Pumping a Large Diameter Well in an Unconfined Aquifer: Finding the Hydraulic Parameters

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ABSTRACT
A 3.66-m diameter well was used to pump an unconfined aquifer. The well storage had a strong influence on the time-drawdown curves, which largely differed from theory. The usual interpretation methods gave questionable values for hydraulic conductivity and storativity. Subsequently, drawdown data were assessed using a finite element analysis that takes into account the non-linear relationships linking the water content and hydraulic conductivity to the pore water pressure and suction.

RÉSUMÉ
Un puits de diamètre 3.66 m a servi à pomper un aquifère à nappe libre. La capacité du puits a eu une forte influence sur les courbes rabattement-temps, très différentes de la théorie. Les méthodes usuelles d'interprétation ont donné des valeurs douteuses pour la conductivité hydraulique et l'emmagasinement. Par la suite, les données de rabattement ont été évaluées à l'aide d'un logiciel d'éléments finis qui tient compte des relations non linéaires reliant la teneur en eau et la conductivité hydraulique à la pression d'eau et la succion.

1 INTRODUCTION

The construction of large buildings in cities means problems associated with the lowering of the water table. In old cities, a common problem is that of old wooden pile foundations of adjacent older buildings, which may be exposed to air and decay. This decay leads to differential settlement with the risk of damaging water and gas pipes.

This paper is about a dewatering problem for which all identification has been removed at the owner's request. It describes merely the constant rate pumping test performed in a large diameter well in an unconfined aquifer. First, it recalls the theories used to interpret steady and transient data, and their limitations. Then, the drawdown data of the pumping and monitoring wells (PW and MW) are analyzed. The usual interpretation had raised several questions while providing questionable results for the hydraulic parameters. Later on, a numerical analysis of the pumping test was performed to try to clarify these questions and better assess the hydraulic parameters.

This paper draws the attention of designers to the fact that available theories for unsteady state in unconfined aquifers may be misleading. It provides a detailed list of differences between how the real hydraulic behaviour is schematized in existing theories, and how it should be taken into account. The resulting highly non-linear differential equations are not amenable to closed-form solutions. Consequently, adequate numerical tools are needed to study each case. The paper explains how to use an adequate numerical tool to assess the hydraulic parameters of the unconfined aquifer.

2 TEST CONDITIONS

This study was started by the difficulty to interpret the data of a pumping test with a 3.66-m diameter well. This well was used to dewater a deep excavation reaching 4 m below the water table corresponding to a nearby water body (Fig. 1). The adjacent facilities made it impractical to install a set of pumping wells around the new building, which reached the deepest level of all nearby facilities. Installing a set of pumping wells inside the building perimeter was a hard option, which was not retained because the wells would have created problems during excavation and construction. The contractor used a well of 3.66 m in diameter, from which horizontal drains could be bored to control groundwater during excavation and later. The water collected in the large well had to be returned to the nearby water body.

The well pumped an unconfined aquifer made of densely fractured limestone, with horizontal bedding. The drilling recovery was in the 90-100% range whereas the rock RQD index was lower than 10% in the upper 11 m, and over 50% below. The rock discontinuities had an aperture that was qualified as close (less than 1 mm). Their spacing was in the range 2-6 cm vertically and 0.2-0.9 m horizontally in the upper 11 m. It was difficult to predict the water inflow into the excavation and later into the drainage system for the basement of the building. Local experience with this aquifer indicated that the pumped flow rates were highly variable, and that a pumping test could be interpreted with usual methods for single-porosity homogeneous aquifers at places where this aquifer was confined. The pumping test was performed before installing the horizontal drains.

3 CONSTANT RATE PUMPING TEST, THEORIES

Theories for pumping tests in unconfined aquifers were developed first for steady state, then for transient conditions without well storage effects, and finally for transient conditions with well storage effects (the so-called large-diameter wells). They are briefly presented below.

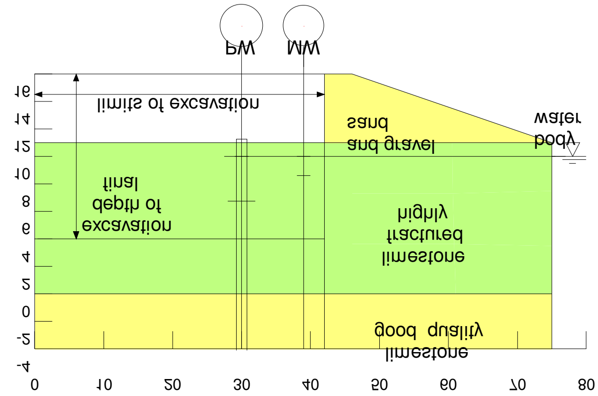

Figure 1. Schematic cross-section with the pumping and monitoring wells (PW and MW), close to a water body (the facilities surrounding the excavation are not shown).

3.1 Steady-state

For steady conditions, the graph of drawdown s versus radial distance r to the well axis is interpreted using the equation of Dupuit (1857, 1863) for a homogeneous, isotropic, unconfined, horizontal aquifer of uniform thickness, fully penetrated by the well. This equation relates the pumped flow rate, Q, to the saturated horizontal hydraulic conductivity, k_r, and the saturated thicknesses, b_1 and b_2 at radial distances r_1 and r_2 from the pumping well axis

$$Q = \pi k_r \frac{(b_2^2 - b_1^2)}{\ln(r_2/r_1)} \quad [1]$$

The transmissivity T of the unconfined aquifer is defined as $T = k_r b_i$, where b_i is the initially saturated thickness. Because the drawdown s at a radial distance r is defined as $s = b_i - b$, then

$$b_2^2 - b_1^2 = (b_i - s_2)^2 - (b_i - s_1)^2 = 2b_i\left[\left(s_1 - \frac{s_1^2}{2b_i}\right) - \left(s_2 - \frac{s_2^2}{2b_i}\right)\right] \quad [2]$$

Jacob (1944) introduced a corrected drawdown, s_c, defined as

$$s_c = s - \frac{s^2}{2\,b_i} \quad [3]$$

As a result, equation (1) simplifies to

$$Q = 2\pi\, k_r\, b_i \frac{(s_{c2} - s_{c1})}{\ln(r_2/r_1)} \quad [4]$$

This linear relationship between s_c and ln(r) is more easily presented in a graph than Eq. [1]. In addition, Eq. [4] is similar to the equation of Thiem (1906) relating s and ln (r) during steady state pumping in an ideal confined aquifer. The previous equations are known to be reliable when the drawdown is measured in the lower third of the aquifer (Charny 1951). They are less reliable for the water table position, especially close to the pumping well, because their assumptions do not consider that the equipotentials are curved close to the pumping well, and that there may be a seepage face (Chenaf and Chapuis 2007).

3.1 Unsteady-state, no well capacity

When the pumping test lowers the water table, certain aquifer volumes that were initially saturated become unsaturated. A very small volume of water is rapidly released by "elastic" settlement (storativity S) and then a larger volume of water is slowly released by vertical drainage (specific yield S_y). Vertical drainage is an unsaturated process involving Darcy's law, the mass conservation equation, the relationship between the volumetric water content, θ, and pore water pressure, u_w, and the relationship between the unsaturated hydraulic conductivity function, k, and u_w. These two relationships are highly non linear.

Several theories have been developed to try to obtain analytical solutions. However, these theories ignore the non-linearity of the unsaturated drainage process, and thus greatly simplify it to be able to get a solution. In general, they predict that the curves of s versus t should have a slanted S-shape. Early and late time overlays on type A and B theoretical curves should provide S, S_y, radial (horizontal) and vertical saturated hydraulic conductivities, k_r and k_z. Ideally, the two overlays should provide two k_r values that are equal. The usual double superimposition method is described in the Standard D5920 (ASTM 2008).

In practice, few field data have the so-called S-shape (see for example Chapuis et al. 2005). Experienced practitioners have little confidence in the theoretical methods for transient conditions. They tend to trust only the late portion of a drawdown curve to estimate k_r and S_y, neglecting the initial portion. The Standard D5920 (ASTM 2008) confirms the know-how of experienced practitioners who tend to trust only the late data. Usually, the derived k_r is considered as reliable (Chen et al. 1999). The derived S value is frequently over-estimated by one to two orders of magnitude, a difference that could be due to water storage in the pumping and observation wells (Moench 1997a, 1997b). The derived values of S_y are usually between 0.03 and 0.13 for sand and gravel aquifers, whereas they should be between 0.2 and 0.3 (Nwankwor et al. 1984, 1992). Recent theoretical improvements have shown limited ability to improve the estimation of S_y (Chen and Ayers 1998; Chen et al. 1999).

During a test with many monitoring wells, Chapuis et al. (2005) observed that none of the drawdown curves had the S-shape. Existing theories provided too small values for S_y, which varied with distance. In addition, the transient and steady-state interpretations yielded large differences in k_r values. Using finite element modeling, the k_r value obtained using the Dupuit equation for steady-state, and realistic complete curves for capillary retention and unsaturated permeability, the authors could reproduce adequately the field drawdown data versus time and radial distance. These published test data and many others (unpublished) underline the need to take into account the highly non-linear unsaturated hydraulic properties of the soil to be able to describe adequately the slow vertical drainage process during pumping tests in unconfined aquifers. This can be done only using numerical tools, considering the difficulty to get analytical solutions for a set of non-linear differential equations. In practice, however, most transient drawdown data are not interpreted using sophisticated numerical tools.

As a result, the only reliable interpretation method for pumping tests in unconfined aquifers is the old steady-state method of Dupuit (1863). However, most pumping tests last 48 or 72 hours, which is usually too short to reach steady state in an unconfined aquifer. Without the help of sophisticated numerical methods to analyze transient conditions and unsaturated drainage, the k_r value is then obtained using superposition methods derided from questionable analytical solutions that are not examined in detail in this paper. Therefore, the k_r value of unreliable transient methods cannot be checked against the k_r value obtained using the reliable steady-state equation of Dupuit (1863). This lack of verification contradicts a basic rule in quality control of input parameters for any designed facility, and thus it may have unpleasant consequences for the professional liability of designing engineers (Chapuis 1995).

3.1 Unsteady-state, well capacity

A large well capacity complicates the theoretical analysis of transient conditions. During early time, a significant portion of the pumped volume comes from water stored in the well pipe (well storage). This modifies the early portion of the log s vs. log t curves to be superimposed on type A curves (early time). Several equations have been proposed to interpret the transient data of pumping tests in unconfined aquifers, and take into account the well storage. All analytical solutions have used rough simplifications to describe the unsaturated drainage. Instead of considering the exact highly non-linear differential equations with variable coefficients, the authors simplified the equations to make them linear with constant coefficients and thus, be able to get analytical solutions. Here, we consider only an early analytical solution, which can be found in many textbooks: it was the available solution about 20 years ago when the examined pumping test was performed. This solution (Boulton and Streltsova 1976) involves a well function W of u_A and six other parameters, to describe the effect of well capacity for unconfined anisotropic aquifers pumped by a partially penetrating well:

$$s = \frac{Q}{4\pi k_r b_i} \mathrm{W}(u_A, \textit{six parameters}) \quad [5]$$

The parameter u_A is defined as

$$u_A = \frac{r^2 S}{4 k_r b_i t} \quad [6]$$

Since this solution is an extension of other simplified solutions (without well capacity) that are known to perform poorly as explained before (Nwankor et al. 1992; Akindunni and Gilham 1992; Halford 1997; Chapuis et al. 2005), its reliability is poor. This is why only this solution will be examined, considering that more recent solutions have used similar simplifications, which do not describe adequately the highly non linear drainage process, and are thus similarly unreliable.

3.3 Real and assumed behaviours

Table 1 summarizes the differences between the real behaviour of the aquifer material and the schematized behaviour as assumed in theories. In light of Table 1, one can easily understand why existing transient theories give questionable assessments of hydraulic parameters, and why this paper has not considered them in detail.

4 INTERPRETING FIELD DATA WITH THEORIES

4.1 Transient condition theories

During the 48-hr pumping test, the flow rate was 0.303 m^3/min (80 US gpm). The drawdown data appear in Fig. 2 for the pumping well (PW) and in Fig. 3 for the monitoring well (MW). The graphs on logarithmic scales include field data and displaced data to be matched with the type A theoretical curves of Boulton and Streltsova (1976), which are used to find first the ratio β defined as

$$\beta = \left(\frac{r}{b_i}\right)^2 \frac{k_z}{k_r} \quad [7]$$

For the pumping well data (Fig. 2), the best visual fit gives β = 0.15, and as a result k_z is close to k_r. For the monitoring well data (Fig. 3), the best visual fit gives β = 1, and also that k_z is close to k_r. Matching coordinates t, s, $1/u_A$ and W (u_A, six parameters) are then used in [5] and give k_r values of 2.0 x 10^{-3} m/s and 3.6 x 10^{-4} m/s for the pumping and monitoring well respectively. It should be noted that the theoretical curves were developed with the single value S = 10^{-3}, assuming as usual that the S value has little influence on type A curves, and thus could not be determined accurately.

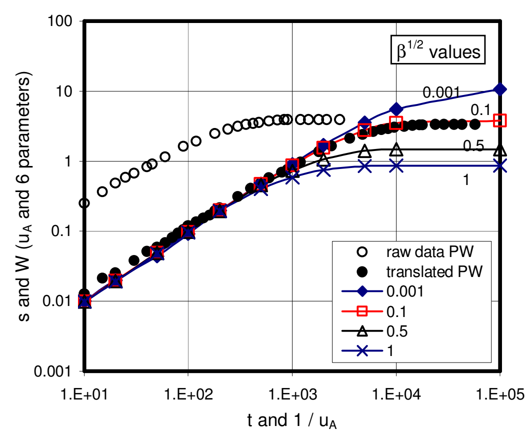

Figure 2. Graph of log s versus log t and type A curves for the pumping well.

Table 1. Comparison of real behaviour and assumed behaviour in theories

Equation, assumption	Real conditions, real data	Assumed conditions
conservation equation	2D (r, z)	1D (r only) for earlier theories 2D (r, z) for more recent theories
conservation equation	$div(\mathbf{k}\, grad\, h) = \frac{\partial \theta}{\partial t}$ in which k and θ are highly non linear functions	$div(\mathbf{k}\, grad\, h) = S_s \frac{\partial h}{\partial t}$ in which k and S_s are constant, $b_i\mathbf{S}_s$ equals either S or ($S + S_y$)
unsaturated flow	important above the water table	unsaturated k is not considered
water table	the locus where $u_w = p_{atm} = 0$ (it is not a boundary)	viewed as a moving boundary such as $\frac{\partial h}{\partial z} = -\frac{S_y}{k_z}\frac{\partial h}{\partial t}$
specific storage S_s	constant or not at positive pore water pressure u_w	always constant
specific yield S_y	function of drawdown and capillary retention curve, acting in the vadose zone	constant acting only at the moving water table boundary
numerical codes (examples)	Seep/W, Hydrus, SVFlux	Modflow
	Observed behaviour	**Theoretical behaviour**
log s vs. log t	most have not a slanted S-shape	they have a slanted S-shape
s vs. log t	most have no central zone; when they have one, it is not flat and the two side slopes differ	there is a flat central zone and the two side slopes are equal, leading to a single k_r value

After having found the best match with type A (early times) curves, a second match had to be found with type B (late time) curves. However, the second overlay was not possible because the drawdown was constant after about 20 hours. Initially, it was supposed that the test duration was too short, and that the second branch needed more time to develop. Later, the contractor used the same pumping rate for several weeks (before boring the horizontal drains) but the drawdown was not increased, thus confirming that a steady-state condition had been reached after about 20 hours.

Consequently, the S_y value could not be obtained by the superimposition of type B curves. Thus, the previous k_r value (type A curve) could not be confirmed. The field data seemed to match only half of the transient theory. However, the k_r values, which differed by a factor of 5.6 for the PW and the MW, could be checked with the reliable equation of Dupuit (1863) for steady-state.

4.2 Steady-state interpretation

The graph of corrected drawdown s_c vs. log r is plotted in Fig. 4 for pumping times t of 45 and 120 min, and for steady-state. The extrapolated r value, for $s_c = 0$, defines the radius of influence R_0 as a function of time t. For steady-state, $R_0 = 49$ m by extrapolation, which is close to the physical value $R = 45$ m, the shortest distance between the well and the water body, as could be anticipated for a homogeneous material. However, the k_r value derived from Eq. 4 is 8.5 x 10^{-5} m/s, which is 24 and 4 times lower than the values derived using the transient theory. Since the equation of Dupuit (1863) is the only one that can be trusted, its k_r value was deemed to be the correct one.

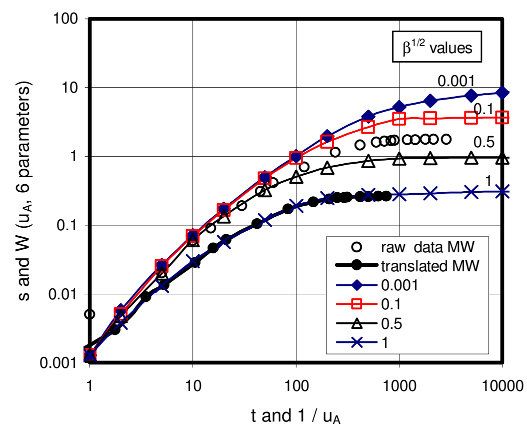

Figure 3. Graph of log s versus log t and type A curves for the monitoring well.

4.3 Alternate transient interpretation

An estimate of S_y was given by Eq. 8 (Weber 1927)

$$R_0(t) = 3\sqrt{\frac{k_r\, b_i\, t}{S_y}} \qquad [8]$$

Using the k_r value for steady-state, R_0 (t = 45 min) = 20.15 m gave S_y = 6.4 10^{-4}, R_0 (t = 120 min) = 23.0 m gave Sy = 1.3 x 10^{-3}, and reaching steady-state at t = 600 min (end of transient conditions, defined by the intersection of straight-lines shown in Fig. 5) gave S_y = 1.4 x 10^{-3}. This value seemed too small for the drained volume of water-bearing fractures in the rock aquifer.

When plotted as s versus log t, the field data (Fig. 5) looked like theoretical graphs for confined aquifers (Cooper and Jacob 1946). The influence of the pumping well capacity is shown by a plot of $Q\,t / \pi\, r_w^2$, i.e. the pumped volume at time t divided by the well cross sectional area, which represents the drawdown that would occur in a hypothetical "well" with solid wall and bottom. The difference between this theoretical plot and the plot of well drawdown, s_w, represents the aquifer part to the pumped volume, which is only about 50% after 100 min.

The equations of Cooper and Jacob (1946), when applied to the linear portions of the graph (Fig. 5), yielded S_y = 4.1 x 10^{-2} for the MW, k_r = 3.3 x 10^{-5} m/s for the PW, and k_r = 6.6 x 10^{-5} m/s for the MW. Even if the equations are for confined aquifers, without well pipe capacity, they provided k_r values that were much closer to the steady-state value (Dupuit equation) than those obtained with the superimposition of type A theoretical curves.

The interpretation difficulties can be summarized as follows. The field drawdown data corresponded only partly to the theory for transient conditions in an unconfined aquifer. Using this theory, different k_r values were obtained for the pumping and monitoring wells. In addition, the k_r value derived from steady-state was 24 and 4 times lower than the k_r values derived using the theory for transient condition. The discrepancies could be due to any set of the numerous reasons (Table 1) why the transient theory solves a physically ill-defined problem. To try to understand the reasons for such differences, a finite element modeling of the pumping test was performed, taking into account all the real physical features that should be considered for such a test.

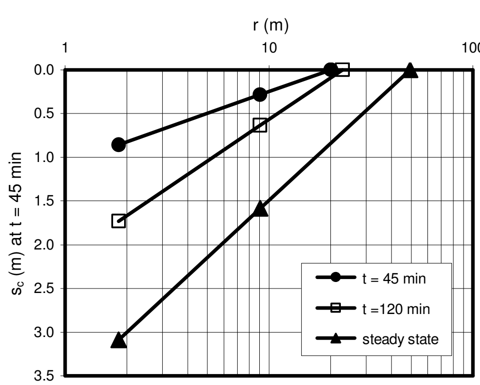

Figure 4. Curves of s_c versus log r.

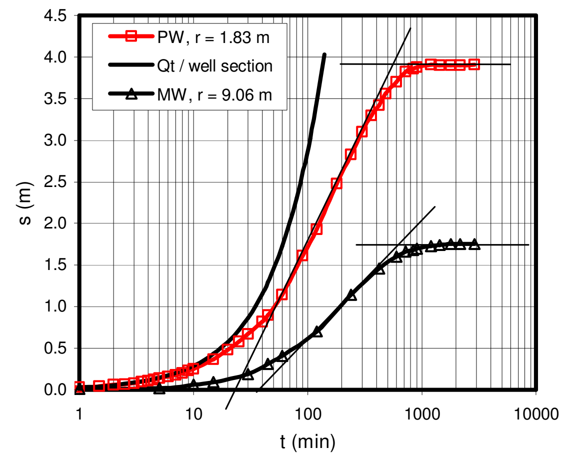

Figure 5. Curves of s versus log t.

5 NUMERICAL MODELLING

The retained numerical code had to satisfy the requirements of the central column of Table 1. This was important because the purpose of numerical modelling was to verify whether a realistic representation of the hydraulic properties and delayed drainage would provide numerical drawdown data similar to the field data. Using a numerical code that works with the same unrealistic assumptions as the simplified theory could not help to clarify the interpretation problems.

The finite element code Seep/W (Geo-Slope International 2003) was retained because it passed the tests in a detailed study by Chapuis et al. (2001). This code solves steady- and unsteady-state problems in unsaturated or saturated materials. It uses the functions $k(u_w)$ and $\theta(u_w)$, where k is the hydraulic conductivity function, u_w is the pore water pressure, and θ is the volumetric water content ($\theta = nS_r$ where n is the total porosity and S_r the degree of saturation in water). The equations of Darcy and conservation (Richards 1931) are solved as u_w-based equations for saturated and unsaturated seepage. The total stresses are assumed constant, and the air phase is at atmospheric pressure. This code was used to study pumping tests (Chesnaux et al. 2006; Chesnaux and Chapuis 2007; Chapuis et al. 2006, 2005), variable-head permeability tests (Chapuis 1998a, 1998b, 2005; Chapuis and Chenaf 2002, 2003; Chapuis et al. 2007a), drainage tests (Chapuis et al. 2007b), and seepage through dikes (Chapuis and Aubertin 2001). In the case of pumping tests (Chapuis et al. 2001), this code gave results matching the theoretical equations of Theis (1935) for unsteady state in confined aquifers, Thiem (1906) for steady state in confined aquifers, and Dupuit (1863) for steady state in unconfined aquifers. In the latter case, it provided also the position of the seepage face (Chenaf and Chapuis 1998, 2007) and helped to confirm that the solution of Dupuit is respected at any r value when the drawdown s is given by monitoring wells having a short screen in the lower third of the saturated zone.

5.1 Hydraulic parameters

In the numerical study, the isotropic aquifer had a constant k at $u_w > 0$. The initially saturated thickness was about 10 m (Fig. 1). Examples of functions $k(u_w)$ and $\theta(u_w)$ are provided in Figs 6 and 7.

The specific yield S_y is defined as the volume of water released by storage per unit surface area of the unconfined aquifer and per unit change in hydraulic head (or water table position). Thus, S_y is mathematically defined as the surface between two positions of the capillary retention curve (Fig. 8), plus elastic storage S, divided by the change in total head or drawdown. As long as the slow drainage is not achieved, S_y physically depends on time. Contrarily to elastic storage S, which is rapidly mobilized through stress transfer and can be viewed as a physical constant, S_y is not a physical constant. When the slow drainage is achieved, S_y usually depends on the initial and final positions of the water table with respect to the ground surface, and the shape of the capillary retention curve where θ_{sat} and θ_r are the saturated and residual volumetric water contents. When the slow drainage is achieved, S_y can become constant if the curve $\theta(z)$ was already reaching θ_r between the initial water table and the ground surface. It is then equal to the difference ($\theta_{sat} - \theta_r$). The conditions of the examined pumping test (Fig. 1) correspond to such a case of constant final S_y.

The examined $\theta(u_w)$ functions were assumed to be fully developed between 0 and -10 kPa, and provided final S_y values (after full completion of transient drainage) between 1 and 5% (Fig. 6). The saturated volumetric content of the aquifer was assessed as θ_{sat} =10%. Note that the numerical simulations are not influenced by the value of θ_{sat}, but only by the value of ($\theta_{sat} - \theta_r$) and the shape of the $\theta(u_w)$ function. Each $k(u_w)$ function was obtained from $\theta(u_w)$ using the van Genuchten (1980) method which is one of the methods available in the numerical code. The well volume was treated as a very pervious material ($k = 10^2$ m/s) or pipe element, in which θ dropped from 100% to 0.01 % when u_w dropped from 0 to -1 kPa. This simulated the storage capacity in the pumping well while the hydraulic head was kept constant within 0.02 mm (Chapuis 2009).

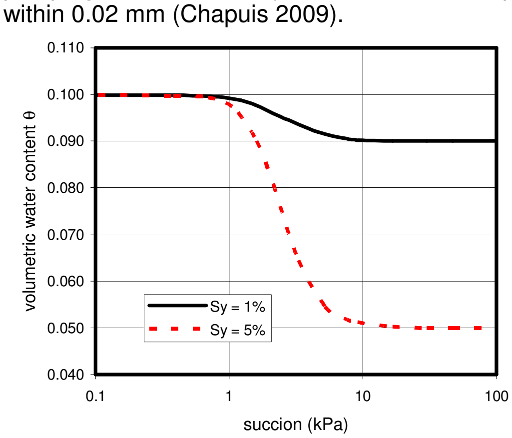

Figure 6. Example of functions $\theta(u_w)$.

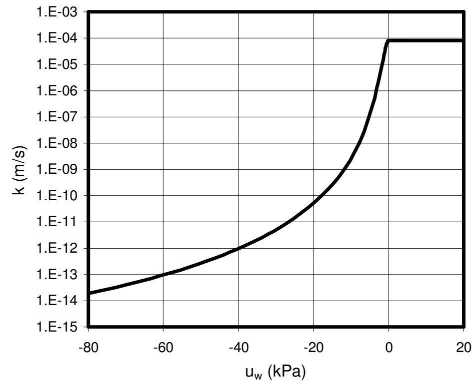

Figure 7. Example of function $k(u_w)$.

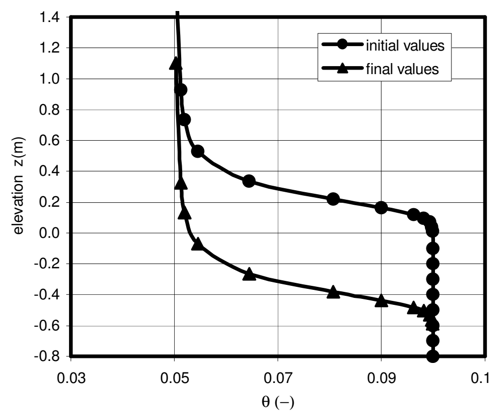

Figure 8. Mathematical definition of specific yield S_y.

5.2 Grid and boundary conditions

The finite elements were small (2.5 cm in the vertical direction, z, within the well and 10 cm within the aquifer) in the unsaturated zone to facilitate convergence when there are sharp variations in $\theta(u_w)$ and $k(u_w)$. This can be verified by using several methods (Chenaf and Chapuis 1998). The grid had 3729 nodes. Boundary conditions included a constant pumping rate imposed inside the well, and a constant head at a distance of 45 m from the pumping well, to simulate the water body (Fig. 1).

5.3 Parametric study

The influence of S was first investigated. In theory, the influence of S can be felt only at early times. It was found that the numerical drawdown curves were almost unchanged (maximum difference of 1.5%) for S values between 10^{-4} and 2×10^{-3}, because early time data are controlled mainly by the well storage capacity, and thus the influence of S is non-significant.

Once knowing that S has almost no influence, the influence of k_r (saturated horizontal k value) was then investigated, using $S = 10^{-3}$. The drawdown curves are highly sensitive to k_r as shown in Fig. 9. The steady-state drawdown values depend only on k_r. Since the equation of Dupuit (1863) is reliable, it is thus important to continue a pumping test until steady-state, the plot of stabilized s

against log (r) providing the correct value of k_r. However, the steady-state equation of Dupuit defines some equivalent k_r for the saturated thickness, which includes the small contribution of unsaturated flow above the water table. As a result, the numerical match gave a saturated k_r value of 8.3 x 10^{-5} m/s, slightly smaller than 8.5 x 10^{-5} m/s as obtained directly using equation of Dupuit (1863).

Once the saturated k_r value was known, the influence of S_y was examined. As explained above, the S_y value was defined as ($\theta_{sat} - \theta_r$). It was found that the duration of the transient condition was markedly influenced by S_y, but the drawdown ended at the same final steady-state value (Fig. 10).

As seen in Fig. 10, a S_y value (as previously defined) of either 1 or 2% does not make a difference for the pumping well. However, the monitoring well drawdown data helped to make the distinction. The best fit was obtained using a S_y value close to 1.7%.

6 CONCLUSION

A 3.66-m diameter well was used to pump an unconfined fractured rock aquifer. A single monitoring well was available. The time-drawdown curves were influenced by the large well storage. They could be matched to the theoretical type A transient curves but not to the type B curves, the steady-state condition being reached after about one day. When applied to the field data, the theoretical overlay gave k_r values of 2.0 x 10^{-3} m/s for the pumping well and 3.6 x 10^{-4} m/s for the monitoring well.

However, the graph of steady-state corrected drawdown against radial distance gave a k_r value of 8.5 x 10^{-5} m/s, which was 24 and 4 times lower than the values obtained using the transient theory. Many reasons were provided (Table 1), which could explain such differences, the usual transient solution corresponding to a physically ill-defined problem. To try to better imitate the real physical problem, a numerical modeling of the pumping test was performed, taking into account all the real physical features that should be considered for such a test. An adequate finite element code was selected, which does not oversimplify the physical reality of unsaturated drainage.

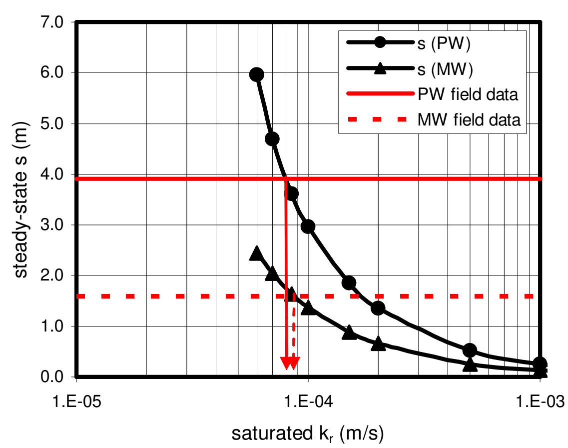

Figure 9. Numerical fitting for steady-state conditions.

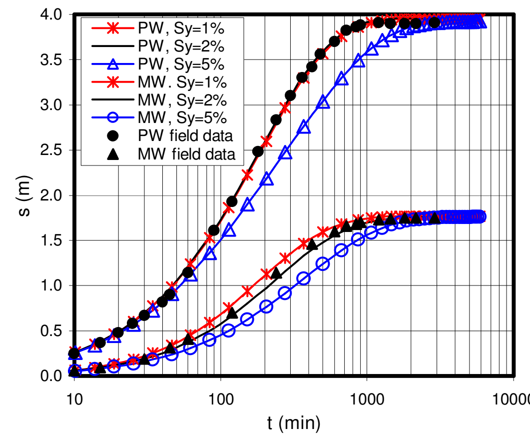

Figure 10. Numerical fitting for different values of S_y defined as ($\theta_{sat} - \theta_r$).

The numerical study has shown that the elastic storativity S has almost no influence for such a pumping test. The steady-state drawdown is controlled by the k_r value, whereas the duration of the transient phase is controlled by the capillary retention curve. According to the numerical study, the saturated k_r was about 8.3 x 10^{-5} m/s and S_y was close to 1.7%.

A practical recommendation for all pumping tests in unconfined aquifers is to continue the test until reaching steady-state conditions. This is deemed important since the Dupuit equation (1863) seems to be the only reliable analytical solution for unconfined aquifers. Reaching steady-state may take many days or weeks. However, according to the author's experience – and as confirmed in this paper – the graph of s_c against log r at large times usually provides a good estimate of k_r, because this graph is slowly modified with time to finally adjust with the reliable steady-state equation.

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