Assessment of groundwater lowering for urban infrastructure works, a case study in Sumbe, Angola/Africa

Avaliação de rebaixamento de lençol freático para implantação de infraestrutura urbana, estudo de caso em Sumbe, Angola, África

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ABSTRACT: This paper describes the hydrogeological study developed for Sumbe city, in Angola, Africa, to evaluate the feasibility of employing existing wells to drawdown the water table in the central area of the city both to facilitate the urban infrastructure implementation and to increase its lifespan. The hydraulic conductivity of the unsaturated zone was estimated based on the open hole and double ring infiltrometer tests conducted in several points of the site to capture the variability of the hydrogeologic condition of the zone of aeration. The transmissivity and the hydraulic conductivity of the saturation zone were estimated based on an pumping test performed in a pumping well and two monitoring wells. Hand auger, SPT and rotary boring were carried out and combined with all field data information to produce a simplified conceptual model of the hydrogeologic system for the study site. Based on the survey data and by applying the Dupuit approximation of the Boussinesq equation for unconfined aquifers, the piezometric surface and the radius of influence of the well were computed for an operational steady condition. The study demonstrated that the entire site location is on a thick layer of clay and employing wells for groundwater lowering is unfeasible for either permanent or temporary purposes. For temporary groundwater lowering, collecting the water drained at the excavation by direct pumping was suggested. Finally, additional considerations regarding the sewerage/stormwater systems and the drainage system for the pavement base were also addressed.

Keywords: groundwater lowering; infrastructure; hydraulic conductivity; influence radius of wells;

RESUMO: Este artigo descreve um estudo hidrogeológico desenvolvido para a cidade de Sumbe, em Angola, África, com o propósito de avaliar a viabilidade de se utilizar poços profundos existentes para efetuar o rebaixamento do lençol freático na área central da cidade de forma a facilitar a implementação das obras de infraestrutura, bem como incrementar a vida útil das instalações. A condutividade hidráulica da zona não saturada foi estimada baseada nos ensaios open and hole e de anéis concêntricos em locais diversos buscando caracterizar a variabilidade das condições hidrogeológicas da zona de aeração. A transmissividade e a condutividade hidráulica da zona saturada foram estimadas com base em um teste de aquífero realizado em um poço de bombeamento e dois poços de observação. Ensaios de sondagem à trado, SPT e rotativa foram realizados e combinados com diversos dados levantados em campo para produzir um modelo hidrogeológico conceitual para área de estudo. Com base nos levantamentos executados e por meio da aproximação de Dupuit para a equação de Boussinesq para aquíferos confinados e semiconfinados, a superfície piezométrica e o raio de influência do poço de bombeamento foram calculados para condição de operação em regime permanente do poço. As avaliações indicaram que toda a área de estudo econtra-se sobre uma camada espessa de argila e a utilização de poços profundos para rebaixamento do lençol freático, seja temporariamente ou de forma permanente, mostra-se inviável. Para o rebaixamento temporário, foi sugerido o uso de bombeamento direto das águas drenadas nas escavações executadas. Por fim, recomendações gerais foram realizadas para os projetos de drenagem, esgotamento sanitário e pavimentação.

Palavras-Chave: rebaixamento de aquíferos; infraestrutura; condutividade hidráulica; raio de influência de poços;

Este artigo é parte integrante da *Revista Recursos Hídricos*, Vol. 42, N.º 2, 9-21 dezembro de 2021.

 ISSN 0870-1741 | DOI 10.5894/rh42n2-cti1

1. INTRODUCTION

Lowering the groundwater level to facilitate subsurface construction is an important task in civil engineering projects such as the construction of stormwater and wastewater systems, pavements, building foundations, mining in deep excavations, slope stabilization structures or any other civil work that requires excavation in areas where the water table is shallow. Besides the inconvenience for civil engineering works, shallow groundwater may also contribute to groundwater flooding, especially in cities located in flood plain areas, such as the city of Oxford on the River Thames, in the UK (Macdonald *et al.* 2012). Another issue of concern in shallow groundwater areas is a higher susceptibility to ground water contamination. Burri *et al.* (2019) presented a review of several threats to groundwater quality caused by anthropic activities such as agriculture, mining, land use practices, solid and liquid waste deposition, industrial activities and wastewater disposals that generate compound chemicals, which may easily reach groundwater, especially in sites without a minimum width of an unsaturated layer as demonstrated in Nyenje *et al.* (2014). Al-Saedy & Abdulhussain (2013) also mention the effect of reduction in the land productivity state caused by an increase in soil salinity and alkalization due to shallow groundwater.

Regardless the reason for water table drawdown, the implementation of a groundwater withdrawal project must be done carefully to avoid undesirable side effects. Angel *et al.* (2015) highlight that controlling groundwater can be costly, not only during construction but also from resulting damages to infrastructure or properties around the working area. Roy & Robinson (2009) reported a case study indicating settlements as large as 36 cm extending to distances of several times the dimension of the area within which actual permanent dewatering was taking place in an underground construction in Vancouver, Canada. Modoni *et al.* (2013) evaluated land subsidence mainly caused by groundwater withdrawal for water supply in the city of Bologna, Italy, for the period 1943-2005, finding maximum cumulated settlements of 4 m in critical spots. The authors also mention several cases around the world in which subsidence induced by groundwater withdrawal has been an issue of concern.

As noticed by Angel *et al.* (2015), different factors must be taken into account to determine the most effective dewatering system. First, a thorough understanding of the subsurface rock and soil conditions at the site is necessary. This is achieved by conducting field tests to identify the soil type, layer thickness and basic soil properties such as the hydraulic conductivity. Then, a general understanding of the hydrological characteristics at the site is essential to determine the source of the water and the water level behavior during the dewatering process. Finally, the effects of dewatering on adjacent structures must be considered in a cost-benefit analysis.

In this way, before proceeding with any cost benefit assessment, a model building process is required. Gupta *et al.* (2012) summarized the conceptualization of a model in three formal stages: conceptual model, mathematical model and computational model. Enemark *et al.* (2019) defined the groundwater system conceptualization as a collection of hypotheses describing the understanding of the different aspects of the groundwater system that are important to the modeling objective. The conceptual model is based on the available geological and hydrological information (observed water levels, estimation of soil parameters by field survey, etc.), but often also include geological insights or expert interpretation (Enemark *et al.* 2019). The mathematical model is related to the equations that will be derived to handle the states, fluxes and parameters of the modeled system and the computational model promotes the boundary conditions and numerical solutions for the system variables.

However, in practical engineering applications, besides the inherent limitations in representing a real process by a model, there are also time and budget constraints that push practitioners to seek simplistic empirical approaches adequate to allow for a successful and economic completion of their projects. In addition, for some conditions, as argued by Cashman & Preene, (2001), there is little practical justification for the use of sophisticated and time-consuming techniques, when simpler methods can give serviceable results, and this fits the context of this work.

In this context, this paper describes a case study to evaluate the feasibility of employing existing wells in the city of Sumbe/Angola to drawdown the water table both to facilitate the urban infrastructure implementation and to increase its lifespan.

2. METHODS

Due to the absence of previous technical studies and surveys in the study region, practical *in situ* tests were performed to allow a better characterization of the groundwater surface, the estimation of the hydraulic conductivity for the unsaturated and saturated zones, as well as a general classification of the material for the soil layers. Initially, site description and a general characterization of the problem are presented. Then, a brief description of the tests and surveys is presented in this section followed by the methods adopted to estimate the radius of influence for a well at the study site. The tests and surveys were chosen based on the time and budget constraints but were found to be sufficient for the study purpose.

2.1 Site Description and Characterization of the Problem

Sumbe is a city located in the Cuanza Sul province in Angola, Africa (Figure 1). The city suffers from shallow groundwater in almost all regions in which urbanization works (pavement, drainage, wastewater and water distribution systems) must be implemented. Several points in the city have groundwater levels less than 80 cm below the surface, in other spots almost no water can be absorbed by the soil and permanent water puddles are found (Figure 2). This condition not only hampers civil works but also brings several flood spots to the city during the rainy season affecting Sumbe citizens.

Given the provided context and the existence of three inactivated wells in the city (Figure 1) that previously worked during the Portuguese colonization period, around the 1970s, the recovery or new implementation of the wells was considered for dewatering the area. However, aside from the oral history of the efficient use of the wells in the past, no technical information was available. Angel *et al* (2015) argue that a thorough understanding of the subsurface rock and soil conditions throughout the identification of the soil types and hydrological characteristics of the site are paramount in the development and implementation of a successful dewatering operation. In this way, to evaluate the possibility of employing the wells for groundwater lowering, at least some basic hydraulic parameters such as the soil hydraulic conductivity and transmissivity had to be determined, in addition to a general understanding of the hydrogeological features of the site.

Considering the limited availability of time for the field survey (15 days), some straightforward *in situ* tests were selected to allow a practical estimation of the hydraulic conductivity. For the unsaturated zone the open and hole and the double ring infiltrometer tests were employed. These tests

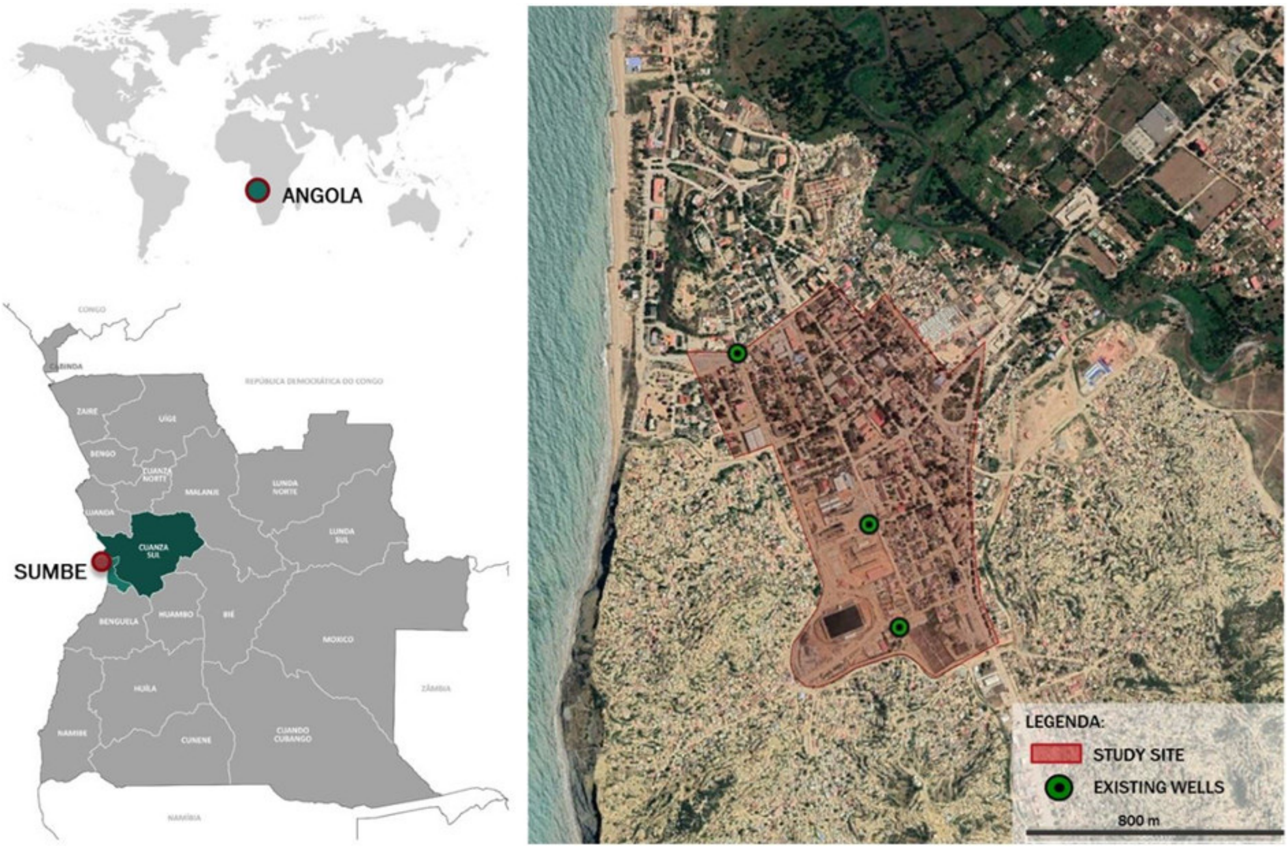

Figure 1. General localization map.

Figure 2. Example of shallow groundwater spots at the study site.

have been reported to determine the hydraulic conductivity for the unsaturated zone in several study cases in Brazil (Carvalho *et al.* 2013, Flori, 2010, Olivia *et al.* 2005). The transmissivity and the hydraulic conductivity of the saturation zone were estimated based on pumping test. Besides the *in situ* tests to estimate the hydraulic conductivity additional surveys and procedures were performed to address the main purpose of this study that was to evaluate the feasibility of using deep wells for dewatering the urban area of the city.

2.2 Double Ring Infiltrometer

The double ring infiltrometer test aims to estimate the vertical hydraulic conductivity of the unsaturated zone. In this method two cylinders with different sizes are concentrically arranged and nailed a few centimeters in the ground to prevent lateral dispersion of the inserted water (Figure 3). The length of the cylinder below the ground surface is measured resulting in the value of I. Initially, the external cylinder is filled with water until the water level stabilizes (when the soil becomes saturated and lateral water flux is avoided). Then, the internal cylinder is filled with water and the distance between the top of the cylinder and the water level is measured (M_i), allowing the computation of the initial water depth (h_0). After some time (around 15 min), this procedure is repeated and the distance between the top of the cylinder and the water level is verified for the second time resulting in the final measurement M_f, and, consequently, the final water depth (h). The vertical hydraulic conductivity (K_z) is computed by Equation (1).

$$K_z = U \cdot \frac{I}{\Delta t} \cdot ln(\frac{h_0}{h}) \tag{1}$$

Where, K_z is the vertical hydraulic conductivity (m/s), U is a unit conversion factor (1/60000), unit conversion factor, Δt is the time between initial and final measurements (s), h_0 is the initial water depth (mm) equals to $H - I - M_i$, h, is the final water depth (mm) equals to $H - I - M_f$, H, is the total height of the cylinder (250 mm), I is the length of the cylinder nailed in the ground (mm), and M_i, and M_f are the initial and final distances between the top of the cylinder and the water level (mm) respectively. Five tests were performed in the study area with locations indicated in Figure 4.

2.3 Open and Hole

The Open and Hole test was also employed to determine the vertical hydraulic conductivity for the unsaturated zone, however, it considers 4 different depths (50, 100, 150 and 200 cm), as shown in Figure 5. Initially, four holes are dug using

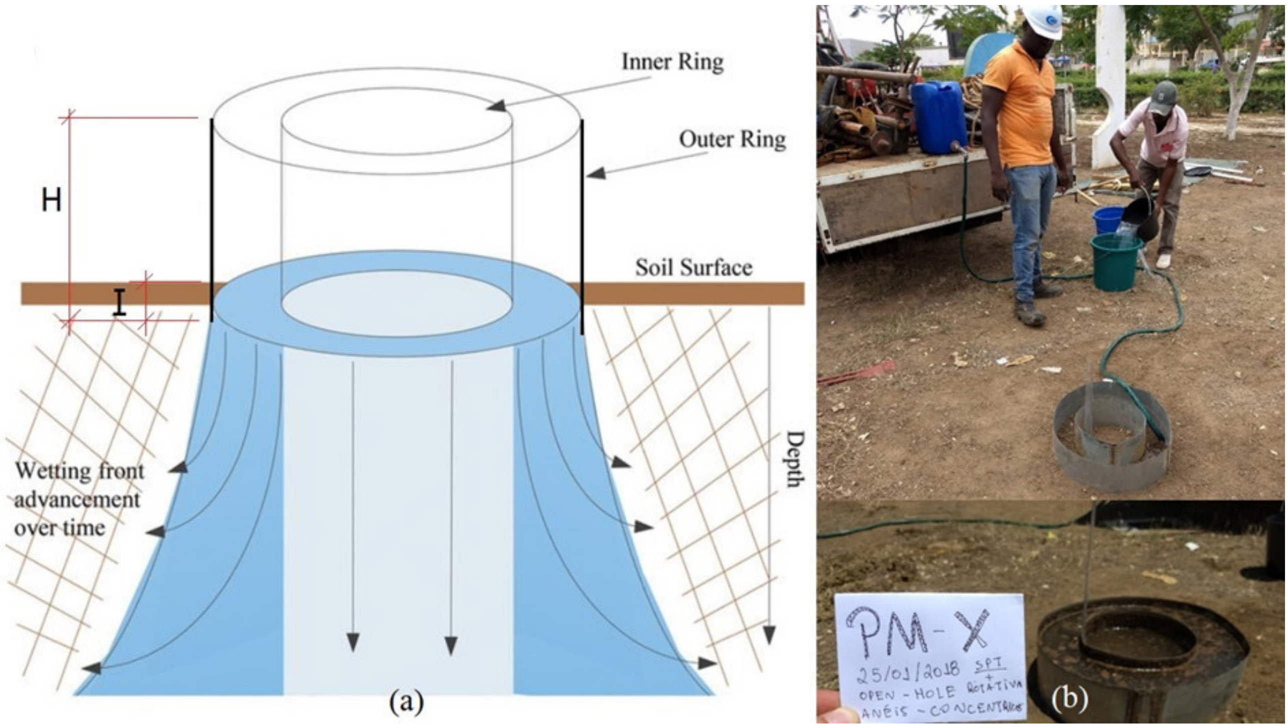

Figure 3. (a) General framework for the Double Ring Infiltrometer Test (adapted from Dixon, et al. 2018), (b) Double Ring Infiltrometer Test performed at the site location.

a hand auger and a PVC pipe with 50 mm diameter is inserted in each hole. Each pipe must be inserted a few centimeters in the ground to minimize lateral water flux during the infiltration. Then, the distance between the top of the pipe and the bottom of the hole inside the pipe is measured (X). Next, each pipe is filled with water and the initial distance between the top of the pipe and the water level is measured (Mi), allowing the computation of the initial water depth (h0). After a time period of , the measurement of the water level is repeated, resulting in Mf , and the final water depth is computed (h). The vertical hydraulic conductivity (Kz) is given by Equation (2).

$$Kz = 2.303 \cdot \frac{r}{b4\Delta t} \cdot log(\frac{h_0}{h}) \qquad (2)$$

Where, K_z = Vertical hydraulic conductivity (m/s), is a unit conversion factor (2.303), Δt is the time interval between initial and final measurements (s), X is the distance between the top of the tube and the bottom of the hole inside the tube (cm), h_0 is the initial water depth (cm) given by X - M_i, r is the final water depth (cm) given by X - Mf , r is the inner radius of the pipe (m), M_i, and M_f are the initial and final distances between the top of the cylinder and the water level (cm) respectively. Nine Open and Hole tests were performed on the study area, the locations are presented in Figure 4.

2.4 Pumping Test

To estimate the hydraulic conductivity for the saturated zone, a pumping test was performed near to an existing well (the central well on Figure 4). The water was pumped in the pumping well (P-2 in Figure 6) at a constant rate of 6 m3/m, while the water level was monitored at two observation wells located at 5 and 10 m from the pumping well. The monitoring wells were designed according to the ABNT (1997) and the test was performed between the second and third days of March/2018, based on the general standard procedures described in the technical literature (Kruseman & de Ridder,1994; Ferris *et al.*,1962; Stallman,1971; CPRM, 2000; Lima & Filho, 2003; ADASA, 2016). The pumping phase took 12 hours while the water table recovery phase was monitored for 6 hours. Figure 6 shows a schematic profile of the monitoring and pumping wells and their respective locations.

2.5 Monitoring Wells

In order to characterize the groundwater depth in the study site, 43 monitoring wells were installed. Each well was composed of a 50 mm PVC pipe with 3 m depth and was dug by a 60 mm diameter hand auger. Starting 50 cm down from the top of the pipe, holes with 0.80 cm diameter were made in a stretch of 1.50 m to allow the water infiltration.

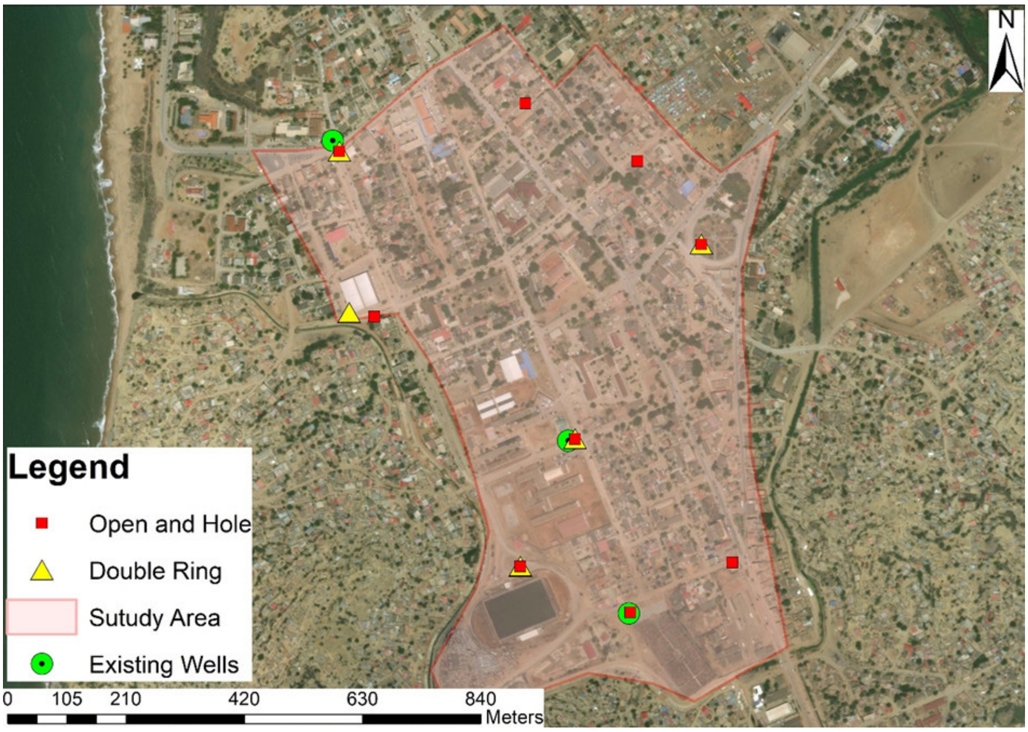

Figure 4. Double Ring Infiltrometer and Open Hole Test spots

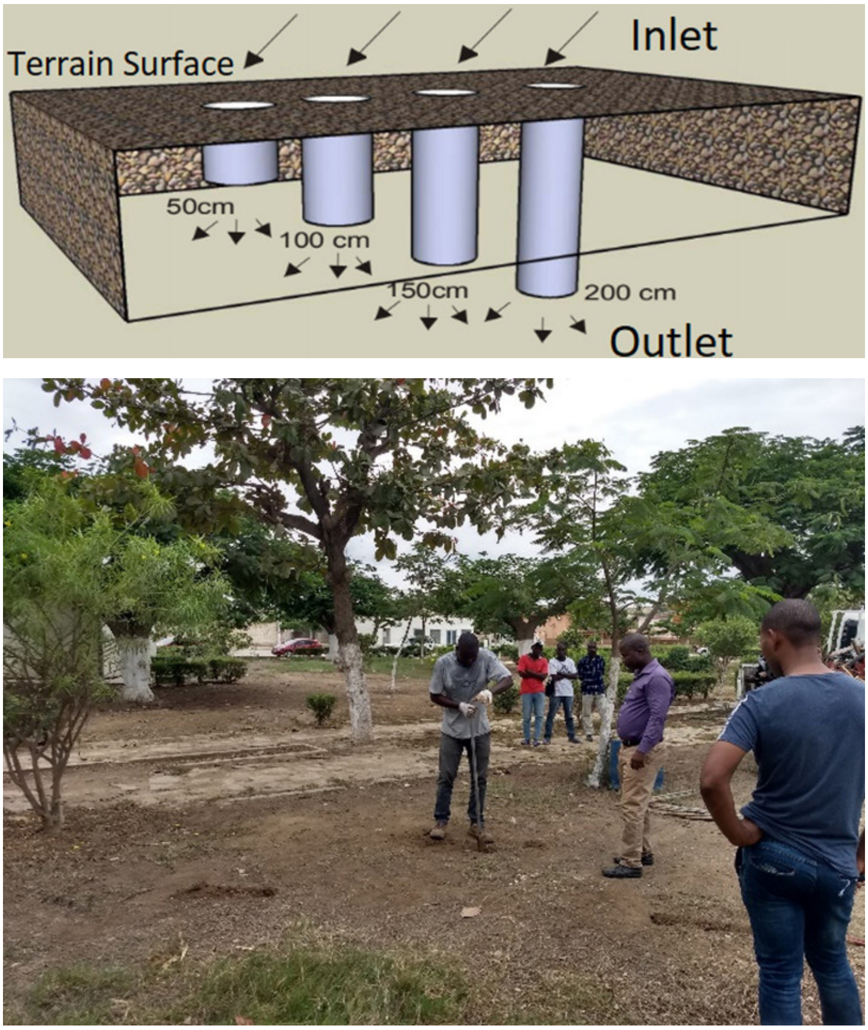

Figure 5. General framework for the Open and Hole Test.

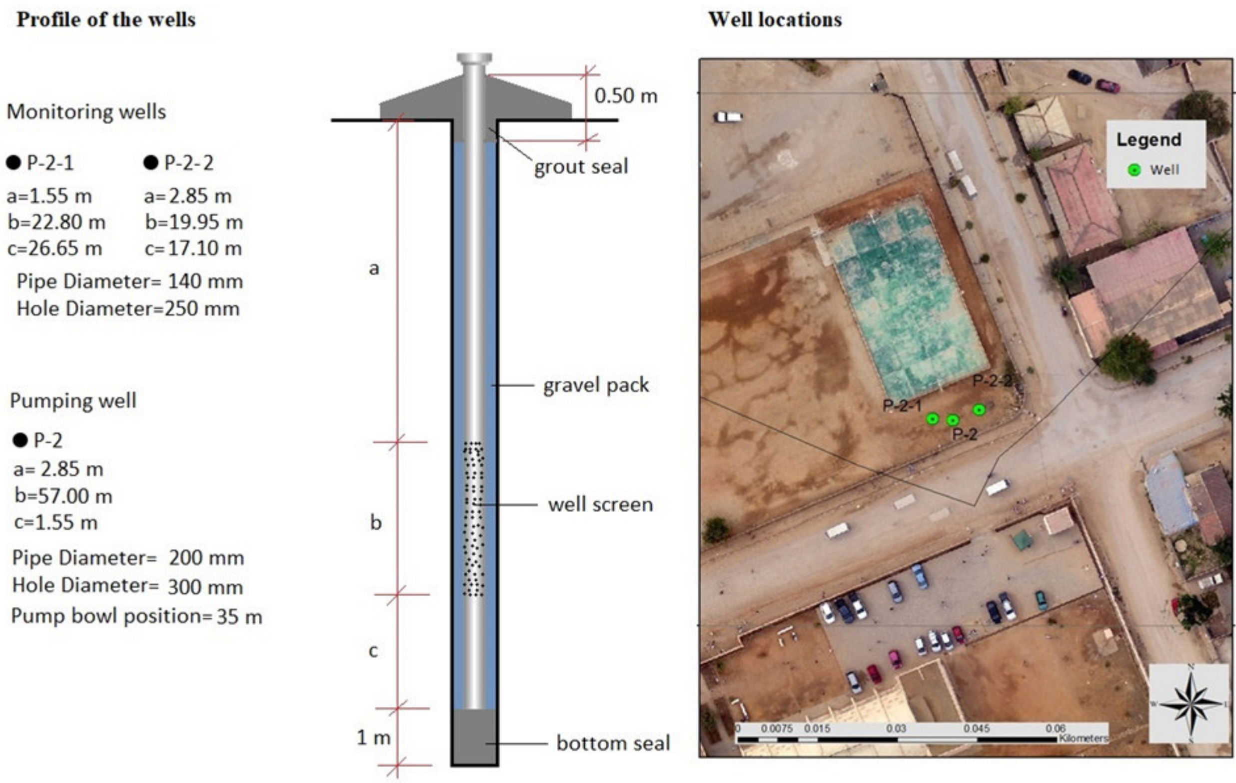

Figure 6. Monitoring and pumping wells profile and locations.

At the bottom of each well a cap was installed and the interface between the pipe and the ground was filled with gravel ranging from 1 to 2 cm diameter. In each well, a topographic landmark was installed and the elevations of the water level were monitored during February 2018. Figure 7 presents a general profile of the monitoring wells.

2.6 Geotechnical Investigations

Geotechnical investigations were carried out at the study site to support the general characterization of the physical properties of the soil, as well as to support a simplified hydrogeologic conceptual model for the site. The hand auger drills of all monitoring wells were used to characterize the superficial soil layer. In addition, three Standard Penetration Tests (SPT) associated with rotary boring were conducted closest to the existing wells, allowing the characterization of deeper soil layers (until 60 m depth). The SPT tests followed the ABNT (2001) guidelines.

2.7 Governing Equation

By assuming a two-dimensional, homogeneous and isotropic aquifer and applying the approximation first suggested by Dupuit (1863), one can obtain the Boussinesq (Equation 3) equation to a steady state condition.

$$\nabla^2(h^2) = \frac{Q_W}{Kb} \tag{3}$$

Where, K is the hydraulic conductivity (m/h) assumed to be equal in any direction, h is the water table elevation or the saturated thickness (m), Q_W is the pumping rate (m^3/h), b is the aquifer thickness (m); and $\nabla^2()$ is the Laplacian operator;

A finite difference scheme was applied for the numerical solution of Equation (3) by assuming a 2-D discretization in a mesh with Δx by Δy resolution. For the mesh nodes without pumping, Equation (4) was employed while for the mesh node with the pumping, Equation (5) was used.

$$\frac{h_{i+1,j} - 2h_{i,j} + h_{i-1,j}}{(\Delta x)^2} + \frac{h_{i,j+1} - 2h_{i,j} + h_{i,j-1}}{(\Delta y)^2} = 0 \tag{4}$$

$$\frac{h_{i+1,j} - 2h_{i,j} + h_{i-1,j}}{(\Delta x)^2} + \frac{h_{i,j+1} - 2h_{i,j} + h_{i,j-1}}{(\Delta y)^2} = \frac{W}{Kb} \tag{5}$$

Where (i,j) represents the node index for the x and y directions. The solutions of Equations (4)

Figure 7. General profile of the monitoring wells.

and (5) were numerically implemented allowing the computation of the groundwater level for any node of the mesh around the pumping well. Table 1 presents the parameters adopted for Equations (4) and (5).

Table 1. Parameters assumed for the numerical solution for Equations (4) and (5)

QW (m^3/h)	K(m/h)	b(m)	Δx = Δy (m)
6,00	0.01015	60	1.5

2.8 Transmissivity and Hydraulic Conductivity

Assuming the same conditions adopted for the Boussinesq equation, the transmissivity (T), and the hydraulic conductivity (K) were computed based on Equations (6) and (7), as follows.

$$T = \frac{0{,}366\, Q_W}{(s_1 - \frac{S_1^2}{2H_0}) - (s_2 - \frac{s_2^2}{2H_0})}\, log(\frac{r_2}{r_1}) \quad (6)$$

$$K = \frac{T}{H_0} \quad (7)$$

Where, Q_W is the pumping rate (m^3/s), s_1 is the water table drawdown at monitoring well 1 (m), s_2 is the water table drawdown at monitoring well 2 (m), r_1 is the distance between the pumping well and the monitoring well 1 (m), r_2 is the distance between the pumping well and the monitoring well 2 (m), H_0 is the aquifer thickness (m), K is the hydraulic conductivity (m/s), and T is the Transmissivity of the aquifer (m^2/s).

2.9 Radius of Influence

The radius of influence (R) for the pumping well was empirically determined by Equation (8), originally proposed by Sichard (1927).

$$R = 3000 \cdot s_w \sqrt{K} \quad (8)$$

Where, s_w is the water drawdown at the pumping well (m), R is the Radius of Influence (m), and K is the hydraulic conductivity (m/s).

3. RESULTS AND DISCUSSIONS

The results for the Double Ring Infiltrometer test are presented in Table 2. The hydraulic conductivity for the superficial soil layer ranged from 4.01E-05 to 9.70E-06 m/s with a mean of 1.25E-05 m/s and was classified as a silt-clay soil.

The results for the Open and Hole test employed to verify the hydraulic conductivity on soil layers ranging from 50 cm to 200 cm depth are presented in Table 3. The mean value for the four depths tested ranged from 1.01E-07 m/s to 1.86E-08 m/s.

Based on samples collected by the hand auger drills and the hydraulic conductivity determined by the *in si*tu tests, the initial soil layer evaluated by the Double Ring Infiltrometer test was classified as silt-clay while the superficial soil layer with depth between 50 to 200 cm tested by the Open and Hole method was classified as a clay-silt soil.

Table 2. Results for the Double Ring Infiltrometer Tests

SPOT	E	N	K(m/s)
POÇO 1	373148	8761547	4.01E-05
POÇO 2	373566	8761037	2.44E-06
PM-X	373790	8761382	8.41E-06
PM-Z	373165	8761261	9.70E-06
PM-3-3	373469	8760811	2.02E-06
		MEAN	**1.25E-05**

Based on the hydraulic conductivity for the superficial soil layers, it is observed that, in general, as the depth increased, the hydraulic conductivity decreased, with values in some locations so small that it may be considered impermeable for practical purposes. The type of soil material identified by the hand auger samples for the soil layers between 100 cm to 200 cm was similar across all locations along the study region. The results correspond to what was expected based on the visual and tactile characterization of the material verified *in situ*. It should be noted that in some spots the presence of groundwater was identified, explaining the null/mearly impermeable values obtained for the hydraulic conductivity for some points.

Considering the parameters presented in Table 4 and Equations (6) and (7), the computed transmissivity (T) was 1.69E-04 m^2/s and the hydraulic conductivity (K) was 2.82 E-06 m/s. Since the geotechnical investigations could not reach bed-rock, the saturated thickness (H0) was estimated at 60 m based on the general geologic features of the study site. The maximum drawdown at the monitoring wells (P-2-1, P-2-2) were obtained from a logarithmic regression model fit to the drawdown curves for each monitoring well (Figure 8). The results suggest a silt-clay material property.

By employing the empirical Equation (8) at the pumping well with a drawdown of 47.82 m and using the aquifer parameters from Table 4, the radius of influence was computed resulting in 61.32 m as shown in Figure 9.

The radius of influence for the pumping well was also estimated based on the numerical solution of Equation (3), computationally implemented via finite differences based on the Equations (4) and (5). This approach shows the water table elevation at each node of the 126 m by 126 m numerical mesh, where the well is positioned at the center of the mesh. Figure 10(a) presents a plan view of the numerical mesh indicating the behavior of the water table elevation by a color gradation pallet. The center of the figure matches the center of the well and the axes indicate the distance from the center of the pumping well.

The mesh resolution is equally sized (1.5 m) and the relevant parameters used for the numerical approach were presented in Table 1. Three hundred iterations were performed to reach the convergence criteria (assumed as an error less than 0.1% between the water table elevations in the last two iterations for the pumping well node). Figure 10(b) shows a profile view for a cross section passing along the x-axis intercepting the center of the pumping well. By the numerical approach, the water table is insensitive to the pumping (for a steady state condition) at distances greater than 40 m from the well. At distances greater than 20 m from the center of the well, the drawdown was found to be less than 1 m.

Table 3. Results for the Open and Hole tests.

SPOT	E	N	K(m/s)				
			50 cm	100 cm	150 cm	200 cm	MEAN
PM-X	373790	8761382	4.62E-08	4.38E-08	1.95E-08	1.43E-08	3.09E-08
PM-3-3	373469	8760811	1.74E-08	6.03E-09	4.13E-10	2.91E-10	6.03E-09
POÇO 1	373148	8761547	1.32E-07	5.09E-09	2.00E-08	1.73E-09	3.98E-08
POÇO 2	373566	8761037	1.69E-09	1.38E-09	9.96E-10	4.23E-09	2.08E-09
	POÇO 3 373664 8760730		2.16E-08	1.11E-08	3.49E-10	0.00E+00	8.24E-09
P-M-J	373677	8761530	1.31E-08	7.28E-08	1.19E-08	2.95E-10	2.45E-08
P-M-Y	373478	8761632	1.58E-07	7.19E-10	4.53E-09	3.57E-09	4.18E-08
P-M-Z	373210	8761254	5.11E-07	8.86E-09	0.00E+00	1.40E-07	1.65E-07
P-M-K	373845	8760818	6.96E-09	3.31E-09	4.15E-08	3.06E-09	1.37E-08
		MEAN	1.01E-07	1.70E-08	1.10E-08	1.86E-08	**3.69E-08**

Table 4. Pumping test parameters and results.

Parameter	P-2-1	P-2-2
Distance from the pumping well - r(m)	5.00	10.00
Maximum drawdown - s (m)	10.87	9.56
H0 (m)	60.00	
Qw (m^3/s)	0.00166	
T (m^2/s)	1.69E-04	
K (m/s)	2.82E-06	

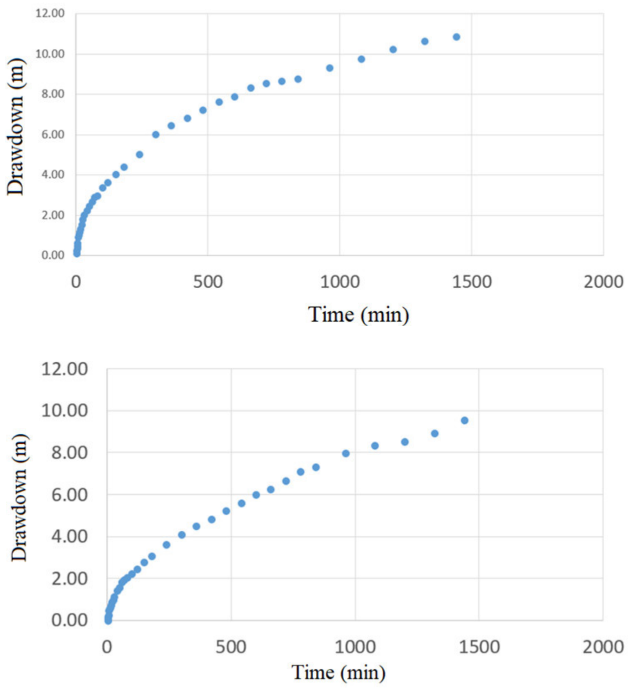

Figure 8. Drawdown curves of the monitoring wells.

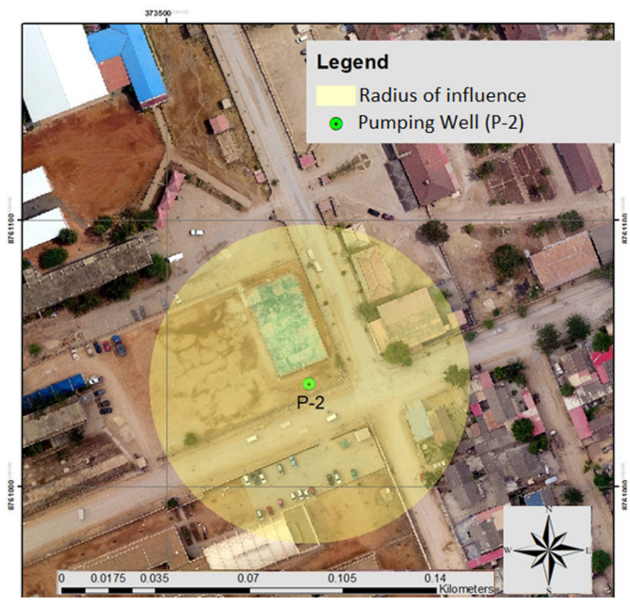

Figure 9. Radius of influence for the pumping well based on the empirical equation.

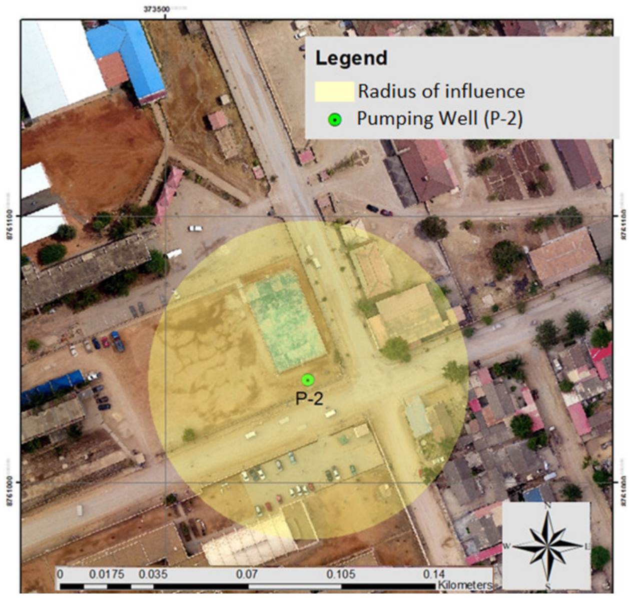

Figure 10. Radius of influence for the pumping well based on the numerical solution for Equation (3).

Based on the numerical approach, one can estimate the radius of influence as 40 m for the pumping well. Although the numerical and empirical approaches presented differing values for the radius of influence (the difference between the results is approximately 35% of the empirical output), for practical purposes both values suggest the use of wells for reducing groundwater levels to be unfeasible.

The mean values for the water elevation at the monitoring wells during February/2018 was used to generate a water table surface for the entire study area. Figure 11 presents the depth of the water table based on the water surface created by a 3D interpolation from the data of the monitoring wells. The dark blue color represents areas where the water table is close to the ground surface (less than 50 cm depth) while the red color indicates deeper groundwater (more than 250 m depth). It is clear that a majority of the study area is underlain by shallow groundwater.

Despite the absence of deeper geological investigations to allow for a detailed characterization of deeper soil layers as well as the bed rock material (which was not reached by the geotechnical investigations), the collected data and the *in situ* tests suggest that the study area is underlain by a deep layer of clay and silt with the potential for the development of a perched water table above the clay layer. Given the geological features of the area, it is believed that below the clay layer there is a sand layer where a confined aquifer may exist with some saltwater intrusion. At the study site, the bedrock is assumed to be deep, perhaps more than 100 meters deep. Figure 12 presents a sketch of the proposed conceptual model for the site.

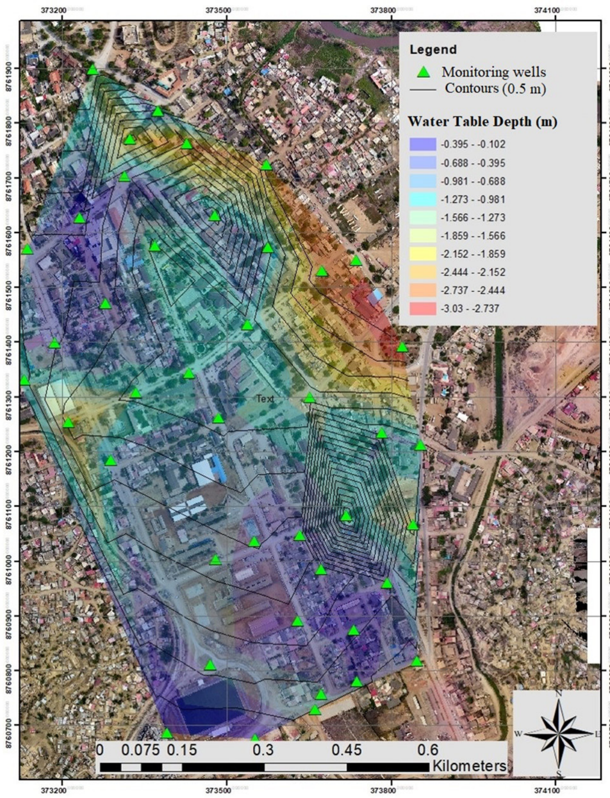

Figure 11. Water table depth and monitoring wells.

Although the proposed conceptual model was useful to indicate the infeasibility of using wells to drawdown the water table for the study site, it should be noted that further investigations are necessary to confirm and improve the suggested model including a detailed characterization of the composition of the layers materials as well as the depth of each layer.

4. CONCLUSIONS

The investigations suggest that most of the urban area in the city of Sumbe/Angola is located on a thick soil layer mainly composed of clay with similar hydrogeologic behavior along all the study area. The pumping test indicated that the area of influence of a pumping well in the study site is extremely localized, reaching no more than 60 m from the center of the well which makes the use of deep wells for lowering the water table inefficient and not cost-effective. Thus, it is recommended that urban infrastructure projects developed in this area must consider solutions that directly deal with shallow groundwater conditions. For drainage and sewage networks, it is suggested that maximum acceptable limits be used for the pipe slopes to preserve the best flow conditions.

For the drainage and sewage networks it is suggested that HDPE (High Density Polyethylene) be used to limit groundwater infiltration into the systems, as well as to limit the weight and provide more flexibility, while reducing possible disruption due to buoyant force on the system. Another alternative, in case of using concrete staves, is to use a residual flow (from 10% to 30%) added to the design discharge to taking into account the percolation of external water on the network. In addition, an annular filler with gravel and geotextile could be added to relieve external pressures on the network due to buoyancy. At specific spots in the design network, the external drainage flow should be pumped or conducted into the network.

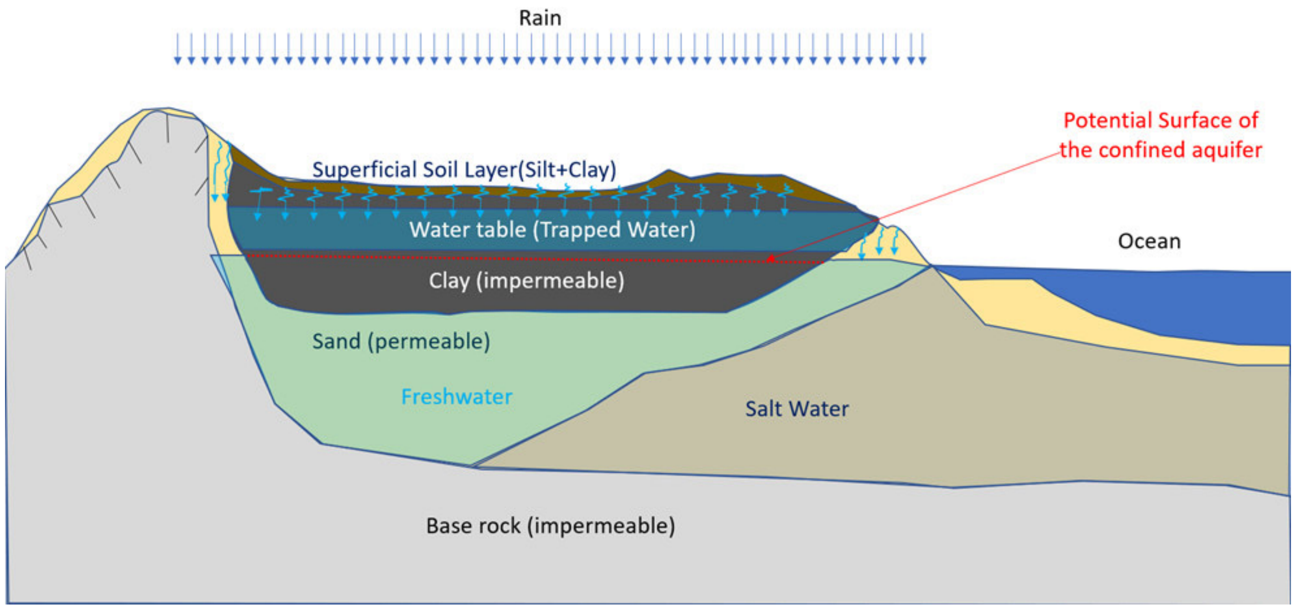

Figure 12. Conceptual hydrogeological model for the study site.

For paving, it is suggested that a gravel drainage layer wrapped by geotextile blanket be placed under the base of the paving with a slight slope from the center of the paving towards its margins. Longitudinally to the paving margins, it is recommended that drainage pipes be installed to conduct the drained water under the paved towards manholes. Such recommendations aim to extend the lifespan of the entire system given the exposure to shallow groundwater and salinity. Finally, regarding the construction of the network, it is recommended that direct pumping or wellpoints be used to temporarily lower the water table.

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