Chapter

Underground Excavations Below the Water Table by the Cut-and-Cover Method

Estanislao Pujades, Miao Jing, Chunhui Lu and Anna Jurado

Abstract

Most underground constructions, which are needed to improve mobility and increase available space in urban areas, require excavations that are usually deeper than the water table (e.g., for the construction of stations or underground parking lots). A frequently used technique to develop excavations under these conditions consists in combining the cut-and-cover method with a dewatering system based on deep pumping wells. Retaining walls used for the cut-and-cover method allow excavating between vertical walls and minimizes the inflow of groundwater, while deep pumping wells provide dry and stable conditions. Despite this technique is widely used, some aspects related with the presence of groundwater must be considered to avoid accidents. Dewatering systems must be properly designed to guarantee suitable conditions and to minimize the pumping settlements outside of the working area. In addition, it is required to assess the presence of defects in the retaining walls because the flow of groundwater through them may entail negative consequences. This chapter explains procedures (i) to design efficient dewatering systems considering the working conditions, the stability and the impacts generated in the vicinity of the construction, and (ii) to evaluate the state of the retaining walls by using hydrogeological tools.

Keywords: underground excavation, dewatering system, cut-and-cover method, pumping settlements, bottom stability, weather tightness assessment test

1. Introduction

The 70% of the world's population will be living in urban areas by 2050 [1]. Then, new infrastructures are needed for the development of cities and to cover the demands of the growing population. New infrastructures will improve the mobility (i.e., railway or motorway tunnels, stations, etc.) and the available space (i.e., underground parking lots). These new infrastructures must be constructed underground due to space limitations on the surface of urban areas, thereby, developing urban areas in the vertical direction. Most cities already have underground infrastructures and thus, the new ones should be deeper than the previous ones. Consequently, new infrastructures will probably be developed below the water table and will interact with groundwater.

The interaction between groundwater and the construction of underground infrastructures is bi-directional, meaning that the infrastructure may impact the natural behavior of aquifers (during and after its construction) and that the presence of groundwater hinders their construction, and in certain circumstances, even their utilization. Thus, the interaction between groundwater and underground infrastructures must be properly considered.

There are different construction methods adapted to the different kinds of infrastructures and the requirements of their construction. Among them, the cut-and-cover method is one of the most used in urban areas for constructing linear or non-linear infrastructures [2]. This method involves excavating between vertical retaining walls that minimize the needed space for the construction because, in the absence of retaining walls, the excavation walls must be inclined increasing enormously the needed space in case of deep excavations. The cut-and-cover method must be combined with dewatering facilities when excavations are developed below the water table [3]. Despite this dry conditions can be reached by collecting water at the surface by means of a system of trenches, and pumping it with a sump pump [4], stable conditions at the strata below the excavation bottom can only be reached by using deep pumping wells. The retaining walls (i) allow excavating between vertical walls, (ii) minimize the inflow inside the excavation and (iii) mitigate the potential impacts generated by the dewatering outside [5], whilst pumping wells reduce the piezometric head, ensuring dry conditions and providing stability to the excavation bottom [6]. The main actions to develop constructions by the cut-and-cover method combined with deep pumping wells (summarized in **Figure 1**) are: (1) to construct the retaining walls, (2) to drill the pumping wells and reduce the piezometric head until the required depth, (3) to excavate until the desired depth, (4) to construct the infrastructure and (5) to fill the space between the top of the infrastructure and the surface [7].

The presence of groundwater must be carefully considered during the execution of underground infrastructures by the cut-and-cover method. Groundwater may produce instabilities at the bottom of the excavation giving rise to liquefaction or bottom uplift events [8]. Then, the dewatering system, in addition to providing dry conditions, must reduce the water pressure as much as necessary to ensure stable conditions [6]. However, the hydraulic head should not be lowered more than necessary in order to minimize the impacts outside of the enclosure. The potential impacts produced outside the enclosure are: (i) the loss of hydraulic resources [9], which is relevant in

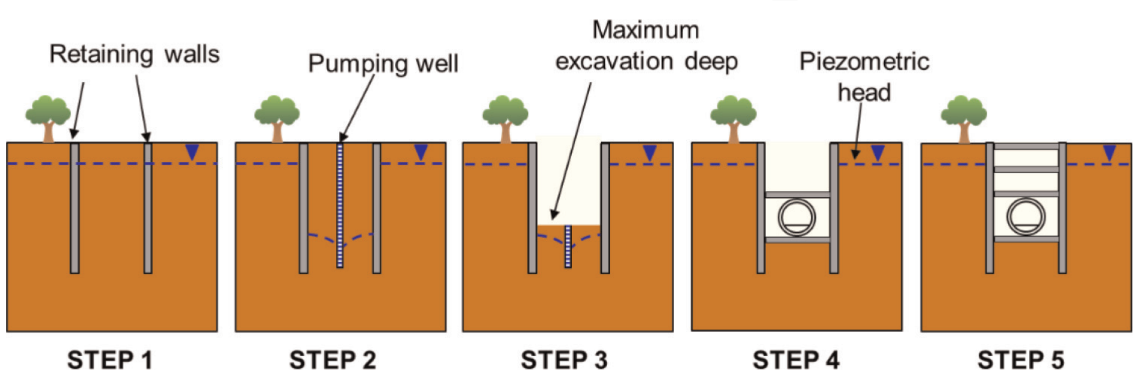

Figure 1.
Cut-and-cover method steps for the construction of an infrastructure below the water table (modified from [7]).

areas where water resources are scarce, and (ii) the soil subsidence (i.e., settlements) [10], which can endanger nearby buildings and infrastructures, either surface or underground. Common practices to minimize the groundwater-related issues consist of: (i) deepening the retaining walls until reaching an impervious layer and (ii) injecting jet-grouting below the excavation bottom to avoid the groundwater flow. However, both practices are risky since the stability of the excavation relies on the permeability of the retaining walls and the jet-grouting block. On the one hand, jet-grouting injection is complex and is difficult to reach a uniform and impervious block of treated soil [11, 12], especially in unconsolidated granular materials. On the other hand, defects in retaining walls, through which groundwater enters into the excavation, are relatively common [13, 14]. Then, the use of deep wells seems to be a more reliable option.

Faulty retaining walls do not block the groundwater and may endanger the construction development because they increase the risk of bottom instabilities, excavation flooding, sinkholes and soil deformations outside the enclosure [7, 15, 16]. Then, it is of paramount importance to determine the hydraulic conditions of the retaining walls before starting the excavation stage to adopt measures if needed (i.e., repair the retaining walls or redesign the dewatering system).

This chapter aims at explaining in detail how to design the dewatering system of deep excavations below the water table carried out by the cut-and-cover method considering groundwater stability issues and the potential impacts generated around the excavated area. In addition, this chapter exposes the consequences of faulty retaining walls and how they can be characterized by means of hydrogeological tools.

2. Dewatering system

The objective of a dewatering system is to provide workable conditions. Dry conditions are reached by dropping the piezometric head, at least, up to the maximum excavation depth, while stable conditions are achieved by reducing the water pressure in the soil located below the excavation bottom to guarantee that it is lower than the total vertical stress. Then, dewatering systems must be designed accounting for the piezometric head distribution above and below the maximum excavation bottom. The impact produced outside the excavation by the lowering of the piezometric head must be also considered when designing a dewatering system. The induced drawdown to ensure working conditions must not be excessive to minimize outer impacts such as (i) the loss of hydraulic resources, and (ii) soil settlements [17].

Groundwater can be extracted by different techniques but deep wells are the most appropriate in the context of urban deep excavations developed by the cut-and-cover method. Sump pumps [4] remove water from the excavation and allow working in dry conditions, but they do not drop enough of the water pressure below the excavation and bottom instabilities may occur. Well points are more adequate for shallow excavations because they extract water by suction and the maximum drawdown produced is lower than 6 m. Then, for developing deep excavations, well points must be installed in successive tiers or stages as excavation advances [18]. As a result, the area occupied by the excavation is usually very large, which is not acceptable in urban environments.

2.1 Bottom stability

Deep excavations developed below the water table can suffer bottom stability problems when pore water pressure (P_W) is too high. Bottom instabilities arise when P_W below the excavation bottom exceeds the total vertical stress (σ_V), then, according to Eq. (1) [19], when the effective vertical stress (σ'_V) is lower than 0,

$$\sigma'_V = \sigma_V - P_W, \tag{1}$$

where σ_V and P_W are defined as:

$$\sigma_V = z\gamma_S, \tag{2}$$

and

$$P_W = h\gamma_W \tag{3}$$

where γ_S and γ_W are the specific weights of the soil and water, respectively, z is the depth where σ_V is calculated, and h is the piezometric head at the same location. The two types of stability problems that excavation may suffer are liquefaction and bottom uplift (**Figure 2**) [8]. Liquefaction occurs when the excavation bottom is made of unconsolidated materials and under unconfined conditions. If σ'_V is lower than 0, the soil completely loses its shear strength and starts to behave like a fluid. Bottom uplift may occur when the excavation bottom is located above a confined aquifer whose P_W is not enough reduced [17]. Liquefaction and bottom uplift may also produce the deformation of the soil outside the excavation since the soil may migrate towards the centre of the excavation when they occur. In order to design dewatering systems to be able to guarantee the bottom stability, a safety factor (SF) is calculated below the excavation at different depths. Ideally, the depth of the points where SF is calculated should increase gradually. SF is defined as:

$$SF = \frac{\sigma_V}{P_W} \tag{4}$$

Stability is guaranteed when $SF > 1$. However, given that errors in the hydrogeological characterization are relatively common, it is advisable to consider

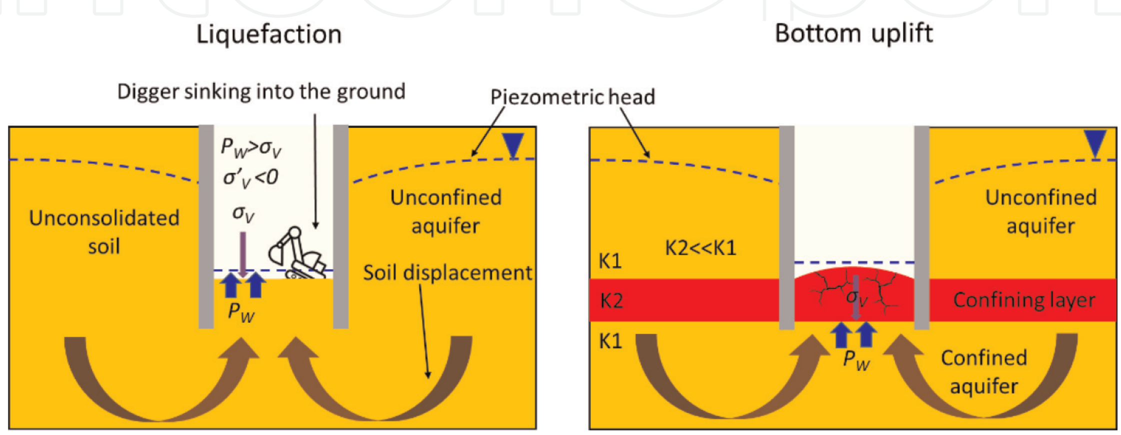

Figure 2.
Schematic description of bottom instabilities, liquefaction (left) and bottom uplift (right).

values of SF larger than 1 [20, 21]. Theoretically, a value of SF equal to or larger than 1.2 [22] should be sufficient as some strengths, like the friction between the soil and the retaining walls, that are opposite to P_W are not considered in Eq. (4). Consequently, SF computed using Eq. (4) is conservative. The stability of the bottom must be assessed for different dewatering schemes by numerically calculating the piezometric head and computing SF below the maximum excavation depth.

2.2 Impacts outside of the construction area

Dewatering systems may induce groundwater drawdown outside the excavation enclosure. Despite this impact is temporal, it must be considered in arid and semi-arid regions where water scarcity is a source of concern. Thus, dewatering systems must modify the piezometric head distribution outside of the enclosure as little as possible, which can be reached by deepening the retaining walls and avoiding oversizing the dewatering system more than needed to ensure stable conditions. Soil subsidence outside the enclosure as a result of the groundwater drawdown is another consequence of the dewatering. Soil subsidence (i.e., pumping settlement) is really feared in urban areas, the reason for which dewatering systems consisting of deep pumping wells are sometimes discarded. Instead, less efficient solutions are adopted like deepening the retaining walls more than needed until reaching deep low conductive layers [23] or using a jet-grouting bottom plug [24]. However, dewatering is not the only cause of soil subsidence occurring outside excavation; there are other construction-related actions that may induce larger soil deformations, like the digging of the retaining walls [20, 25]. Anyway, soil subsidence associated with the dewatering must be estimated during the design phase in order to choose the dewatering scheme that ensures more stability and less settlements. Pumping settlements can be calculated by using hydro-mechanical numerical models but they are time-consuming and a wide variety of parameters (hydraulic and mechanical) are needed, which are difficult to estimate. Therefore, analytical tools to predict the pumping settlements during the design phase in a short period and with acceptable error are of paramount importance.

Settlements can be computed by establishing the relation between the volumetric strain in the vertical direction (ε_z) and the vertical effective stress (σ'_z) as follows:

$$\varepsilon_z = \frac{\Delta z}{b} = \alpha \Delta \sigma'_V \tag{5}$$

where Δz is the length variation in the vertical direction (i.e., the settlement), b is the aquifer thickness and α is the compressibility of the porous material. Assuming a Biot's coefficient equal to 1, which is realistic for soft soils [26], then Eq. (1) is valid and σ_V can be considered constant. Thus,

$$\Delta z = \alpha b \Delta P_W \tag{6}$$

Eq. (6) is commonly used for the quantification of settlements caused by groundwater variations [17]. The meaning of α depends on the mechanical boundary conditions (i.e., displacement constraints). If the soil can only be deformed in the vertical (i.e., it is laterally confined), then α is the vertical compressibility of the soil [27] and is defined as [28–30].

$$\alpha = \frac{1}{(\lambda + 2G)} = \frac{(1+\nu)(1-2\nu)}{E(1-\nu)} \tag{7}$$

where G is the shear modulus, λ is the lame constant (drained conditions), E is the Young's modulus and ν is the Poisson's coefficient. If it is considered that displacements produced by groundwater drawdown occur in the three dimensions (e.g., in the presence of a pumping well [29, 30]), α can be defined as:

$$\alpha = \frac{1}{\left(\lambda + \frac{2}{3}G\right)} = \frac{3(1-2\nu)}{E} \tag{8}$$

Settlements around a single pumping well in steady state assuming a flat porous medium with axial symmetry can be computed from the solution proposed by Bear and Corapcioglu (1981) [30] as:

$$\Delta z = \frac{\rho_W g}{(\lambda + G)} \frac{Q}{4\pi K} \, ln \, \frac{R}{r} \tag{9}$$

where Q is the pumping rate, R is the influence radius of the pumping, r is the distance between the well and the site where the settlement is computed and K is the hydraulic conductivity of the porous medium ($K = T/b$, where T is the transmissivity of the aquifer. ρ_W and g are the water density and the gravitational constant, respectively, and are needed to express the variation of the groundwater head in Pascals. Note that Bear and Corapcioglu (1981) [30], defined α as $1/(\lambda + G)$ that is somewhat inconsistent with the common definition of compressibility shown in Eq. (8).

Previous equations assume that pumping settlements are proportional to groundwater drawdown, but this seems not to be true in the proximity of pumping wells as pointed out by Pujades [29] who modified Eq. (9) to improve the estimation of settlements occurring around pumping wells as follows:

$$\Delta z = \frac{\alpha \rho_W g Q}{4\pi K} = \begin{cases} ln\left(\dfrac{0.3b}{R}\right) & \text{for } r < \dfrac{9b}{70} \\ ln\left(\dfrac{0.3b}{R}\right) + \dfrac{ln^2\left(\frac{70r}{9b}\right)}{4\,ln\left(\frac{7}{3}\right)} & \text{for } r \geq 9b/70 \text{ and } r < 0.7b \\ ln\left(\dfrac{r}{R}\right) & \text{for } r \geq 0.7b \end{cases} \tag{10}$$

Assuming elastic behavior for aquifers, which occurs in the case of overconsolidation, the term $\alpha\rho_W g$ can be replaced by the specific storage coefficient (S_S) by considering the equation for elastic aquifers proposed by Jacob [31, 32]:

$$S_S = \rho_W g\alpha + \rho_W g\theta\beta \tag{11}$$

where θ is the porosity and β is the water compressibility coefficient. The first term of Eq. (12) ($\rho_W g\alpha$) is associated with soil deformation (S_{SE}) while the second term ($\rho_W g\theta\beta$) is related to water (S_{SW}). Considering that shallow-draining soils are much more compressible than water [33, 34], S_{SW} can be neglected. Thus,

$$S_S = S_{SE} = \rho_W g\alpha \tag{12}$$

Using S_S instead of $\rho_W g\alpha$ facilitates the computation of settlements since S_S can be easily derived from pumping tests. Parameters related to the compressibility of the soil can be also obtained from laboratory tests, but results may not be realistic since tested samples (i) cannot be representative of large volumes of aquifer and/or (ii) can be altered during the extraction processes.

Two kinds of settlements can be differentiated depending on their spatial distribution. If settlements are constant and their magnitude does not vary across the space, they are known as 'non-differential settlements', whilst 'differential settlements' occur when the soil deformation varies depending on the location. Potential damage of pumping settlements increases as induced settlements are more differential. Groundwater head distribution far from pumping wells is usually nearly flat, and therefore, large differential settlements are not expected from its variation. Largest hydraulic head variations are expected in the proximities of pumping wells, but, as observed by Pujades [29], pumping settlements close to pumping wells are constant and non-differential as a result of an arch effect that develops around the well.

However, this fact does not mean that pumping settlements are always non-differential and harmless because groundwater drawdown produces differential settlements in several situations, for example, when the aquifer is heterogeneous, which is relatively common. Variations in the hydraulic conductivity, the mechanical parameters or the geometry of the pumped aquifer induce differential settlements that can damage nearby infrastructures. This fact highlights the paramount importance of performing a detailed geological, hydrogeological and geotechnical characterization before designing dewatering systems.

3. Retaining walls

The objectives of the retaining walls are: (i) to allow excavating between vertical walls [5], (ii) to prevent the groundwater entrance into the excavation [35] and (iii) to minimize the impacts of the dewatering outside the enclosure [36]. There are different kinds of retaining walls that can be used for developing deep urban excavations below the water table such as retaining walls made up of concrete piles and panels or jet-grouting piles. Sheet piles can be also used for urban excavations but they are not used to support very deep excavations [37]. Retaining walls are made up of impervious materials to prevent the groundwater flow. However, construction defects in the retaining walls, through which groundwater can easily flow, are relatively common [13, 14]. There are several factors that may cause defects and they depend on the nature of the retaining walls. Defects in jet grouting retaining walls may be related to deviations and variations of the column dimensions [14], which is common in high vertical heterogeneity soils [38] because the area affected by the treatment depends on the properties of the soil. In addition, the presence of coarse sediments may produce shadow effects, leading to zones without treatment and openings [11]. The permeability of retaining walls made up of concrete piles or panels (i.e., diaphragm walls) can also be compromised by construction defects [21, 39] that can lead to openings. Openings may occur when: (i) sediments made of large boulders are drilled, (ii) the walls of the drilled space collapse or (3) the configuration of the slurry wall excavator is not suitable. Note that faulty enclosures commonly have numerous defects because, similar difficulties arise during the drilling of each one of the individual structures (i.e., concrete piles and panels or jet-grouting piles) that constitute an underground enclosure.

3.1 Consequences of faulty retaining walls

Consequences of faulty retaining walls depend on the relative location of the defects with respect to the bottom of the excavation (**Figure 3**). If defects are located above the excavation bottom, groundwater inflow (i) may drag sediments from outside leading to the formation of sinkholes [22] and (ii) may flood the excavation making it difficult for construction tasks. In addition, groundwater drawdown and its associated consequences (i.e., settlements) will increase outside the enclosure. If defects are located below the bottom of the excavation, drawdown and settlements will also increase outside the excavation site. Moreover, the pore water pressure will be higher than expected below the excavation leading to bottom instabilities (i.e., bottom uplift or liquefaction, structure instability and subsidence related to soil migration [15]).

3.2 Detection of defects

If retaining walls have defects, corrective measures can be carried out such as: (i) injecting a sealing substance to repair them, or alternatively, (ii) redesigning the dewatering system considering their actual hydraulic condition. In any case, the state of the retaining walls must be assessed before starting the excavation stage because corrective measures must be carried out before starting the pumping. If the excavation and the dewatering have started and there are attempts to repair the defects, the sealing substance will be dragged towards the pumping wells. As a result, the defects will not be repaired and the pumping wells may be damaged. If the dewatering system needs to be redesigned, new pumping wells should also be drilled before the excavation stage to minimize interference with the construction tasks and avoid bottom instabilities. Pujades et al. [7] and Pujades et al. [16] proposed two methods to assess the hydrogeological behavior of linear and non-linear underground enclosures and locate isolated defects. Both methods are based on pumping tests carried out inside the underground enclosure and their interpretation by using diagnostic plots. Both methods are based on the premise that changes in the flow behavior between linear and radial during the pumping depend on the characteristics of the retaining walls. Two different types of retaining walls are considered depending on the number and

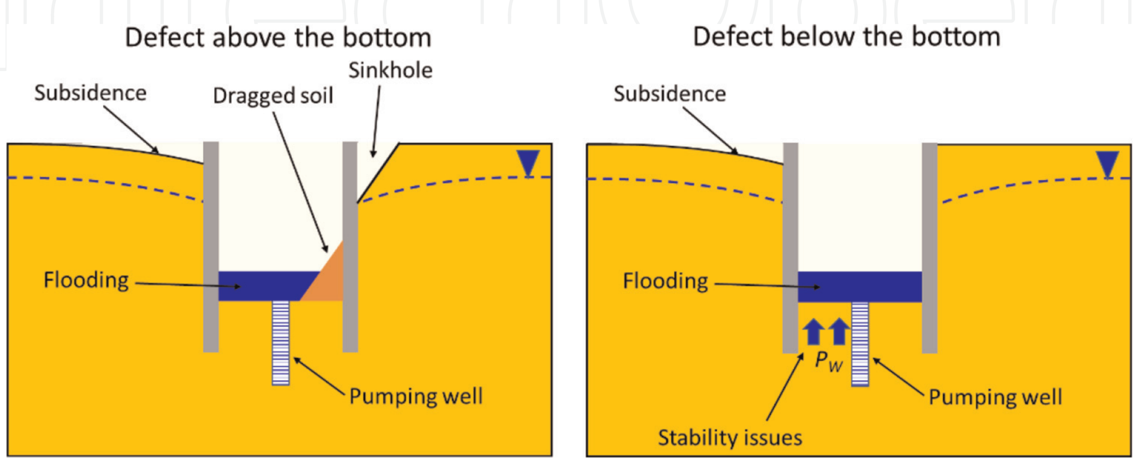

Figure 3.
Consequences of faulty retaining walls if defects are located above (left) and below (right) the bottom of the excavation.

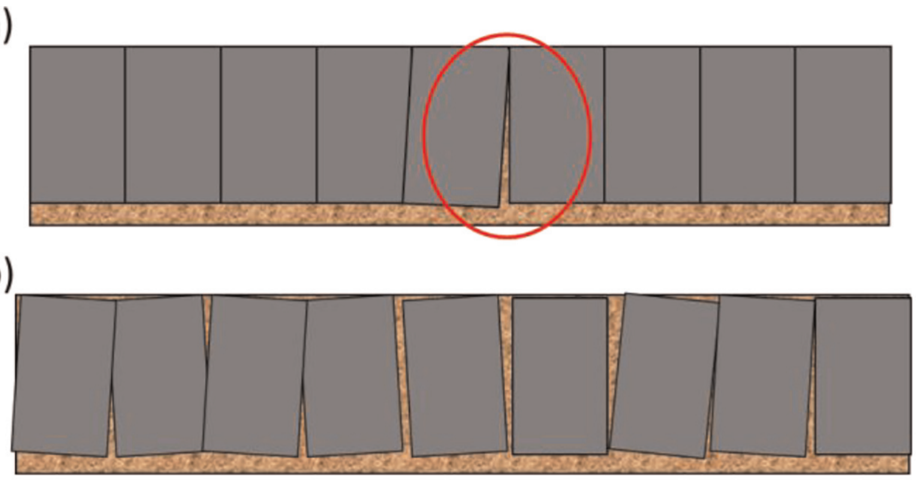

Figure 4.
Example of (a) heterogeneous and (b) homogeneous retaining walls.

distribution of defects (**Figure 4**). If there are isolated defects, retaining walls are named 'heterogeneous', while if there are numerous defects more or less homogeneously distributed, retaining walls are named 'homogeneous'.

3.2.1 Linear enclosures

Linear underground enclosures refer to underground excavations with a linear shape surrounded by retaining walls. This kind of enclosure is used, for example, for constructing tunnels. **Figure 5** illustrates the flow behavior evolution during pumping inside a linear underground enclosure. The flow is radial when the pumping starts because groundwater flows from all directions (**Figure 5a**). The flow is radial until the effect of the retaining walls reaches the pumping well, at this time, $0.1t_{L1}$, the behavior of the flow starts to change from radial to linear. The flow is totally linear at a time equal to t_{L1}. t_{L1} is defined as:

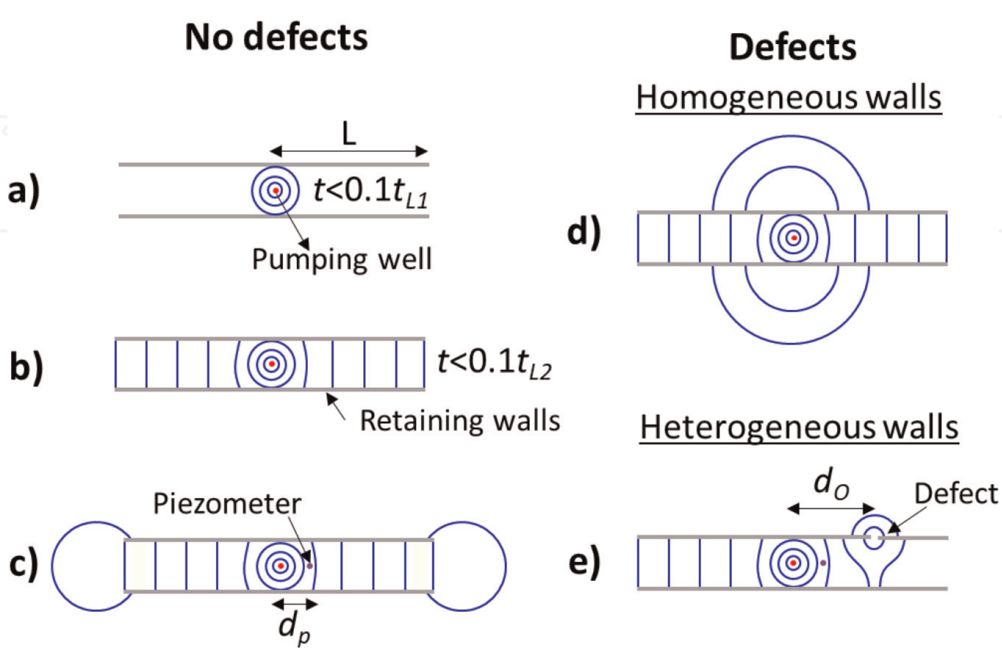

Figure 5.
Schematic description of the flow behavior during a pumping inside a linear enclosure.

$$t_{L1} = \frac{S(d_{dw})^2}{T} \tag{13}$$

where S is the storage coefficient of the aquifer, T is the transmissivity of the aquifer and d_{dw} is the distance between retaining walls. After t_{L1}, the flow is totally linear since pumped groundwater only arrives from the inner part of the enclosure that is drained linearly. If retaining walls are located at the edges of the enclosure, the flow continues to be linear until the enclosure is totally drained. However, if there are no retaining walls closing the edges of the enclosure, groundwater coming from all directions of the aquifer (i.e., with a radial behavior) enters the enclosure through them. Then, when the effect of the opened lateral edges of the enclosure reaches the pumping well the flow behavior at the pumping well changes from linear to radial. The behavior of the flow starts to change from linear to radial at a time equal to $0.1t_{L2}$ and it is totally radial after t_{L2}. t_{L2} is defined as:

$$t_{L2} = \frac{S(2L)^2}{T} \tag{14}$$

where L is the distance from the well until the end of the underground enclosure. If the behavior of the flow is observed in an observation point, t_{L2} is as follows:

$$t_{L2} = \frac{S\left(2L - d_p\right)^2}{T} \tag{15}$$

where d_p is the distance from the well until the observation point. The flow changes faster from linear to radial if there are defects in the retaining walls. If there are numerous defects and the retaining walls can be considered homogeneous (**Figure 4b**), the behavior of the flow starts to change from linear to radial at $0.1t_{L2.1}$, where $t_{L2.1}$ is defined as:

$$t_{L2.1} = \frac{S(1/\lambda)^2}{T} \tag{16}$$

and

$$\lambda = \sqrt{2\alpha_{dw}/Td_{dw}} \tag{17}$$

where $\alpha_{dw} = T_{dw}/w_{dw}$ is the leakage coefficient (conductance) of the retaining wall and T_{dw} and w_{dw} are the transmissivity and the thickness of the retaining walls, respectively. The inverse of λ is the length of retaining walls affected by the pumping when all pumped groundwater enters the enclosure through the defects [7]. Finally, if retaining walls have very few defects and can be considered heterogeneous (**Figure 4a**), the behavior flow changes from linear to a combination of radial and linear. The radial component is related to the groundwater entering the enclosure through the defect while the linear component is related to the groundwater coming from the side of the enclosure where the defect is not located. In this case, the flow does not behave as radial until a characteristic time equal to t_{L2}. The change in the flow behavior starts at a time of $0.1t_{L2.2}$, where $t_{L2.2}$ is:

$$t_{L2.2} = \frac{S(2d_O)^2}{T} \tag{18}$$

d_O is the distance between the pumping well and the defect. If the head evolution is measured in an observation point different from the pumping well, $t_{L2.2}$ is:

$$t_{L2.2} = \frac{S\left(2d_O - d_p\right)^2}{T} \tag{19}$$

Identifying the changes in the flow behavior during a pumping test and applying Eqs. 13 to 19, it is possible to know (i) if the retaining walls are faulty or not, (ii) if they have numerous (homogeneous) or very few (heterogeneous) defects, (iii) their effective hydraulic conductivity and (iv) the position of the defects. However, flow behavior changes are really difficult to identify by observing the drawdown evolution. In this context, the use of diagnostic plots is useful to differentiate the changes in the behavior of the flow. Diagnostic plots are developed by calculating the derivatives of drawdown with respect to the logarithm of time (s') and plotting it versus time. s' is very sensitive to drawdown variations and, for this reason, diagnostic plots allow us to detect variations that are difficult to observe in the drawdown evolution [40].

Figure 6 shows the diagnostic plots calculated by considering infinite impervious retaining walls (black line), finite impervious retaining walls (purple line), faulty homogeneous retaining walls (red line), faulty heterogeneous retaining walls (blue dashed line) and the absence of retaining walls (gray line). The characteristics of the homogeneous and heterogeneous retaining walls have been chosen to ensure that their effective hydraulic conductivity is the same, and thus, 0.1 $t_{L2.1}$ and 0.1 $t_{L2.2}$ occur at the same time. In the beginning, the flow is radial for all scenarios since the retaining walls have not affected the pumping well. When it occurs, the flow behavior becomes linear and separates from the radial tendency (**Figure 6**, gray line). If the retaining walls have no defects (**Figure 6**, purple line), the flow is linear until the pumping well

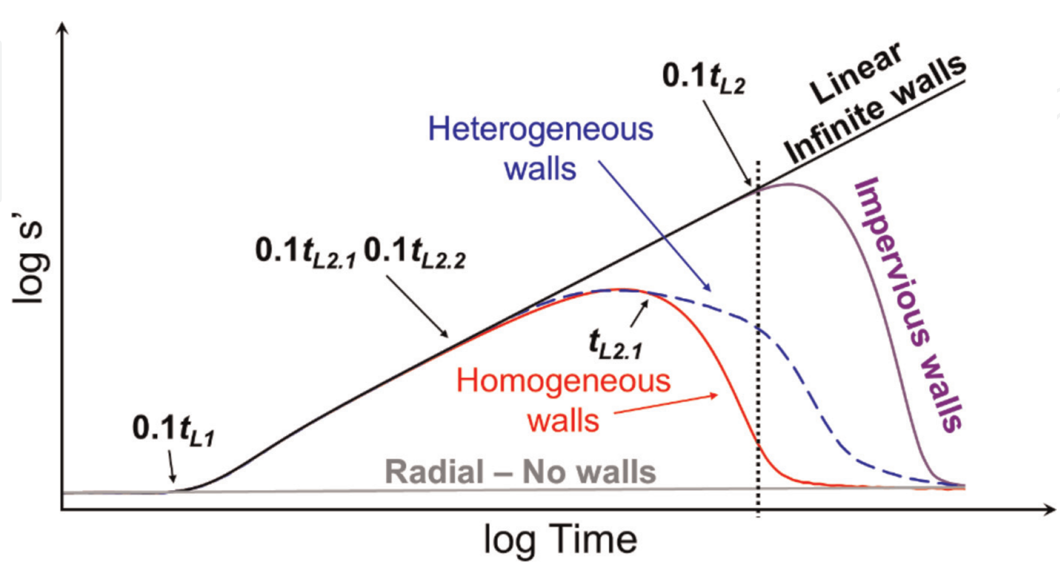

Figure 6.
Evolution of the logarithmic derivative of the drawdown (s') assuming infinite impervious retaining walls (black line), finite impervious retaining walls (purple line), faulty homogeneous retaining walls (red line), faulty heterogeneous retaining walls (blue dashed line) and the absence of retaining walls (gray line).

notices the effect of the end of the enclosure. At this time, *s'* separates from the linear tendency. If retaining walls have defects (**Figure 6**, red and blue dashed lines), the separation from the linear tendency occurs before. The transition period between linear and radial flow is shorter when retaining walls are homogeneous than when they are heterogeneous. If the retaining walls are homogeneous, the effect of radial flow starts to be observed at $0.1t_{L2.1}$ and s' is maximum at $0.5t_{L2.1}$, when half of the pumped flow is radial. This proportion increases with time and s' as the flow behavior becomes more and more radial. If the retaining walls are heterogeneous, the flow behavior separates from the linear tendency at $0.1t_{L2.2}$, but s' decrease more slowly than for homogeneous walls because the linear component of the continues being relatively high as groundwater from the aquifer (i.e., from all directions) only reaches the enclosure through the defect. The decrease rate of s' increases when the effect on the groundwater behavior of the lateral edges of the enclosure starts to be observed in the pumping well, then at $0.1t_{L2}$. **Figure 6** shows that *s'* has a different evolution depending on the kind of retaining walls. Once the retaining walls are classified as homogeneous or heterogeneous, it is possible to calculate their effective hydraulic conductivity (homogeneous retaining walls) or the position of the defect (heterogeneous retaining walls) only by identifying the times when s´ separates from the linear tendency.

Diagram in **Figure 7** aims at clarifying the changes in the flow behavior and the times when they occur. Difference in the flow behavior when the retaining walls are impervious or faulty occurs when the flow changes from linear to radial since t_{L1} is the same in all situations.

3.2.2 Non-linear enclosures

Non-linear underground enclosures refer to underground excavations with a non-linear shape surrounded by retaining walls. This kind of enclosure is used, for example, for constructing stations or emergency shafts for deep tunnels. **Figure 8a** illustrates the evolution of the groundwater flow behavior when pumping inside a non-linear enclosure and **Figure 8b** shows the evolution of the logarithmic derivative of the drawdown (*s'*) considering impervious retaining walls (black line), faulty homogeneous retaining walls (red line) and the absence of retaining walls (gray line). Groundwater flow behaves radially (stage 1) from the beginning of the pumping until the effect of the retaining walls is noticed at the observation point (e.g., the pumping

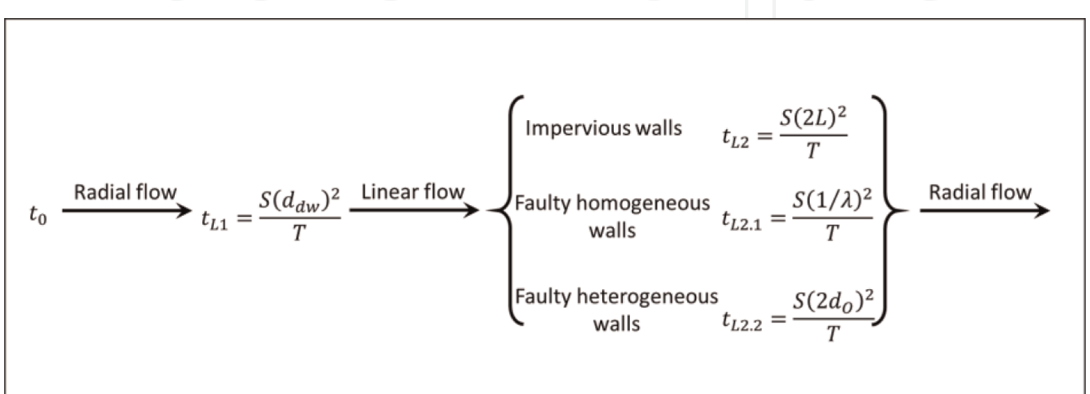

Figure 7.
Diagram of the flow behavior evolution for impervious and faulty retaining walls.

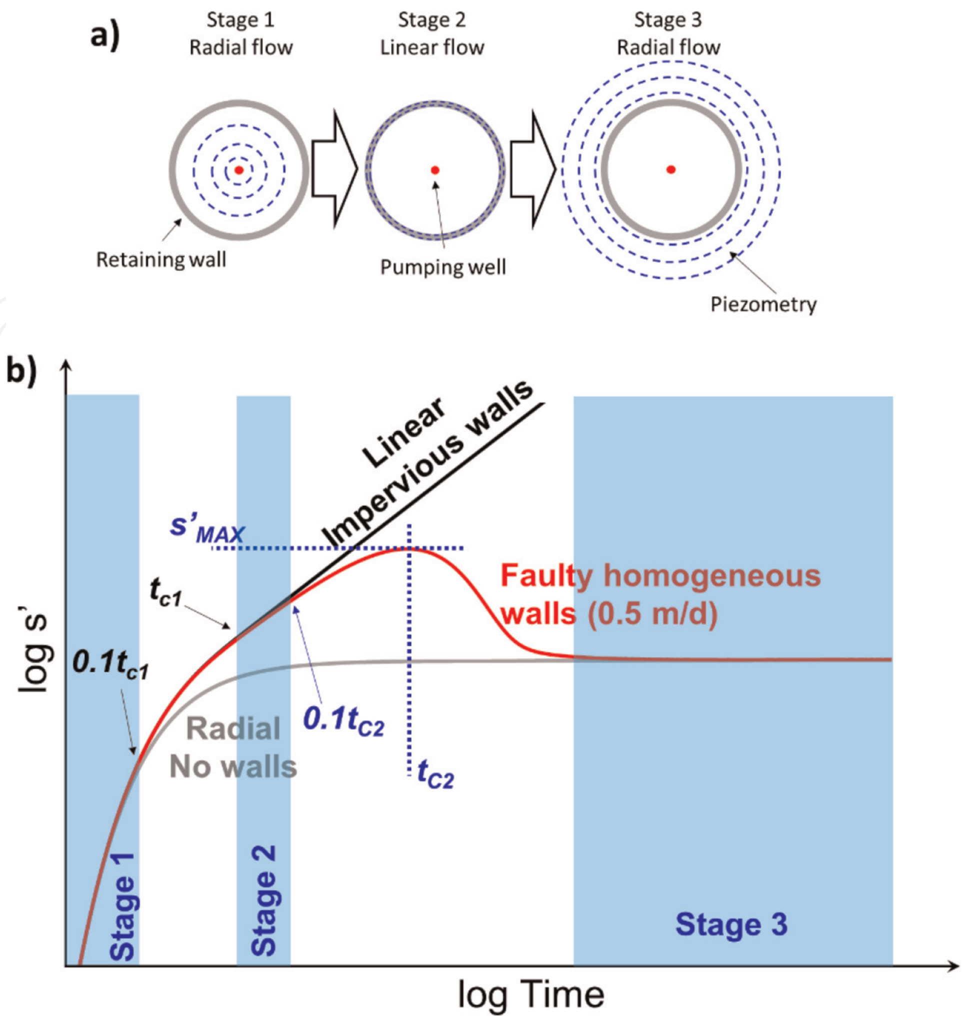

Figure 8.
a) Schematic description of the flow behavior during a pumping inside a circular (i.e., non-linear) enclosure. b) Evolution of the logarithmic derivative of the drawdown (s') assuming impervious retaining walls (black line), faulty homogeneous retaining walls with a hydraulic conductivity of 0.5 m/d (red line) and the absence of retaining walls (gray line).

well), when the flow behavior changes from radial to linear. This change in the flow behavior starts at $0.1t_{C1}$ and the flow is totally linear at t_{C1} defined as:

$$t_{C1} = \frac{S(2r_{exc})^2}{T} \tag{20}$$

where r_{exc} is the radius of the excavation. Note that t_{C1} may vary if the pumping well is not located in the centre of the excavation, but this effect is too short and can be neglected. In **Figure 8b**, $0.1t_{C1}$ occurs when the black and red lines separate from the gray line (i.e., the scenario without retaining walls where the groundwater flow is always radial). From t_{C1} to $0.1t_{C2}$, the flow is linear (stage 2) because the excavation is drained vertically and groundwater only comes from the inner part of the enclosure. The groundwater flow that crosses the retaining walls (Q_W) increases progressively as the piezometric head inside the enclosure decreases. Given that Q_W represents the portion of the groundwater crossing the retaining walls from the aquifer, it is a radial component controlling the flow behavior. Stage 2 finishes when the groundwater entering the enclosure through the retaining walls cannot be neglected and s'separates from the linear tendency (**Figure 8b**). After, there is a transition period until the flow behavior is totally radial (stage 3). During the first part of the transition period from stage 2 to stage 3, the pumped groundwater coming from inside the enclosure (Q_E) is higher than Q_W, and then, *s'* continues increasing. Note that Q_E is a linear component of the flow, and then, the flow behaves linearly as the portion of Q_E increases. During the second part of the transition period, Q_W is higher than Q_E and *s'* decreases. The inflection point in *s'* occurs when Q_W is equal to Q_E at t_{C2}. At this time, the maximum value of *s'* (s'_{MAX}) is reached. t_{C2} and s'_{MAX} are related to the effective hydraulic conductivity of the retaining walls (K_{eff}) as follows:

$$K_{eff} = \frac{Sw_W r_{exc}}{2t_{C2}b}, \tag{21}$$

and

$$K_{eff} = \frac{Q_p w_W}{2\pi r_{exc} s'_{MAX} b} \, ln\, 2.25 \tag{22}$$

Stage 3 (**Figure 8**) is reached when all pumped groundwater comes from outside the enclosure and Q_E is 0 m/d.

The application procedure to ascertain the state of the retaining walls in a non-linear enclosure is as follows [16]:

1. To determine the hydraulic parameters of the subsurface before starting the construction process.

2. To perform a watertightness assessment test (WTAT) [20, 41, 42] before starting the excavation and after drilling the retaining walls. The pumping well and piezometers must be located inside the underground enclosure, and at least, one observation point is needed.

3. To calculate *s'* from the observed drawdown and identify s'_{MAX} and t_{C2}. Depending on the location of the pumping and observation points, the number of observation points and the measured drawdown, different approaches can be adopted:

    a. K_{eff} can be always estimated from t_{C2} by using Eq. (21). Only one observation point is needed, but it is advisable to calculate K_{eff} using different observation points and compare the results.

    b. Pumping well and observation points are randomly distributed (**Figure 9a**): The pumping well is placed anywhere (i.e., it can be centred or not) and the piezometers are located at different distances from it. In this case, the retaining walls can be assessed from s'_{MAX} by using data from more than one observation point. If s'_{MAX} is equal in all observation points, the retaining walls behave homogenously and their K_{eff} can be computed by Eq. (22). If s'_{MAX} varies within observation points, (i) the enclosure is faulty and behaves heterogeneously or (ii) the retaining walls are homogeneous and their $K_{eff} \geq 0.01$ K. In this case, K_{eff} can only be estimated from t_{C2} by applying Eq. (21).

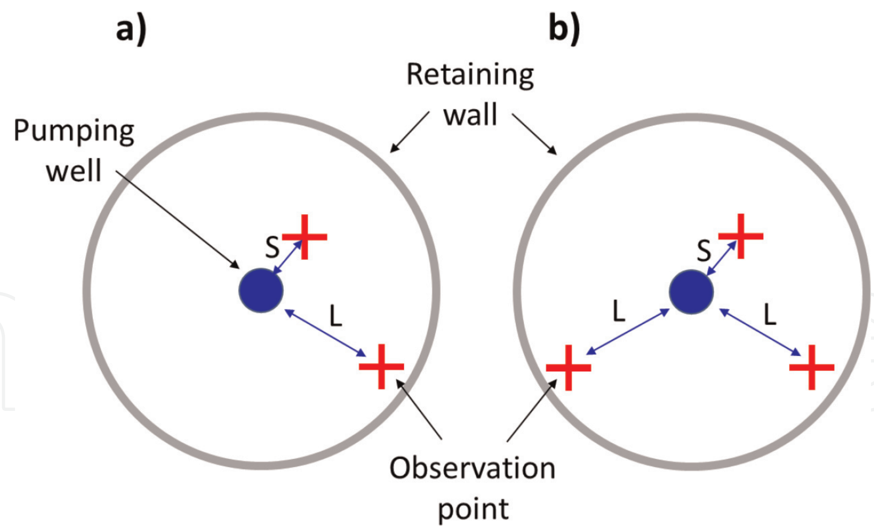

Figure 9.
Schematic description of the pumping well and observation points needed to characterize the state of a circular enclosure by a WTAT. ***Figure 9a*** *Shows a random distribution while* ***Figure 9b*** *shows a strategic distribution of all the required elements. S and L represent the distance from the well to the piezometers.*

c. Pumping well and observation points are strategically distributed (**Figure 9b**): The retaining walls can be assessed using s'_{MAX}. In this case, it is advisable to drill, at least, three piezometers and one centred pumping well. Two of the piezometers should be located equidistant from the pumping well whilst the other one should be drilled at a different distance.

   i. If s'_{MAX} is the same in all observation points, K_{eff} can be calculated by applying Eq. (22).

   ii. If s'_{MAX} is only equal in the observation points located at the same distance, the retaining walls are homogeneous and $K_{eff} \geq 0.01\,K$. Thus, K_{eff} cannot be estimated from s'_{MAX}.

   iii. If s'_{MAX} is not the same at any observation point, even at those located at the same distance from the pumping well, the retaining walls behave heterogeneously and K_{eff} cannot be estimated from s'_{MAX}. In the hypothetical and unlikely case that there is a defect located at the same distance to the observation points equidistant to the pumping well, s'_{MAX} will be equal at both observation points. Then, another piezometer at the same distance as the previous ones would be needed to establish the behavior of the retaining walls.

4. Conclusions

This chapter is focused on the interaction between deep excavations performed by the cut-and-cover method and groundwater. The first part of the chapter explains how to design dewatering systems to bring about workable conditions (i.e., dry and stable conditions) and estimate settlements generated outside the excavation as a result of lowering the piezometric head. Despite there being different kinds of dewatering systems, deep pumping wells seem to be the most adequate in the context of deep excavations. Deep pumping wells are useful to provide dry conditions, but also to reduce the water pressure below the excavation bottom bringing about stable conditions and avoiding liquefaction or bottom uplift events. The bottom stability must be anticipated during the design phase of dewatering systems by computing, for example, the safety factor as the ratio between the total vertical stress and the water pressure. The chosen dewatering system must be the one that, providing adequate conditions for the excavation, causes the minimum impacts outside of the construction area. Pumping settlements that occur as a result of lowering the piezometric head are really feared, especially in urban areas, because they can damage nearby infrastructures. Therefore, they must be estimated when designing dewatering systems. Hydromechanical models allow for estimating soil deformations precisely, but they are time-consuming and numerous parameters are needed. Thus, analytical equations like those proposed in this chapter are really useful to approximate the magnitude of the pumping settlements generated by a dewatering system.

The second part of the chapter describes how to ascertain the state of the retaining walls before starting the excavation phase. Retaining walls are essential to develop deep excavations in urban areas since they allow excavating within the enclosure of vertical walls (minimizing the needed space) and obstruct the flow of groundwater towards the excavation. However, defects in retaining walls are relatively common. If defects are not detected and remediation actions are not taken before starting the excavation phase, several issues may arise like the development of sinkholes in the outer part of the excavated enclosure, bottom instabilities or excavation flooding events. Here, two methods based on the interpretation of pumping tests are explained for the assessment of linear and non-linear enclosures. The methods allow to determine if the retaining walls have many defects or not. In the case where there are many defects, the hydraulic properties of the retaining walls can be known by identifying times when the behavior of the groundwater flow changes and applying the proposed equations. If there are few isolated defects, they can be located by analyzing the evolution of the groundwater flow behavior and using the proposed equations.

Acknowledgements

This research has been supported by the grant CEX2018-000794-S funded by MCIN/AEI/ 10.13039/501100011033, the grant RYC2020-029225-I funded by MCIN/AEI/10.13039/501100011033 and by 'ESF Investing in your future'. EP gratefully acknowledges the support received from the Barcelona City Council through the Award for Scientific Research into Urban Challenges in the City of Barcelona 2020 (20S08708) and the Spanish Ministry of Science, Innovation and Universities (MCIN/AEI /10.13039/501100011033/) and FEDER "one way to make Europe" through the project PID2021-128995OA-I00. M. Jing acknowledges the financial support from the National Natural Science Foundation of China (52109012).

Conflict of interest

The authors declare no conflict of interest.

Appendices and nomenclature

P_W	Water pressure
σ_V	total vertical stress
σ'_V	effective vertical stress
γ_S	specific weight of soil
γ_W	specific weight of water
z	soil/aquifer depth
h	piezometric head
SF	safety factor
ε_z	volumetric strain in the vertical direction
Δz	settlement − length variation in the vertical direction
b	aquifer thickness
α	compressibility of the porous material
G	shear modulus
λ	lame constant (drained conditions)
E	Young's modulus
ν	Poisson's coefficient
Q	pumping rate
R	influence radius of a pumping
r	radial distance from the pumping well
K	hydraulic conductivity
T	transmissivity of the aquifer
ρ_W	water density
g	gravitational constant
S_S	specific storage coefficient
θ	(porosity)
β	water compressibility coefficient
S_{SE}	portion of the specific storage coefficient associated with soil deformation
S_{SW}	portion of the specific storage coefficient associated with water
t_{L1}	time at which the behavior of the flow changes from radial to linear in the case of linear enclosures
d_{dw}	distance between retaining walls in liner enclosures
t_{L2}	time at which the behavior changes from linear to radial in the case of linear enclosures
L	distance from the well until the end of the linear underground enclosure
d_p	distance from the well until the observation point
$t_{L2.1}$	time at which the behavior of the flow changes from linear to radial behavior in case of faulty homogeneous retaining walls
$t_{L2.2}$	time at which the behavior of the flow changes from linear to radial behavior in case of faulty homogeneous retaining walls
α_{dw}	conductance of the retaining walls
T_{dw}	transmissivity of the retaining walls
w_{dw}	thickness of the retaining walls
d_O	distance between the pumping well and the defect
d_p	distance between the pumping well and the piezometer
s'	derivate of drawdown with respect to the logarithm of time
t_{C1}	time at which the behavior of the flow changes from radial to linear in the case of non-linear enclosures

r_{exc} radius of the excavation
Q_W pumped groundwater that reaches the inner part of the excavation by crossing the retaining walls
Q_E pumped groundwater coming from inside the enclosure
s'_{MAX} maximum value of s'
t_{C2} time when s'_{MAX} occurs

Author details

Estanislao Pujades[1*], Miao Jing[2,3], Chunhui Lu[3,4] and Anna Jurado[1]

1 Department of Geosciences, Institute of Environmental Assessment and Water Research (IDAEA), Severo Ochoa Excellence Center of the Spanish Council for Scientific Research (CSIC), Barcelona, Spain

2 School of Earth Science and Engineering, Hohai University, Nanjing, China

3 State Key Laboratory of Hydrology-Water Resources and Hydraulic Engineering, Hohai University, Nanjing, China

4 Yangtze Institute for Conservation and Development, Hohai University, Nanjing, China

*Address all correspondence to: estanislao.pujades@idaea.csic.es

References

[1] United Nations, Department of Economic and Social Affairs, Population Division. World Urbanization Prospects: The 2018 Revision. New York: UN; 2019

[2] Konda T. Shield tunneling method. Civil Engineering, Japan Society of Civil Engineers. 2001;**39**:23-27

[3] Forth RA. Groundwater and geotechnical aspects of deep excavations in Hong Kong. Engineering Geology. 2004;**72**(3):253-260

[4] Puller M. Deep Excavations: A Practical Manual. London: Thomas Telford; 1996. p. 474

[5] Xanthakos PP, Abramson LW, Bruce DA. Ground Control and Improvement. New York: John Wiley & Sons; 1994 938 p

[6] Linde-Arias E, Harris D, Davis A. Depressurisation for the excavation of Stepney green cavern. Proceedings of the Institution of Civil Engineers - Geotechnical Engineering. 2015;**168**(3): 215-226

[7] Pujades E, Carrera J, Vázquez-Suñé E, Jurado A, Vilarrasa V, Mascuñano-Salvador E. Hydraulic characterization of diaphragm walls for cut and cover tunnelling. Engineering Geology. 2012; **125**:1-10

[8] Jurado A, De Gaspari F, Vilarrasa V, Bolster D, Sánchez-Vila X, Fernàndez-Garcia D, et al. Probabilistic analysis of groundwater-related risks at subsurface excavation sites. Engineering Geology. 2012;**125**:35-44

[9] Guo F, Wang GH, Li ZC. Influence of artificial recharge in a phreatic aquifer on deep excavation dewatering: A case study of Dongguantou Nan Station in Beijing, China. Hydrogeology Journal. 2022;**30**(2):673-689

[10] Wu YX, Lyu HM, Han J, Shen SL. Dewatering—Induced building settlement around a deep excavation in soft deposit in Tianjin, China. Journal of Geotechnical and Geoenvironmental Engineering. 2019;**145**(5):05019003

[11] Vilarrasa V, Carrera J, Jurado A, Pujades E, Vázquez-Suné E. A methodology for characterizing the hydraulic effectiveness of an annular low-permeability barrier. Engineering Geology. 2011;**120**(1):68-80

[12] Pujades E, Vázquez-Suñé E, Carrera J, Vilarrasa V, De Simone S, Jurado A, et al. Deep enclosures versus pumping to reduce settlements during shaft excavations. Engineering Geology. 2014;**169**:100-111

[13] Bruce DA, De Paoli B, Mascardi C, Mongilardi E. Monitoring and quality control of a 100 metre deep diaphragm wall. In: Pilling and Deep Foundations. AA Balkema Rotterdam. 1989. p. 23-32

[14] Croce P, Modoni G. Design of jet-grouting cut-offs. Proceedings of the Institution of Civil Engineers - Ground Improvement. 2007;**11**(1):11-19

[15] Xu YS, Shen SL, Du YJ. Geological and hydrogeological environment in Shanghai with geohazards to construction and maintenance of infrastructures. Engineering Geology. 2009;**109**(3):241-254

[16] Pujades E, Jurado A, Carrera J, Vázquez-Suñé E, Dassargues A. Hydrogeological assessment of non-linear underground enclosures. Engineering Geology. 2016;**207**:91-102

[17] Cashman PM, Preene M. Groundwater Lowering in Construction: A Practical Guide. London: CRC Press; 2001. p. 498

[18] Shaqour FM, Hasan SE. Groundwater control for construction purposes: A case study from Kuwait. Environmental Geology. 2008;**53**(8): 1603-1612

[19] Terzaghi K, Peck RB, Mesri G. Soil Mechanics in Engineering Practice. New York: John Wiley & Sons; 1996. p. 579

[20] Pujades E, Vàzquez-Suñé E, Carrera J, Jurado A. Dewatering of a deep excavation undertaken in a layered soil. Engineering Geology. 2014;**178**: 15-27

[21] Wu YX, Shen SL, Xu YS, Yin ZY. Characteristics of groundwater seepage with cut-off wall in gravel aquifer. I: Field observations. Canadian Geotechnical Journal. 2015;**52**(10): 1526-1538

[22] Pujades E, Jurado A. Groundwater-related aspects during the development of deep excavations below the water table: A short review. Underground Space. 2021;**6**(1):35-45

[23] Baxter DY. Mechanical Behavior of Soil-Bentonite Cutoff Walls [Thesis]. USA: Virginia Tech; 2000

[24] Modoni G, Flora A, Lirer S, Ochmański M, Croce P. Design of jet Grouted Excavation Bottom Plugs. Journal of Geotechnical and Geoenvironmental Engineering. 2016; **142**(7):04016018

[25] Pujades E, Vázquez-Suñé E, Culí L, Carrera J, Ledesma A, Jurado A. Hydrogeological impact assessment by tunnelling at sites of high sensitivity. Engineering Geology. 2015;**193**:421-434

[26] Verruijt A. Consolidation of soils. In: Encyclopedia of Hydrological Sciences. Chichester, UK: John Wiley & Sons, Ltd; 2008

[27] Verruijt A. Theory of consolidation. In: Verruijt A, editor. An Introduction to Soil Dynamics. Dordrecht: Springer Netherlands; 2010. p. 65-90. DOI: 10.1007/978-90-481-3441-0_4

[28] Merxhani A. An introduction to linear poroelasticity. arXiv preprint. 2016; arXiv:160704274

[29] Pujades E, De Simone S, Carrera J, Vázquez-Suñé E, Jurado A. Settlements around pumping wells: Analysis of influential factors and a simple calculation procedure. Journal of Hydrology. 2017;**548**:225-236

[30] Bear J, Corapcioglu MY. Mathematical model for regional land subsidence due to pumping: 2. Integrated aquifer subsidence equations for vertical and horizontal displacements. Water Resources Research. 1981;**17**(4):947-958

[31] Jacob CE. Flow of Groundwater in Engineering Hydraulic. In: Rouse H, editor. Engineering Hydraulics. New York: John Wiley & Sons; 1950. pp. 321-386

[32] Ferris JG, Knowles DB, Brown RH, Stallman RH. Theory of aquifer tests. Theory of Aquifer Tests. Washington: U. S. Government Print Office; 1962, (Water Supply Paper; vols. 1536-E). Report No.: 1536-E. Available from: h ttp://pubs.er.usgs.gov/publication/ wsp1536E [Aceessed: April 4, 2022]

[33] Scott RF. Principles of Soil Mechanics. Reading, Mass: Addison-Wesley Pub. Co.; 1963

[34] Helm DC. Horizontal aquifer movement in a Theis-Thiem confined

system. Water Resources Research. 1994;**30**(4):953-964

[35] Wu YX, Shen SL, Yuan DJ. Characteristics of dewatering induced drawdown curve under blocking effect of retaining wall in aquifer. Journal of Hydrology. 2016;**539**:554-566

[36] Zeng CF, Song WW, Xue XL, Li MK, Bai N, Mei GX. Construction dewatering in a metro station incorporating buttress retaining wall to limit ground settlement: Insights from experimental modelling. Tunnelling and Underground Space Technology. 2021;**116**:104124

[37] Singh AP, Chatterjee K. Ground settlement and deflection response of cantilever sheet Pile Wall subjected to surcharge loading. Indian Geotech Journal. 2020;**50**(4):540-549

[38] Modoni G, Croce P, Mongiovì L. Theoretical modelling of jet grouting. Géotechnique. 2006;**56**(5):335-347

[39] Wu YX, Shen SL, Yin ZY, Xu YS. Characteristics of groundwater seepage with cut-off wall in gravel aquifer. II: Numerical analysis. Canadian Geotechnical Journal. 2015;**52**(10):1539-1549

[40] Renard P, Glenz D, Mejias M. Understanding diagnostic plots for well-test interpretation. Hydrogeology Journal. 2009;**17**(3):589-600

[41] Serrano-Juan A, Pujades E, Vázquez-Suñè E, Velasco V, Criollo R, Jurado A. Integration of groundwater by-pass facilities in the bottom slab design for large underground structures. Tunnelling and Underground Space Technology. 2018;**71**:231-243

[42] Serrano-Juan A, Pujades E, Vázquez-Suñè E, Crosetto M, Cuevas-González M, Leveling vs. InSAR in urban underground construction monitoring: Pros and cons. Case of la sagrera railway station (Barcelona, Spain). Engineering Geology. 2017;**218**:1-11