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Field performance of four vibrating-wire piezometer installation methods

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Abstract

Vibrating wire piezometers provide a number of advantages over the traditional hydraulic piezometer design. There are many methods and configurations for installing vibrating-wire piezometers, with the most common being: single piezometers in sand packs (SP), multilevel piezometers in sand packs (MLSP), and fully-grouted multilevel piezometers using either bentonite (FGB) or cement-bentonite grout (FGCB). This study assesses the performance of these four different installation methods for vibrating wire piezometers at a field site possessing complex stratigraphy, including glacial and marine sediments. Pore pressure data recorded between December 2017 and July 2019 were analyzed to accomplish this objective. Data indicate that SP, MLSP, and FGB piezometers performed well. This determination is based on the fact that piezometers installed at the same depth with these arrangements recorded similar pressure variations that were coherent with the hydrogeological setting. Of the two fully-grouted installations using cement-bentonite grout, one installation failed completely due to a hydraulic short circuit, caused either by shrinkage of the grout or flow occurring along the wires of the embedded instruments. While the FGB-type piezometers used in this study worked correctly, the lack of standard methods concerning both the construction of fully-grouted piezometers is concerning. Furthermore, the lack of a standard method for mixing cement-bentonite grout likely contributed to the failure of the FGCB installations. Thus, due to the lack of guidance for both construction and grout preparation, the use of a bentonite grout removes a degree of uncertainty when fully-grouted installation techniques are used.

RÉSUMÉ

Keywords:

1 Introduction

In the province of Quebec, 89% of the population lives within the extent of the Champlain Sea formations, which were deposited by seawater invasion after the Wisconsin glaciation and are prone to landslides (Demers et al. 2014). In response to the landslide risk posed by these formations, the Quebec Ministry of Transport (MTQ) has deployed an extensive network of vibrating-wire piezometers (VWP) to monitor spatial and temporal variation of pore pressures in clay slopes (Cloutier et al. 2017). This type of piezometer provides a number of advantages over the traditional hydraulic piezometer design, very small amounts of liquid are required for measurement; measurement accuracy is very high; response times to changes in water pressure are very quick; and it is possible to transmit observed responses over long distances (McKenna, 1995). However, the largest advantage of using of VWPs is the ability to deploy multiple instruments within a single borehole. These multilevel VWP installations are created by mounting several piezometers to a single grout tube, allowing for multiple depths to be instrumented quickly and inexpensively.

Currently, there are many methods of constructing multilevel VWP installations, however the most common are multilevel piezometers in sand packs, and fully-grouted multilevel piezometers using either bentonite or cement-bentonite grout. Historically, the most common method for installing a vibrating-wire piezometer involves encasing a single instrument within a sand pack (referred to as the SP method, hereafter), which is then sealed above and below the measurement interval by layers of bentonite (Obbink, 1969; Anochikwa et al. 2011). Piezometers installed in this manner can either be installed as a single instrument, or multiple sand pack/bentonite caps can be utilized within a single borehole (i.e., multilevel VWPs with sand packs, MLSP; Germain et al. *Submitted*). While the use of sand packs has long been the standard method for installation, concerns over cost, as well as the long-term integrity of sand packs and bentonite caps have resulted in fully-grouted methods becoming increasingly popular in recent years (Simeoni, 2012; Marefat et. 2018).

Fully-grouted methods result in a piezometer that is completely encased by either bentonite or cement-bentonite grout, and does not include a sand pack (Mikkelsen, 2002). When these methods are used, the VWPs are first attached to a grout pipe that is then lowered into the base of the borehole. Grout is then pumped through the pipe, completely filling the borehole and encasing the piezometers (Contreras et al. 2008). Advocates of this approach cite faster installation times and a lower potential for water to be routed to the piezometer from above or below the targeted installation depth (i.e., “short-circuiting”; McKenna, 1995). Furthermore, the ability to mount multiple instruments to a single grout pipe allows for a nest of VWPs to installed in the absence of a sequence of bentonite seals and sand packs (Mikkelsen and Green, 2003).

While fully-grouted methods have distinct advantages, there are potential drawbacks which have not been thoroughly explored. Specifically, fully-grouted methods require precise preparation of the grout mixture, and the grout hydraulic conductivity (K) needs to be calibrated to ensure that it is at least 2-3 orders of magnitude lower than the surrounding formation (McKenna, 1995). Improperly mixed grout may crack during curing, resulting in short circuiting. Meanwhile, grout that is too permeable relative to the surrounding formation can cause measurement errors (Marefat et al. 2018). While each installation method has associated risks and advantages, their comparative performance has not been extensively documented in the scientific literature.

This study focuses on how different installation methods impact the long-term reliability of multilevel piezometer installations, as well as the accuracy of the observations they produce (i.e., the performance of each installation). Specifically, this work was performed to assess the risks and benefits of using either sand packs or fully-grouted methods. As such, the objective of this study is to (1) assess the performance of different methods for constructing multilevel VWP installations at a field site possessing a complex stratigraphy, and (2) to recommend the most suitable methods for installing and configuring an observation network for monitoring groundwater conditions. To accomplish this goal, multilevel VWP installations were constructed at a field site in the municipality of Sainte-Anne-de-la-Pérade (Québec, Canada) using four different methods. The results of these multilevel installations were then compared to a reference dataset, gathered from individual vibrating-wire piezometers in sand packs (SP). An analysis of pore pressure data recorded between December 2017 and July 2019 was then conducted to compare the performance of the different piezometers.

2 Study area

The study site is located close to St-Anne-de-la-Pérade, about 100 km west of Quebec City, on the north shore of the St-Lawrence River in the Province of Quebec, Canada (Figure 1).

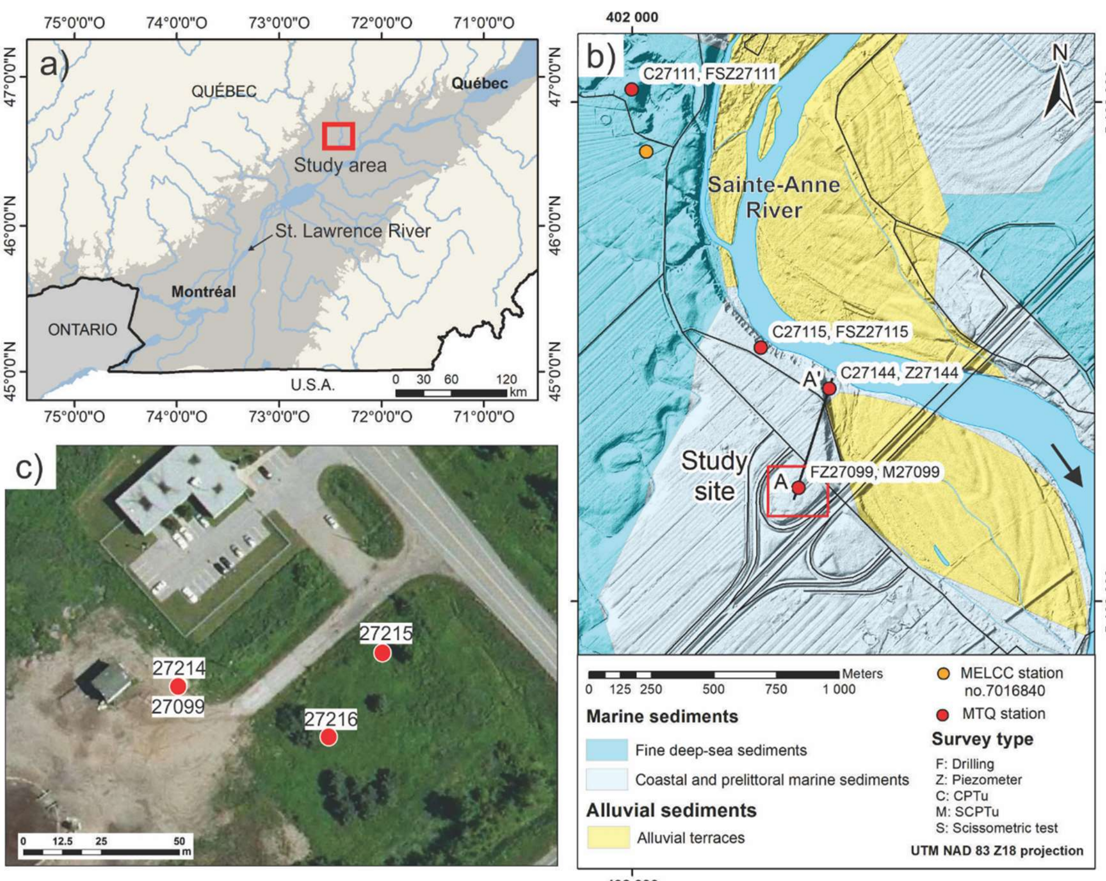

Figure 1. a) Site location within the St. Lawrence Lowlands basin, showing the area invaded by Champlain Sea (dark gray). b) Digital elevation model overlain with the surficial geology of the study area. The location of the study site is shown within the red box. c) Map of the study site showing the location of the monitoring instrumentation used in this study.

The study area is located within the St. Lawrence Lowlands basin, and the bedrock unit present at the study site, the Utica Shale, is composed of calcareous shale and clayey limestone. Sitting atop the shale is a 35-m thick complex succession of marine sediments as the area was invaded by the Champlain Sea following the last glacial maximum. This sequence constitutes a clay plain that is locally covered by littoral sediments (Figure 2). More modern sedimentary deposits can be found along the Sainte-Anne rivercourse, and the river is deeply incised within its current channel.

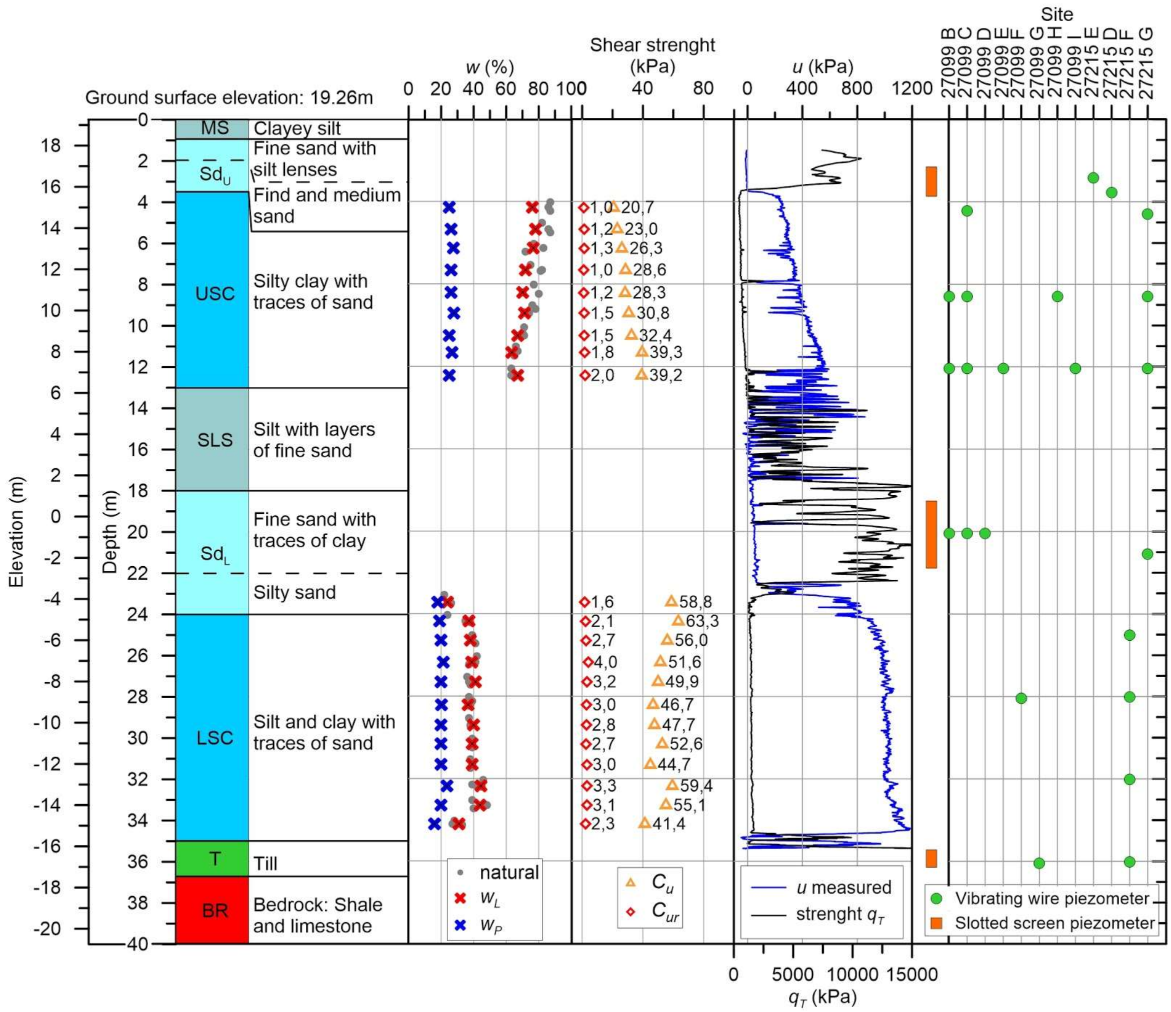

Figure 2: Geotechnical profile at location 27214 (located on Figure 1c) showing site stratigraphy, liquid and plastic limits, undrained shear strength, pore pressure and corrected tip resistance from CPTu and piezometers depth.

The sedimentary units present on site begin with a 2-m-thick layer of till (T), followed by thick silt and clay deposits with traces of sand that span about 10 meters (LSC). The silt is overlain with a silty-sand layer (2 m thick) that is followed by a fine sand layer that contains traces of clay (4 m thick). Since these two units are hydrostratigraphically similar, they are combined into a single hydrostratigraphic unit, Sd_L. Above unit Sd_L is a 4.5-m-thick silt unit that contains layers of fine sand (SLS). This unit is followed by an 8-m-thick layer of silty clay (USC). The upper 3 m of the site is composed of a sequence of fine and medium sands that are overlain by a thin fine sand unit containing silt lenses. As with unit Sd_L, because this sequence of units displays similar hydrogeologic characteristics, they are combined into the hydrostratigraphic unit Sd_U. This unit is then being capped by a layer of clayey silt, which forms the modern surficial material (MS). extensive discussion of the site geology, hydrogeology, and geotechnical properties can be found in Germain (2019) and Diene (1989).

The water table on site is found about 2.1 m below the surface in the layer of fine to medium sand. The upper part of the sequence constitutes a shallow unconfined aquifer, while the fine sand layer found at depth (unit Sd_L, 18-24 m depth; Figure 3) can be considered a deep aquifer. Despite its depth, previous work has suggested that a phreatic water table may be present, as the sand layer was found to be only partially saturated at certain points in the year (Figure 3, unit SLS; Germain et al. *Submitted*).

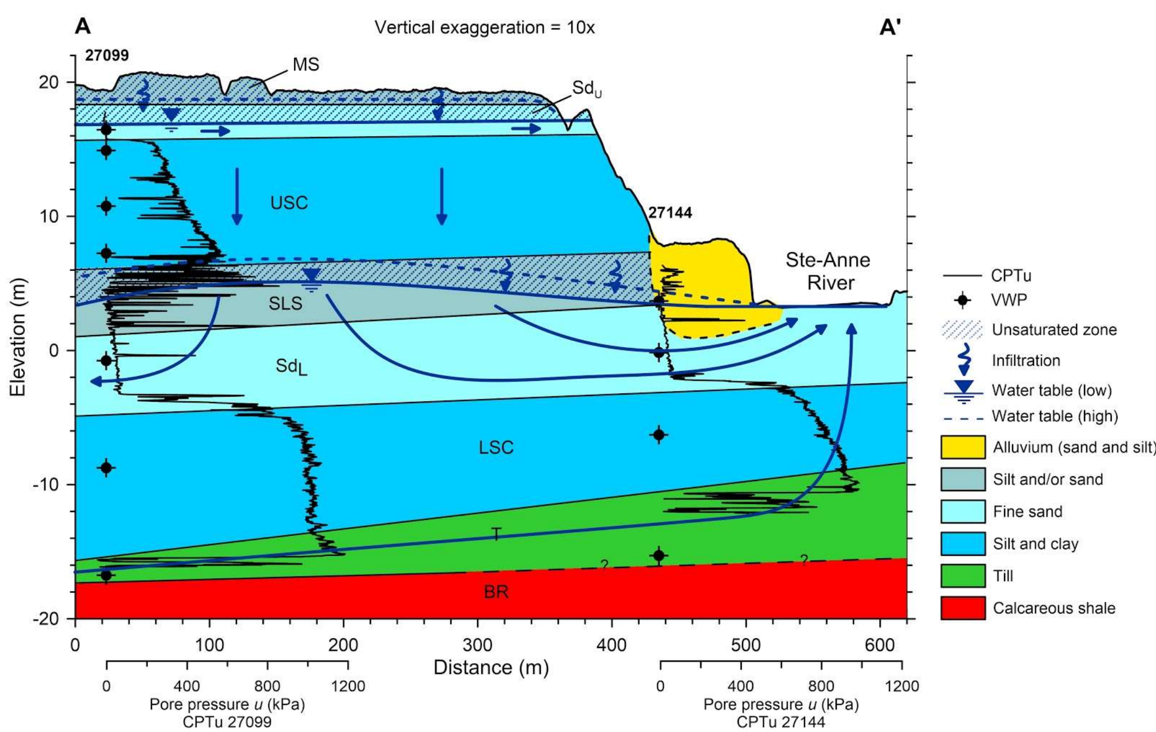

Figure 3: Cross section showing site hydrogeology and the distribution of geologic units between the field site (location A) and the far bank of the Sainte-Anne River (location A')

The field site is located about 500 m away from the St. Anne River on level ground. While the site was chosen because it is located far from the river and any important topographic features, Germain et al. (submitted) demonstrated that seasonal river stage variations influences pore water pressure at the site, due to a hydraulic connection within the deep fine sand layer (unit Sd_L).

3 Methods

3.1 Piezometer installation methods

The field site was instrumented with Geokon model 4500S vibrating-wire piezometers. The piezometers were then connected to a datalogger (Geokon, LC-2 Series, Model 8002-16) that converts the vibration frequencies measured by the instruments into pore pressures through the use of a parabolic equation. The sampling rate for pore pressure measurements was initially set to 12 hours from the period of December 2017 to June 2018. The sampling rate was refined to 15 minutes for a pumping test (performed between June 22 and August 6, 2018), as well as the entire fall of 2018. The rate was then coarsened to 1 hour for the period from January 6, 2019 to the present.

The original set of piezometers installed on-site were individual piezometers in sand packs (SP). In order to compare the performance of multilevel piezometers installed using different methods, four multilevel piezometer nests were constructed using three different methods: multilevel piezometers in sand packs (MLSP), and two different fully-grouted methods: one using bentonite grout only (FGB) and one using a cement-bentonite grout (FGCB). Data collected from the multilevel installations were then compared to data gathered from the SP-type piezometers to assess similarity of performance.

3.1.1 Individual piezometers sand packs

This installation type consists of a single vibrating-wire piezometer installed in a sand pack (SP) at the bottom of a borehole. The VWP is located in the middle of a 60 cm (2 foot) long sand pack made of industrial silica. The sand pack is overlain by a 60- cm-long bentonite plug before the borehole was filled with a bentonite-cement grout (Figure 3a). The grout used had water:cement:bentonite proportions of 5:1:1.25.

3.1.2 Multilevel piezometers in sand packs (MLSP)

This installation type consists of four multilevel vibrating-wire piezometers within a single borehole. The VWPs are located in the middle of 60 cm (2 foot) long sand packs made of industrial silica. The sand packs are underlain and overlain by a 60 cm-long bentonite plug. Between the bentonite plugs, the borehole is filled with well-sorted crushed stone (Figure 3b).

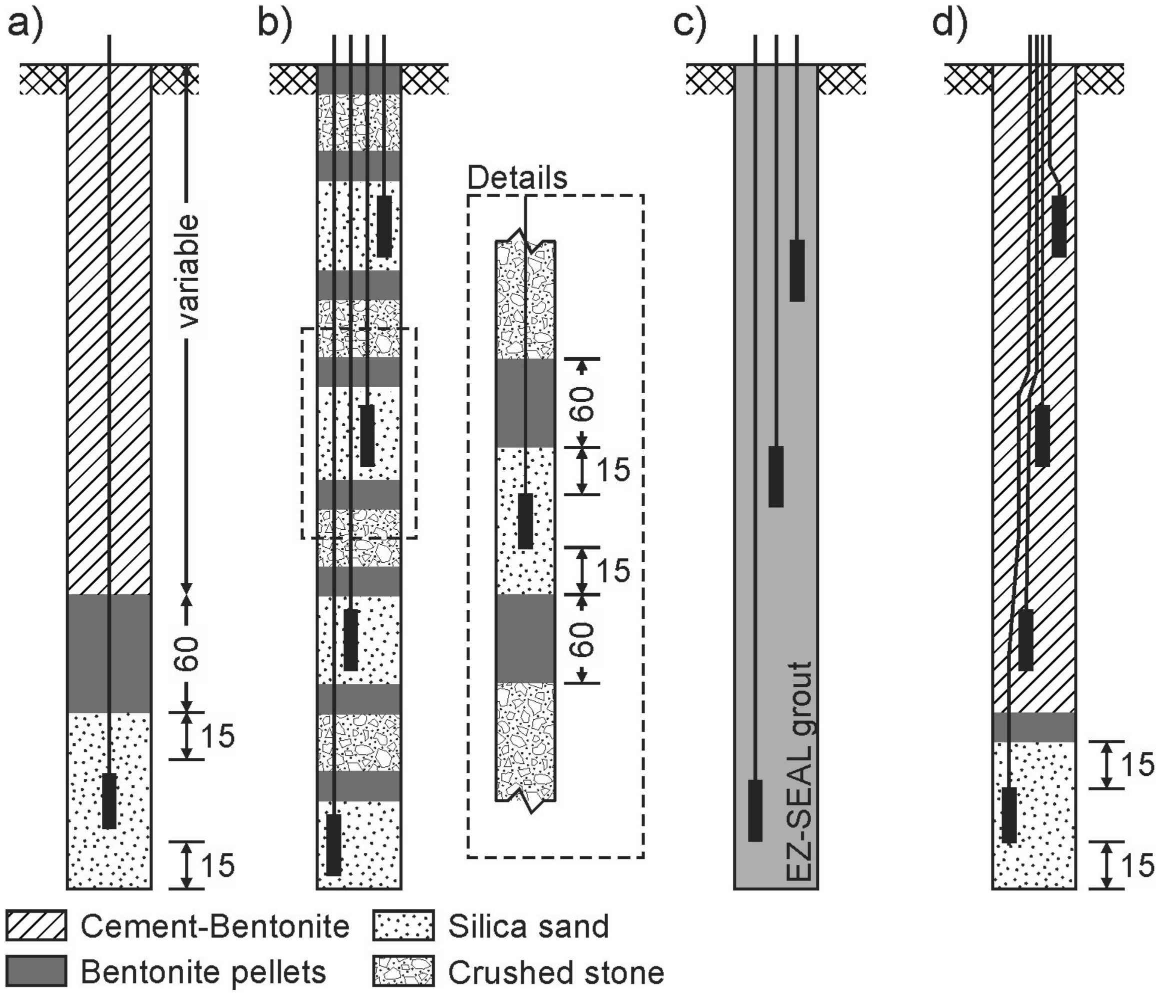

Figure 3: Schematic diagram of (a) a piezometer installed in a sand pack (SP), (b) a multilevel piezometer nest where each piezometer is installed in a sand pack (MLSP), (c) a fully-grouted piezometer nest within a single borehole (FGB), and (d) a cement-bentonite-grouted piezometer nest within a single borehole (FGCB). Length is in cm.

3.1.3 Fully-grouted multilevel piezometers (bentonite grout; FGB)

This installation type consists of four multilevel, fully-grouted, vibrating-wire piezometers within a single borehole. The grout is a mixture of granular bentonite (EZ-SEAL, Baroid Industrial Drilling Products) and water mixed at 20% solid content. According to the manufacturer datasheet, the typical hydraulic conductivity of this mixing ratio is 1×10^{-11} m/s. When installing these piezometers, the four VWP wires are attached using tie wraps, and then suspended at the desired depth in the open borehole, within the drill rods. Then, the grout mixture is pumped from the bottom of the borehole toward the surface, and the drill rods are progressively removed, leaving the piezometers encased in grout (Figure 3c).

3.1.4 Fully-grouted multilevel piezometers (cement-bentonite grout; FGCB)

This type of installation is similar to the fully-grouted multilevel piezometers, except that a mixture of cement and bentonite is used, and the bottom piezometer is located in a sand pack instead of being fully grouted (Figure 3d). The sand pack is overlain by a 60-cm-thick bentonite plug. The grout mixture used has water:cement:bentonite proportions of 5:1:1.25, and using tables from Mikkelsen and Green (2003) the resulting mixture would have a hydraulic conductivity of 6×10^{-9} m/s. Aside from these differences, the installation is the same as the method detailed in Section 3.1.3.

3.1.5 Installation depth

The installation depths of the piezometers for each configuration is presented in Table 1 and Figure 2. Comparison of different configurations is made possible by instances where piezometers of different arrangements are located at the same depths. For example, at the depths of 8.5 m and 12 m, there is a piezometer in each of the three configurations, and at 20-m-depth, there are piezometers in the FGB and MLSP configurations.

Table 1: Installation depths for each installation method, as well as the lithology of the material containing each piezometer. Shaded cells indicate the depths where piezometers were installed. Note that there are two different FGCB installations, while there is only one installation each for the MLSP and FGB-type piezometers. Further note that for the single piezometer, sand pack (SP) installation method, each shaded cell represents an individual borehole. The hydrostratigraphic units referenced in Column 3 of the Table correspond with those presented in Figure 2.

Installation depth (m)	Piezometer installation method				Instrumented material (hydrostratigraphic unit)
	SP	MLSP	FGB	FGCB	
2.8					Sand (Sd_U)
3.5					
4.35					Clay (USC)
4.5					
8.5					
12					
20					Sand (Sd_L)
21					
25					Clay (LSC)
28					
32					
36					Till (T)

Legend:

Installation Method:

SP	Single piezometer in a sand pack
MLSP	Multilevel piezometers with sand packs
FGB	Fully-grouted multilevel piezometer with bentonite grout
FGCB	Fully-grouted multilevel piezometer with cement-bentonite grout

Additional details:

FGCB installation 1 (depths: 4.5m, 8.5m, 12m, 21m)

FGCB installation 2 (depths: 25m, 28m, 32m, 36m)

3.2 Barometric compensation

Barometric compensation was applied to the VWP data in order to remove atmospheric interference from the pore pressure data. This study utilizes the linear method due to its relative simplicity and accurate results. According to this method, the barometric compensation of measured VWP pore pressures is described by (Germain et al. *Submitted*):

$$u_t^* = u_t - LE(B_t - B_{ave}) \tag{1}$$

where u_t* is the corrected pore pressure at time *t*, u_t is the measured pore pressure at time *t*, *LE* is the loading efficiency, B_t is measured barometric pressure at time *t* and B_{ave} is the average barometric pressure measured for the site. The loading efficiency correspond to the fraction of barometric pressure that is transmitted to pore pressure through soil compressibility, and is defined for undrained conditions as (Equation 1b):

$$LE = \frac{m_v}{m_v + n\beta_w} \tag{1b}$$

where m_v is the soil vertical compressibility, *n* is the soil porosity and β_w is the water compressibility. Practically, it is obtained from the ratio of pore pressure change to a change in barometric pressure (Equation 1c):

$$LE = \frac{\partial u_w}{\partial B} \tag{1c}$$

Following barometric compensation, the pore pressure was converted to hydraulic head (m) by adding the pore pressure to the elevation of the VWP using the mean sea level datum.

3.3 Numerical modelling

A 2-D, radial-coordinate numerical model was used to explore two questions, (1) what is the impact of the grout permeability on transient head values in the piezometers, and (2) can anomalies within the data could be explained by the presence of a hydraulic short-circuit. The model extent is restricted to the silty clay unit (USC; 3-13 m depth) where simple groundwater flow conditions were found (i.e., downward vertical flow). Furthermore, because the sandy unit below unit USC (Sd_L) is partially-unsaturated and hydraulically connected to the Sainte-Anne river, simulating the entire sediment sequence would require the use of a significantly larger-scale model that includes the dynamics of the river, which is beyond the scope of the present study (Germain et al., *submitted*).

Simulations of the groundwater flow were performed using the model HydroGeoSphere (HGS; Therrien et al. 2006). For the conditions considered in this study (fully-saturated conditions with transient water levels) this software simulates the flow of groundwater using the control-volume, finite-element method. The general flow equation solved by HGS is:

$$K \frac{\partial^2}{\partial x^2} = S_s \frac{\partial h}{\partial t} \quad (2)$$

where K is hydraulic conductivity (m/s), h is hydraulic head, S_s is specific storage (m^{-1}), and t is time (s). Given the geometry of the system under study, and for reasons of numerical efficiency, this equation is solved in radial coordinates with axisymmetric geometry.

Two sets of simulations were conducted. First, natural conditions (i.e., without a grouted borehole) were simulated to obtain the theoretical transient head values within the upper clay layer (USC; Figure 2) as influenced by head variations in the overlying unconfined aquifer. Second, a borehole with a permeable grout was included in the clay layer, and grout permeability was varied until a reasonable match was obtained with observed heads. Note that no formal calibration exercise was performed, as the objective was to test whether the excessively-permeable grout hypothesis can explain the observations in the fully grouted piezometers.

The simulated vertical domain is 26 m long, with prescribed head boundaries at the top and bottom of the model domain. The hydraulic head time series measured in a single VWP located in the sand aquifer (Sd_u; Figure 2) above unit USC is prescribed as a time-varying boundary condition at the top boundary of the model, while the bottom boundary is prescribed a constant head value of 0 m. This value was chosen in order to produce a vertical downward gradient of about 0.65 m/m in the clay unit, as measured in the field. While the actual thickness of the clay layer is 10 m, an additional 16 m of clay was simulated in order to prevent the appearance of boundary effects in the simulated piezometers (Fig. 4).

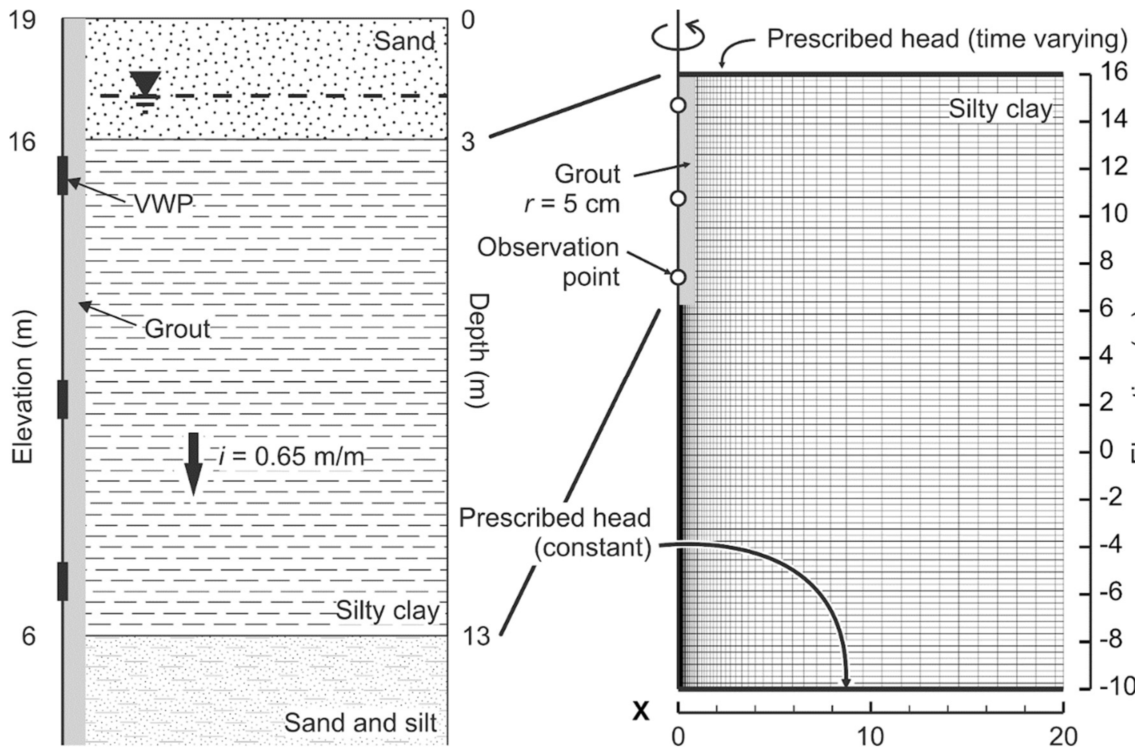

Figure 4. Conceptual model of the simulated domain, along with mesh and boundary conditions used for the numerical simulations.

The model has 60 horizontal elements, the width of which is refined from 1 m at the right boundary of the model, to 1 cm, near the well (left boundary). There are 260 vertical elements with a constant height of 10 cm, regardless of width. The hydraulic conductivity of the clay is considered isotropic with a value of 6.5×10^{-10} m/s, and the specific storage is 3.8×10^{-5} m^{-1} ($m_v = 3 \times 10^{-10}$ kPa^{-1}; Germain et al., *submitted*). These values were obtained using both permeability tests in a triaxial cell, and the interpretation of the effects of barometric variations on the hydraulic heads measured at the site. The grout material has a radius of 5 cm (2 in.) and a specific storage value similar to the clay was used. As mentioned above, the hydraulic conductivity was modified until a reasonable fit was obtained with the observed data. In order to do this, observation points were inserted in the model at 4.35, 8.5 and 12 m depth (model elevation of 14.65, 10.5 and 7 m respectively), which are the same depths as the piezometers in the FGCB installation.

3.4 Events influencing hydraulic head measurements

3.4.1 Pumping test

A pumping test was carried out on-site during the period from June 22 to August 6, 2018 at location 27215 (vertical shaded region, Figures 5-11). During this 45-day period, the various slotted-screen piezometers at the study site and the vibrating-wire piezometers at Location 27215 (Figure 1) were used as observation points. Consequently, pumping associated with the test may have influenced the pressure measurements recorded by the various piezometers on-site. However, the extent to which pumping may have influenced measurements in an individual piezometer is highly dependent on the horizontal and vertical distance from the pumping well. The highest impacts are expected in the deep fine sand unit where the pumping well was located. Potential effects resulting from the pumping test will be considered when comparing pore pressures for piezometers located at the same depth, but installed in different configurations. Further details on the pumping test can be found in Germain (2019).

3.4.2 Grounding

For the multilevel piezometer using sand packs (MLSP) and the fully-grouted installation using bentonite-only grout (FGB), a pressure anomaly was observed beginning June 1 at midnight until lasting until June 23, 2018. This anomaly was the result of improperly-grounded dataloggers, which were influenced by a thunderstorm that was present over the site from 6 p.m. to 7 p.m. on June 1, 2018 (denoted by a vertical dashed line, Figure 5-11). Measurements returned to normal on June 23 after the installation of a grounding wire. Thus, it is expected that the pressures measured by the

MLSP and FGB configurations will differ from those measured by the other piezometer configurations for the aforementioned time period.

4 Results

The results are presented as follows: first, the hydraulic head values from the single piezometers in sand packs (SP) are presented as a reference dataset that will be used assess the performance of the multilevel installations. Next, the head profiles at individual depths across all four types of installations are compared. The hydraulic head profiles from all piezometers within a particular piezometer nest are then presented in order to better identify any malfunctioning piezometers. Finally, results from the numerical model are shown.

All results include differential head variations since the beginning of the monitoring period in order to better compare the measured hydraulic head values between installations. Additionally, all figures include water table variations measured with a single VWP enclosed in a sand pack (referred to hereafter as $SP_{2.8}$), located within the unconfined upper aquifer at 2.8 m depth. This head profile allows for comparisons to be made between the dynamics of the water table in the unconfined aquifer and the head profiles measured by each of the piezometer nests. Note that all figures highlight the presence of the thunderstorm (vertical dashed line, Figures 5-11) and the pressure anomaly (vertical shaded region, Figures 5-11) caused by the pumping test as described in Section 3.4.1.

4.1 Reference dataset: individual piezometers in a sand pack (SP)

While not a true piezometer nest, piezometers of the same construction located at different depths can be superimposed to detect possible measurement anomalies in the multilevel installations. When plotted together, data from the SP-type piezometers show that increases in hydraulic head during the spring recharge event are largest near the surface, and then progressively dampen with depth (Figure 5, left panel).

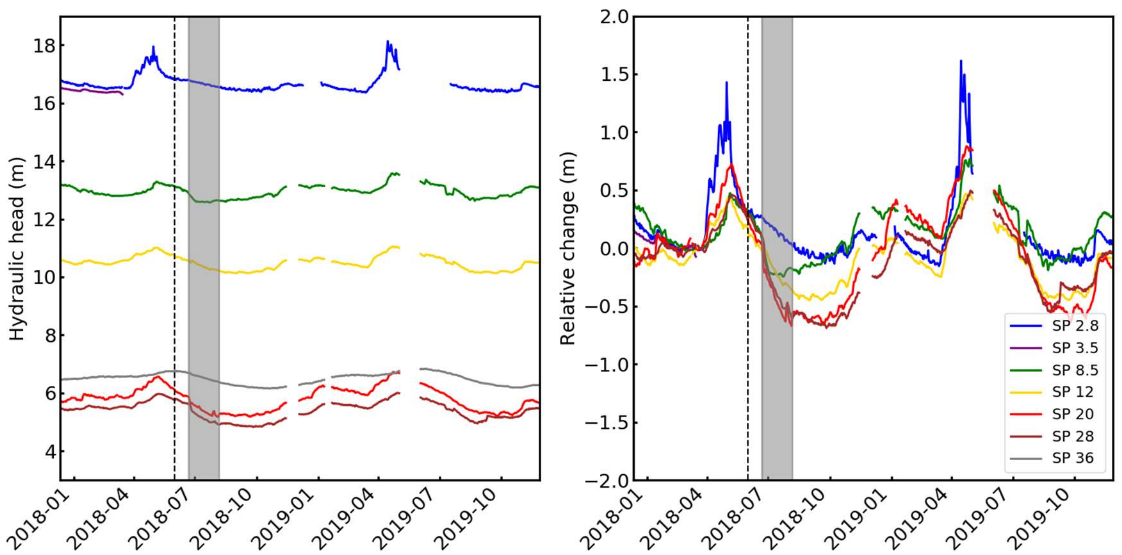

Figure 5. Hydraulic head profiles (left panel), and observed changes in hydraulic head (right panel) for 2018 and 2019 measured by the individual piezometers with sand packs. Note that the data from $SP_{3.5}$ (purple line) were omitted after April, 2018 due to a malfunction in the piezometer.

There is also an observable phase lag in the hydraulic head peaks which becomes more pronounced with depth (Figure 5, right panel). Interestingly, the amplitude of the increase for SP_{20} remains higher than that of SP_{12} and $SP_{8.5}$, indicating that vertical infiltration is not the only process influencing changes in hydraulic head within the piezometers.

4.2 Comparison of head variations in piezometers at the same depth across different installations

In all piezometers, regardless of depth or installation method, there is an increase in hydraulic head which occurs towards the end of March/beginning of April, before reaching a maximum in late April/early May (Figure 6). This increase in head corresponds with the spring snowmelt, which is the largest annual groundwater recharge event in this area. Maximum head values are followed by a recession period that continues until the fall, where slight increases in hydraulic head are observed. The effect of the pumping test is only observed in piezometers located within units Sd_L or LSC, at depths of 20m and deeper (Figure 6, third row). Similarly, the effect of the electrical storm can also be observed (e.g., MLSP at a depth of 12m; Figure 6, second row; and FGB piezometers at a depth 20 m, Figure 6, third row).

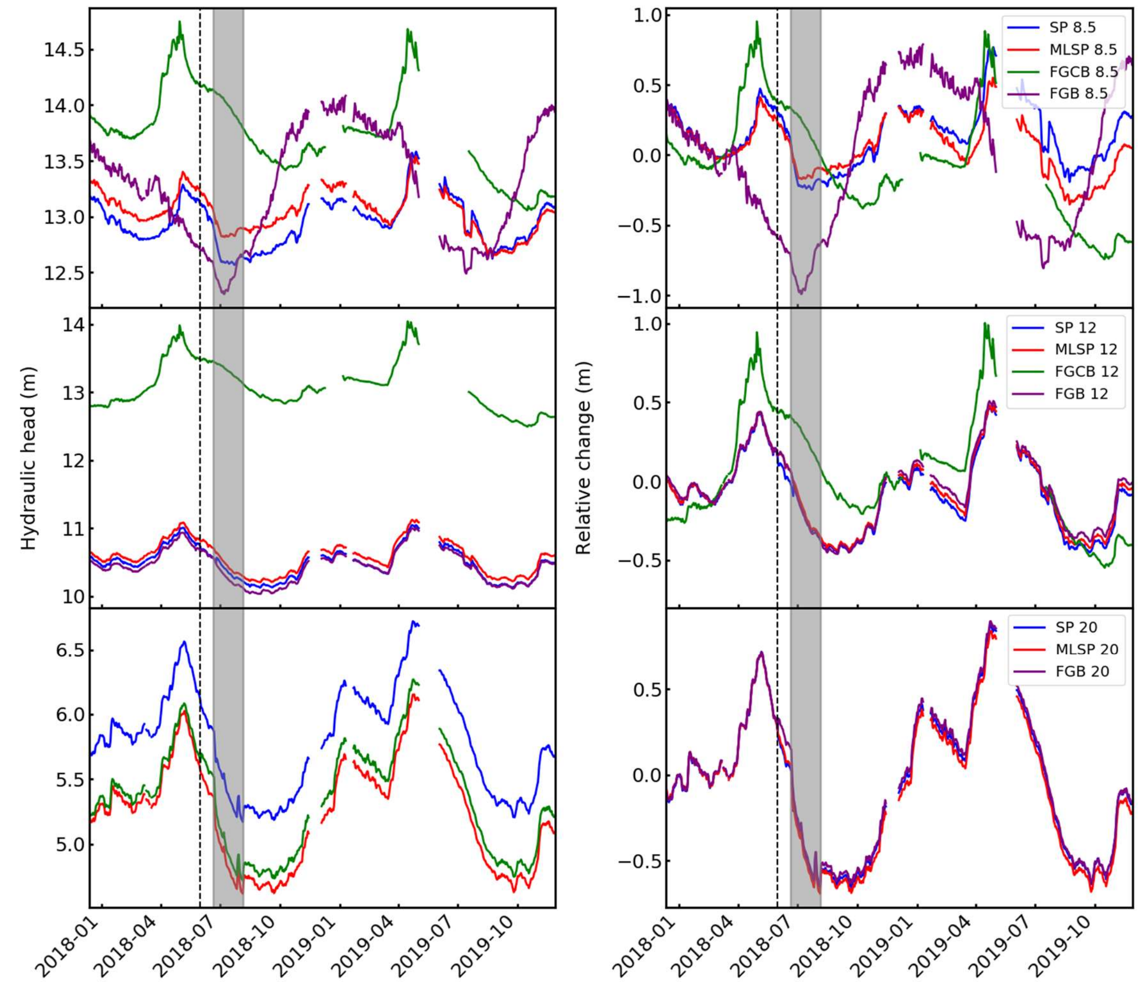

Figure 6: Hydraulic head profiles (left) and the change in hydraulic head values (right) for piezometers at depths of 8.5m, 12 m, and 20 m (first-third rows, respectively).

At 8.5 m (Figure 6, first row), the piezometers with sand packs show markedly different responses when compared with the fully-grouted methods. For instance, the spring rise in head (derived from snowmelt at the end of March) is not felt by the FGB$_{8.5}$ piezometer (Figure 6, first row, green line). Additionally, the hydraulic head values measured by the FGCB-type piezometer, are about 0.5 m higher than those observed in the sand-pack type piezometers, MLSP$_{8.5}$ and SP$_{8.5}$. Furthermore, FGCB$_{8.5}$ has a larger increase in hydraulic head in the spring when compared with all other installation methods, and the shape of the head profile is significantly different.

At 12 m (Figure 6, second row), large differences (up to 2 m for hydraulic head and 0.5 m for the change in head) are observed in the $FGCB_{12}$, while the responses are smaller in the three other arrangements. The head profiles and change in head curves are almost perfectly superimposed for FGB_{12}, $MLSP_{12}$, and the SP installation. The curves for FGB_{12} and $MLSP_{12}$ show the influence of the thunderstorm on June 1, 2018, as head variation curves between a difference of approximately 0.1 m until the grounding pins were installed four weeks later.

At 20 m (Figure 6, third row), the head profile curves have similar shapes, but the values measured differ by between 0.2 and 0.5 m across the three installation types that were present. In addition, a sudden increase in head occurs from June 2 in the FG_{20} piezometer, at the time of the thunderstorm. This increase dissipates on June 22, 2018 at the start of the pumping test. The relative head change curves for the three installations are almost identical.

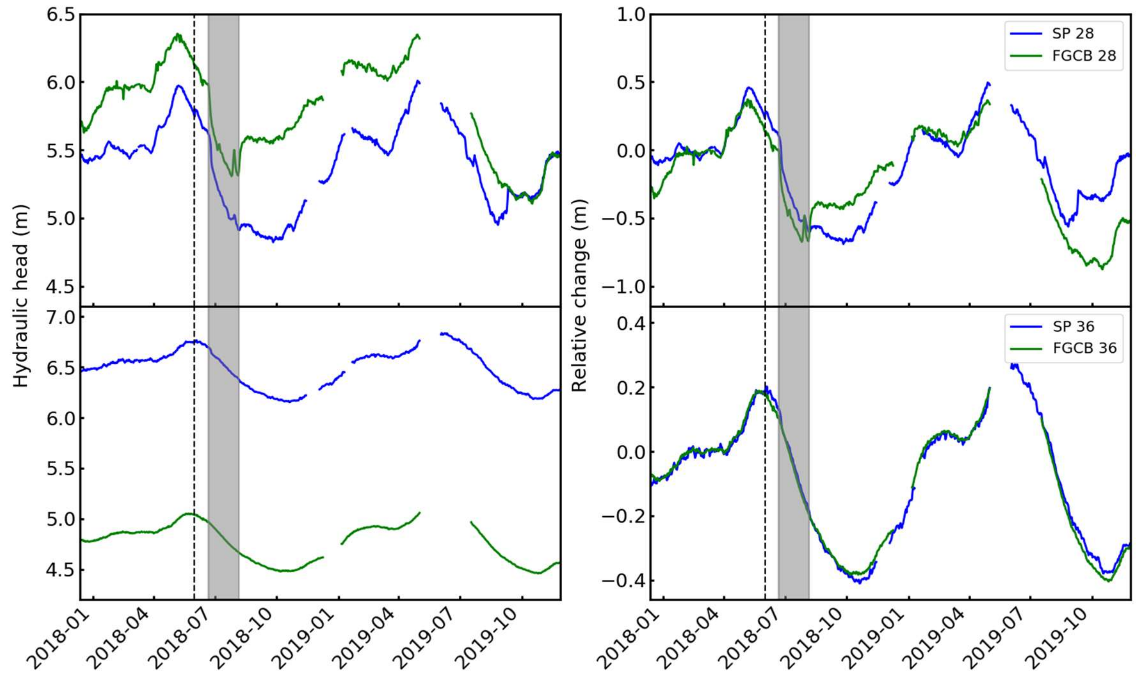

Figure 7: Barometrically-compensated hydraulic head values (left) and the change in head (right) for the piezometers at depths of 28 m and 36 m.

At 28 m (Figure 7, first row), the head profiles have a generally similar shape, but with notable differences in June 2018, likely due to the pumping test. Otherwise, the two piezometers display a difference of approximately 0.7 m throughout the period studied, despite a difference in elevation of only 0.1 m (Table 1). The curves for the relative change in head are also roughly similar, however the hydraulic head peaks of March and May reach a different magnitude and are slightly offset in time.

At 36 m, the head profiles have a very similar shape, but there is a difference in head of about 1.7 m between the two piezometers (Figure 7, second row). This is a large deviation given the difference in elevation between the two piezometer installations is about 0.1 m (Table 1). On the other hand, the curves showing the changes in hydraulic head are very similar, the head peaks have the same magnitude, and there is no apparent time lag, which means both installations react identically to the spring recharge event. A slight increase in head for the piezometer SP_{36} can be observed in the relative change curves following the electrical storm, but this variation is small, and is eventually muted by the start of the pumping test (Figure 7, right panel). Since there were no co-located FGB or MLSP piezometers below 20 m depth, the comparative performance of the deep FGCB installation was not able to be assessed.

4.3 Hydraulic head variations in piezometer nests using three different construction methods

Hydraulic head profiles for all piezometers in each type of installation were compared to both assess piezometer performance relative to the head changes in the surficial aquifer, and detect the presence of any hydraulic short circuits in a piezometer nest. Operating under the initial assumption that groundwater at the study site is exclusively vertical, it is expected that the head variations in each piezometer nest are related to the head variations in the surficial aquifer, as measured by well $SP_{2.8}$.

4.3.1 Multilevel piezometers with sand packs (MLSP)

Data from all of the piezometers within the MLSP installation show an increase in hydraulic head resulting from the spring recharge event, regardless of depth. There are, however, variations in the magnitude of the change in head to this event. The data show that the hydraulic head peaks in $MLSP_{4.35}$ are slightly higher than those observed in the surficial aquifer ($MLSP_{4.35}$ and $SP_{2.8}$; Figure 8). Since this scenario is not physically possible, the differences are quite small, and the wells are approximately co-located (~10 m apart), such results are likely the product of measurement uncertainty in either the depth or barometric measurements.

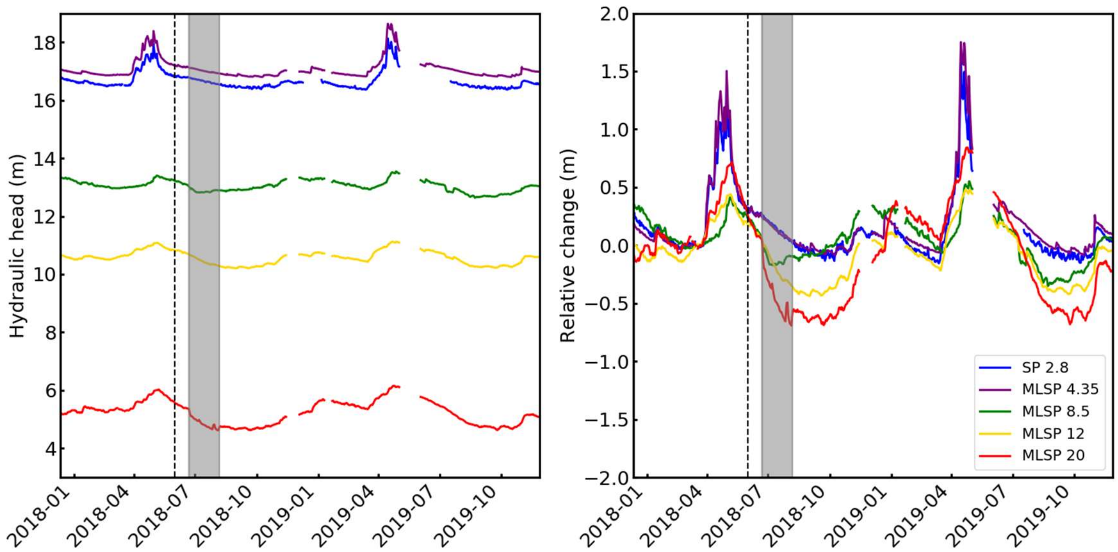

Figure 8. Hydraulic head profiles (left panel) and observed changes in hydraulic head (right panel) for 2018 and 2019 in the piezometer nest that utilized multi-level piezometers with sand packs. The observed changes in hydraulic head in the upper aquifer ($SP_{2.8}$; blue line) are included for comparison.

When the data from the MLSP installation are compared with the hydraulic heads in the surface aquifer ($SP_{2.8}$), the shape of the head profiles for the MLSP-type piezometers are similar to that of the surface water table, but with the high-frequency events filtered out. Furthermore, the amplitude of the hydraulic head changes progressively decreases with depth. The timing of the hydraulic head peak in $MLSP_{4.35}$ is synchronous with that of the surficial aquifer. For the deeper piezometers, the hydraulic head peaks are still fairly synchronous, however there is an 8-day phase lag between these piezometers and $MLSP_{4.35}$/ $SP_{2.8}$.

4.3.2 Fully-grouted piezometers with bentonite grout (FGB)

Comparing the responses of the different piezometers within a single FGB installation reveals a number of notable differences compared to the MLSP reference dataset. (Figure 9).

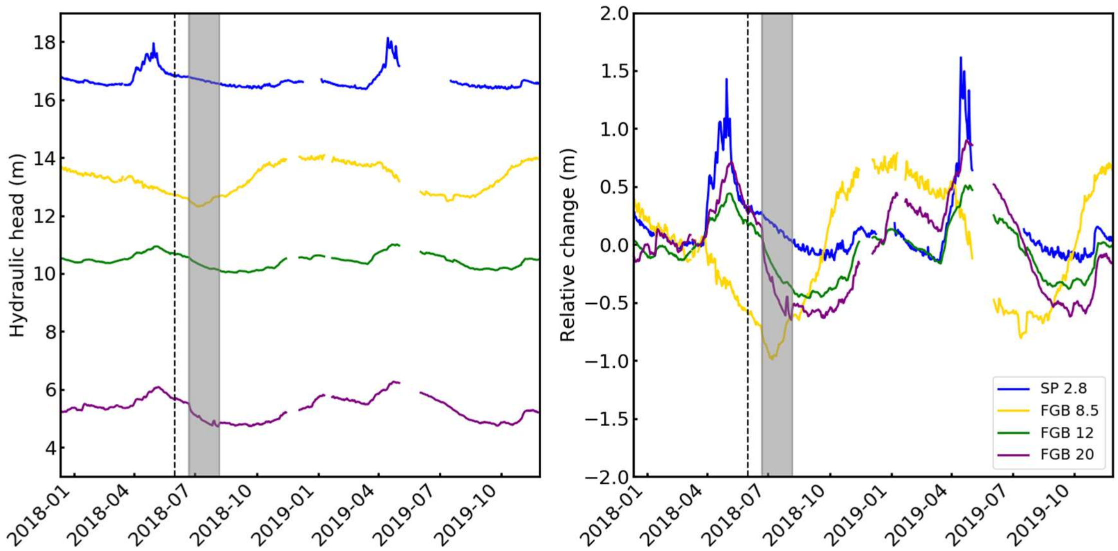

Figure 9. Hydraulic head profiles (left panel) and observed changes in hydraulic head for 2018 and 2019 (right panel) in a fully-grouted piezometer nest using bentonite grout (FGB). Hydraulic head changes in the surficial aquifer above are shown for reference ($SP_{2.8}$).

The most prominent observation is that the hydraulic head profile of the $FGB_{8.5}$ piezometer does not show any response to the spring recharge event (Figure 9, left panel, yellow line). Additionally, piezometers FGB_{12} and FGB_{20} exhibit responses similar to the head fluctuations in the surficial aquifer (Figure 9 right panel, green and purple lines, respectively), though the high frequency events are largely filtered out. There is a small (~ 3 day) time lag between the peak water level observed by the FGB piezometers and those observed by the reference well. The peak for FGB_{12} arrives approximately 3 days earlier than that observed in FGB_{20}, however, the increase in hydraulic head that occurs in response to the spring recharge event is higher for FGB_{20} than for FGB_{12}. This difference is significant, as the opposite would be expected if the increase in head was due solely to vertical water infiltration from the surface.

4.3.3 Cement-bentonite-grouted multilevel piezometers (FGCB)

There are two FGCB installations, one shallow (4.5-21 m depth) and one deep (25-36 m depth). For the shallow nest, the hydraulic head profiles of the three piezometers are located in the upper silty clay layer (unit USC; Figure 2). These piezometers, $FGCB_{4.5}$, $FGCB_{8.5}$, and $FGCB_{12}$, respectively, have shapes that are very similar to that of the piezometer located in the surficial aquifer (Figure 10, right panel). Furthermore, the amplitude of the head variations in response to the spring recharge event are similar for these three piezometers, though the effect dampens with depth. The apparent synchronization of the peak head values further demonstrates the similar behavior of these three piezometers (Figure 10, left panel).

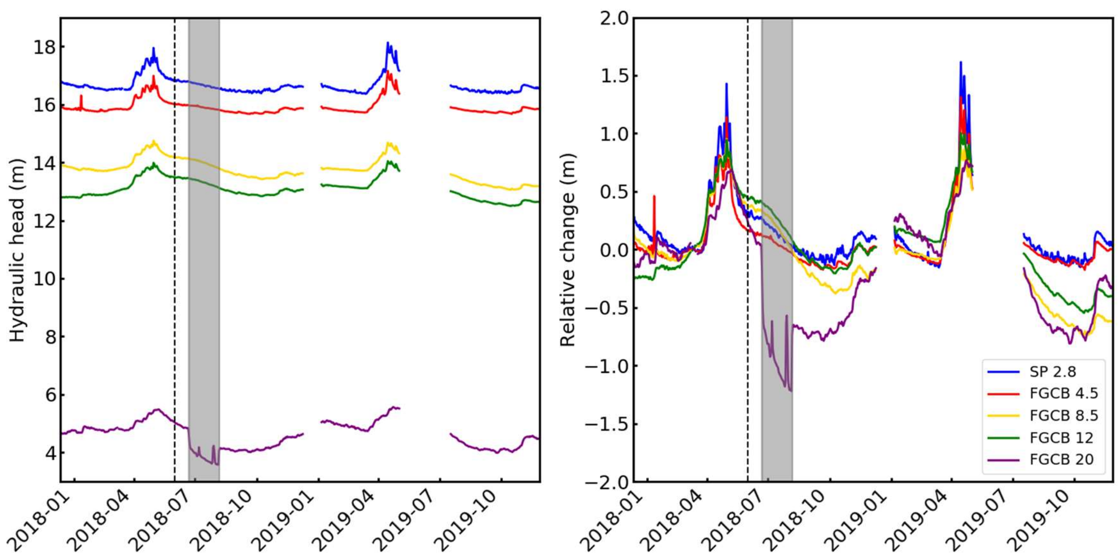

Figure 10. Hydraulic heads (left panel) and observed change in hydraulic head for 2018 and 2019 in the shallow, fully-grouted piezometers using cement-bentonite grout (FGCB). The observed changes in hydraulic head within a piezometer in the upper aquifer ($SP_{2.8}$; blue line) is included for comparison.

The behavior of piezometer $FGCB_{21}$, located in the intermediate fine sand layer (Sd_L; Figure 2) is similar to that of the aquifer in the surface aquifer, but it does not have the same peaks in the head profile as seen in $FGCB_{4.5}$ and $FGCB_{8.5}$. The amplitude and date of the head change peak are noticeably different compared to the other three piezometers, as the amplitude of the peak is smaller and it occurs out of phase with the other peaks by about 6 days. Most notably, $FGCB_{21}$ is the only piezometer that exhibits changes in head during the pumping test.

The deep FGCB nest is located within the deeper layer of clay silt (LSC) and till (T; Figure 2). Unlike the shallow FGCB nest, there is substantial time lag in the observed head peaks between the shallowest piezometer in the nest, at 25 m depth, and the deepest (36 m depth). There is also a marked attenuation of the amplitude of the head profiles (Fig. 11, left panel)

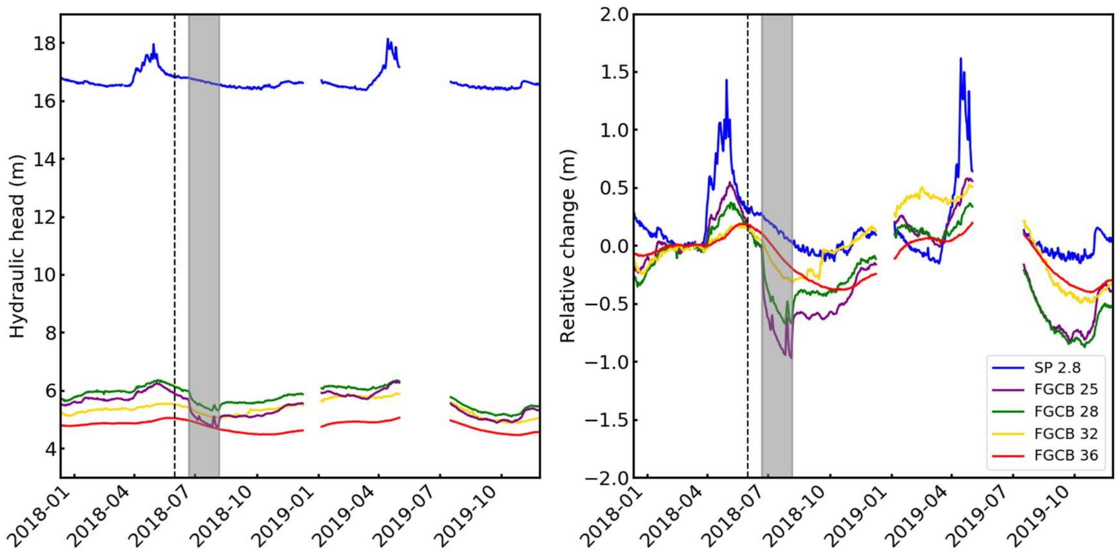

Figure 11. Hydraulic head profiles (left panel) and observed changes in hydraulic head (right panel) in the deep fully-grouted piezometer nest using cement-bentonite grout for 2018 and 2019. The observed head changes for a piezometer in the upper aquifer ($SP_{2.8}$; blue line) are included for comparison.

The head profiles also have a smoother shape with respect to the profile of the reference well. The phase lag between head peaks increases with the depth from the surface, up to a maximum of 25 days, as observed in $FGCB_{36}$ in 2018. It should also be noted that the pumping test has a marked effect on the head profiles from the piezometers at 25, 28, and 32 m.

4.3 Numerical results

Hydraulic heads were simulated at depths of 4.35, 8.5 and 12 m, for undisturbed conditions (dark lines; Figure 12, right panel), and after the installation of a fully-grouted piezometer where the grout hydraulic conductivity 500 times higher than that of the surrounding clay (pale lines; Figure 12, right panel). The modeling results are then compared with observations at the same depths made with FGCB (pale lines; left panel) and SP-type piezometers (dark lines; left panel).

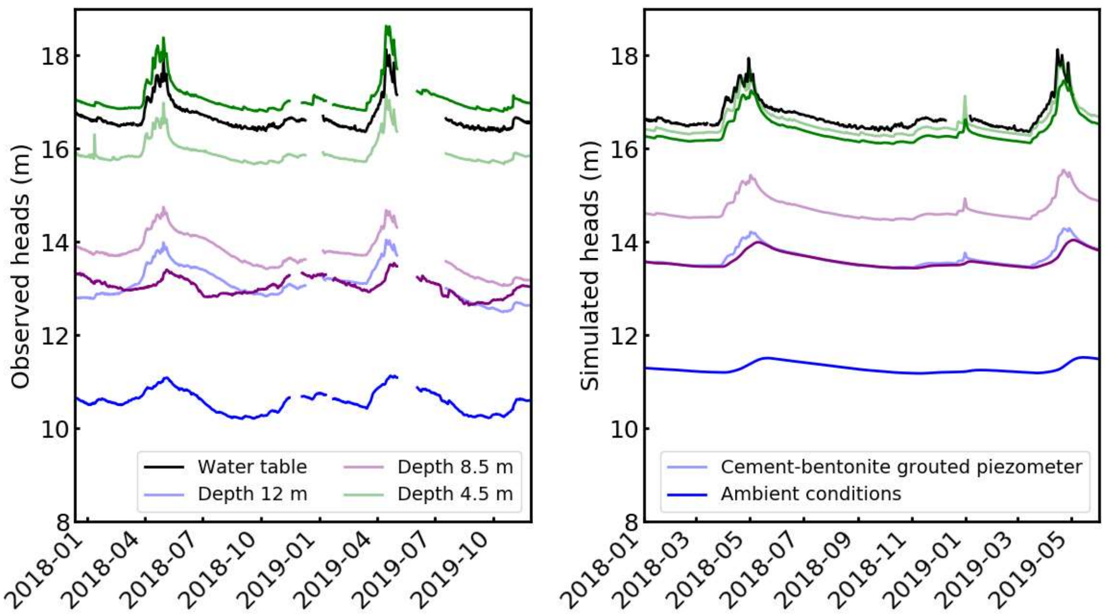

Figure 12. Observed vibrating-wire piezometer data for installations using sand packs (dark lines) and fully-grouted methods (pale lines). Simulated hydraulic heads for ambient conditions (i.e., no piezometer; dark line) and in a FGCB piezometer.

Model results show that the simulated ambient conditions are similar to the observed head profiles in the SP-type piezometers. Meanwhile, when highly permeable grout is used to simulate the effects of a hydraulic short circuit, model results strongly resemble the observed head profiles from the FGCB-type piezometers. In these profiles, high frequency variations quite visible and heads values are higher than those observed under ambient conditions. These results suggest that the SP-type piezometers located on site are functioning properly and the observed head profiles from these installations can be considered largely representative of undisturbed conditions. They also suggest that the observations from the FGCB installation correspond to a hydraulic short circuit, resulting in an effective permeability around the piezometer which is almost 500 times that of the host formation.

5 Discussion

The fully-grouted installation method has received considerable support in current geotechnical and hydraulic engineering literature (Marefat et al. 2018; Marefat et al. 2017; Smith et al. 2013; Simoni, 2012). However, there are currently no standard methods for proper installation of fully-grouted piezometers (e.g., ASTM guidelines). Furthermore, data from both piezometer nests using a form of fully-grouted installation indicate that some of the piezometers may not be functioning properly.

The majority of piezometers recorded a signal compatible with the hydrogeological context, regardless of installation method. These piezometers recorded an episode of significant spring recharge, followed by a period of summer recession. A second recharge episode occurs in the fall, which is then followed by a winter recession. There were, however, a few piezometers that exhibited obvious malfunctions when compared with other piezometers at similar depth, or when compared with expected hydraulic head/pressure curves for the hydrogeologic context. Data indicate that two specific piezometers are not functioning correctly, and that an entire FGCB piezometer installation appears to be malfunctioning.

5.1 Identifying malfunctioning piezometers

Within the FGB piezometer nest, $FGB_{8.5}$ exhibits a very different hydraulic head curve from the other piezometers in the installation (Figure 10). Furthermore, the response of this piezometer is incompatible with the hydrogeological context, as the rate of change in hydraulic head is much greater than that measured in other types of piezometers located at the same depth (Figure 6, first row).

While there was evidence that an individual piezometer within the FGB may be recording anomalous values, it appears that the entire FGCB installation is not functioning properly. As shown in Fig. 8, the head values measured at 8.5 and 12 m were 1 to 2 m higher than those measured by other piezometers at the same depth but installed with different methods. In addition, the hydraulic head curves from these two piezometers, as well as the piezometer located at 4.5m, were practically identical to that measured in the surface aquifer. These observations are indicative of a hydraulic short circuit (McKenna, 1995). However, in this string of FGCB piezometers, the deepest piezometer does not seem to exhibit the same problems, probably due to the fact that it is located in a permeable unit and that it is enclosed in a sand pack.

There are two hypotheses that could explain the hydraulic short circuit observed in the FGCB piezometer nest. The first posits that there is an issue with the permeability of the bentonite-cement grout. Based on the work of Marefat et al. (2017), the theoretical vertical hydraulic conductivity value calculated for the grout from the water-cement-bentonite proportions (water = 5 portions, c = 1 portion, b = 1.25 portion) used on the site gives approximately 6×10^{-9} m/s. This value is an order of magnitude greater than that of the vertical hydraulic conductivity of the clay layer measured in the laboratory (6×10^{-10} m/s; Germain et al. *submitted*). According to Marefat et al. (2019), hydraulic short circuits occur when the hydraulic conductivity of the grout is two orders of magnitude higher than that of the geological formation. Thus, at our field site, it is unlikely that the short circuit resulted from excessively permeable grout. However, it should be noted that the calculation of the hydraulic conductivity is an estimate based on the proportions used to prepare the grout, which are approximate.

The second explanation concerns the existence of a preferential flow path between instruments created during installation. This preferential pathway can exist along the space between the sheath cables, or it can result from a space between the grout and the formation due to excessive shrinkage during curing. While the presence of such features can indicate installation error, preferential flow paths have also been reported in studies where great care was taken to ensure proper installation (Wan and Standing 2014). Indeed, many studies suggest using a grout pipe with the VWP attached radially outside the pipe, while here, a suspended bundle was used. Depending on the grout viscosity, the space between the cables may have not been filled and cause a preferential pathway. While Marefat et al. (2018) observed grout shrinkage of samples after partially curing under atmospheric conditions, it is less likely to happen at depth where saturated conditions are present.

While the exact cause is unknown, the preferential flow path hypothesis fits the observations presented in Figure 7, where the hydraulic head curves for $FGCB_{4.5}$, $FGCB_{8.5}$, and $FGCB_{12}$ vary synchronously and with the same amplitude as the surface aquifer. Indeed, if the grout was of appropriate permeability and did not contain a preferential path, the energy loss associated with the downward flow of groundwater would result in both a time lag and reduction in amplitude of the hydraulic head peaks, like shown with the numerical model. This behavior is documented by Marefat et al. (2019), which shows that the signal recorded by a piezometer 22 m deep installed in a grout similar to the surrounding geology is out of phase by several days, even several weeks, compared to the surface signal. In addition, it shows that during a hydraulic short circuit due to the permeability of the grout, there is still a decrease in the amplitude of the signal by about 0.2 m (Marefat et al., 2019).

5.2 Suggestions for future installations in Champlain clay-type deposits

Analysis of the different piezometer nests at the Sainte-Anne-de-la-Pérade site shows that the multilevel piezometers with sand packs (MLSP) and the fully-grouted piezometer nests sealed with bentonite (FGB) installation methods are the most reliable for use in massive clay deposits. This determination is based on the fact that piezometers installed at the same depth with these arrangements recorded similar variations in hydraulic head. While some piezometers within individual installations were found to not be functioning properly, these issues were likely due to instrument problems as opposed to a fundamental issue with the installation method. Furthermore, while one string of the cement-bentonite-grouted piezometers (FGCB) appeared to function properly, the hydraulic short circuit detected in the second string (likely due to a grout-related issue) suggests that using this method introduces an unnecessary degree of risk to the long-term functionality of a given piezometer installation. The increased risk of failure largely stems from a lack of standard methods concerning both the construction of fully-grouted piezometers and the mixture of cement-bentonite grout.

While some fully-grouted piezometers used in this study worked correctly, the lack of a standard method is concerning. Consulting different studies yields a number of different installation methods, however there is not significant documentation on how these methods may influence piezometer performance and the presence of comparative studies on the topic is limited (Mickelsen and Green 2003; Contreras et al. 2012; Marefat et al. 2018). Furthermore, while there are a number of different given “recipes” and guidelines for mixing cement bentonite grout, many of these methods employ subjective or qualitative descriptors for the recommended grout consistency (Mickelson 2002; Marefat et al. 2018). Thus, because of the lack of guidance for both construction and grout preparation, using bentonite grout removes a degree of subjectivity from the construction process, thereby potentially reducing the risk of piezometer failure resulting from construction error. Furthermore, there is evidence that cement-bentonite grout may interact with some types of VWP filters, which may ultimately render this type of grout inadvisable for use in low-permeability formations (Simonsen and Sorenson, 2018).

The MLSP method performed well, and installations using sand packs have traditionally been used by the Quebec Department of Transport to monitor hydraulic heads/pore pressures within clayey slopes. There is also extensive documentation from both governmental and academic sources which demonstrate the continued successful performance of these methods (Lafleur et al., 1988; Chapuis et al. 2012; Germain et al. *Submitted*) As such, these installations can be considered adequate and should be used in the future. However, we would recommend using bentonite instead of crushed stones between the bentonite plugs. Yet the fact that the fully grouted piezometers (particularly those using cement-bentonite grout) performed less well should not be interpreted as a need to abandon the use of this installation method when building infrastructure for monitoring clay slopes across the province. The FGB-type layout has two main advantages over “conventional” installation methods: ease and speed of installation

(McKenna, 1995; Mikkelsen, 2002). Furthermore, the fact that FGB-type installations can use either one or multiple piezometers within a single borehole filled with a single material (grout) results in a cost-efficient installation method that can instrument a large area both laterally as well as vertically. However, the relative lack of guidance on the proper implementation of this technique means that it must be undertaken with considerable care, particularly if cement-bentonite grout method is used.

5.3 Continuing work and directions for future research

A series of laboratory investigations were started in the fall of 2019 in order to provide additional guidance when using cement-bentonite grout. These experiments, which are still ongoing as of April, 2020, seek to measure the hydraulic conductivity, viscosity, and shrinkage of grout mixtures when different proportions of cement and bentonite are used. While laboratory experiments can help to assess grout performance, additional piezometer installations using fully-grouted methods (using both bentonite and cement bentonite grouts) are also recommended. These additional installations can be used to better document the performance of specific fully-grouted installation methods with regards to piezometer performance and the rate of installation-related malfunctions. Additionally, a long-term monitoring study is recommended to compare the performance of FGB-type piezometers with sand-pack-type piezometers over extended time periods.

6 Conclusion

This study assessed the performance of different methods for installing multilevel vibrating wire piezometers at a field site in the municipality of Sainte-Anne-de-la-Pérade that possessed a complex stratigraphy. Comparison of the three different installation methods showed the use of sand packs or bentonite grout resulted in similar performance. Of the two fully-grouted installations using cement-bentonite grout, one installation failed completely due to a hydraulic short circuit. This short circuit is likely the result of either shrinkage of the grout, or flow occurring along the wires of the instruments.

While the bentonite-grouted piezometers used in this study worked correctly, the absence of a standard method for installation is concerning. A review of the literature provides a number of different installation methods, as well as a number of different recipes for mixing cement-bentonite grout—many of which contained a number of subjective or qualitative descriptions. Thus, due to the lack of guidance for both construction and grout preparation, the use of a bentonite grout will likely remove a degree of uncertainty when fully-grouted installation techniques are used. However, the fact that the fully grouted piezometers using cement-bentonite grout performed less well does not mean that all fully-grouted methods should be avoided. The relative ease, speed of installation, cost efficiency of these types of piezometers means that they can be used to instrument a large area, both laterally as well as vertically.

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