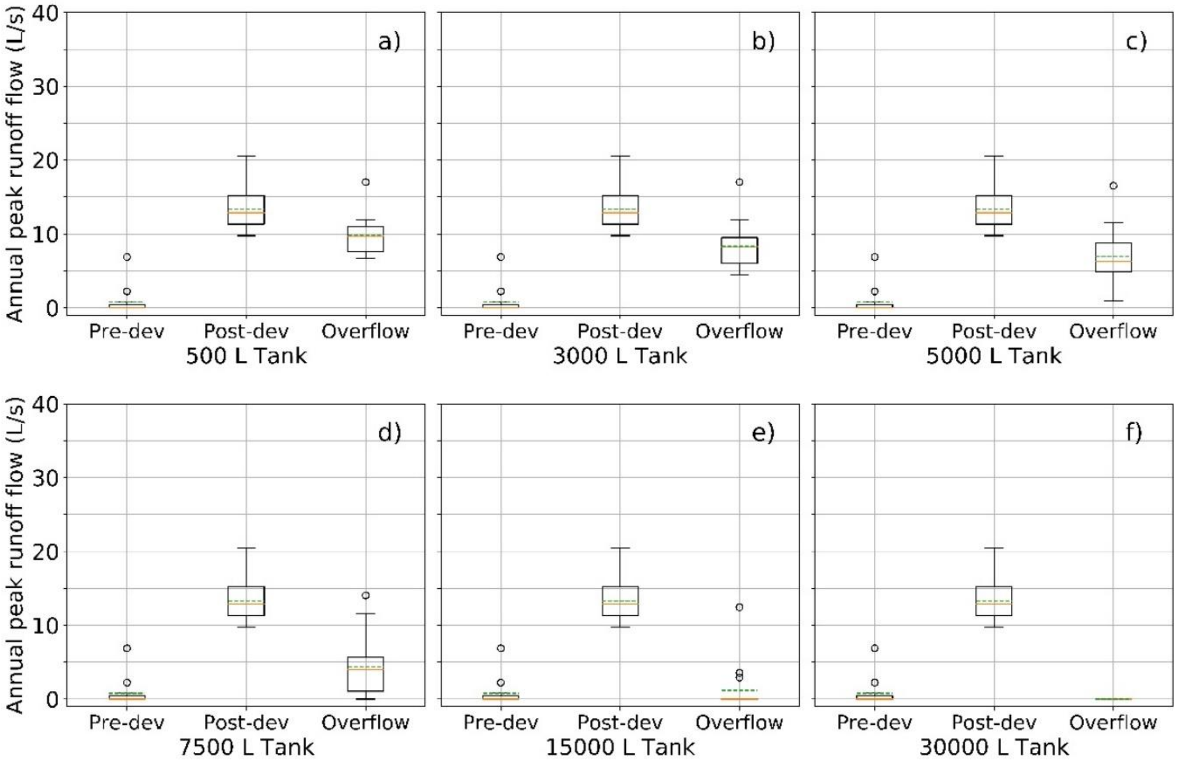

Fig. 10. Annual peak runoff flow of pre- and post-development scenarios and of overflow (L/s) considering a 500 m² catchment. Orange lines and green dashed lines stand for median and mean, respectively.

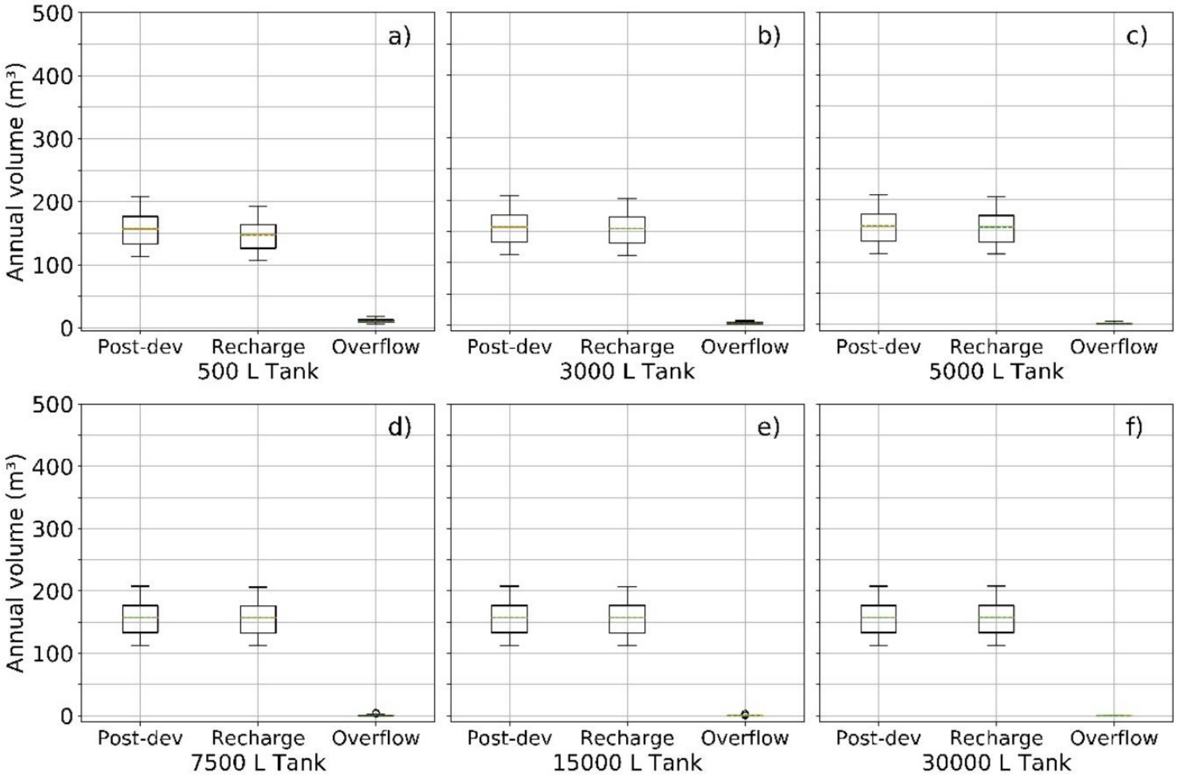

Fig. 11. Annual volumes of pre- and post-development scenarios and of annual overflow (m³) considering a 500 m² catchment. Orange lines and green dashed lines stand for median and mean, respectively.

Fig. 12, Fig. 13 and Fig. 14 are analogous to Fig. 9, Fig. 10 and Fig. 11, respectively, but they present results of RWH systems conveying surface water runoff from a 1,000 m² rooftop. In this case, it is explicit that the impact of the RWH system as a flood mitigation solution is small, since it was not able to prevent the system from overflowing following a 5-year return period rainfall event (Fig. 12). The mean annual peak overflow was similar to the post-development's (27 L/s/year) for most water tanks studied, varying from 24 to 21 L/s/year for tanks from 0.5 to 7.5 m³ (Fig. 13a to Fig. 13d). The largest tanks produced higher reductions in these means (to 18 and 8 L/s/year for the 15 and 30 m³ tanks, respectively; Fig. 13e and Fig. 13f). In any case, the overflow runoff series resulted in frequent peak values much higher than the ones from the pre-development peak runoff overflow series. On the other hand, similarly to what was observed concerning the 500 m² catchment (Fig. 11), even the RWH system using a 0.5 m³ water tank was able to result in satisfactory rates of mean annual aquifer recharge (250 m³/year) compared to the volume of rainwater conveyed by the 1,000-m² rooftop annually, on average (310 m³/year). The greater the capacity of the tank, the closer the distribution of annual aquifer recharge was to the annual volume of surface runoff in the post-development scenario. It is noteworthy that these configurations could work properly to capture rainwater from a 1,000 m² catchment only if flood control is negligible. If flood control is crucial, here it would be the case to expand the injection capacity, e.g. by increasing the number of injection wells and/or increasing the screen length, since further enlarging the capacity of the tank would be too prohibitive.

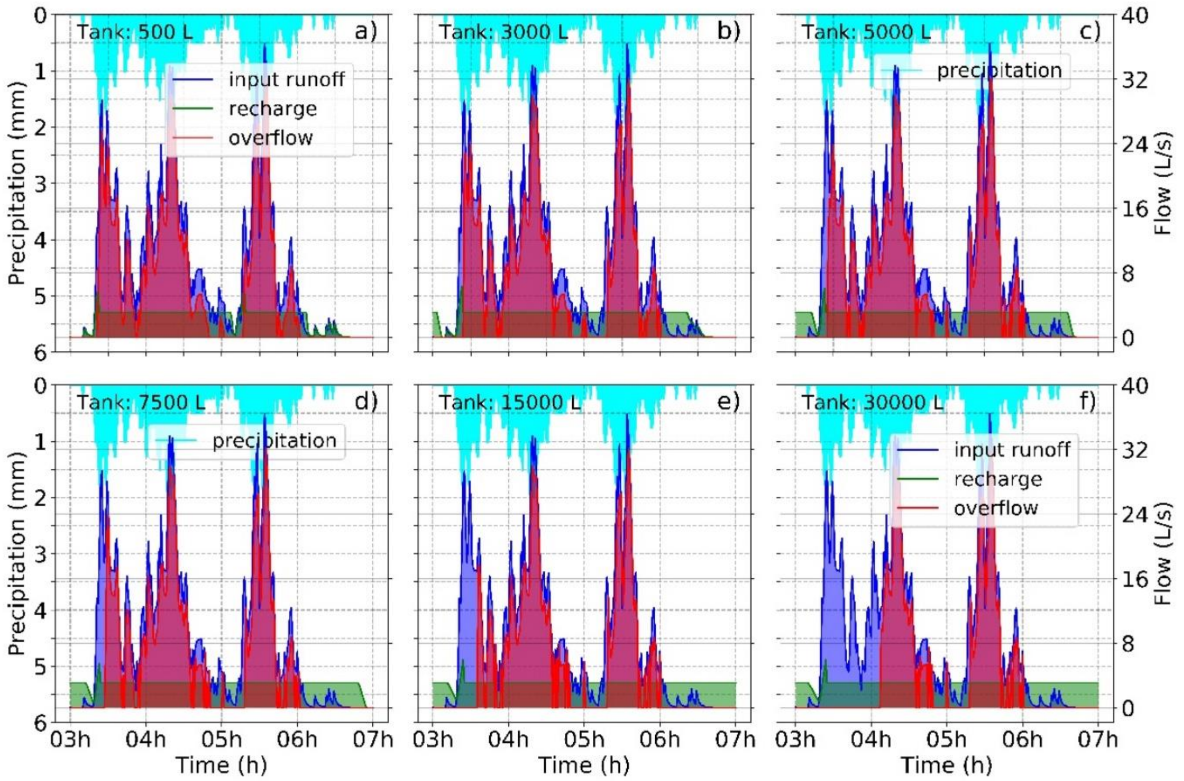

Fig. 12. Simulations of RWH systems for runoff conveyed by a 1,000 m² catchment and a rainfall event with intensity of 37 mm/h and a 5-year return period.

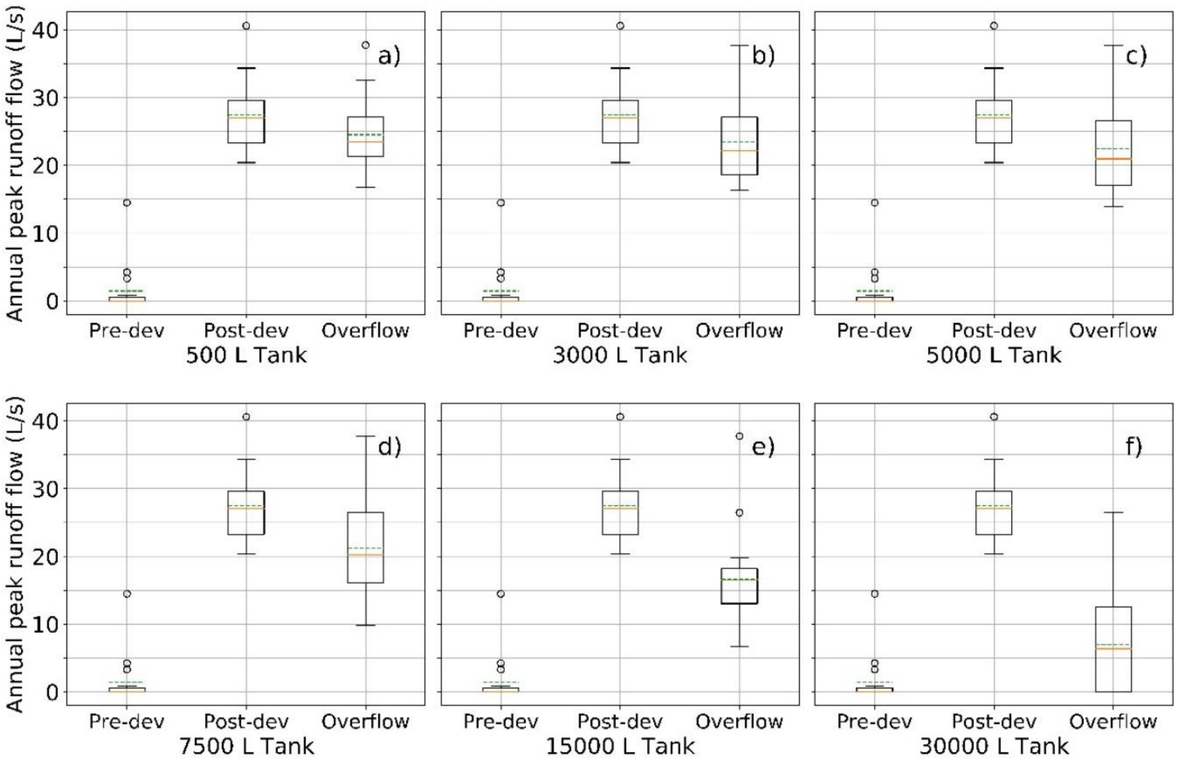

Fig. 13. Annual peak runoff flow of pre- and post-development scenarios and of overflow (L/s) considering a 1,000 m² catchment. Orange lines and green dashed lines stand for median and mean, respectively.

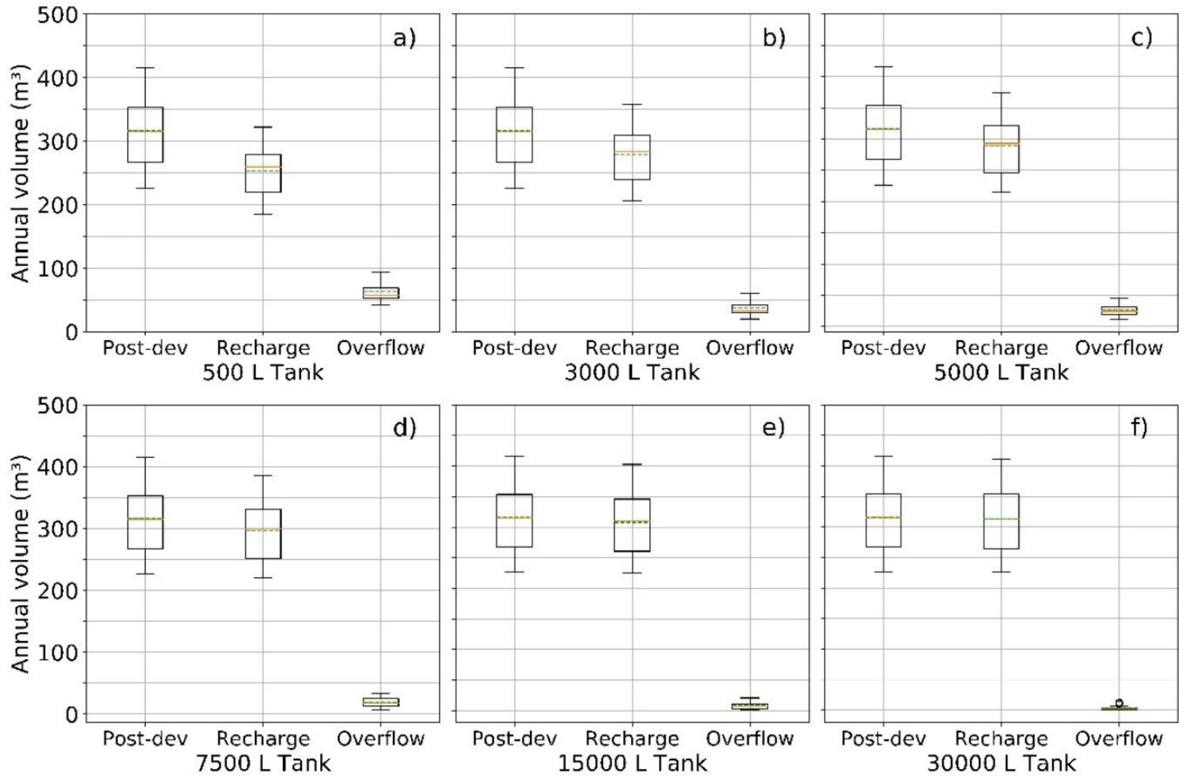

Fig. 14. Annual volumes of pre- and post-development scenarios and of annual overflow (m^3) considering a 1,000 m^2 catchment. Orange lines and green dashed lines stand for median and mean, respectively.

Fig. 15 shows contour plots of the mean annual aquifer recharge and the difference between simulated aquifer recharge and tank overflow (m^3/year) in the function of water tank capacity (0.5 – 30 m^3) and rooftop catchment area (10 – 5,000 m^2). Each combination of the water tank and rooftop catchment provided a unique point in the plot and the whole set was used to interpolate values in the contour plots. It can be seen that each water tank had a unique catchment area which maximized the mean annual difference, thus represented with asterisks. Fig. 15a displays the mean annual volumes of aquifer recharge provided by the RWH systems. For the largest catchments, the water tank capacity played a major role in the proportion of rainwater that is annually retained for aquifer recharge, being this influence attenuated with increasing tank capacities. This fact has been observed in distributions for the 1,000-m^2 catchment (Fig. 14), a behaviour that is not as sensitive for smaller catchments (Fig. 11).

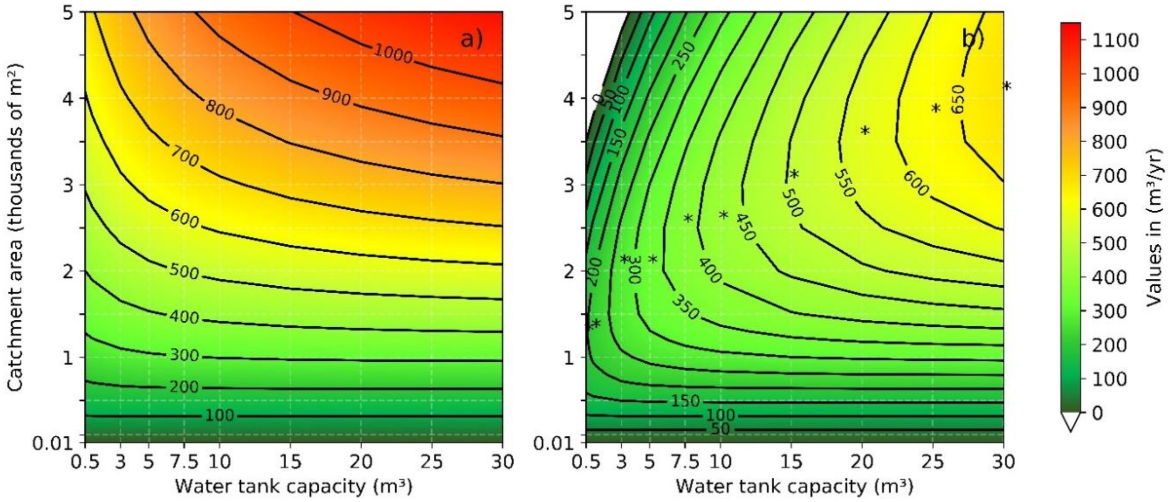

Fig. 15. Mean annual a) recharge and b) difference between recharge and overflow volumes as function of the water tank capacity and rooftop catchment area. The * symbol stands maximized differences of each tank.

Mean annual post-development runoff and overflow volumes, from each combination of the rooftop catchment area and water tank, were used to calculate the rainwater harvesting retention *R* (Fig. 16a). Mean annual peak flows (pre-development, post-development, and overflow) were used to calculate the rainwater harvesting efficiency *E* (Fig. 16b).

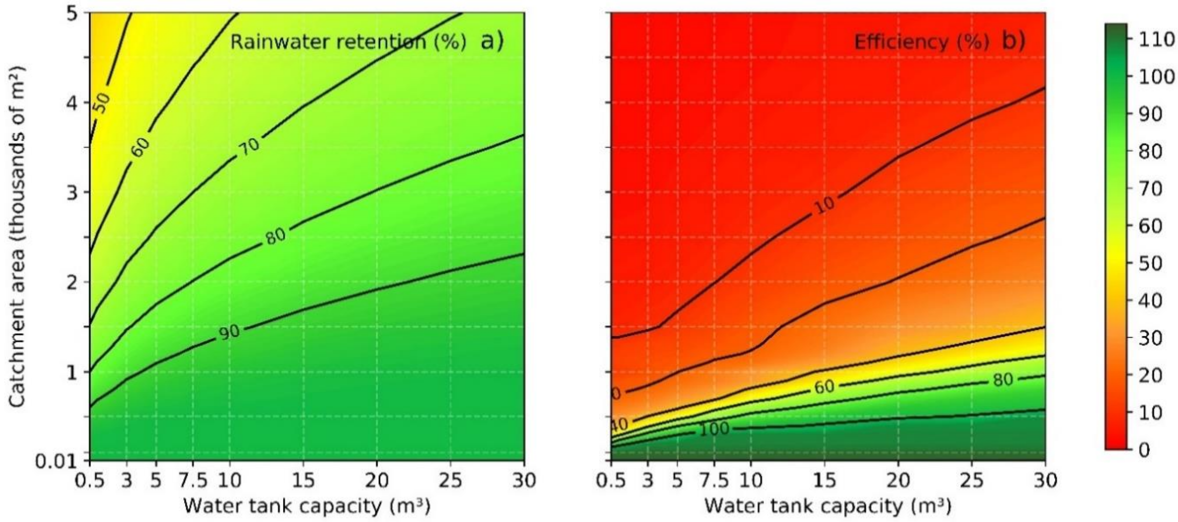

Fig. 16. Contour plots of a) rainwater retention *R* and b) efficiency *E* in the function of water tank capacity and rooftop catchment area.

The mean annual post-development runoff has a linear relationship with the rooftop catchment area, while mean annual tank overflow and aquifer recharge relationships are asymptotic (Fig. 17). The mean annual aquifer recharge volume tends to fit an almost horizontal asymptote when the rooftop catchment area tends to +∞, while the mean annual tank overflow tends to fit a line parallel to the line that corresponds to the mean post-development runoff. In all cases, both curves intersect at some point that represents the combination of the rooftop catchment area and interim storage which produces an equal division of rainwater, on average: half recharges the unconfined aquifer and half overflows to the current drainage system. If flood control is not among the goal of an RWH system, then the optimal value of the catchment area is found below the one from the half-to-half division, when the volume difference of recharge to overflow is maximized, depending on the interim storage capacity. This is best visualized in Fig. 15b. The values of rainwater retention *R* for these combinations of water tanks and catchment areas lies around 75% (Fig. 16a). Further increasing the catchment area, for any water tank, is not interesting since it will not further significantly increase the annual recharge amount (in fact, this would reduce the mean annual value of rainwater retention *R*, as stated in Fig. 16a).

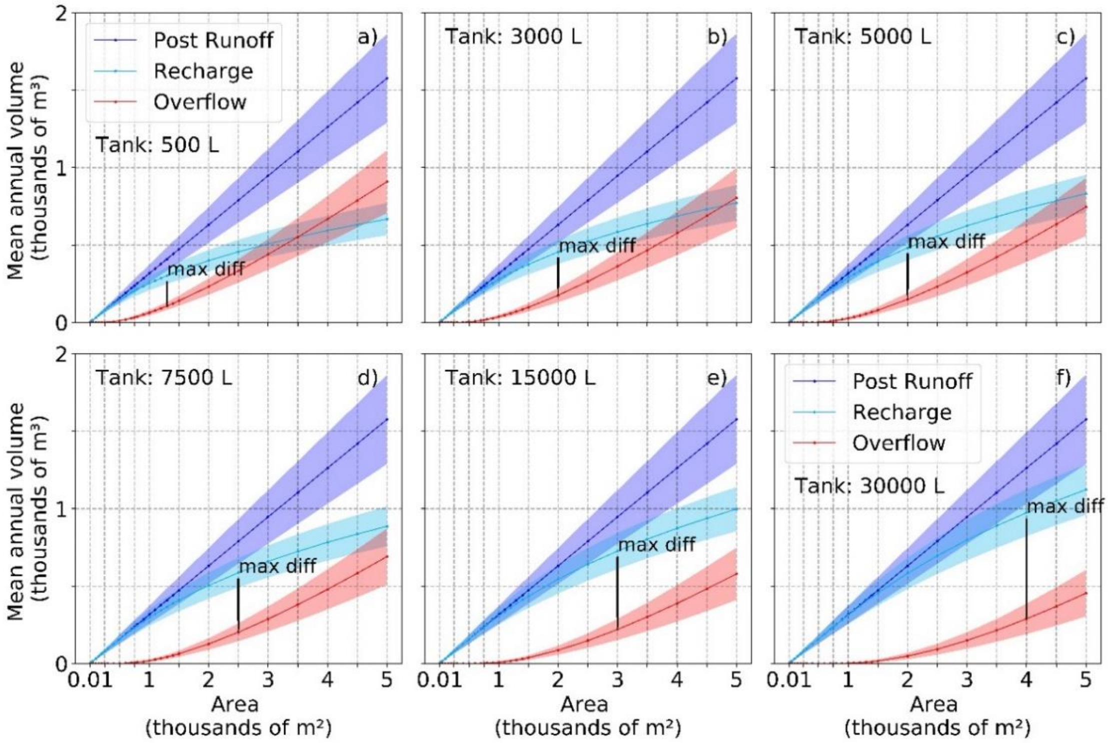

Fig. 17. Mean annual volume of recharge, overflow, and post-development runoff as function of the water tank capacity and rooftop catchment area.

The mean annual peak pre- and post-development runoff flows have a linear relationship with the rooftop catchment area as well (Fig. 18), however, values of mean annual peak overflow tended to reach close to post-development's with much smaller catchment areas for any tank studied. This shows that when stormwater management is the focus of an RWH system, the catchment area needs to be reduced, for a given water tank, in comparison to a situation when flood control is not being attempted. For example, Fig. 17**Erro! Fonte de referência não encontrada.**b shows a 3 m³ tank has a maximized annual difference between recharge and overflow conveying rainwater from a 2,000 m² rooftop whereas in this case, the mean annual peak overflow will be close to the post-development's. To provide reliable stormwater management, the catchment area would need to be reduced to 200 m² (Fig. 18b).

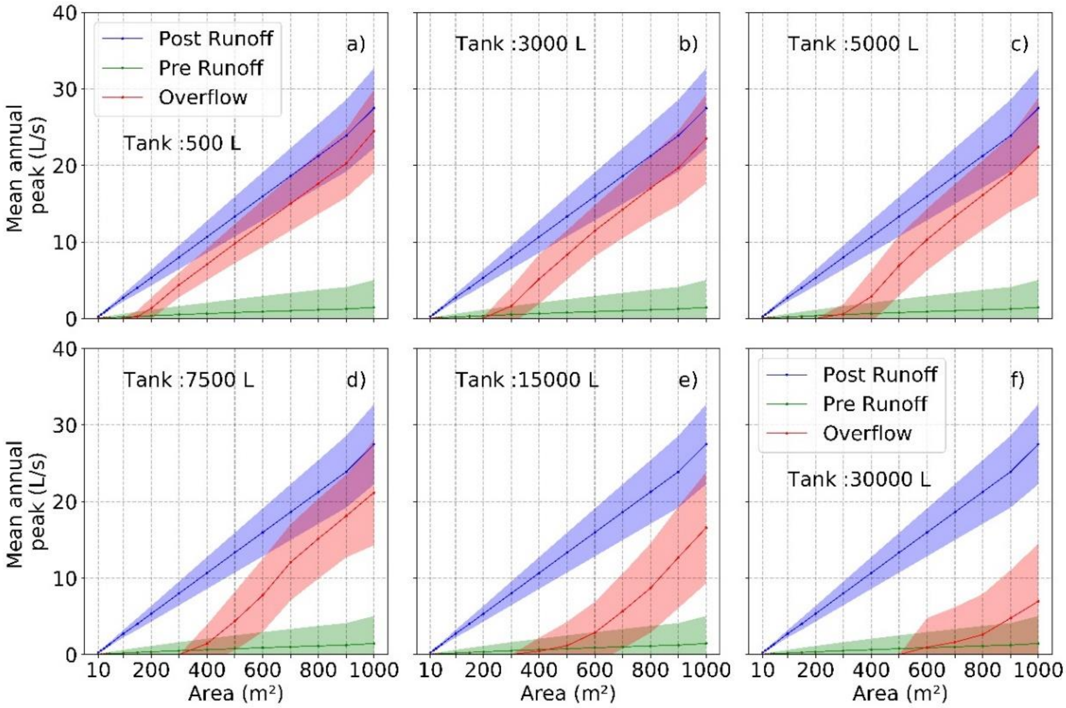

Fig. 18. Mean annual peak pre- and post-development runoff flow and peak overflow (L/s/year) in the function of the water tank capacity and rooftop catchment area.

Table 5 shows the optimal catchment area for each water tank used in this study when flood control is not being attempted. On the other hand, in terms of flood control, optimal rooftop catchment areas according to specific interim storage volumes are much more restricted than when the overflow amount to existing drainage systems can be neglected. This is true because what matters concerning flood control is not only the total rainfall volume that overflows but its distribution with time and thus the impact in reducing the peak runoff flow. In general, for the RWH system to work properly, when flood control is important, the peak tank overflow should not surpass the peak pre-development runoff flow, which represents the natural drainage condition before the implementation of the urban development. Fig. 16a provides values of optimal catchment areas when efficiency is optimized (efficiency *E* of 100%) for the collection of water tanks used in this study. These catchment areas represent the limit of the RWH system when flood control is being prioritized (Table 5).

Water tank (m³)	Optimal catchment for MAR with flood control (m²)	Optimal catchment for MAR without flood control (m²)
0.5	158	1,230

1	174	1,271
3	234	2,018
5	290	2,017
7.5	348	2,492
10	369	2,531
15	400	3,001
20	477	3,504
25	510	3,768
30	571	4,026

Table 5. Optimal catchment areas of the RWH system, with and without flood control

When analysing the behaviour of the metrics used in this study, one can conclude the region where the rainwater retention *R* is found within the 90%-100% interval (Fig. 16a) is the same region where the efficiency *E* face a much stronger gradient: it varies from 110% to 20% (Fig. 16b). Then, only the first 10% decrease in the rainwater retention (thus an increase from nil to 10% in overflow) is responsible for dramatically reducing the efficiency of the RWH system in mitigating flooding. In other words, any overflow in the system tends to rapidly-produce floods in such a magnitude that the system will not be able to control it. For a given catchment area whose potential runoff is meant to be destined for an RWH system, its interim storage volume will depend on the MAR system objective. If flood control can be neglected, relatively small interim storage tanks will be able to produce satisfactory rainwater retention. If flood control is important, larger tanks will be demanded.

The initial investment costs of the RWH systems were compared to the cost of standard buildings with optimal catchment areas concerning each water tank capacity, for MAR with and without flood control (Table 5). The methodology for computing the costs is described in section 3.5. The optimal catchment areas for MAR with flood control, which ranged from 158 to 571 m² in the function of the water tanks, were assumed to be from high standard single-family residences (code R1-A, with a basic unit cost of 301.02 USD per m²). On the other hand, the optimal catchment areas for MAR without flood control, which ranged from 1,230 to 4,026 m², were assumed to be from low standard popular buildings (tanks: 0.5 to 5 m³; code PP-B, with a basic unit cost of 186.50 USD per m²), normal standard popular buildings (tanks: 7.5 and 10 m³; code PP-N, with a basic unit cost of 227.18 USD per m²), and from normal standard multi-family residences (tanks: 15 to 30 m³; code R8-N, with a basic unit cost of 199.18 USD per m²). Fig. 19 shows the relative costs of each RWH system concerning their optimal catchment areas for each water tank, for MAR with and without flood control.

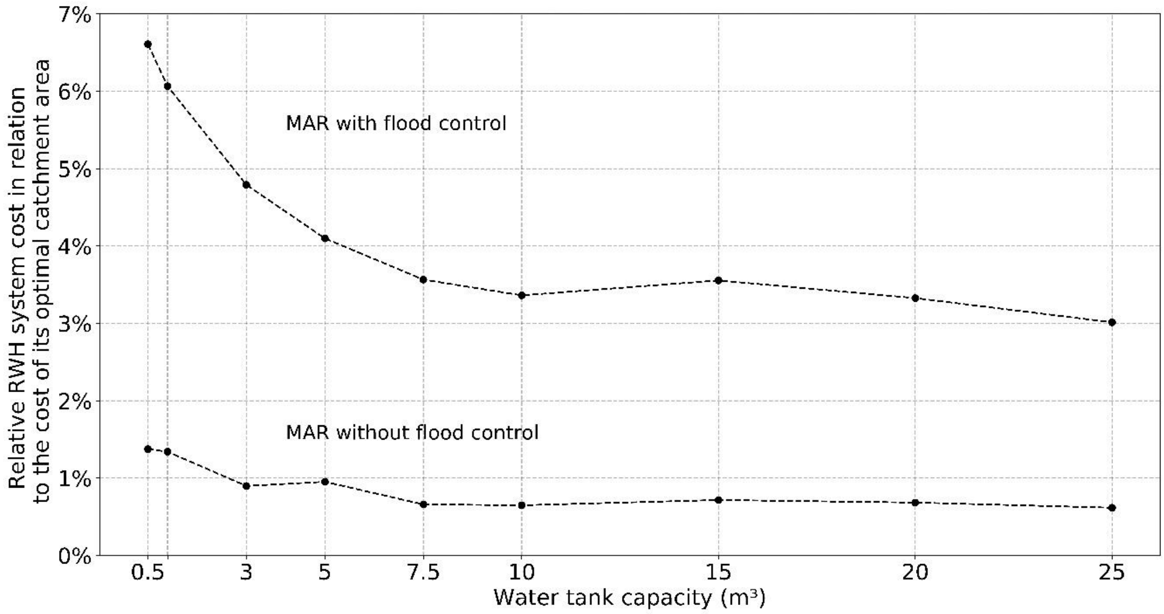

Fig. 19. RWH systems relative costs concerning their optimal catchment areas for each water tank capacity, for both objectives contemplated in this study.

One can state that the relative costs of the RWH systems, for MAR without flood control, lied around 1%, no matter the water tank being considered, concerning its optimal catchment area. On the other hand, in comparison, the costs of MAR with flood control were quite higher, being further high for the smaller water tanks. For the larger tank capacities, the costs of the RWH systems for MAR with flood control were around 3.5% of the cost of their optimal catchment. This means that for an RWH system with any given water tank capacity, the optimal catchment area is much more restricted when flood control is being attempted and therefore the system's cost will share a larger amount of the total cost, including the catchment area's. Other expenses concerning the RWH system are not being considered here (pre-treatment, gutters and downspouts adaption, the structure supporting the water tank, pipeline connecting it to the injection well, etc.) hence the percentages shown in Fig. 19 are expected to be higher in the case of employment of an RWH system in the study area.

Considering the largest rooftop catchment available in the study area (~580 m², from the Hydraulics Laboratory), results from this study point out that a 0.5 m³ tank would be enough to recharge, on average, 90% of the annual rainfall that is conveyed by the given rooftop. However, the remaining 10%, not retained, includes all extreme rainfall events, representing the share that most impacts in terms of flooding, leading the RWH system with a 0.5 m³ tank to perform poorly, on average, with an efficiency around 30%. For sustainable stormwater management, the RWH system connected to the Hydraulics Laboratory must have interim storage higher than 30 m³, which would lead, on average, to both rainwater harvesting retention and efficiency close to 100%. On the other hand, if only half of the given rooftop is connected to the RWH system (~290 m²), a 5 m³ tank would be enough to recharge, on average, 100% of the annual rainfall and provide an efficiency of 100% in terms of stormwater management. In this case, the relative costs of the RWH system (3,580.66 USD, from Table 3, in addition to the assumed cost of the injection well) would be around 4% the cost of a typically high standard single-family residence in João Pessoa, costing around 87,296.91 USD (code R1-A, from Table 4). This example illustrates how stormwater management affects the managed aquifer recharge scheme. Particularly, there is a strong negative gradient in efficiency values of the RWH system when the rooftop catchment area is raised from 10 m² up to 1,500 m² (Fig. 16b), which is not followed by the rainfall retention (Fig. 16a). Hence, prioritizing the stormwater management component of a proposed RWH system has the benefit of maximizing its aquifer recharge potential, without leading to a substantial increase in the costs of the RWH system.

5. Conclusions

This paper evaluated the technical feasibility of rooftop rainwater harvesting (RWH) systems as a tool for managed aquifer recharge aimed at sustainable stormwater management in the João Pessoa city (Paraíba, Brazil). It was demonstrated that the RWH system can efficiently 1) detain surface water runoff, 2) dampen its peak values, and 3) reduce the volume that would, otherwise, be directed into downstream drainage network whereas enhancing groundwater recharge.

Results from this study require validation based on empirical, real-time monitoring, even though the level of reliability provided by the results (high-temporal resolutions, based on *in situ* injection tests) are expected to provide initial guidelines on the dimension of a proposed RWH pilot system in the study area – as is being currently studied in the scope of the SMART-Control project. Although not considered in this study, water quality aspects related to the rainwater and groundwater should not be neglected – these may affect in the long-term performance of the RWH systems if clogging mechanisms are triggered by injection. Albeit the risks of clogging and aquifer contamination in the long-term are expectedly low, preventive measures to minimize them are required.

Improvements to the proposed RWH systems can be achieved in at least three ways. The former alternative is by studying MAR schemes for integrated water conservation and stormwater management. For this matter, both passive and active RWH systems require further study, but passive systems are expected to offer less resistance by householders since they can be designed based solely on gravity. Another viable alternative is by recharging rainwater in multiple injection wells simultaneously aiming at increasing the capacity of the system. The experimental tests have shown that it is unlikely that the recharge rate in the injection well P02 will be impaired if another recharge occurs in the vicinity at least at a 10-metre distance. The latter alternative suggested is by carrying hypothetical and empirical research combining the proposed RWH systems with green roofs, since these have been extensively studied and are useful for stormwater peak runoff flow reduction. Moreover, the usage of green roofs can provide other benefits, such as reduced energy consumption, reduced heat island effect, reduced dioxide carbon emissions, improved air quality and landscape, etc. hence making the RWH system more attractive to householders and the public. All abovementioned suggestions for improvement may provide increases in the optimal areas for MAR with and without flood control (Table 5).

Overall, this study demonstrated the potential of implementing urban MAR systems for flood control. This is a promising alternative to enhance the groundwater supply whereas mitigating flooding and its impacts. It was made clear in this study that stormwater management requirements tend to be a restrictive component of a MAR scheme, but also an incentive towards MAR implementation in urban areas that endure frequent flooding events. Besides, the stormwater management component tends to maximize the aquifer recharge potential. Sub-daily records (rainfall and field investigation data) were crucial to this study, enabling the conclusion that even unexpected water tank volumes (e.g. 0.5 m³) were able to provide great rainwater retention estimates (90% of annual rainfall) in the study area, a fact that would not be apprehended by common sense.

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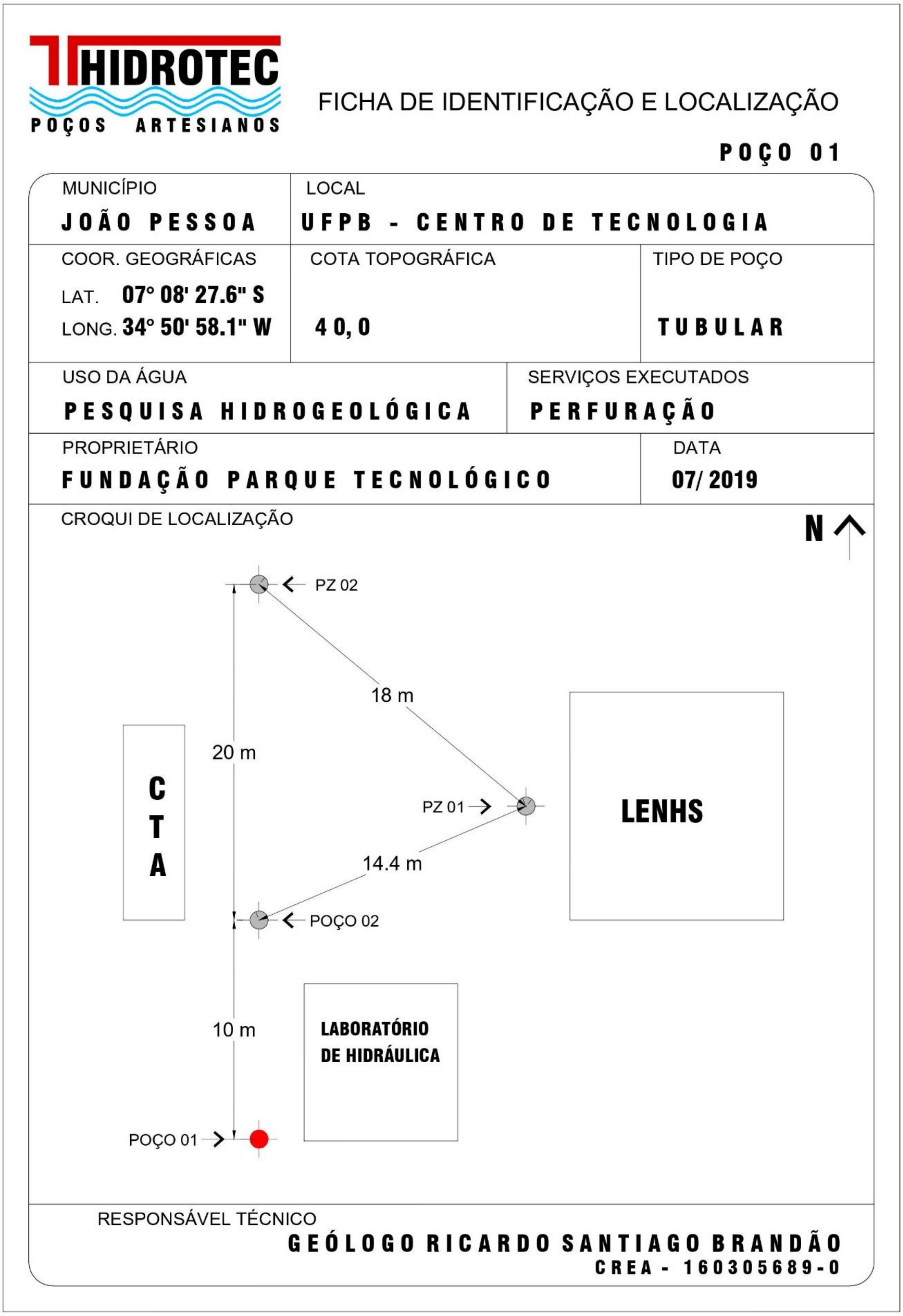
ANNEXE B

Drilling report concerning wells drilled at the study area (page 1/10) (ABNT 2006)



HIDROTEC
POÇOS ARTESIANOS

FICHA DE IDENTIFICAÇÃO E LOCALIZAÇÃO

POÇO 01

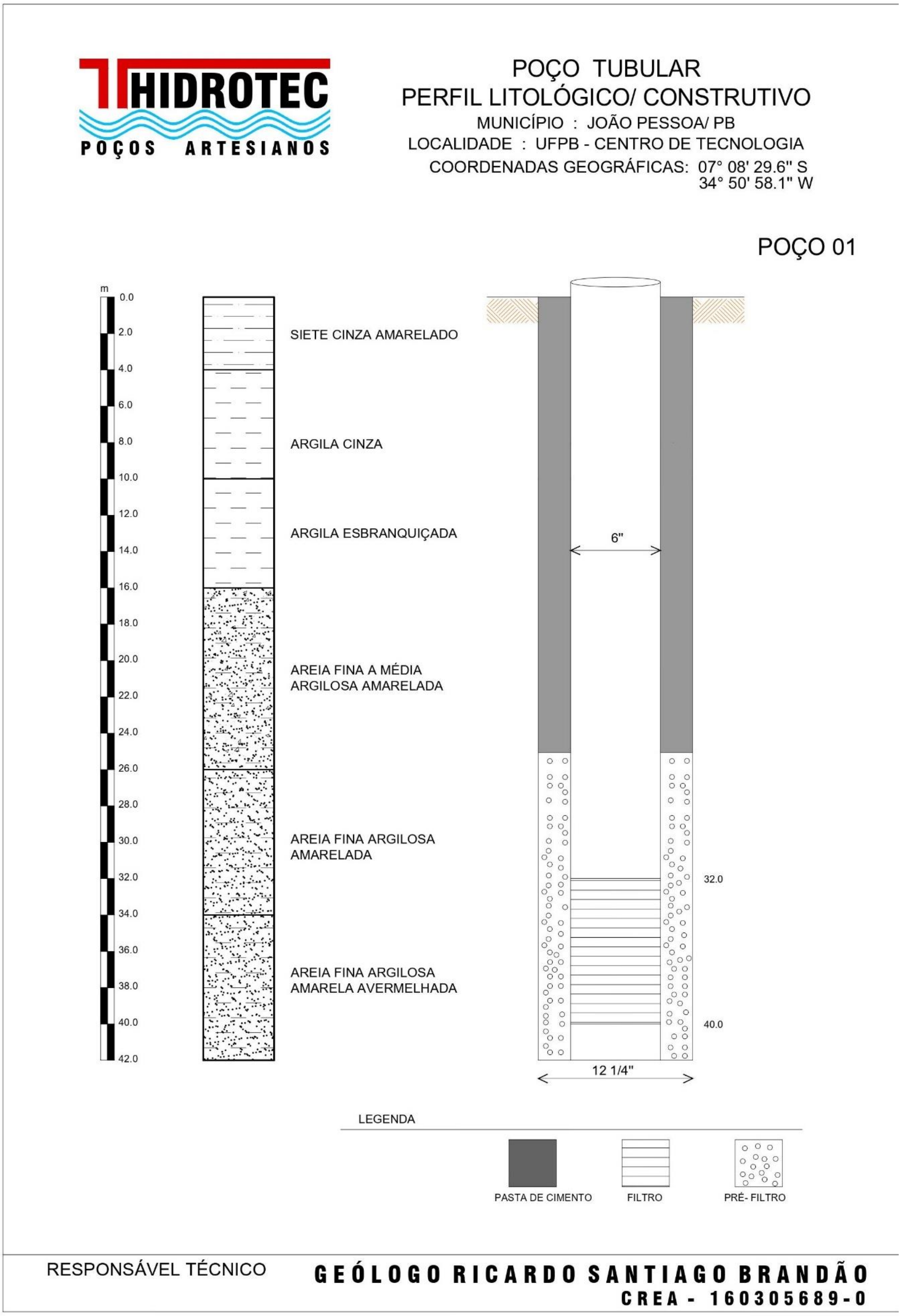
MUNICÍPIO	LOCAL	
JOÃO PESSOA	UFPB - CENTRO DE TECNOLOGIA	
COOR. GEOGRÁFICAS	COTA TOPOGRÁFICA	TIPO DE POÇO
LAT. 07° 08' 27.6" S LONG. 34° 50' 58.1" W	40,0	TUBULAR
USO DA ÁGUA	SERVIÇOS EXECUTADOS	
PESQUISA HIDROGEOLÓGICA	PERFURAÇÃO	
PROPRIETÁRIO	DATA	
FUNDAÇÃO PARQUE TECNOLÓGICO	07/ 2019	

CROQUI DE LOCALIZAÇÃO

N
PZ 02
18 m
20 m
CTA
PZ 01
LENHS
14.4 m
POÇO 02
10 m
LABORATÓRIO DE HIDRÁULICA
POÇO 01

RESPONSÁVEL TÉCNICO
GEÓLOGO RICARDO SANTIAGO BRANDÃO
CREA - 160305689-0

Drilling report concerning wells drilled at the study area (page 2/10) (ABNT 2006)



Drilling report concerning wells drilled at the study area (page 3/10) (ABNT 2006)

HIDROTEC
DESDE 1989
(83)3244-2135

TESTE DE PRODUÇÃO

UFPB POÇO 01

POÇO TIPO: TUBULAR	PROF: 42,00 m	EQUIPAMENTO: BOMBA SUBMERSA	
MUNICÍPIO: JOÃO PESSOA	CRIVO: 40,00 m		
LOCAL:UFPB - C.T	NE: 28,20 m	INÍCIO:	24/05/2019
	ND: 36,90 m		
TB: 12:00 HORAS	Q: 6,20 m^3/h	TÉRMINO:	24/05/2019

REBAIXAMENTO						RECUPERAÇÃO			
HORA	T (min)	ND (m)	S (m)	Q (m^3 / h)	Q/S (m^3 / h m)	T'	ND	S	Tb/T' + 1
07:01	1	31,600	3,40	6,43	1,89	1	30,190	1,99	721,00
07:02	2	33,300	5,10	6,43	1,26	2	29,130	0,93	361,00
07:03	3	34,180	5,98	6,43	1,08	3	28,870	0,67	241,00
07:04	4	34,700	6,50	6,43	0,99	4	28,740	0,54	181,00
07:05	5	35,000	6,80	6,43	0,95	5	28,600	0,40	145,00
07:06	6	35,200	7,00	6,43	0,92	6	28,520	0,32	121,00
07:08	8	35,470	7,27	6,43	0,88	8	28,470	0,27	91,00
07:10	10	35,520	7,32	6,43	0,88	10	28,420	0,22	73,00
07:15	15	35,580	7,38	6,20	0,84	15	28,350	0,15	49,00
07:20	20	35,670	7,47	6,20	0,83	20	28,320	0,12	37,00
07:30	30	35,820	7,62	6,20	0,81	30	28,280	0,08	25,00
07:40	40	35,930	7,73	6,20	0,80	40	28,250	0,05	19,00
07:50	50	36,030	7,83	6,20	0,79	50	28,240	0,04	15,40
08:00	60	36,080	7,88	6,20	0,79	60	28,230	0,03	13,00
08:20	80	36,190	7,99	6,20	0,78	80	28,220	0,02	10,00
08:40	100	36,290	8,09	6,20	0,77	100	28,210	0,01	8,20
09:00	120	36,330	8,13	6,20	0,76	120	28,200	-	7,00
09:30	150	36,410	8,21	6,20	0,76				
10:00	180	36,500	8,30	6,20	0,75				
11:00	240	36,600	8,40	6,20	0,74				
12:00	300	36,650	8,45	6,20	0,73				
13:00	360	36,630	8,43	6,20	0,74				
14:00	420	36,610	8,41	6,20	0,74				
15:00	480	36,800	8,60	6,20	0,72				
16:00	540	36,830	8,63	6,20	0,72				
17:00	600	36,920	8,72	6,20	0,71				
19:00	720	36,900	8,70	6,20	0,71				
	840								
	960								
	1080								
	1200								
	1320								
	1440								

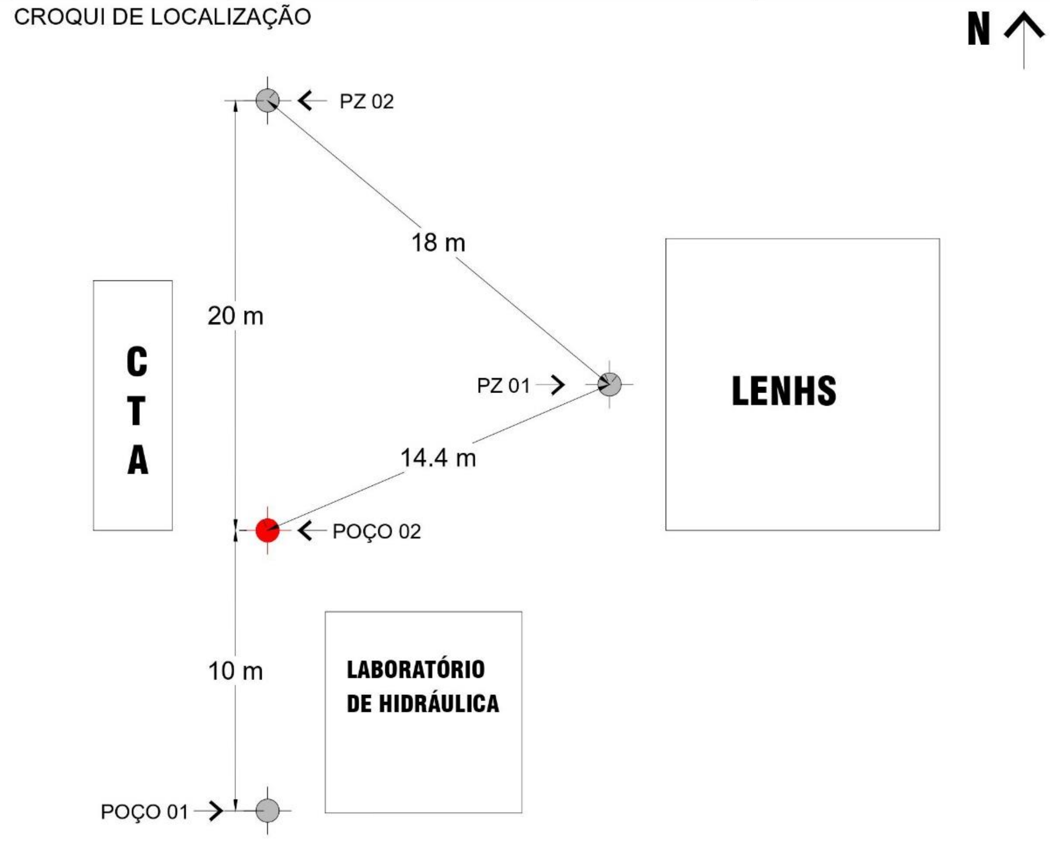
HIDROTEC
POÇOS ARTESIANOS

FICHA DE IDENTIFICAÇÃO E LOCALIZAÇÃO

POÇO 02

MUNICÍPIO	LOCAL	
JOÃO PESSOA	**UFPB - CENTRO DE TECNOLOGIA**	
COOR. GEOGRÁFICAS	COTA TOPOGRÁFICA	TIPO DE POÇO
LAT. **07° 08' 30.8" S** LONG. **34° 50' 58.4" W**	**40,0**	**TUBULAR**
USO DA ÁGUA	SERVIÇOS EXECUTADOS	
PESQUISA HIDROGEOLÓGICA	**PERFURAÇÃO**	
PROPRIETÁRIO	DATA	
FUNDAÇÃO PARQUE TECNOLÓGICO	**07/ 2019**	

CROQUI DE LOCALIZAÇÃO

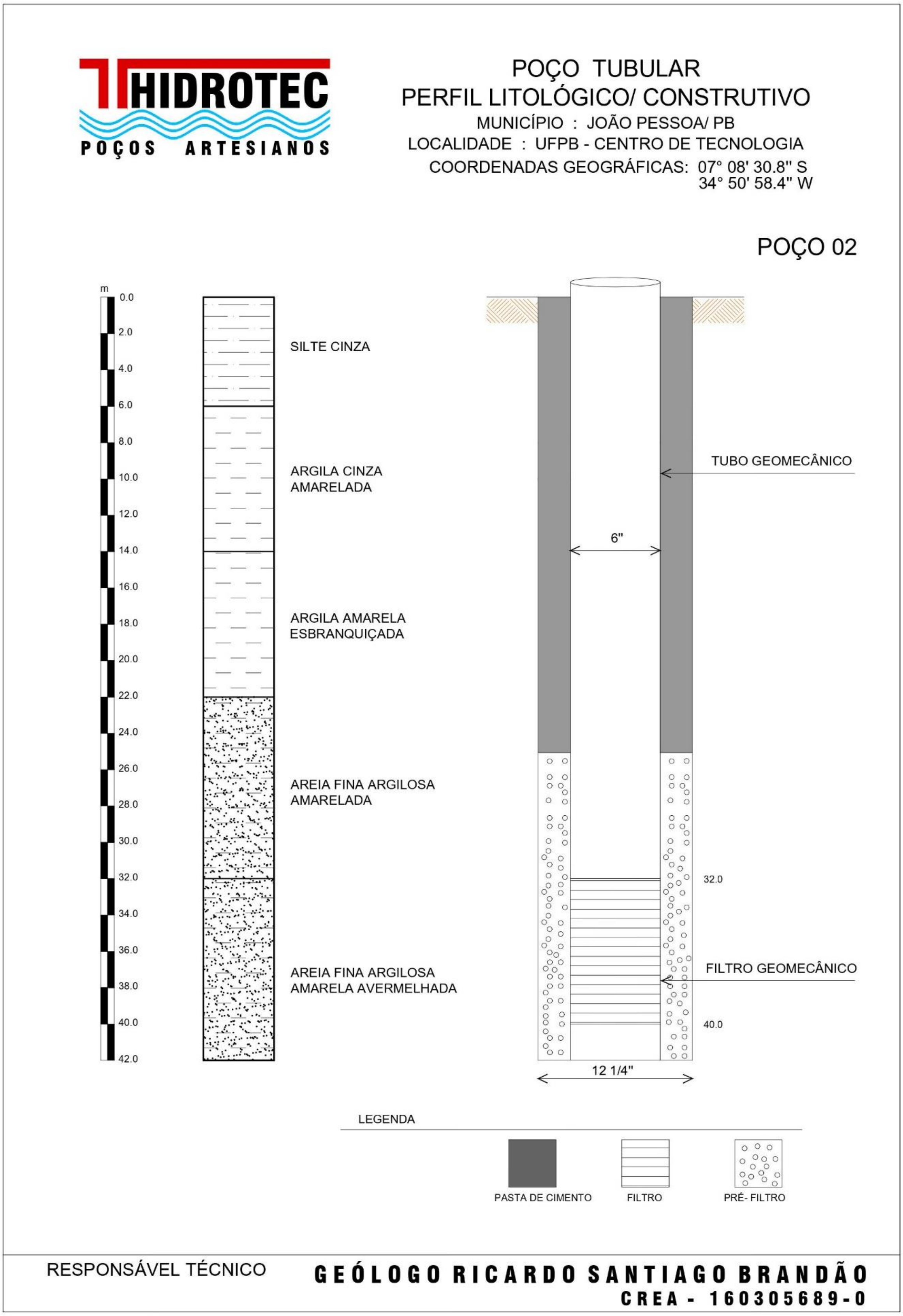


RESPONSÁVEL TÉCNICO

GEÓLOGO RICARDO SANTIAGO BRANDÃO

CREA - 160305689-0

Drilling report concerning wells drilled at the study area (page 5/10) (ABNT 2006)



Drilling report concerning wells drilled at the study area (page 6/10) (ABNT 2006)

HIDROTEC
DESDE 1989
(83)3244-2135

TESTE DE PRODUÇÃO

UFPB POÇO 02

POÇO TIPO: TUBULAR	PROF:	42,00 m	EQUIPAMENTO: BOMBA SUBMERSA		
MUNICÍPIO: JOÃO PESSOA	CRIVO:	40,00 m			
LOCAL:UFPB - C.T	NE:	28,30 m	INÍCIO:	23/05/2019	
	ND:	35,45 m			
TB: 12:00 HORAS	Q:	10,91 m^3/h	TÉRMINO:	23/05/2019	

REBAIXAMENTO						RECUPERAÇÃO			
HORA	T (min)	ND (m)	S (m)	Q (m^3 / h)	Q/S (m^3 / h m)	T'	ND	S	Tb/T' + 1
07:01	1	31,230	2,93	11,25	3,84	1	30,210	1,91	721,00
07:02	2	33,830	5,53	11,25	2,03	2	28,870	0,57	361,00
07:03	3	34,500	6,20	11,25	1,81	3	28,630	0,33	241,00
07:04	4	34,860	6,56	11,25	1,71	4	28,500	0,20	181,00
07:05	5	35,150	6,85	11,25	1,64	5	28,420	0,12	145,00
07:06	6	35,210	6,91	10,91	1,58	6	28,400	0,10	121,00
07:08	8	35,340	7,04	10,91	1,55	8	28,390	0,09	91,00
07:10	10	35,350	7,05	10,91	1,55	10	28,380	0,08	73,00
07:15	15	35,340	7,04	10,91	1,55	15	28,370	0,07	49,00
07:20	20	35,370	7,07	10,91	1,54	20	28,360	0,06	37,00
07:30	30	35,360	7,06	10,91	1,55	30	28,350	0,05	25,00
07:40	40	35,380	7,08	10,91	1,54	40	28,340	0,04	19,00
07:50	50	35,390	7,09	10,91	1,54	50	28,330	0,03	15,40
08:00	60	35,400	7,10	10,91	1,54	60	28,320	0,02	13,00
08:20	80	35,380	7,08	10,91	1,54	80	28,310	0,01	10,00
08:40	100	35,370	7,07	10,91	1,54	100	28,300	-	8,20
09:00	120	35,390	7,09	10,91	1,54				
09:30	150	35,400	7,10	10,91	1,54				
10:00	180	35,410	7,11	10,91	1,53				
11:00	240	35,420	7,12	10,91	1,53				
12:00	300	35,430	7,13	10,91	1,53				
13:00	360	35,440	7,14	10,91	1,53				
14:00	420	35,430	7,13	10,91	1,53				
15:00	480	35,440	7,14	10,91	1,53				
16:00	540	35,450	7,15	10,91	1,53				
17:00	600	35,440	7,14	10,91	1,53				
19:00	720	35,450	7,15	10,91	1,53				
	840								
	960								
	1080								
	1200								
	1320								
	1440								

Drilling report concerning wells drilled at the study area (page 7/10) (ABNT 2006)

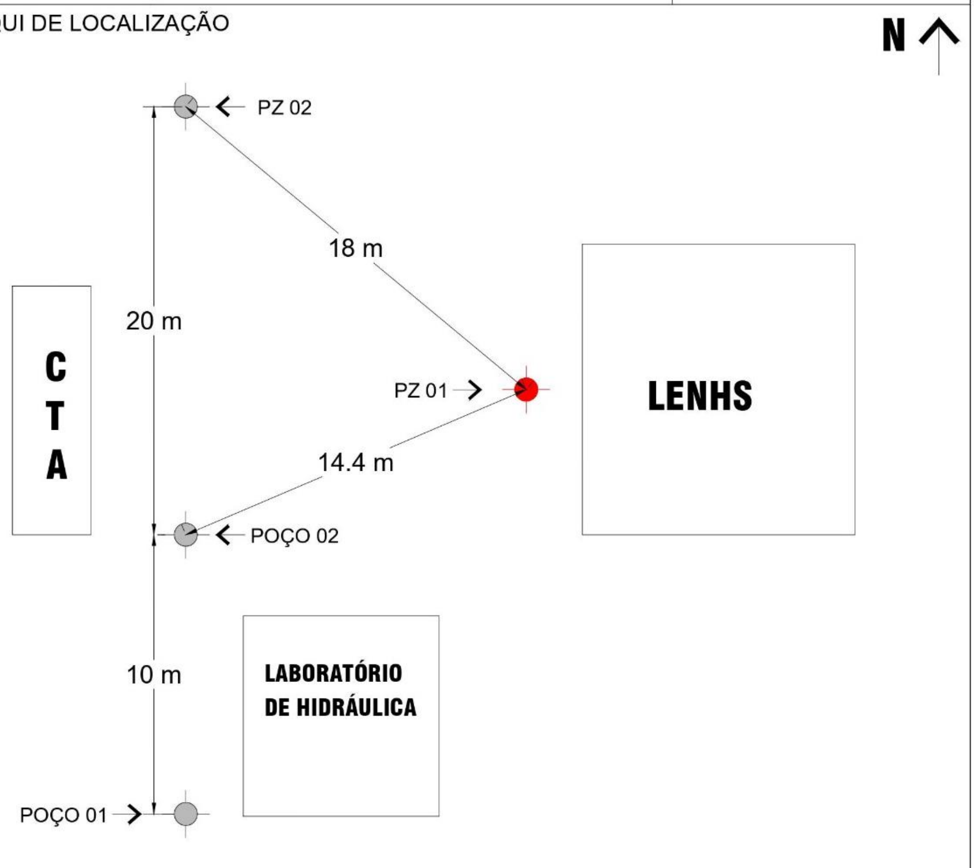
HIDROTEC
POÇOS ARTESIANOS

FICHA DE IDENTIFICAÇÃO E LOCALIZAÇÃO

PIEZÔMETRO 01

MUNICÍPIO	LOCAL	
JOÃO PESSOA	**UFPB - CENTRO DE TECNOLOGIA**	
COOR. GEOGRÁFICAS	COTA TOPOGRÁFICA	TIPO DE POÇO
LAT. **07° 08' 30.5" S** LONG. **34° 50' 58.2" W**	**40,0**	**TUBULAR**
USO DA ÁGUA	SERVIÇOS EXECUTADOS	
PESQUISA HIDROGEOLÓGICA	**PERFURAÇÃO**	
PROPRIETÁRIO	DATA	
FUNDAÇÃO PARQUE TECNOLÓGICO	**07/ 2019**	

CROQUI DE LOCALIZAÇÃO

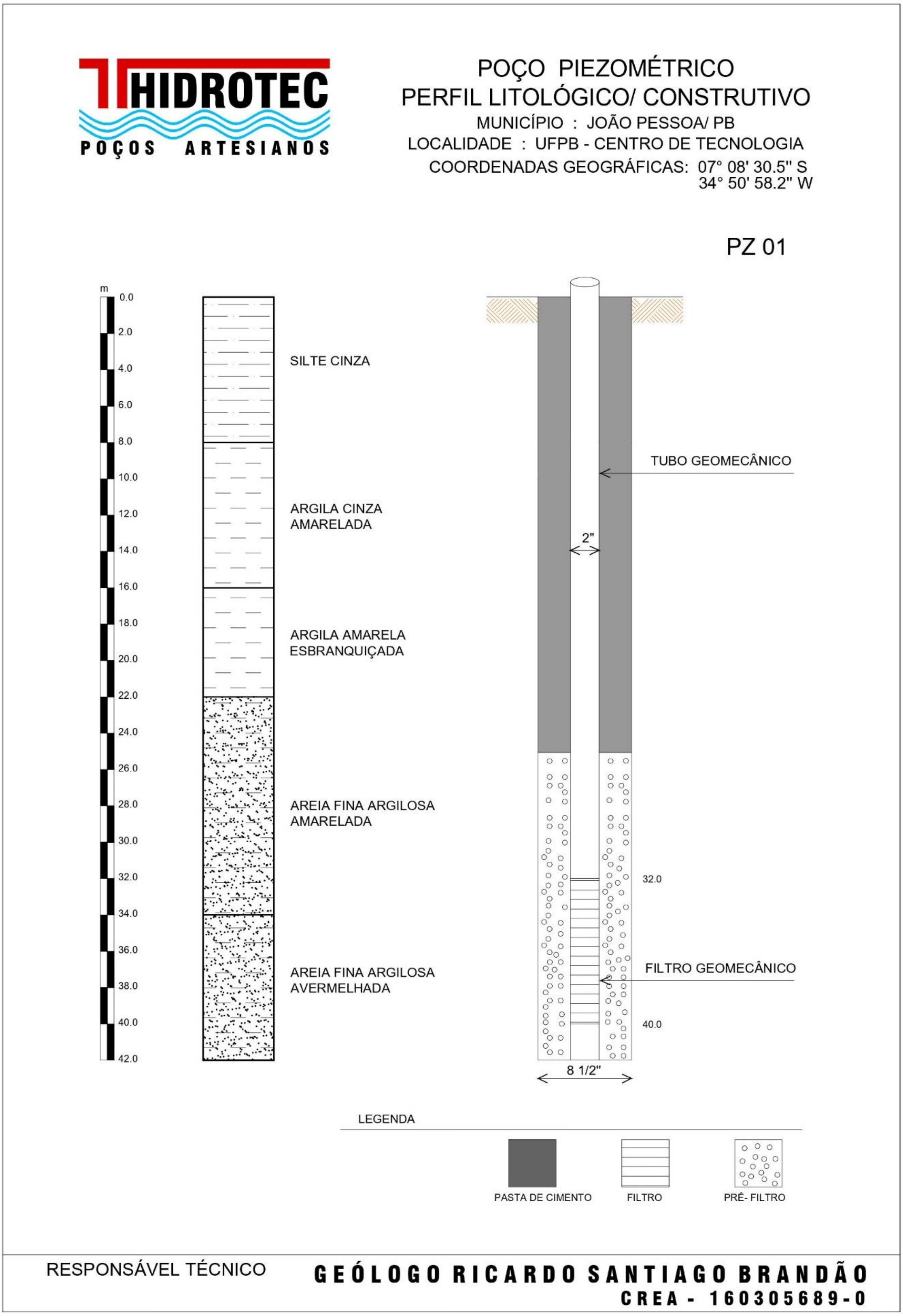


RESPONSÁVEL TÉCNICO

GEÓLOGO RICARDO SANTIAGO BRANDÃO

CREA - 160305689-0

Drilling report concerning wells drilled at the study area (page 8/10) (ABNT 2006)



Drilling report concerning wells drilled at the study area (page 9/10) (ABNT 2006)

HIDROTEC
POÇOS ARTESIANOS

FICHA DE IDENTIFICAÇÃO E LOCALIZAÇÃO

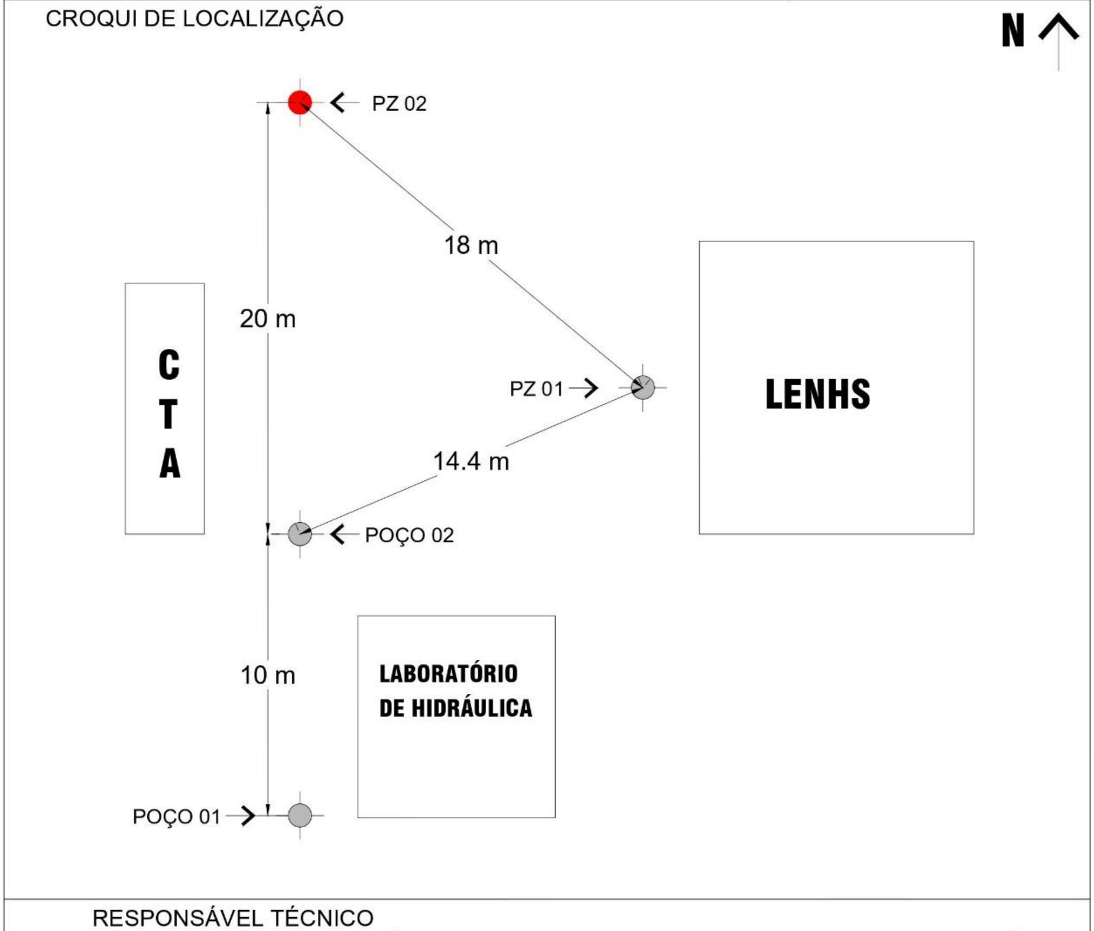
PIEZÔMETRO 02

MUNICÍPIO	LOCAL	
JOÃO PESSOA	**UFPB - CENTRO DE TECNOLOGIA**	
COOR. GEOGRÁFICAS	COTA TOPOGRÁFICA	TIPO DE POÇO
LAT. **07° 08' 30.1" S** LONG. **34° 50' 58.7" W**	**4 0, 0**	**TUBULAR**

USO DA ÁGUA	SERVIÇOS EXECUTADOS
PESQUISA HIDROGEOLÓGICA	**PERFURAÇÃO**

PROPRIETÁRIO	DATA
FUNDAÇÃO PARQUE TECNOLÓGICO	**07/ 2019**

CROQUI DE LOCALIZAÇÃO

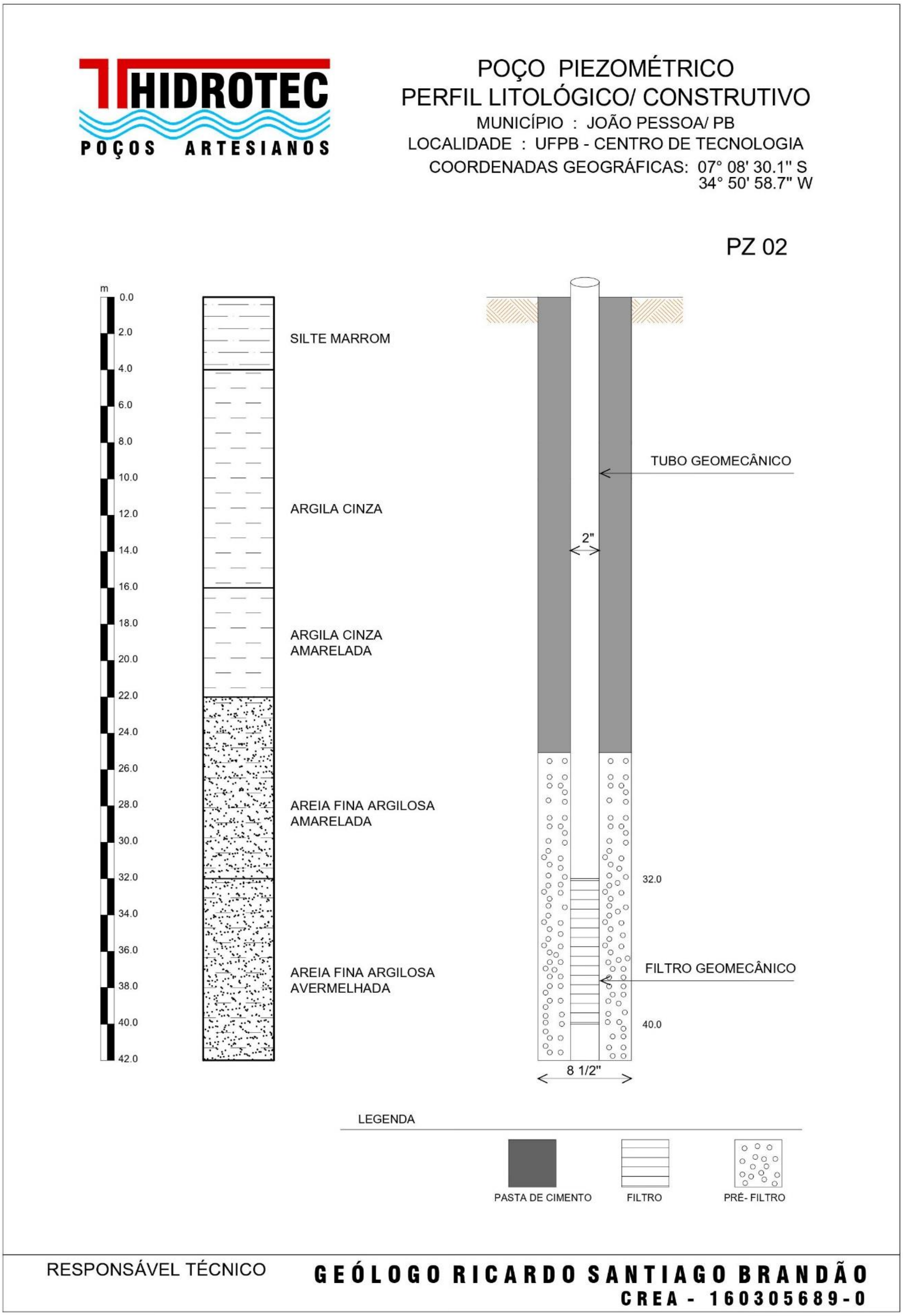


RESPONSÁVEL TÉCNICO

GEÓLOGO RICARDO SANTIAGO BRANDÃO

CREA - 160305689-0

Drilling report concerning wells drilled at the study area (page 10/10) (ABNT 2006)



ANNEXE C

Results from a brief survey run in June 2020 to find the average price of water tanks

Volume (L)	Type	Brand	Provider	Price	Date
500	Polyethylene	Fortlev	Leroy Merlin	R$ 154.90	2020-06-16
500	Polyethylene	Equation	Leroy Merlin	R$ 134.90	2020-06-16
500	Polyethylene	Fortlev	Ferreira Costa	R$ 169.00	2020-06-16
500	Polyethylene	Fortlev	Telha Norte	R$ 194.90	2020-06-16
500	Polyethylene	Acqualimp	Telha Norte	R$ 184.90	2020-06-16
1,000	Polyethylene	Fortlev	Ferreira Costa	R$ 269.00	2020-06-16
1,000	Polyethylene	Fortlev	Leroy Merlin	R$ 254.90	2020-06-16
1,000	Polyethylene	Fortlev	Telha Norte	R$ 359.90	2020-06-16
1,000	Polyethylene	Acqualimp	Telha Norte	R$ 319.90	2020-06-16
3,000	Polyethylene	Fortlev	Ferreira Costa	R$ 1,439.00	2020-06-16
3,000	Polyethylene	Fortlev	Copafer	R$ 1,244.78	2020-06-16
5,000	Polyethylene	Fortlev	Leroy Merlin	R$ 2,263.54	2020-06-16
5,000	Polyethylene	Fortlev	Ferreira Costa	R$ 2,459.00	2020-06-16
7,500	Polyethylene	Fortlev	Leroy Merlin	R$ 3,843.23	2020-06-16
7,500	Glass fiber	Bakof Tec	Cassol	R$ 2,459.00	2020-06-16
10,000	Polyethylene	Fortlev	Leroy Merlin	R$ 3,995.53	2020-06-16
10,000	Polyethylene	Caixa Forte	Caixa Forte	R$ 2,260.00	2020-06-17
15,000	Polyethylene	Fortlev	Leroy Merlin	R$ 7,710.90	2020-06-16
15,000	Polyethylene	Caixa Forte	Caixa Forte	R$ 4,220.00	2020-06-17
20,000	Polyethylene	Fortlev	Amoedo	R$ 10,900.00	2020-06-16
20,000	Polyethylene	Caixa Forte	Caixa Forte	R$ 6,040.00	2020-06-17
25,000	Polyethylene	Caixa Forte	Caixa Forte	R$ 7,740.00	2020-06-17

ANNEXE D

Main features of standard designs of single- and multi-family residences (ABNT 2006) – page (1/2)

Low standard single-family residence	Normal single-family standard residence	High standard single-family residence
Code: R1-B	Code: R1-N	Code: R1-A
Residence consisting of two bedrooms, room, bathroom, kitchen and laundry area	Residence consisting of three bedrooms, one suite with bathroom, toilet, living room, circulation, kitchen, laundry area with balcony (carport)	Residence consisting of four bedrooms, one suite with bathroom and closet, another with bathroom, toilet, living room, dining room, and intimate room, circulation, kitchen, full-service area and balcony (carport)
58.64	106.44	224.82

Projects of social interest	Low standard popular building	Normal standard popular building
Code: PIS	Code: PP-B	Code: PP-N
Ground floor and four type floors. *Ground floor:* Hall, staircase and four apartments per floor with two bedrooms, living room, bathroom, kitchen, and service area. Outside are the guard room with bathroom and central measurement. *Type floor:* Hall, staircase and four apartments per floor with two bedrooms, living room, bathroom, kitchen, and service area	**Ground floor and three type floors.** *Ground floor:* Entrance hall, staircase, and four apartments per floor with two bedrooms, living room, bathroom, kitchen, and service area. Outside are located in the garbage room, guardhouse, central gas tank, toilet, and sixteen uncovered car spaces. *Type floor:* Circulation hall, staircase, and four apartments per floor with two bedrooms, living room, bathroom, kitchen, and laundry area	**Garage, pilotis and four type floors.** *Garage:* Stairs, elevators, thirty-two covered parking spaces, trash room, storage room, and sanitary installation. *Pilotis:* Stairs, elevators, entrance hall, ballroom, eating area, two bathrooms, gas central and guardhouse. *Type floor:* Circulation hall, staircase, elevators, and four apartments per floor with three bedrooms, one suite, living/dining room, bathroom, kitchen and laundry area with bathroom. and balcony
Typical area (m²)		
991.45	1,415.07	2,590.35

Main features of standard designs of single- and multi-family residences (ABNT 2006) – page 2/2

Low standard multi-family residence	Normal standard multi-family residence	High standard multi-family residence
Code: R8-B	Code: R8-N	Code: R8-A
Ground floor and seven type floors. *Ground floor:* Entrance hall, elevator, staircase and four apartments per floor with two bedrooms, living room, bathroom, kitchen, and tank area. Outside are the trash room and thirty-two uncovered parking spaces. *Type floor:* Circulation hall, staircase, and four apartments per floor with two bedrooms, living room, bathroom, kitchen, and tank area	**Garage, pilotis, eight type floors.** *Garage:* Stairs, elevators, sixty-four covered parking spaces, junk room, and sanitary installation. *Pilotis:* Stairs, elevators, entrance hall, ballroom, eating area, two bathrooms, gas central and guardhouse. *Type floor:* Circulation hall, staircase, elevators, and four apartments per floor with three bedrooms, one suite, living/dining room, social bathroom, kitchen and laundry area with bathroom and balcony	**Garage, pilotis, eight type floors.** *Garage:* Stairs, elevators, forty-eight covered parking spaces, trash room, storage, and sanitary installation. *Pilotis:* Stairs, elevators, lobby, ballroom, games room, eating area, two bathrooms, central gas, and guardhouse. *Type floor:* Circulation halls, stairway, elevators, and two apartments per floor four bedrooms, one suite with bathroom and closet, another with bathroom, toilet, living room, dining room, and intimate room, circulation, kitchen, full-service area and balcony
Typical area (m²)		
2,801.64	5,998.73	5,917.79

Normal standard multi-family residence	High standard multi-family residence
Code: R16-N	Code: R16-A
Garage, pilotis and sixteen type floors. *Garage:* Stairs, elevators, one hundred and twenty-eight covered parking spaces, rubbish bin and sanitary installation. *Pilotis:* Stairs, elevators, entrance hall, ballroom, eating area, two bathrooms, gas central and guardhouse. *Type floor:* Circulation hall, staircase, elevators, and four apartments per floor with three bedrooms, one suite, living/dining room, social bathroom, kitchen and laundry area with bathroom and balcony	**Garage, pilotis and sixteen type floors.** *Garage:* Stairs, elevators, ninety-six covered parking spaces, trash room, storage, and sanitary installation. *Pilotis:* Stairs, elevators, lobby, ballroom, games room, eating area, two bathrooms, central gas, and guardhouse. *Type floor:* Circulation halls, stairs, elevators, and two apartments per floor four bedrooms, being one suite with bathroom and closet, another with bathroom, toilet, living room, dining room, and intimate room, circulation, kitchen, full-service area and balcony
Typical area (m²)	
10,562.07	10,461.85

Source: ABNT - Associação Brasileira De Normas Técnicas (2006)