Field performance of fully grouted piezometers

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1 Introduction

In the 19th century the Adige River (Northern Italy) was straightened to prevent flooding and to allow the building of the Verona-Brenner railway by the construction of embankments. At present the river flows between embankments for about 140 km and occasionally collapses have occurred, with the most dangerous in 1981 when the village of Salorno was mostly underwater. Since January 1st, 2000 the Autonomous Province of Bozen/Bolzano - South Tyrol has been responsible for the management of the Adige River located in the region. For this purpose it started to install piezometers to monitor the pore water pressures in the embankments and in the soil foundations.

The valley of the Adige River is made of alluvial soils of lacustrine or fluvial origin. Fine sand is the major class of soil, but coarser soil is found in the present and in the ancient river bed. Because of the great soil heterogeneity at any cross-section the back-analysis of the pore water pressure distribution has resulted the most efficient method for the comprehension of the stability conditions of the river embankments. Moreover, the piezometer measurements have been used for controlling the efficiency of the remedial works. To date more than 100 piezometers have been installed at the top, mid and toe of the embankments. One fifth of these uses sensors with automatic readings.

In order to test an easier and faster installation, laboratory and field tests were carried out to investigate the performance of fully grouted piezometers. In particular, two nested fully-grouted piezometers were installed in a river embankment and data were compared to adjacent conventional installed piezometers with sand pack and bentonite seal.

Even though Vaughan first proposed in 1969 the use of piezometers without the sand pack, only recently McKenna (1995), Mikkelsen et al. (2003) and Contreras et al. (2007) firmly suggested to use piezometers in fully grouted boreholes. The omission of the sand

packs makes the installation easier and faster, and prevents piezometer failures during the placement of the sand pack and the overlying bentonite seal.

2 Laboratory testing

The laboratory testing was aimed at investigating the role played by the grout in controlling the response time of fully-grouted diaphragm piezometers (Simeoni, submitted). Cylindrical samples of cement-bentonite mixture were investigated.

2.1 Samples

The samples were of cement-bentonite mixture with water - cement - bentonite ratio by weight of 2,5 - 1 - 0,3. All the samples had a diameter of 7 cm, but different heights of 2 cm, 4 cm and 8 cm. According to Mikkelsen (2002) the grout was prepared by mixing the cement with the water first, and then adding the bentonite because this procedure makes the stiffness of the grout more stable. The grout was tremied to stainless steel moulds and left to rest for a curing time of no less than 28 days. Three series of samples were investigated (Table 1), paying attention that sample preparation and curing were carried out in deaired and distilled water in order to reduce the presence of air in the sample.

Table 1. Investigated samples.

Sample	S2-01	S4-01	S8-01	S2-02	S4-02	S8-02	S4-03	S8-03
Height [cm]	2	4	8	2	4	8	4	8
Series	1	1	1	2	2	2	3	3
Curing time [d]	43	30	37	43	36	38	50	48
Pressure changes	18	6	7	18	18	18	28	28

2.2 Testing apparatus

The testing apparatus was similar to a triaxial cell (Figure 1). The cylindrical surface of each sample was covered by a rubber membrane sealed by rubber O-rings, while at the ends porous disks were placed to apply a uniform vertical seepage. The disk placed at the bottom was connected to a pore water pressure control system with tubing when flow was permitted or directly to a pore pressure transducer when drainage was not allowed. The

disk placed at the top was connected to a pore water pressure transducer by two polythene tubes. During the placement of the porous disks, the tubes and the transducers special attention was taken to avoid air being trapping by flushing the porous disks and making connections with flowing deaired and distilled water. The pore water pressure transducers had diaphragm electrical sensors.
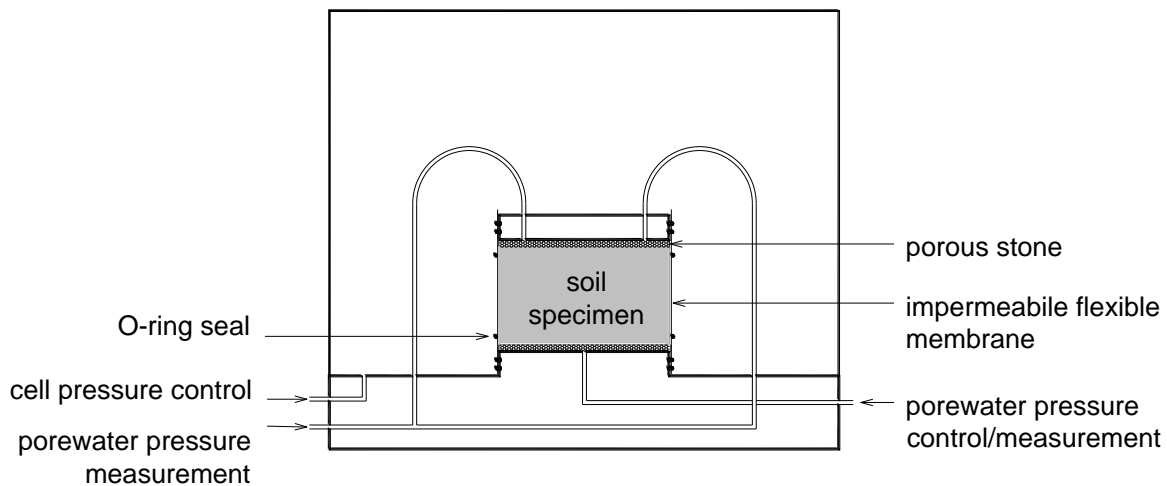

Figure 1. Sketch of the testing apparatus.

2.3 Testing procedure

Initially the samples were subjected to a uniform all-around cell pressure of 150 kPa, and the two porous disks flushed repeatedly with deaired and distilled water. The samples were then consolidated by applying a back pressure of 20 kPa and increasing the cell pressure up to 600 kPa. Consequently in the steps that follow the samples behaved as overconsolidated and their volume change could be assumed negligible or very small. When consolidation was completed the pore water pressure at the bottom of the sample was incrementally increased up to 500 kPa with increments ranging between 10 kPa and 200 kPa. A total of 141 changes in pore water pressure at the bottom of 8 different samples was applied (Table 1).

2.4 Test results

A typical water pressure response at the top of the sample for an imposed water pressure increment at the bottom is shown in Figure 2 for a 8 cm height sample and an increment of 100 kPa.
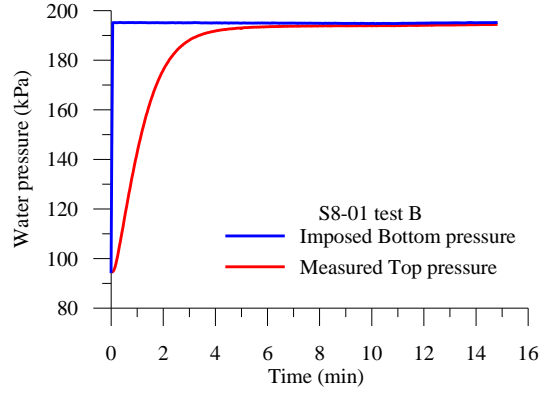
Figure 2. Water pressure response for an imposed increment of 100 kPa.

A response time of about 6 minutes was observed at the top of the sample, while the pressure at the bottom reached a constant value in about 10 seconds. The latter was the time required to equilibrate the diaphragm electrical sensor. It was significantly shorter than the time recorded at the top because there was not delayed due to the seepage through the grout.

To compare the response time for different pressure changes and different heights, the degree of excess water pressure at the top of the sample was defined as

$$U_{top,t} = 1 - \frac{u_{etop,t}}{u_{etop,0}}$$
(1)

where $u_{etop,t}$ is the excess of water pressure at the top of the sample at time t, and $u_{etop,0}$ is the initial excess of water pressure. The response time t_{95} was accordingly defined as the time to dissipate the 95% of the excess water pressure, corresponding to a degree of excess water pressure at the top U_{top} equal to 0.95.

Figure 3 shows the relation between the response time t_{95} versus the average value between the initial and final water pressure at the top of the sample. Closed and open symbols refer to "uw" and "dw" tests. Tests "uw" refer to increments of water pressure causing an upward flow, tests "dw" refer to decrements of water pressure causing a downward flow. t_{95} clearly increases with the height of the sample and decreases with the increasing average water pressure.
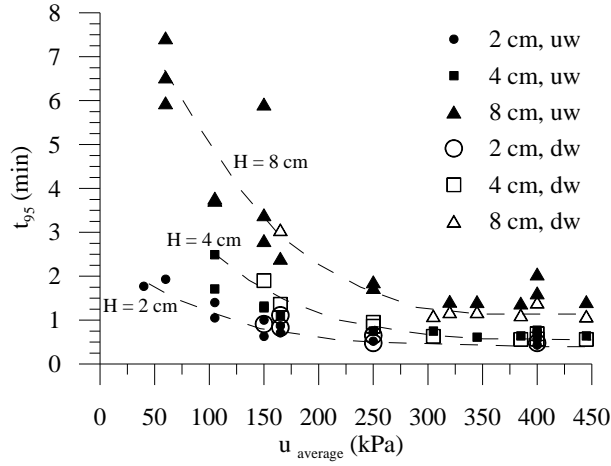
Figure 3. t_{95} versus the average value of water pressure at the top of the sample. Closed symbols: upward flow (uw); open symbols: downward flow (dw).

If the response time was simply depending on the equalization volume of the transducer diaphragm (McKenna, 1995), the response time would change only with the sample height. Therefore, the dependence on the water pressure should be explained by assuming the soil is quasi-saturated, where the assumptions of incompressibility and saturation of the soil could not be valid, and more correctly the equation of continuity for the one-dimensional flow should be written taking the volumetric water content into account.

By introducing the Darcy's law, the equation of continuity may be written as:

$$\frac{\partial}{\partial z}\left(-k_g \frac{\partial}{\partial z}\left(\frac{u_e}{\gamma_w} + z\right)\right) = -\frac{\partial \vartheta}{\partial u_e}\frac{\partial u_e}{\partial t} \tag{2}$$

where u_e is the excess water pressure at time t and position z, $k_g = k_g(\vartheta)$ is the hydraulic conductivity at the volumetric water content ϑ. If for the sake of simplicity both k_g and $\frac{\partial \vartheta}{\partial u_e}$ are assumed constant for a given range of excess water pressure u_e, equation (2) reduces to the equation:

$$\frac{k_g}{\gamma_w \cdot \dfrac{\Delta \vartheta}{\Delta u_e}} \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t} \tag{3}$$

This equation is similar to the well-known Terzaghi's equation for one-dimensional consolidation (Terzaghi, 1923), and the solution relies on the value of the coefficient of consolidation c_v:

$$c_v = \frac{k_g}{\gamma_w \cdot \dfrac{\Delta \vartheta}{\Delta u_e}} \tag{4}$$

In Figure 4 the best fitting of the experimental data with the one-dimensional consolidation model is shown for a sample 3 cm height subjected to a water pressure increase of 100 kPa. A very good agreement was obtained for a c_v value of 27 cm^2/min.
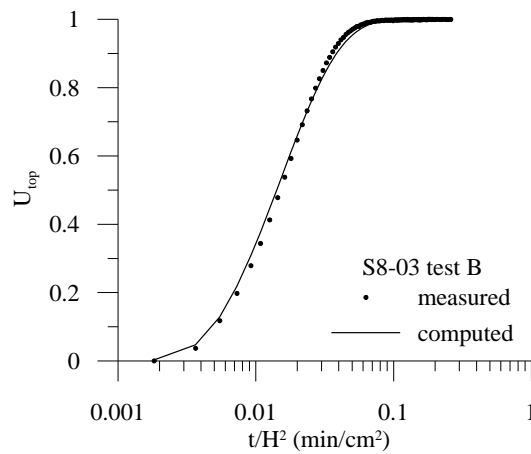

Figure 4. Measured and computed degree of U_{top} for an upward flow test on a 3 cm height sample.

The values of c_v estimated for upward and downward flow tests carried out on samples 2, 4 and 8 cm height ranged between 2.5 cm^2/min to 86.3 cm^2/min, with higher values for higher water pressures.

According to equations (3) and (4) the response time may be evaluated as:

$$t_{95} = \frac{1.129 \cdot H^2}{c_v} \tag{5}$$

3 Field testing

Successful field tests of fully-grouted piezometers have already been presented in McKenna (1995) and Contreras et al. (2007). McKenna (1995) compared the hydraulic heads calculated from pneumatic piezometers installed with the traditional sand pack method or with the fully-grouted method. Because the piezometers were generally installed in low-permeable soils and days elapsed between measurements, the data were used mainly to investigate the installation procedures or the ability of the fully-grouted piezometers to measures the actual value of pore water pressure. Then, the field tests did not investigate the response time of the fully-grouted piezometers. Contreras et al. (2007) demonstrated the efficiency of the fully-grouted method with the installation of nested vibrating-wire piezometers to describe upward or downward flow conditions. Again, the Authors did not investigate the response time of the fully-grouted piezometers.

The field test described in this paper was aimed at investigating if the response time of fully-grouted piezometers was comparable to that of traditional sand pack piezometers. In particular, the response time should be short enough to measure the pore water pressure changes in the embankment due to the fluctuation of the river level. As suggested by Contreras et al. (2008), the trial application used both the fully-grouted method and the traditional sand pack method in adjacent boreholes.

3.1. Location and soil profile

The piezometers were installed in the right-side embankment of the Adige River at Egna, 20 km South of Bolzano-Bozen (Northern Italy) and nearly 300 m South of the section C monitored with piezopress piezometers (Simeoni et al., 2011).

The soil profile is shown in Figure 5. The embankment (Unit A) is a gravel with silty sand, which overlain a fluvial-lacustrine deposit (Unit C). The upper ground C1 is a sandy silt with an horizontal hydraulic conductivity k_h equal to 10^{-5} m/s; the deeper ground C2 is a silty sand with gravel with $k_h = 10^{-4}$ m/s. The upper piezometers were installed in C1, the lower piezometers in C2. Grain size distributions of ground C1 and C2 are shown in Figure 6. Ground C1 clearly appears more heterogeneous than ground C2. In fact, narrow levels of sand could be identified alternatively to levels of clayey silt.
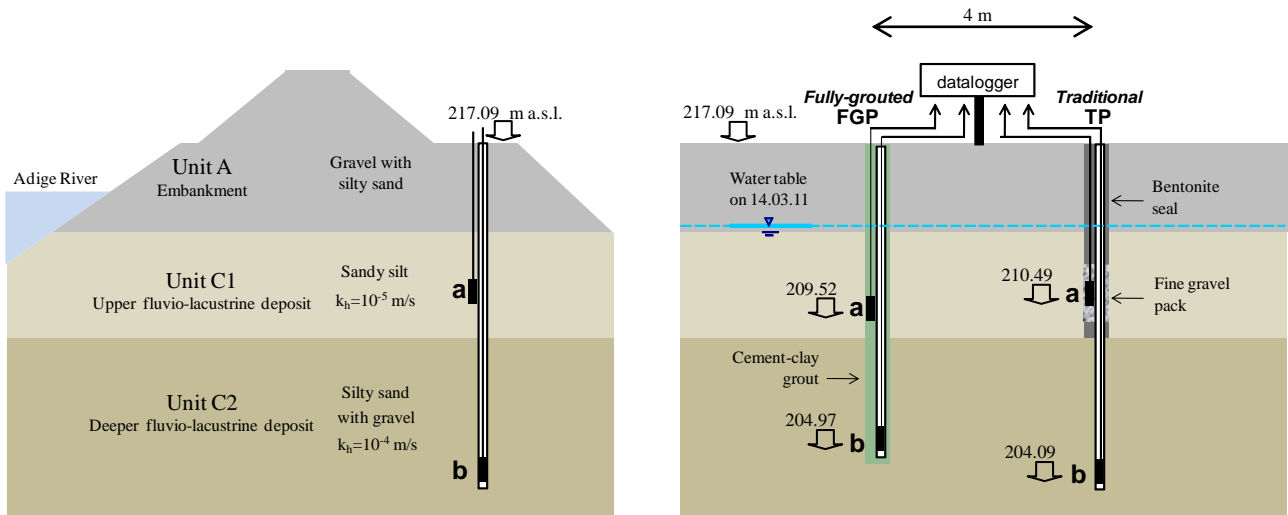
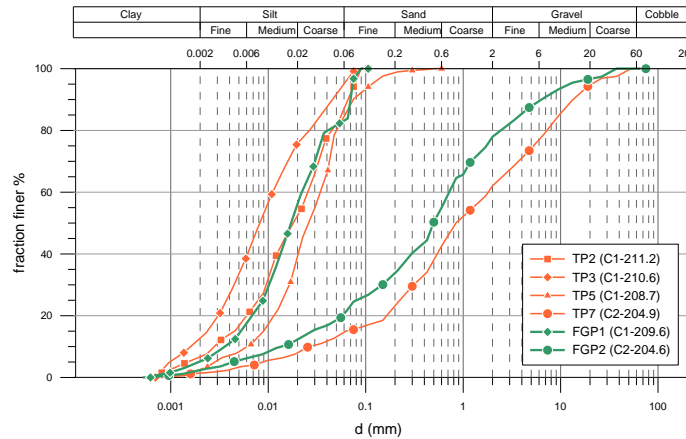Figure 5. Soil profile and piezometers fully-grouted FGP and traditional TP.

Figure 6. Grain size distributions of ground C1 and ground C2.

3.2. Instrumentation and installation procedure

Two 15 m boreholes were drilled at the beginning of March 2011, within 4 m of each other, using a 140 mm diameter drill casing. Nested piezometers with two measurement points were installed in each borehole. The upper pore water pressure transducers "a" were grouted or installed in the sand pack; while the deeper transducers "b" were pushed in a Casagrande filter of high density polyethylene tube, at the bottom of 1.5" blind pipes (Figure 7). The latter installation method allows the transducer recovering if it fails or if calibration needs to be checked.

The upper pore water pressure transducers were resistive, with HAE value ceramic filter (pore size 0.25 μm) and a full-scale reading range of 100 kPa (type Sisgeo P235S1). The filters, already saturated by the manufacture, were screwed on the transducer under water

in a bucket (Figure 8). Then the transducers were fastened upside down to their cables, to minimize the possibility of desaturation.

The deeper transducers were resistive, with sinterised filter (pore size 40 μm) and a full-scale reading range of 200 kPa (type Sisgeo P252C). The transducer tips were conical with o-rings to prevent any leakage when pushed in the filter units.

Both type of transducers were absolute (not vented). The grout was a cement-clay mix, with water - cement - bentonite ratio by weight of 2,5 - 1 - 0,45. According to EN-197 the cement was type Portland 32,5 R, CEM II/A-L. The clay was composed for 60% kaolin, 37% illite, 3% smecite, it also included a small percentage of silt. The cement-clay grout was used in error instead of cement-bentonite grout. However it had the advantage that particles were initially suspended and settled without entrapping air bubbles (fully saturated grout).

Figure 7. Deeper piezometer pushed in the filter (Sisgeo, 2011).

The disadvantage was a 30% of bleed water, that was taken into account in the mixing process. The transducers "a" were fastened to the standpipes while they were lowering into the boreholes (Figure 9).
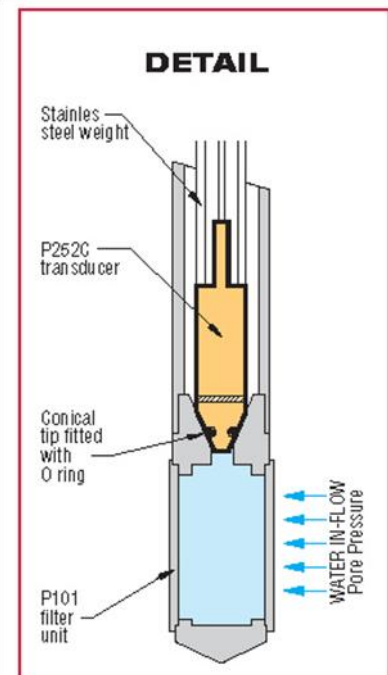

Figure 8. Pore water transducer.

Figure 9. Transducer fastened upside-down to the standpipe.

With the fully-grouted method (piezometer FGP) the grout was pumped bottom up into the borehole with a tremie pipe withdrawn as grouting proceeded. Once the grout reached the

top, the drill casing was retrieved and the upper part of the borehole filled with bentonite pellets. The use of the standpipe allowed the exact knowledge of the piezometer depths (Figure 5).

The conventionally installed piezometer (traditional piezometer TP) used fine gravel (2-6 mm) packs and bentonite pellets. For the deeper piezometer "b" the fine gravel pack was omitted because the borehole in the ground C2 self-closed while retrieving the drill casing. The gravel pack of piezometer TPa was 1.25 m height.

Eventually, the transducers "b" were lowered into the standpipes by using their electrical cables and pushed in the filter units. The transducers have been kept fixed by stainless steel weights coaxial to the cables.

Even though care was taken during installation, differences up to 1 m in the elevation of corresponding piezometers were obtained. Consequently, differences in hydraulic head were expected.

3.3. Grout hydraulic conductivity

The hydraulic conductivity of the grout was measured in the laboratory by a falling head test. In order to prevent any seepage between the specimen and the wall of the test mould, the wall of the test mould was lined with silicon grease before treming the grout. Moreover, the sample was incrementally loaded to a vertical pressure of 100 kPa before the grout set-up.

Values of hydraulic conductivity are shown in Figure 10. They decreased with time until reached a constant value of 1.3×10^{-10} m/s after 15 days.
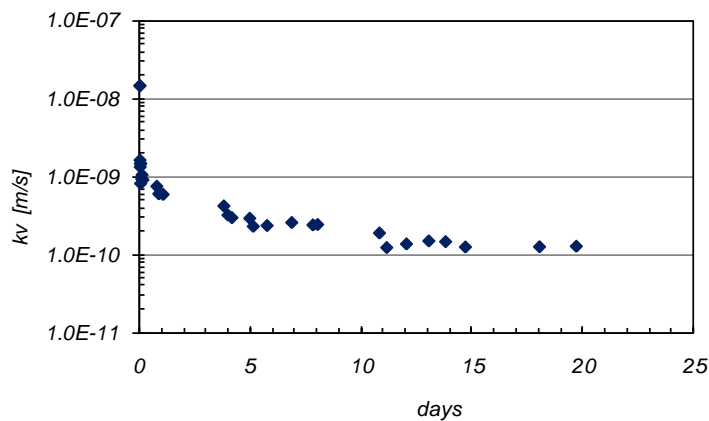

Figure 10. Grout hydraulic conductivity.

3.4. Test results

Hydraulic head at the four piezometers are shown in Figure 11 and Figure 12.

Figure 11 shows the hydraulic head calculated two months after the installation, from May 19 to May 31, 2011. The Adige River level is also reported. It is seen that hydraulic heads at the upper piezometers, installed in ground C1 (sandy silt), were slightly different, but both of them started increasing at the same time when the river rose. The deeper piezometers, installed in ground C2 (silty sand with gravel), practically exhibited parallel values of hydraulic head, revealing a similar response time. It is worth noting that on May 22 the upper traditional piezometer TPa provided a decrease of the hydraulic head similar to those measured at the deeper piezometers. These decreases should be due to water extraction for irrigating the orchards (Simeoni et al. 2011) and could not influence the upper and lower permeable ground C1. If this was the case, it may be supposed that on May 22 the bentonite seal between piezometer TPa and piezometer TPb had not already set-up and the two piezometers were still influenced to each other. In fact, because the elevation of piezometer TPb is lower than elevation of FGPb, and the seepage should have a downward component from the river, it was expected that hydraulic head at TPb would be lower than at FGPb. On the contrary, in Figure 11 hydraulic head at TPb was higher than that at FGPb and a connection with TPa was suspected.

a)   b)
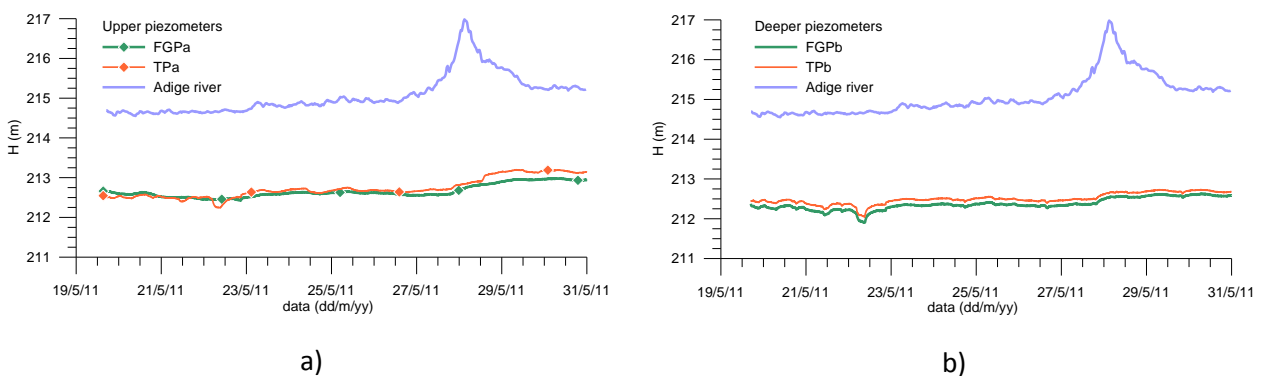
Figure 11. Hydraulic head and river level from May 19 to May 31, 2011.

Figure 12 shows the hydraulic head calculated from June 10 to July 11, 2011. The hydraulic head at TPb gradually became lower than that at FGPb, and the hydraulic head at TPa was always higher than at FGPa as expected from their elevations and seepage direction. The deeper piezometers, installed with the Casagrande filter and without the gravel pack, provided very similar pore water pressure changes, while the upper

piezometers, the fully grouted FGPa and the traditional with the 1.25 m gravel pack TPa, showed some differences, especially during the increase on June 19 and on June 24. It is worth noting that both fully-grouted and traditional piezometers were able to measure the daily fluctuation of the river, due to the discharge from power stations.
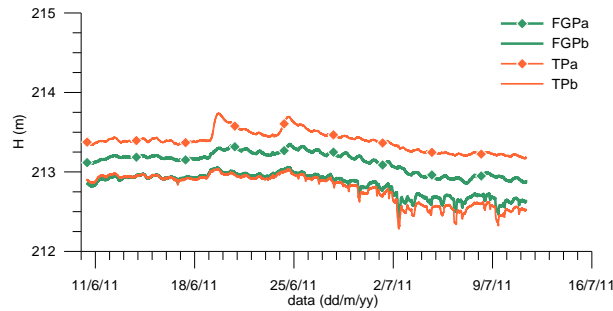

Figure 12. Hydraulic head from June 10 to July 11, 2011.

Differences in hydraulic head from fully-grouted piezometers and traditional piezometers are shown in Figure 13 from July 4 to July 11, 2011. Differences are due to different elevations, to the size of the gravel pack and to different response times. It is seen that the upper piezometers exhibited higher changes in the differences of the hydraulic head, revealing either different response times or an influence of the size of the gravel pack. According to equation (5) the fully grouted response time varies with the square of the drainage distance, then piezometer FGPa should mainly measure the pore water pressure applied closer to their diaphragm. On the contrary, the traditional piezometer measured an average value of the pore water pressure applied on the border of the 1.25 m gravel pack.
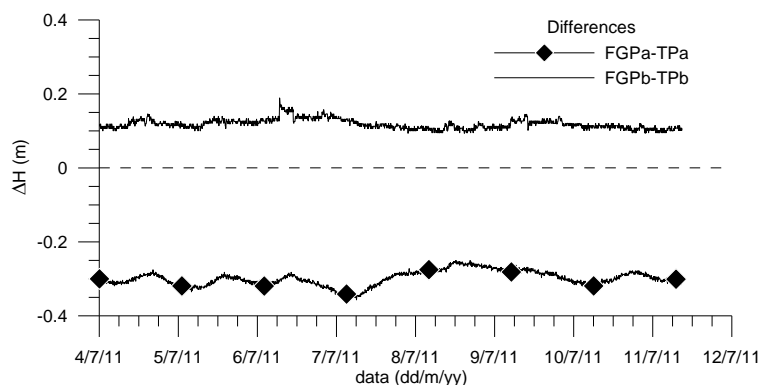

Figure 13. Hydraulic head differences from July 4 to July 11, 2011.

4 Conclusions and perspectives

Laboratory and field testing confirmed that fully grouted piezometers have a very good conformance in terms of response time. Laboratory tests on cement-bentonite samples provided response times of the order of some minutes. They could be evaluated with coefficients of consolidation c_v between 2.5 cm^2/min to 86.3 cm^2/min, with higher values for higher water pressures since the mixture resulted quasi-saturated. Different values of c_v should be evaluated for different mixture, degree of saturation and void ratio. According to the one-dimensional theory of consolidation, response time increased with the square of the drainage distance.

Field testing with two fully-grouted piezometers and two traditional piezometers installed in a nested configuration, revealed that both methods are able to provide reliable measurements of pore water pressure. Response time of fully grouted piezometers was similar to that of traditional piezometers. In fact, differences in piezometric head less than 0.1 kPa were evaluated. They could be referred to different response times or to the effect of the gravel pack size in the traditional piezometer.

The differences in hydraulic head were bigger at the beginning of the monitoring, two months after the piezometer installation, probably because there was hydraulic connectivity between the upper and deeper piezometers through the bentonite seal that had not already set-up.

The fully-grouted method is easier and faster than the sand-pack method, as well as it is more likely to succeed in measuring the correct pore water pressure, specially for deep or nested piezometers. A major difficulty in the traditional piezometers is the control of depths during the pouring of sand or bentonite pellets or chips, with the consequence that the measurement points are not in the expected position or sand packs allow the transmission of water between soil layers. In the fully-grouted method the position of piezometers is a-priori well defined, when the transducers are fastened to a sacrificial grout pipe, to a blind pipe of the deeper piezometer or to a fiberglass rod. In nested piezometers attention should be taken in separating the cables around the pipe in order to prevent any seepage path along a bundle of cables. Contreras et al. (2008) stated that they have successfully installed up to four piezometers tips in a borehole. In this paper, a blind pipe with a Casagrande filter at the bottom was effectively used to carry the upper piezometer. It introduced the further advantage of using a pore water pressure transducer with a large pore sinterised filter and therefore of preventing the difficulties in saturating the filter.

On the contrary, the fully-grouted method should be avoided when the average value of pore water pressure in an interval depth is needed. For examples in slightly fissured rocks, where the fully grouted piezometer could be installed close to the massive rock instead of a fissure and it does not provide any reliable value.

Acknowledgements

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