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What maximum permeability can be measured with a monitoring well?

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ABSTRACT

The PVC screens of recent monitoring wells (MWs) have thin slots and a low open area, usually in the 2–8% range. The MW screen and filter pack may cause important head losses which are not taken into account when interpreting the data of permeability tests performed using the MW. The equivalent hydraulic conductivity K of usual PVC screens was defined by hydraulic tests in a water tank, which have shown that gas microbubbles, a common problem in MWs and filter packs, contribute to increase the parasitic head losses. Closed-form equations and numerical models are used to explain by how much a field permeability test in a MW under evaluates an aquifer K value due to parasitic head losses in the screen and filter pack. The MW can properly measure the local soil K value only if it is markedly lower than the maximum MW value as obtained in a water tank. The MW measuring capacity can reach 5×10^{-3} m/s for large slots and deaired water, but is most often between 10^{-5} and 10^{-4} m/s for small slots in field conditions, and it can be only 10^{-6} m/s for poorly designed and installed MWs. The limited measuring capacity may yield artificial permeability scale effects as often registered in environmental studies.

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1. Introduction

Many engineering geology problems, with or without environmental issues, require installing monitoring wells (MWs) in specific aquifer or aquitard layers. Each MW is characterized by several parameters for the pipe, screen, filter material around the screen, and sealants for the annular space between the borehole and the MW pipe. Many regulating agencies have guides on how to design and instal MWs. For example, the design and installation of MWs in aquifers are described in detail in ASTM (2010) whereas their development is described in ASTM (2013).

Monitoring wells are used to measure the hydraulic head h and perform field permeability tests to assess the hydraulic conductivity K of the natural soil surrounding the filter pack, and also to sample groundwater. When analyzing field permeability test data, the hydraulic head losses in the filter pack and screen are usually neglected. However, these head losses may be not negligible.

For example, in 1982, an excavation in the Quebec City harbour required a huge pumping system to evacuate a tide-dependent inflow rate reaching 114 m^3/min. Most water inflow came from rockfill behind and under the wharfs, and around a pressure sewer pipe 2.1 m in diameter (Chapuis et al., 1991). The rockfill K value was determined using several methods (pumping test, tracer tests, tidal effects, and predictive methods): it was close to 6 cm/s. However, constant-head permeability tests in MWs gave much lower values of about 5×10^{-2} cm/s. Further testing of a MW screen in a 45-gallon barrel, without any soil around the screen, gave flow rates equal to those registered during the field tests, which proved that all hydraulic head losses occurred within the screen slots, and not within the rockfill. This case and other unpublished cases led us to examine the head-loss problem and its conditions.

Many pumping wells have stainless steel screens with large slots, surrounded by pea gravel which is the usual filter material: the "filter pack" is then a "gravel pack". Monitoring wells are not installed to continually extract water. They typically have PVC pipes (51 mm of inside diameter) and screens with small slots, usually 0.25 to 0.50 mm: the filter pack is made of uniform natural sand. The open area OA of most MW screens is in the 2–8% range (Table 1), whereas most pumping well screens (e.g., wire-wound type) have an OA in the 15–40% range. According to physical tests and numerical studies, it is recommended to select a screen with an OA of at least 10–15% and preferably 40% to minimize head losses in the screen slots (Peterson et al., 1953; Garg and Lal, 1971; Williams, 1981; Clark and Turner, 1983; Singh and Shakya, 1989).

However, there are no such criteria for hydraulically efficient MWs. A screen with fine slots and a small OA, and fine filter sand, may induce significant head losses during a permeability test which gives a K_{test} value for the tested soil. To assess the MW hydraulic efficiency, one must consider seepage through the screen, filter pack and natural soil. Thus, the permeability test is influenced by the screen hydraulic conductivity K_1, and the hydraulic conductivities of filter sand K_2 and natural soil K_3. Simply stated, an efficient MW must provide a K_{test} value very close to the sought K_3 value.

This article examines how the screen and the filter sand influence the K_{test} value of a permeability test performed in a monitoring well, and by

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Table 1
Characteristics of commonly used and tested MW screens.

Screen name	0.02d +	0.02d −	0.01d +	0.01d −	0.006d +	0.006d −
Slot size f (mm)	0.508	0.508	0.254	0.254	0.152	0.152
Screen length (mm)	270	278	285	280	285	280
Open area (%)	6.62	3.31	3.56	1.78	1.15	0.57

how much the K_{test} value differs from the sought K_3 value for the tested soil. First, the common rules for designing and installing a MW are briefly presented. Then, the paper explains how the equivalent hydraulic conductivity K_1 of each screen was determined. The hydraulic conductivity K_2 of the filter sand is obtained using either laboratory tests or predictive equations. Afterwards, closed-form equations are provided to relate K_{test} to K_1, K_2 and K_3, in the case of a constant-head test performed in a MW fully penetrating a confined aquifer. The influence of K_1, K_2 and K_3, was also studied using analytical and numerical (finite element) methods, for constant- and variable-head (slug) tests, full and partial penetrations. The effects of K_1 on K_{test} were studied first before studying the combined influence of K_1 and K_2 on K_{test}.

2. Basic rules for installing MWs

This section provides basic rules to select the filter material and the screen slot size. Five steps must be followed: (1) obtain the correct grain size distribution curve (GSDC), (2) select the filter material, (3) select the screen slot size, (4) instal the MW, and (5) document in detail the MW installation.

2.1. Establish the soil GSDC

Different drilling methods may be used to instal MWs. Also, different soil sampling methods may be used. Well drilling methods are more soil disruptive than geotechnical drilling methods. According to in-hole geophysics, hollow-stem augering is more disruptive than rotary drilling, and driving a flush-joint casing yields the least disturbance (Morin et al., 1988). The drilling fluids may erode and wash out the fines of a soil, producing poor quality samples. The recovered soil sample may be either denser or looser than the in-situ soil, it may be coarser (loss of fines), or it may be an artificial mix of two or more soil layers. Being unaware of these problems and using directly the GSDC of a remoulded sample will yield a poor estimate of the field K value (Chapuis, 2012a,b). Methods are available to assess whether the sample has lost fines, and whether it contains a mix of several layers (Chapuis et al., 2014). It is important to use these methods before trying to assess the sample K value, and before selecting the filter sand and the screen slot size.

2.2. Select the filter material

The filter sand is selected among commercially available, washed and screened, natural siliceous sands, with rounded abrasion-resistant grains, and appropriate GSDC. An example of GSDCs for commercial filter sands is shown in Fig. 1. The filter material must correctly filter the finest particles of the natural soil in order to have clear (no turbidity) groundwater samples.

First, to avoid segregation when falling in water, uniform soils are used; their uniformity coefficient C_U is less than 2.5. Second, the filter pack particles must be round to facilitate placement and avoid arching, and hard to resist mechanical stresses. Thus the GSDC must not be modified by a compaction test. Crushed materials are not used as filters because they are prone to arching, have high porosities with some large individual pores, and their time-evolving GSDC is a source of problems such as internal erosion. Third, the filter GSDC is selected to perform against the finest natural soil facing the filter sand: a filter pack that is suitable for the finest soil is suitable for all soils. Fourth, the filter sand is selected using the criteria of Table 2. Criteria for MWs installed in aquifers

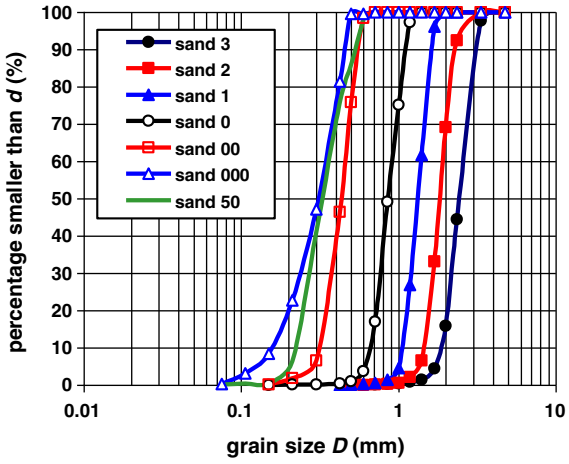

Fig. 1. Examples of grain size distribution curves (GSDCs) for commercial filter sands.

are those of the Bureau of Reclamation (1977). Criteria for MWs installed in silt and clay are those of geotechnical engineers for the safety of dam cores, liners and covers.

2.3. Select the screen slot size

For any MW and pumping well, the screen slot size, f, must be smaller than the D_{10} of the filter material, D_{10}(fm). This means that less than 10% of the filter pack mass can pass the screen slots.

There is a clear difference between a pumping well and a MW. When a pumping well is installed without a filter pack, a common rule is to select a screen slot size f equal to the D_{60} of the aquifer material, D_{60}(am). This means that a large mass of fine particles will pass through the screen during development, about 150 to 300 kg/m of screen length (Wendling et al., 1997). The so-called natural development of the pumping well moves the in-situ grains to create a filter pack, which ultimately provides, between the screen and the in situ soil, smooth transitions in GSDCs and K: near the screen the K value may be 30 to 50 times the intact field K value (Wendling et al., 1997). During development, using for example a double-action piston, the hydraulic gradient may exceed values of 300 close to the screen. As a result, very high horizontal effective stresses are generated and a space of several centimetres appears between the screen and the in-situ soil when water is pushed into the aquifer, as visualised by Wendling et al. (1997): this corresponds to soil failure in a pressuremeter test. When water is withdrawn from the aquifer, the soil solids are violently pushed back against the screen. The brutal outward and backward

Table 2
Criteria for selecting the filter material to be used in the filter pack of a monitoring well.

Aquifer material (am)	Filter material (fm)
If C_U(am) ≤ 2.5	Preferable solution: C_U(fm) ≤ 2.5 with D_{50}(fm) ≤ 6 D_{50}(am) Acceptable solution: 2.5 ≤ C_U(fm) ≤ 5 with D_{50}(fm) ≤ 9 D_{50}(am)
If 2.5 ≤ C_U(am) ≤ 5	Preferable solution: C_U(fm) ≤ 2.5 with D_{50}(fm) ≤ 9 D_{50}(am) Acceptable solution: 2.5 ≤ C_U(fm) ≤ 5 with D_{50}(fm) ≤ 12 D_{50}(am)
If C_U(am) > 5	In the grain size distribution graph, at y = 30% locate the points of abscissa $6D_{30}$(am) and $9D_{30}$(am). Through these points draw two parallel lines representing filter materials with C_U(fm) ≤ 2.5. Select a filter material with a GSDC that falls between the two lines.
Aquitard	Select a fine uniform natural sand with D_{15}(fm) ≤ 0.3 mm for silty materials and D_{15}(fm) ≤ 0.2 mm for clayey materials.

displacements sieve mobile particles through the screen: the sieved grains fall into the solid trap forming the screen deep end.

In the case of a MW, high stresses and brutal displacements must be avoided during development otherwise the soil may be severely damaged (ASTM, 2013). A hydraulic short-circuit can be created between two aquifers (Chapuis and Sabourin, 1989; Chesnaux et al., 2006), or the *K* value of a clay or silt can be greatly increased by producing fissures (secondary porosity), which is detrimental in materials that are used as hydraulic barriers. For example, in sensitive clay or clayey till, the MW can produce only a few cubic centimetres of water per day and should never be developed, nor sampled. Besides, this MW should never be pumped down to the screen for a water level recovery test, because this severely damages the natural or man-made hydraulic barrier. Only MWs in aquifers should be developed using caution, whereas a limited drawdown should be used in all cases for the recovery test, following the principle of low-flow low-drawdown pumping and sampling (ASTM, 2002).

2.4. Instal the MW

Centering guides (centralizers) must be used to centre the casing and screen string in the borehole. This ensures that the filter pack fills the annular space evenly around the screen. This ensures also that the sealing materials fill the space evenly around the solid riser pipe.

The filter sand may be installed through a tremie pipe which is withdrawn in stages as the sand is placed. The filter material should not be poured dry into the annular space or the tremie pipe, otherwise the emplacement may be poor, and large quantities of gas are trapped in the filter pack as adhering tiny bubbles that may occupy over 15% of the filter total pore space. After several days, these bubbles are enlarged by coalescence, may rise and get trapped in the upper part of the annular space between the wall of the borehole, the solid riser pipe just above the screen and the annular sealant. It is wise to soak the filter material in water and pour it with water. A cleaned stainless steel tamper helps to place the filter material, and the bentonite pellets or chips.

Do not forget to respect filter criteria between the filter pack material and the grout or bentonite above. In many cases, it means that several filter materials must be used. For example, assume that the filter material is pea gravel, the filter pack fully intercepting a gravel aquifer. If a grout is placed above the filter pack, it will easily seep into the void space of the filter material and thus, will clog the MW. Similarly, if bentonite pellets are placed directly above the pea gravel, hydrated bentonite will move into the void space of the filter material and all ground water samples will be turbid. Between the pea gravel and the bentonite (or grout), it is necessary to instal two (secondary) filter materials, first at least 15 cm of coarse sand that will not move into the pea gravel (and must a filter material for the natural soil forming the borehole wall at this depth), then at least 15 cm of fine sand that will not penetrate the coarse sand. The fine sand gradation must be selected according to the criteria for filtering fine-grained soil particles (see Table 1) and prevent the grout or bentonite particles from reaching the filter pack.

2.5. Document installation

During step 4, the installation must be fully documented. As the filter pack is placed, the casing and the tremie are withdrawn in stages of preferably 15 cm, which avoids leaving an unsupported borehole wall higher than 15 cm. The volumes and weights of placed materials (filter and sealants) must be precisely measured and recorded, preferably for each 15 cm. The top of the filter pack or the top of the bentonite pellets or chips is measured using the tremie pipe, the tamper line or a weighted line. A detailed installation data sheet must be completed. Placing larger than anticipated quantities between two consecutive depths (this is common with rotary drilling) indicates at once that there has been overexcavation (the borehole diameter is much larger than the nominal borehole size) and by how much. Similarly, placing lower than anticipated quantities indicates that there has been local collapse (sloughing or caving) during the placement. If a bridge or obstruction occurs within the annular space, it is directly detected by successive depth measurements in the annular space. Bridging occurs with the filter pack material and also bentonite pellets or chips. The bridge must be broken (normally the tamper can break it) prior to adding more filter material or bentonite.

Organizations for which the facilities safety is very important have installation sheets and procedures (Hydro-Québec, 1990). Working for such organizations means that installing a MW takes 4 h or more, the installation process being fully quantified and documented. However, environmental contracts are often given to the lowest tender. This has sadly yielded slack attitudes, some MWs being installed in 10 to 30 min according to field logs. A hurried and cheap installation is made without measuring poured quantities versus depth and without completing a detailed installation data sheet. This is really a poor practice that should be discouraged by regulating agencies.

According to Nielsen and Schalla (2005): "*On the basis of an examination of sampling results from thousands of wells, the authors estimate that more than 65% of ground-water monitoring wells installed in North America since the late 1970s suffer from more than one of the aforementioned problems, and thus are improperly designed for their intended purpose. As a result, many of these wells are producing water-level data, hydraulic conductivity test data, and ground-water samples that are not representative, in terms of the data expected from them.*"

After having presented basic rules about the filter material and the screen apertures for MWs, the paper explains how the equivalent hydraulic conductivity K_1 was determined for different screens.

3. Method to obtain the equivalent K_1 for the screen

3.1. Equipment and methods

Six common PVC screens for MWs were selected to be hydraulically tested. The screens have an internal diameter of 51 mm (2 in.). All screens have a wall thickness close to 3 mm. Their slot sizes, *f*, have values between 0.508 mm (0.020 in.) and 0.152 mm (0.006 in.), and their open areas have values between 0.6 and 6.6%. The geometric characteristics are listed in Table 1.

The equivalent hydraulic conductivity K_1 of each screen was obtained using steady-state pumping tests in a water tank (Figure 2). A constant flow rate *Q* was pumped in the MW. Water seeping through the screen slots produced a hydraulic head loss, *H*. As a result, there was a difference between the water level in the MW and that in the tank. This difference was monitored versus time until reaching a stable value. For each tested screen, 3 to 6 constant flow rates were used: they provided as many couples (Q_j, H_j) where the j index can take a value between 1 and 6. These data were used to obtain K_1.

For the steady-state pumping tests in a water tank, the screen was treated as a porous material of equivalent hydraulic conductivity K_1. In our pumping tests, the flow rates were small enough to have no turbulence, as for permeability tests with MWs. Other previous studies have differently quantified the flow resistance through the slots of pumping well screens in which they create production loss and added cost of production. For example, Barrash et al. (2006) assumed that the slots were acting like fissures in a fractured material; they calculated K_1 by pondering the values for each slot over the screen area. Other studies have used the orifice flow hydrodynamic equations (Peterson et al., 1953; Garg and Lal, 1971). Singh and Shakya (1989) performed tests similar to ours, and interpreted their pumping well data using a non-linear law.

During permeability tests in MWs, Darcy's law applies because all fluid velocities are small enough to have low Reynolds numbers and no turbulence, which was verified. As a result, two methods were retained to estimate the K_1 value of MWs, the Thiem's equation and numerical modelling. Both use Darcy's law and the conservation equation (Richards, 1931).

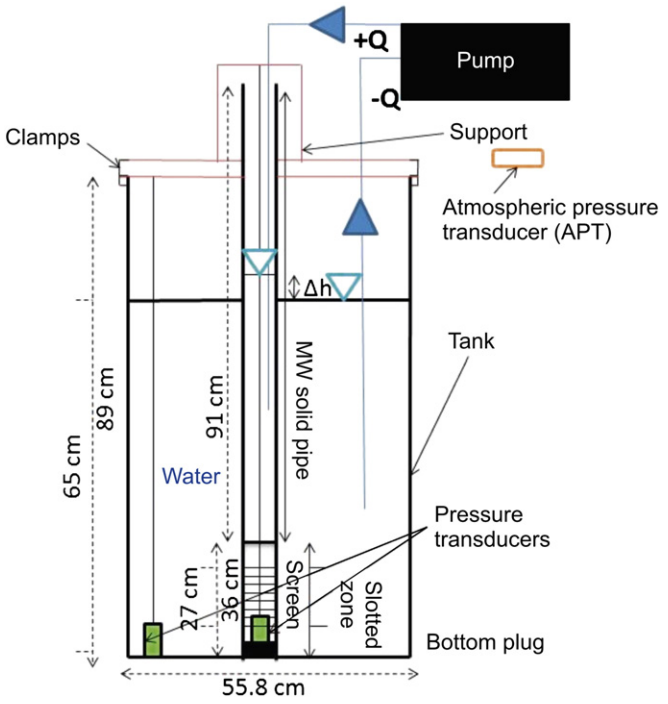

Fig. 2. Sketch of the experimental set-up to determine K_1.

3.2. Interpretation of results: Thiem's equation

The Thiem's equation (Eq. (1)) is for steady-state pumping in an ideal horizontal confined aquifer of hydraulic conductivity K_1 and thickness b. Eq. (1) relates the flow rate Q to the drawdown values s_1 and s_2 at radial distances r_1 and r_2 measured from the well vertical axis:

$$Q = 2\pi K_1 b \frac{(s_2 - s_1)}{\ln(r_2/r_1)} = 2\pi K_1 b \frac{(s_{out} - s_{in})}{\ln(r_{out}/r_{in})}. \quad (1)$$

Here the aquifer thickness b is equal to the screen height; s_1 at r_1 is measured on the screen inner wall (thus giving s_{in} at r_{in}), whereas s_2 at r_2 is measured on the screen outer wall (thus s_{out} at r_{out}). During the tests the pumped water was returned to the tank by the pumping hose (closed loop). No drawdown was registered by the electronic transducers within the water tank.

3.3. Interpretation of results: numerical method

A steady-state pumping test with the screen alone is also a constant-head permeability test: all head losses occur within the screen (K_1) and none in the filter (K_2) and soil (K_3). The test conservation equation is the Laplace's equation. Its solutions are named harmonic functions, the special properties of which being known long before the Hvorslev's equation (Chapuis, 1998) in which H is the constant difference in hydraulic head and c is a shape factor which depends up on the geometry of the MW and tested layers, including boundary conditions (Chapuis, 1989). According to Hvorslev (1951):

$$K_{test} = Q/cH. \quad (2)$$

For each tested screen, the c value was obtained using a finite element method (Geoslope, 2007) to obtain the K_{num} value for the numerical test. Thus, the exact geometry for the screen (position, length, and diameters) and the water tank were taken into account. The water in the tank was treated as a very pervious material ($K = 100$ m/s). The numerical code provided a linear correlation between the H values (boundary conditions) and the Q values (results): this gave the value of the product cK_{num} using Eq. (2). The K_1 value in the numerical code was adjusted to find a K_{num} value matching the experimental value K_{test}. This inverse method provided the sought after K_1 value.

If a field permeability test is performed in a MW equipped with this screen, the field hydraulic head losses occur in the screen, the filter pack, and the tested soil. Therefore the field K_{test} value for this MW is always influenced by the K_1 value for the screen.

4. Method to study the influence of K_1 and K_2 on K_{test}

The influences of the filter pack (K_2) and the screen (K_1) on K_{test} were studied in two steps: first, the influence of the screen alone, and then the combined influences. Each step used an analytical solution and the numerical solution.

4.1. Analytical solution

Consider a vertical MW, fully screened in a horizontal confined aquifer of constant thickness, thus a simple case of radial symmetry. The following notations are used: r_{jo} and r_{ji} are the outer and inner radii of material j (index $j = 1$ to 3, 1 for the screen, 2 for the filter pack and 3 for the tested soil); h_{jo} and h_{ji} are the hydraulic heads at r_{jo} and r_{ji} respectively. We want to establish how the ideal K_{test} value is related to the real K_3 value of the tested soil, the one that is sought after when performing a constant-head permeability test. Using the Thiem's solution for the concentric materials (screen wall, filter pack, natural soil), it comes:

$$\frac{Q}{2\pi b} = K_1 \frac{h_{1o} - h_{1i}}{\ln\left(\frac{r_{1o}}{r_{1i}}\right)} = K_2 \frac{h_{2o} - h_{2i}}{\ln\left(\frac{r_{2o}}{r_{2i}}\right)} = K_3 \frac{h_{3o} - h_{3i}}{\ln\left(\frac{r_{3o}}{r_{3i}}\right)} = K_{test} \frac{h_{3o} - h_{1i}}{\ln\left(\frac{r_{3o}}{r_{3i}}\right)}. \quad (3)$$

The differences in hydraulic head, $\Delta h_j = h_{jo} - h_{ji}$ (index j = 1, 2, 3) can be added, which gives the rule of composition for this geometry with axial symmetry. Therefore, it comes:

$$\frac{\Delta h_1}{\frac{\ln(r_{1o}/r_{1i})}{K_1}} = \frac{\Delta h_2}{\frac{\ln(r_{2o}/r_{2i})}{K_2}} = \frac{\Delta h_3}{\frac{\ln(r_{3o}/r_{3i})}{K_3}} = \frac{h_{3o} - h_{1i}}{\frac{\ln(r_{3o}/r_{3i})}{K_{test}}} = \frac{\Delta h_1 + \Delta h_2 + \Delta h_3}{\sum_{x=1}^{3} \frac{\ln(r_{xo}/r_{xi})}{K_x}}. \quad (4)$$

As a result, the K_{test} value is found as a function of K_1, K_2 and K_3:

$$K_{test} = \frac{\ln(r_{3o}/r_{3i})}{\sum_{x=1}^{3} \frac{\ln(r_{xo}/r_{xi})}{K_x}}. \quad (5)$$

Eqs. (3)–(5) are valid for a constant-head permeability test with a fully penetrating screen. The case of a variable-head permeability (slug) test is not solved analytically in this paper. It will be briefly examined for comparison only, using a numerical code, in Section 5.2.3.

If K_1 and K_2 are much higher than K_3, then their terms in Eq. (5) are negligible before the term of K_3, and thus Eq. (5) yields $K_{test} = K_3$. In all other cases, K_{test} under evaluates K_3.

If the screen is pervious enough but the filter pack is not pervious enough, then K_2 is smaller than K_1 and K_3, and the K_{test} value is limited

by the K_2 value. For very high K_3 values, the limit value (asymptote) of K_{test} is equal to:

$$K_{test} = \frac{\ln(r_{3o}/r_{3i})}{\sum_{x=1}^{3} \frac{\ln(r_{xo}/r_{xi})}{K_x}} \leq K_2 \frac{\ln(r_{3o}/r_{2o})}{\ln(r_{20}/r_{2i})}. \quad (6)$$

If the screen is not pervious enough but the filter pack is pervious enough then K_1 is much smaller than K_2 and K_3, and the K_{test} value is limited by the K_1 value. At the limit, for very high K_3 values, the limit value (asymptote) of K_{test} is equal to:

$$K_{test} = \frac{\ln(r_{3o}/r_{3i})}{\sum_{x=1}^{3} \frac{\ln(r_{xo}/r_{xi})}{K_x}} \leq K_1 \frac{\ln(r_{3o}/r_{2o})}{\ln(r_{10}/r_{1i})}. \quad (7)$$

For these special cases, Eqs. (6) and (7) give the maximum K_{test} value that can be obtained with a screen and a filter pack. Most often, the general Eq. (5) must be used to compare K_{test} with K_3, and obtain by how much the field test in the MW under evaluates the aquifer K_3 value.

4.2. Numerical method

Contrary to Eq. (5), which applies only for an ideal geometry and constant-head tests, the numerical method can study any geometry, for example a screen partially penetrating an aquifer. To study how K_1 alone influences K_{test}, a few constant-head tests were simulated. The K_1 value was that obtained using steady-state pumping tests in a water tank. The numerical models used these K_1 values and a range of K_3 values to quantify their impact on K_{test}. The filter soils were those available in the Montreal area. Filters in other areas are quite similar. The hydraulic conductivity K_2 of filter soils was obtained using the predictive model of Chapuis (2004, 2012a) for sand and gravel, using the filter GSDC and porosity.

The numerical results were compared to the analytical solution for constant-head permeability tests using fully penetrating MWs. In the field, most often variable-head tests or slug tests are performed in partially penetrating MWs. The case of a variable-head test is not solved analytically in this paper. It is numerically examined in Section 5.2.3.

5. Results

5.1. Equivalent K_1 for the screen

In the water tank, a few tests were made just after filling it, or the day after: these tests created many tiny air bubbles in the screen slots. Other tests were made after letting the water rest for at least two days: these tests created less air bubbles. The air content in water influenced the K_1 value, which was markedly lower in recent water than in old water.

With less aired water, the head losses were too small for 3 of the 6 screens to be precisely measured. Fig. 3 compares the K_1 values obtained using Eq. (1) with those obtained using the numerical model. The two methods gave similar results. The K_1 value was between about 5×10^{-5} and 1.5×10^{-4} m/s for the small slots in aired (recent) water, and 2.5×10^{-3} m/s for large slots and deaired water, but in the field, the water in the screen and filter pack is not deaired.

Having more tiny air bubbles decreases K_1 by about one order of magnitude. In all cases, the screen 0.006d−, with an open area OA of 0.57%, had the smallest K_1 value. The other screens, routinely used for soils, with OA in the 1–4% range, have similar K_1 values of about 1×10^{-4} m/s. Therefore, the most often used screens for soils have an equivalent K_1 similar to that of sand. These values can be compared to those obtained for turbulent flow in screens. Singh and Shakya (1989) used non-linear head loss equations: for OA in the 1.7–5.2% range, they found K_1 values similar to ours for laminar flow.

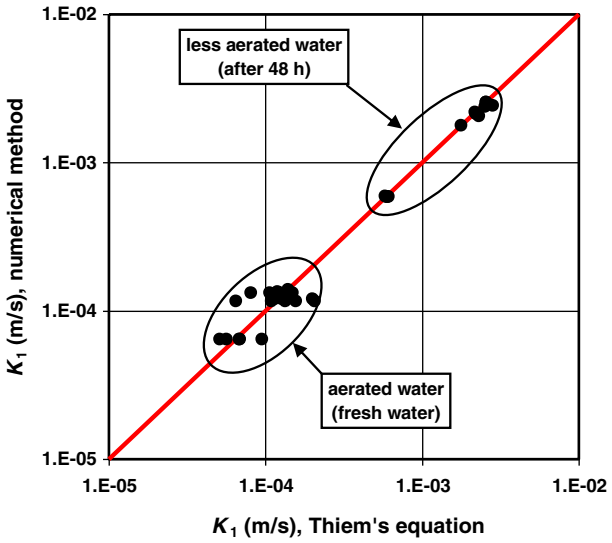

Fig. 3. Correlation between K_1 values obtained either numerically or using Eq. (1) for unclogged screens in a water tank.

The previous values were obtained with a screen in clean water. In field conditions, the screen slots are partially blocked or clogged by solid particles. To assess the performance of pumping well screens after full development the clogging factor is usually taken as 50% (Todd and Mays, 2005): this is a theoretical value. For MWs, the development process is much less severe than for pumping wells, and as a result the clogging factor exceeds 50%. Harich (2009) found clogging factors of 75% for slots of 0.51 mm (0.020 in.) and 85% for slots of 0.25 mm (0.010 in.). The sizes of screen pores, between the solids trapped either against or inside the slots, also decreased, which diminished the K_1 value.

As a result, the field K_1 value of a screen in a soil is lower than that in Fig. 3 (screen in water). For example, Singh and Shakya (1989) performed tests with two screens (OAs of 2.6 and 5.2%) installed in gravelly sand. The K_1 values with soil were much smaller than those in water: they were reduced by 86 and 70% respectively. More clogging would have been found with sand containing traces of silt.

The reduction of K_1 may be proportional to or lower than the OA reduction by slot clogging, which is a major cause of skin effects for pumping wells and MWs (Barrash et al., 2006). Using numerical models, these authors have found field K_1 values in the 10^{-5} to 10^{-6} m/s range, about 1 or 2 orders of magnitude below the values obtained in a water tank (Figure 5). This is a significant reduction.

5.2. Influence of K_1 and K_2 during a field test

5.2.1. No filter pack around the screen

When there is no filter pack, the obtained K_{test} value depends up on the tested soil K_3 value and the screen K_1 value. Fig. 4 shows the analytical (solid lines) and numerical results (symbols) for constant-head tests. The results are very close, the numerical results being slightly below the theoretical ones for an ideal geometry. The screen yields a K_{test} value lower than the real K_3 value when K_1 is smaller than K_3. Each curve in Fig. 4 has three parts: (1) when K_3 is much smaller than K_1, K_{test} equals K_3; (2) when K_3 becomes slightly higher than K_1, then K_{test} is smaller than K_3; and (3) when K_3 becomes much higher than K_1, then K_{test} reaches an asymptote, the screen measuring capacity, as if there was no soil around the screen but only water. As a result, the highest measuring capacity of 2-in.-PVC screens would be the ideal values for unclogged screens (US) in a water tank, thus between 10^{-3} and 5×10^{-2} m/s. However, since the MW slots are partially clogged (CS) and have a clogged K_1 value 10–100 times lower than the

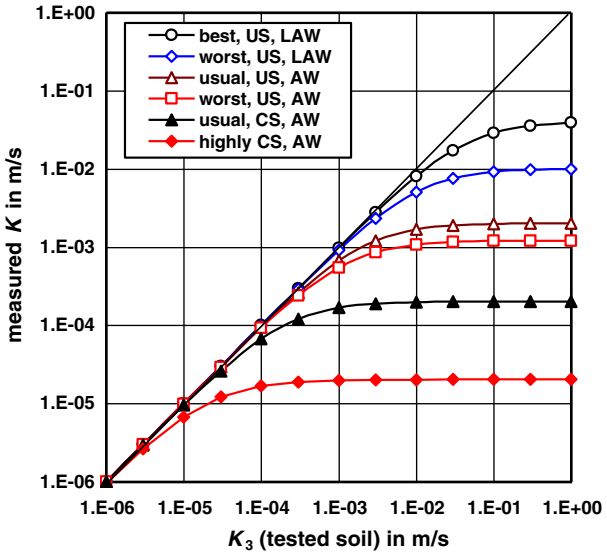

Fig. 4. Influence of screen (K_1) on the K_{test} value when there is no filter pack: analytical results (Eq. (5)) and numerical results are similar; worst for $K_1 = 6 \times 10^{-5}$ m/s; usual for $K_1 = 10^{-4}$ m/s; best for $K_1 = 2 \times 10^{-3}$ m/s; US = unclogged screen; CS = clogged screen; AW = aerated water; LAW = less aerated water.

unclogged K_1 value, the field measuring capacity is in the range of 10^{-5} to 5×10^{-3} m/s.

5.2.2. Filter pack around the screen

The obtained K_{test} depends up on K_3 (tested soil), K_2 (filter pack) and K_1 (screen). A few cases are used here to show by how much K_{test} differs from K_3 for constant-head tests (Figure 5). The screens 0.006d− and 0.010d+ are used with filter sands SF50, SF00 and SF1. Five cases are for unclogged screens, the K_1 value being that given by the tests in the water tank (K_1 in the range of 10^{-4} m/s, see Figure 3). To represent theoretical field clogging conditions, the K_1 values for the clogged screens were taken as 10^{-5} m/s, to investigate the field range of reduced K_1 value by a factor of about 10.

If K_1 (screen) is smaller than any K_2 (filter sand), then K_2 has little influence and K_1 has the chief influence on K_{test}. The results (Figure 5) are similar to those of Fig. 4: the K_{test} value underestimates the tested soil K_3 value when K_3 is higher than K_1. The results place the asymptote at about 50 K_1, thus about 5×10^{-4} m/s for this screen and the limited clogging condition of case 1.

If K_1 is higher than K_2 the obtained results (Figure 5) are also similar to those of Fig. 4, but they now depend only on the K_2 value: K_{test} starts to underestimate K_3 when K_3 is higher than K_2, and the asymptote or measuring capacity has a lower value than that of the case with the same K_1 value but without filter sand. Thus, the filter sand is a major obstruction yielding the K_3 underestimate.

When K_2 is higher than K_1, the influence of K_2 on K_{test} is small but not negligible. The head losses in the filter sand become negligible when K_2 is about 30 times higher than K_1. For lower K_2 values, the K_{test} value is influenced by both K_1 and K_2, the asymptote value being lower than that obtained for the case without filter sand. The field measuring capacity is in the range of 10^{-5} to 10^{-3} m/s.

5.2.3. Numerical examples of variable-head tests

A few variable-head tests were numerically simulated using the code Seep/W (Geoslope, 2007). The screen had an internal radius r_{1i} of 2.5 cm and an external radius r_{1o} of 3.0 cm, whereas r_{2i} was 3.0 cm and r_{2o} was 7.5 cm for the filter pack. A few combinations of K_1 (screen) with K_2 (filter) and K_3 (soil) were analysed. The screen was 60 cm long, the filter pack was 80 cm long, and it was in the middle of a 2 m thick aquifer. The zone influenced by the slug test was numerically found to be very limited, which confirmed the similar findings of Guyonnet et al. (1993). The models had an external boundary at 3 m from the centre of the MW axis. The MW pipe was treated as a rigid and very pervious material in which the functions for volumetric water content and unsaturated permeability were those of a pipe element (Chapuis, 2009a). Numerically, the slug test was modelled in three steps. The first step gives the initial conditions, a constant hydraulic head within the aquifer. The second step is rapid infilling (2 s) of the MW solid pipe. The third step is the slug test, the water level in the solid pipe slowly returning to equilibrium. The numerical data yielded straight velocity graphs (Chapuis, 2001; Chapuis and Chenaf, 2002), which yielded perfectly straight semi-log graphs (Hvorslev, 1951).

Numerical results for slug tests with an unclogged screen ($K_1 = 2 \times 10^{-4}$ m/s) and a clogged screen ($K_1 = 10^{-5}$ m/s) appear in Fig. 6. They gave K_{test} values which are similar to those predicted by Eq. (5)

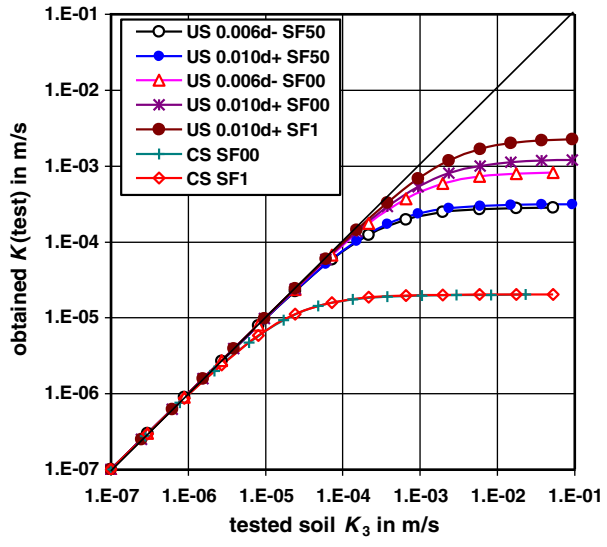

Fig. 5. Influence of K_1 (screen) and K_2 (filter sand) on K_{test} for constant-head tests with fully penetrating screens. US = unclogged screen, CS = clogged screen with $K_1 = 10^{-5}$ m/s; 0.006d− is the type of screen; SF50 the type of filter sand.

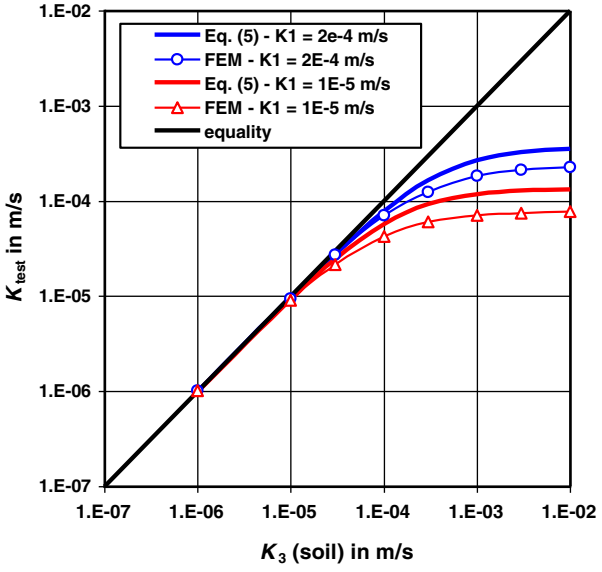

Fig. 6. Influence of K_1 (screen) and K_2 (filter sand) on K_{test} for variable-head (slug) tests and one case of partially penetrating screen (FEM = finite element method).

for constant-head tests with fully penetrating MWs, but slightly lower MW measuring capacities. The differences can be explained by two reasons:

1. The slug test was performed using a partially penetrating MW whereas the constant-head tests were performed using a fully penetrating MW.
2. The slug test has a smaller radius of influence due to the short duration of initial slugging.

It means that for slug tests in MWs, the maximum K_{test} value is about 10^{-3} m/s for large slots and correct filter sand, and they may be as low as 5×10^{-6} m/s for fine slots and some field clogging exceeding the theoretical rule of 50% clogging that is used for fully developed pumping well screens.

5.2.4. Cases of properly designed MWs

After having shown that usual MWs have a field measuring capacity in the range of 10^{-5} to 10^{-3} m/s, let us look at perfectly designed and installed MWs, following the rules of Table 2. Consider first the case of uniform aquifer and filter materials (indexes am and fm): if the GSDCs are parallel, then the D_{50} ratio is equal to the D_{10} ratio and thus the ratio K_2/K_3 is nearly equal to the square of the D_{10} ratio (Chapuis, 2004, 2012a). As a result, K_2 is higher than about 25 K_3. This means that the hydraulic head losses in the filter material can be neglected for a field permeability test. In this first case, the MW field measuring capacity is controlled exclusively by the screen.

Consider a second case of an aquifer material with the following characteristics: D_{10} (am) = 0.92 mm, D_{60} (am) = 0.74 mm, C_U (am) = 8, and D_{30} (am) = 0.15 mm. According to the criteria of Table 2, since C_U is higher than 5, in the graph (Figure 7) and at y = 30% we locate the points of abscissa $6D_{30}$(am) and $9D_{30}$(am). The filter material, with C_U(fm) ≤ 2.5, must fall between these points, thus filter sand 1 is selected for the data in Fig. 7. This selection assumes a homogeneous aquifer material. However, the aquifer material GSDC has a double curvature, which may indicate that the aquifer material is stratified and formed of layers of two uniform materials. The decomposition method which gives the GSDCs of the finer and coarser layers is explained in Chapuis et al. (2014). In such a case, the filter sand 0 must be selected to stop the migration of solid particles in the finer material layers, otherwise the MW screen will be severely clogged and the sampled groundwater may remain turbid.

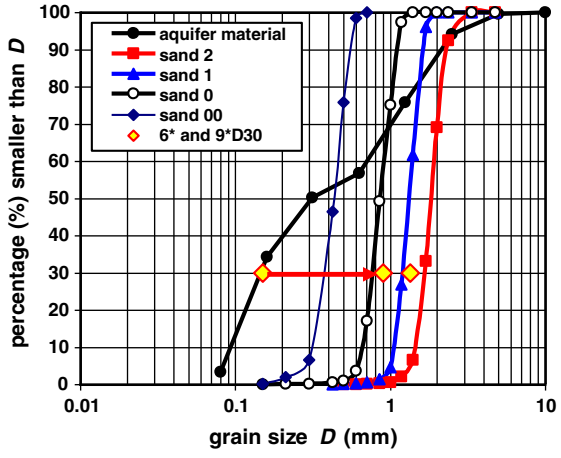

Fig. 7. Selection of the filter sand according to the criteria of Table 2. If the aquifer material is homogeneous, the filter sand 1 is appropriate. However, if the aquifer material is stratified, bimodal, the filter sand 0 must be selected, otherwise the MW screen will be severely clogged and the sampled groundwater will be turbid.

6. Discussion

The aquifer K values may be assessed using several methods, and at different scales. The small scale is that of soil samples: their quality must be assessed and their grain size distribution curve (GSDC) analysed to check for mixes of sub-layers, before using reliable methods to predict the K values. The middle scale is that of field permeability tests in driven flush-joint casings and monitoring wells (MWs), for which reliable methods must be used (Chapuis and Chenaf, 2002). The large scale is that of pumping tests.

This paper has studied only permeability tests with MWs (middle scale evaluation). It has determined how the parasitic head losses in the screen and filter pack influence the K_{test} value and by how much K_{test} differs from the true K_3 of the tested soil. These findings are valid only for permeability tests in MWs. If the same MWs are used to monitor a pumping test, the screen and filter will have a different influence on the extracted K (pumping test) value, which will be mostly due to some increased time-lag in the drawdown curve. The study of this influence during pumping test is outside the scope of the paper.

Because pumping tests involve large volumes, which are more likely to meet preferential flow paths, one may believe that they may yield larger K values than tests at a smaller scale (Bradbury and Muldoon, 1990; Rovey and Niemann, 2001). Thus, the K value may increase with the tested volume, which is viewed as a scale effect. However, there is no consensus about this scale effect. Many studies tried to check or challenge academic opinions, examining testing techniques, investigated scales, and geologic media. Alas, the quality of each K value was not questioned. Regrettably, the quality control of groundwater parameters, which must be methodically completed for engineered facilities, is rarely done for environmental studies (Chapuis, 1995). Butler and Healey (1998) rejected the idea of scale effect, judging that it results from artefacts linked to incomplete well development and low-K skins around well screens, but they did not study what produce a positive or negative skin. The skin phenomena and their effects on the K_{test} value are important for safety issues, and thus are more studied in geotechnique than in geosciences (Chapuis and Chenaf, 2010). This paper has documented by how much poorly installed MWs underestimate the highest range of field K values and thus, decrease the middle scale K values, which may contribute to create an artificial scale effect.

7. Conclusion

Head losses in the MW screen and filter pack may strongly influence the permeability test results. Most often, this leads to underestimate the sought K_3 value of the tested material. Any screen-filter couple has an influence on the K_{test} value obtained by a permeability test. The K_{test} value is always equal to or smaller than the true K_3 value of the tested material. The underestimate depends up on the hydraulic conductivities K_1 and K_2 of the screen and filter material. Predictive equations give the filter K_2 value. This paper gives the equivalent K_1 of the screen. Several common screens were tested in a water tank. The test results for K_1 were shown to depend up on the time that water rested in the tank. Freshly poured water has the highest content of dissolved gas, and yields the lowest K_1 values. If water had been at rest for a few days, its dissolved gas constant had decreased to come closer to the equilibrium gas content as given by Henry's law, and the K_1 value was increased. Field condition differs from laboratory condition in a water tank. To decrease the air content (hardly visible micro bubbles) in the filter pack, the filter material must be deaired in a water bucket before being slowly poured with water into the drilling casing using a tremie. This precaution may increase the MW measuring capacity by a factor of 10 or more. The impact of high gas content in the filter pack was examined elsewhere (Chapuis, 2009b).

For studies to assess the field K_3 values, the filter material must be selected according to known design rules. However, for environmental studies, it is common to select a too fine filter pack. This supposedly

prevents fine solid migration and turbid groundwater samples (Houben and Hauschild, 2011). This is correct for groundwater sampling but it has negative effects for permeability testing.

With known K_1 and K_2 values, the MW measuring capacity is assessed using the provided closed-form equations, which have been verified using numerical models. A graph may also be drawn to visualise by how much K_{test} under evaluated the tested material true K_3 value.

When evaluating groundwater conditions at any site, the permeability data must be examined with caution because the permeability tests may have been performed in poorly designed and installed MWs with limited measuring capacities. This field condition results in under evaluating the field K values, and thus in under evaluating the seepage and contaminant velocities.

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Notations

AW: aerated water
b: vertical thickness (m) of the ideal confined aquifer
CS: clogged screen
C_U: coefficient of uniformity (non-dimensional)
D_x: grain size (mm) corresponding to x% passing
GSDC: grain size distribution curve
h_{1i}: hydraulic head (m) at the inner face of the screen (this is a boundary condition)
h_{1o}: hydraulic head (m) at the outer face of the screen
h_{2i}: hydraulic head (m) at the inner face of the filter sand (usually $h_{2i} = h_{1o}$)
h_{2o}: hydraulic head (m) at the outer face of the filter sand
h_{3i}: hydraulic head (m) at the inner face of the in situ ideal soil (usually $h_{3i} = h_{2o}$)
h_{3o}: hydraulic head (m) at the outer face of the tested soil (this is a boundary condition)
K_1: equivalent hydraulic conductivity (m/s) of the screen
K_2: hydraulic conductivity (m/s) of the filter sand
K_3: hydraulic conductivity (m/s) of the in situ ideal soil
K_{test}: calculated hydraulic conductivity (m/s) using the field permeability test data when hydraulic head losses in the screen and in the filter pack are not taken into account
LAW: less aerated water
MW: monitoring well
OA: open area (%)
Q: pumped flow rate, m^3/s
US: unclogged screen
r_{1i}: radial distance (m) to the inner face of the screen
r_{1o}: radial distance (m) to the outer face of the screen
r_{2i}: radial distance (m) to the inner face of the filter sand ($r_{2i} = r_{1o}$)
r_{2o}: radial distance (m) to the outer face of the filter sand
r_{3i}: radial distance (m) to the inner face of the in situ ideal soil ($r_{3i} = r_{2o}$)
r_{3o}: radial distance (m) to the outer face of the in situ ideal soil
Δh_1: hydraulic head loss (m) in the screen
Δh_2: hydraulic head loss (m) in the filtering sand
Δh_3: hydraulic head loss (m) in the in situ soil