Geotechnical Instrumentation NEWS

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John Dunnicliff

Introduction

This is the fifty-fifth episode of GIN. Two subjects this time, both based on papers that were presented at the International Symposium on Field Measurements in Geomechanics (FMGM) in Boston in September last year. I selected these for re-publication in GIN because I want to maximize the audience for two very important contributions. There is also a review of a book about fibre optic sensing.

Those of you who have already read the FMGM paper on fully-grouted piezometers by Contreras et al—please replace it with this version, in which several issues have been corrected.

The Use of the Fully-grouted Method for Piezometer Installation

The first article, in two parts, is by Iván Contreras, Aaron Grosser, and Richard Ver Strate of Barr Engineering Company in Minneapolis. I've been waiting for this for 39 years, ever since I read Peter Vaughan's 1969 technical note in Géotechnique, "A Note on Sealing Piezometers in Boreholes." That may sound flippant, but it's true!

In 1949 the Journal of the Boston Society of Civil Engineers published Arthur Casagrande's paper, "Soil Mechanics in the Design and Construction of Logan Airport". He described the installation of open standpipe piezometers ("Casagrande piezometers") in boreholes by surrounding them with a sand pack and placing bentonite pellets over the sand. The drill casing was left in place and the bentonite seal was within the casing, so no grout was placed over the bentonite. A few years later it became normal practice to withdraw the drill casing and place grout over bentonite pellets or chips, and we're still doing this for piezometers installed in boreholes.

For **open standpipe** piezometers the sand pack is necessary because a sizable intake volume is required for obtaining a pore water pressure reading without significant time lag. So this "normal practice" is still appropriate today, except that in my view the grout should be placed directly over the sand, omitting the bentonite pellets or chips—I explain this in my discussion of the article by Contreras et al.

Since the development of **diaphragm piezometers**, usually pneumatic or vibrating wire, most of us have followed this same "normal practice", with a sand pack, bentonite pellets or chips, and overlying grout. **Forget the sand and bentonite seal! This is no longer the way to go! Use the fully-grouted method!** The fully-grouted method entails installing a piezometer tip in a borehole which is backfilled entirely with cement-bentonite grout. It's taken several years of discussion and argument for me to arrive at this conclusion because I feared that the grout surrounding the tip might prevent the piezometer from responding correctly to changes in pore water pressure. If you have the same fears, read the article, the discussion and the authors' reply to the discussion, and become a believer!

Forget the sand and bentonite seal! This is no longer the way to go! Use the fully-grouted method!

If any reader has other data, pro or con, about the fully-grouted method, I'd very much welcome hearing about it, and will consider it for publication in a later episode of GIN.

Geotechnical Alarm Systems

The second article is by Kevin O'Connor, and focuses on alarm systems based on the technology of time domain reflectometry (TDR).

My reason for regarding this as a "very important contribution" is because the message is an 'umbrella' one, applicable to **all** alarm systems, not only those based on monitoring with TDR.

I have very clear memories of Kevin

laying down some ground rules both at the 2007 instrumentation course in Florida and during his presentation of this subject at the FMGM symposium in Boston. He talked about responding to alarms and said, with enormous emphasis, "**If there is an alarm, you <u>have</u> to respond. Failure to respond is <u>not</u> an option**".

Closure

Please send contributions to this column, or an article for GIN, to me as an e-mail attachment in MSWord, to john@dunnicliff.eclipse.co.uk, or by fax or mail: Little Leat, Whisselwell, Bovey Tracey, Devon TQ13 9LA, England. Tel. and fax +44-1626-832919.

Instrument sent to me from Australia by Craig Johnson: "I was in an antique store on the weekend, saw this artifact (circa late 20th century) and thought of you. Enjoy!" Thank you Craig.

The Use of the Fully-grouted Method for Piezometer Installation Part 1

Iván A. Contreras
Aaron T. Grosser
Richard H. Ver Strate

Introduction

The fully-grouted method described in this article entails installing a piezometer tip in a borehole which is backfilled entirely with cement-bentonite grout. Part 1 of this article presents a detailed discussion of the fully-grouted method, including the installation procedure and theoretical background, as well as a seepage-model analysis used to evaluate the impact of the difference in permeabilities between surrounding ground and cement-bentonite grout. Part 2 describes laboratory test results for six cement-bentonite grout mixes and field examples of applications of the fully-grouted method. Both parts of this article are based on the paper, "The Use of the Fully-grouted Method for Piezometer Installation," presented at FMGM 2007: Seventh International Symposium on Field Measurements in Geomechanics, and are published in GIN with permission from ASCE.

A crucial parameter for the success of the fully-grouted method is the permeability of the cement-bentonite grout. Vaughan (1969) postulated that the cement-bentonite grout should have a permeability no greater than two orders of magnitude higher than the surrounding soil in order to obtain representative pore-water pressure readings. Unfortunately, there is limited published data on the permeability of cement-bentonite grout mixes.

Figure 1a shows the typical piezometer installation commonly known as a Casagrande or standpipe piezometer. With this installation, the tip of the piezometer (e.g., slotted PVC pipe or porous stone filter) is surrounded with a high permeability material, commonly referred to as sand pack. Above the sand pack is a bentonite seal typically consisting of bentonite chips or pellets. The installation is finished with cement-bentonite grout to the ground surface. This installation relies on a sizable intake volume and a narrow riser-pipe diameter to obtain a pore-water pressure reading in the riser pipe without significant time lag (Hvorslev, 1951).

With the development of diaphragm piezometers (e.g., pneumatic and vibrating wire), the method developed for standpipe piezometers was used for diaphragm piezometer installations (Dunnicliff, 1993). This has been a common practice for decades and the resulting installation is shown on Figure 1b. However, because of the

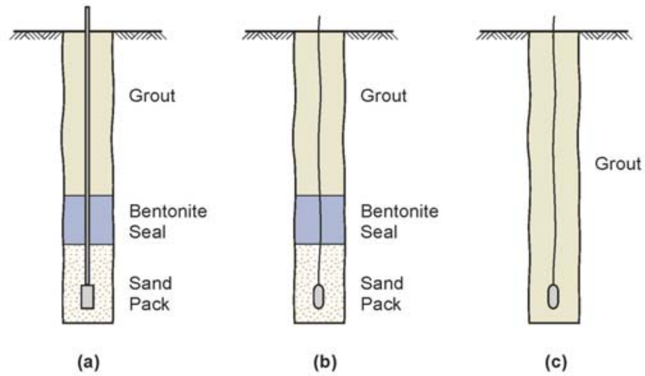

Figure 1(a). Traditional standpipe piezometer with sand pack.
Figure 1(b). Diaphragm piezometer with sand pack.
Figure 1(c). Fully-grouted piezometer.

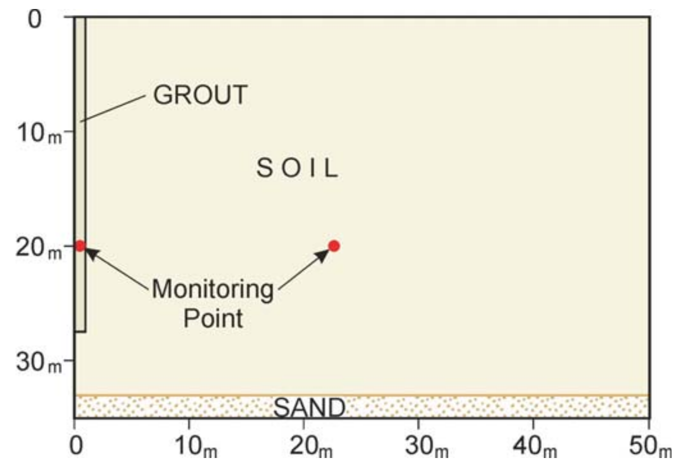

Figure 2. Schematic computer model to simulate seepage around a fully-grouted piezometer (borehole not to scale).

low-volume operation of diaphragm piezometers, the sand pack around the instrument tip is unnecessary, and the diaphragm piezometer can be installed in the borehole surrounded by cement-bentonite grout. This procedure is commonly known as the fully-grouted method (Mikkelsen and Green, 2003) and is shown on Figure 1c.

Fully-grouted Method

Figure 1c shows a piezometer installation using the fully-grouted method, in which a diaphragm piezometer tip is set in a drilled borehole and entirely backfilled with cement-bentonite grout. The following is a detailed description of the installation procedure for a vibrating-wire sensor tip in typical geotechnical boreholes (i.e., 140 mm), including preparation of piezometer assembly and materials, grout mixing, piezometer construction, and theoretical background.

Piezometer Assembly

Construction of the piezometer assembly commonly begins with attachment of the sensor tip to a sacrificial grout pipe. The sacrificial grout pipe, which can be either belled-end electrical conduit or threaded PVC well casing, is generally constructed or laid out on the ground in manageable lengths for handling. The piezometer location is selected by reviewing the soil stratigraphy. The sacrificial grout pipe will generally extend to the bottom of the borehole for support; therefore, it is possible to determine the location of the piezometer tip from the top or bottom of the borehole since the pipe is left in place.

After drilling a borehole, the piezometer tip is attached to the grout pipe at the appropriate location. For boreholes with a diameter of 140 mm, a typical grout pipe (such as 25.4-mm diameter PVC well casing) is used. Large-diameter or stronger grout pipe may be required for deeper installations with higher pumping pressures.

The sensor tip, which has been saturated following the manufacturer's directions, is typically set with the sensor in the upward position to minimize the possibility of desaturation. The cable connected to the sensor tip is attached to the pipe at approximate intervals along the grout pipe, leaving some slack in the line. The grout pipe, sensor tip, and cable are then lowered into the borehole with the grout pipe placed on the bottom for support. The piezometer tip is now located within the desired monitoring zone. The cable is brought to the surface where readings are taken with a readout device.

One advantage of the fully-grouted method is that it can be used for installation of nested piezometers. In a nested piezometer configuration, more than one piezometer tip is attached to the sacrificial grout pipe. The authors have successfully installed up to four piezometer tips in a borehole. During installation the drill casing should be removed carefully to prevent damage to the cables and the cables should be separated around the grout pipe to prevent a direct seepage path along a bundle of cables.

Another advantage of the fully-grouted method is the feasibility of using a single borehole to install more than one type of instrument. For example, the piezometer tips can be attached to an inclinometer casing, and a single borehole is used for measuring both deformation and pore-water pressures, resulting in reduced drilling costs. However, the inclinometer casing joints must be sealed. This technique has been used successfully by the authors on several projects.

Materials

The cement-bentonite mixes described in this article use Type I Portland cement and sodium bentonite powder such as Baroid Aquagel Gold Seal or Quickgel. The water used in the mixes should be potable water to prevent possible interaction of chemical constituents in the water with the cement-bentonite mixture.

Grout Mixing

The mixing procedure described in this article assumes the availability of a capable drill-rig pump and a high-pressure, jet-type nozzle attachment on the end of a mixing hose. In most cases, the drill-rig pump provides enough pressure for the jet-mixing required to obtain a desirable mixture. Other methods may use actual grout-mixing plants. Generally, the cement-bentonite mix is prepared in a barrel or mud tank using the drill-rig pump to circulate the batch with a suction hose and return line. Occasionally, a hydraulically-operated, propeller-type mixer is used. However, it has been the authors' experience that, in some cases (depending on the mix viscosity, pump operability on the drill rig, or grout volume), the use of a grout mixer/pump may be required. Typical batch sizes are 200 to more than 2,000 liters.

The mixing process begins with calculation of the amount of grout required to fill the borehole. A measured quantity of potable water is pumped into the mixing barrel first and circulation begins. During circulation, the water and cement are mixed first so that the water:cement ratio remains fixed and the properties of the grout mix are more predictable. The measured quantity of cement is gradually added to the water until both components have been thoroughly mixed. This is the most important step in the mix preparation and runs contrary to the common practice of mixing bentonite and water first. An initial measured quantity of powdered bentonite, based on a mix design, is added into the barrel near the jet stream to minimize the formation of clumps within the mix. Typically, additional bentonite is added as mixing continues to achieve a "creamy" consistency. Mikkelsen (2002) describes the consistency as "drops of grout should barely come off a dipped finger and should form "craters" in the fluid surface."

Piezometer Construction

At the completion of the grout-mixing process, and after measuring the final density of the mix, the piezometer tip assembly is lowered into the borehole. In shallow boreholes (e.g., typically less than 30 m deep), grout is then pumped into the borehole through the sacrificial grout pipe until it reaches the ground surface. In deeper boreholes, staged grouting using multiple grout pipes or multiple port pipes may be required so the piezometers are not over-pressurized during installation. In cased boreholes, the drill casing is slowly retrieved so that no gap is left between the top of the grout and the bottom of the casing. Typically, the entire process takes approximately one hour for a 30-m borehole. The hole is typically completed with concrete and a protective top.

The field engineer should take pressure readings during and immediately after installation. One benefit of vibrating-wire technology is that readings can be taken quickly. The readings obtained during grouting can be used to determine if the device has been over-pressurized during grouting. The measured pressures should approximately correspond to the pressure exerted by the column of grout above the tip, provided the sensor and grout are at nearly the same temperature, as temperature equalization may take several minutes. However, with time, this pressure decreases as the cement-bentonite mix sets up and pore-water pressure readings are taken at the tip locations. Typically, grout set-up takes one to two days.

Theoretical Background

McKenna (1995) clearly describes the two basic requirements for any piezometer to perform its function. The measured pore-water pressure must be fairly representative of the actual pore-water pressure at the measurement location (i.e., small accuracy error), and the hydrodynamic time lag must be short. At first glance, it does not appear that the fully-grouted method will satisfy these requirements. It would seem that the cement-bentonite grout surrounding the tip might prevent the piezometer from responding quickly to changes in pore-water pressures in the ground due to its low permeability. On the other hand, if the cement-bentonite grout is too permeable to enhance short hydrodynamic time lags, there would be significant vertical fluid flow within the cement-bentonite grout column.

However, the fully-grouted method does satisfy both of McKenna's requirements. A diaphragm piezometer, such as a vibrating wire piezometer, generally requires only a very small volume equalization to respond to water pressure changes (10^{-2} to 10^{-3} cm^3), and the cement-bentonite grout is able to transmit this small volume over the short distance that separates the piezometer tip and the ground in a typical borehole. A practical way to reduce this distance is to set up the tip close to the wall of the borehole by reducing the thickness of grout between the tip and ground using pre-manufactured, expandable piezometer assemblies.

Grout Permeability Requirements

Vaughan (1969) introduced the fully-grouted method and developed closed-form solutions which showed that the error in the measured pressure is significant only when the permeability of the borehole backfill is two orders of magnitude greater than the permeability of the surrounding ground. If the permeability of the cement-bentonite grout is lower than the permeability of the surrounding ground, measured pressures will be without error. As a result, for the fully-grouted method to work, the grout mix used to backfill the borehole must meet certain permeability requirements. A seepage model was developed by the authors to better understand those requirements.

Computer Modeling

A finite-element computer model simulating seepage conditions around a fully-grouted piezometer installation was used to evaluate the impact of grout permeability on the accuracy of the piezometer reading. The seepage model was conducted using SEEP/W, a computer-modeling program developed by GEO-Slope International.

Figure 2 shows the conceptual model developed to simulate the seepage around a piezometer installed using the fully-grouted method. The axisymmetric flow model includes a 7-cm radius, cement-bentonite-grout column surrounded by soil of constant permeability. The simulated cement-bentonite grout column extends 27.5 m and the soil layer extends 33 m below the ground surface with a radius of 50 m. Underlying the soil, a sand layer was incorporated to simulate the lower boundary conditions.

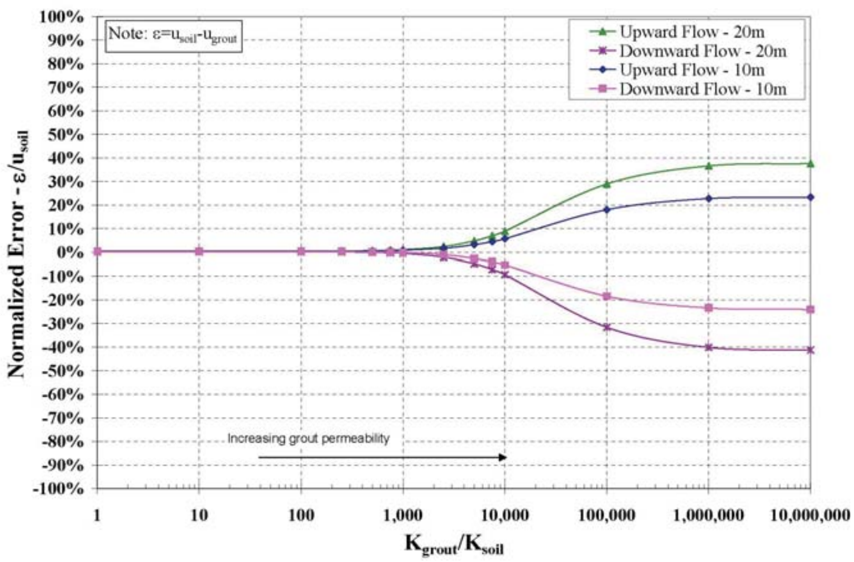

Figure 3. Normalized error versus permeability ratio.

The seepage analyses were performed simulating upward and downward flow using two sets of imposed total head conditions (i.e., 10 and 20 m) that induce flow under steady-state conditions. This set of boundary conditions corresponds to the one-dimensional flow condition in the vertical direction. In all cases, fully saturated conditions were used for all the materials in the model. The error, ε, defined as the difference in computed pore-water pressure between the soil and the grout, was determined during each model run at points in the soil and grout 20 m below the ground surface, as shown on Figure 2.

Results of Computer Modeling

Several model runs were made in which the permeability ratio, k_{grout}/k_{soil}, was varied from 1 to 10^7. Figure 3 shows the results of the seepage simulations in terms of the normalized error, i.e., ε divided by the pore-water pressure in soil, u_{soil}, against the permeability ratio. Figure 3 also shows that the normalized error is zero for all practical purposes up to permeability ratios of 1,000 for downward and upward flow and the two sets of imposed total heads. As the permeability ratio increases beyond 1,000, the normalized error increases up to about ±10 percent at permeability ratios of 10,000. As the permeability ratio continues to increase to 10^7, the normalized error also increases up to about 23 and 40 percent, respectively, for the 10-m and 20-m imposed total heads.

In summary, the finite-element computer model revealed that the permeability of the grout mix can be up to three orders of magnitude greater than the permeability of the surrounding ground without introducing significant error. This finding differs from previous assessments, which indicated that the permeability of the grout mix should only be one or two orders of magnitude greater than the permeability of the surrounding ground. The minimum permeability that is likely to be encountered in natural soils is on the order of 10^{-9} cm/s. As a result, the cement-bentonite grout mix used in the fully-grouted method needs to have a permeability of, at most, 10^{-6} cm/s.

Part 2 of this article will discuss laboratory test results of six cement-bentonite grout mixes and field examples of applications of the fully-grouted method.

The Use of the Fully-grouted Method for Piezometer Installation Part 2

Laboratory Testing Program

A laboratory testing program was developed to evaluate the range in permeability and strength of cement-bentonite grout for piezometer installationsusing the fully-grouted method. The test program was designed so that small batches of grout could be mixed in a controlled environment without large grout-batch mixing equipment. Six mix designs were chosen to represent a wide range of values that would reasonably be used on projects.

Sample Preparation

Mixing the grout used for laboratory testing began with calculating the desired quantities of material and thenweighing individual portions of cement, water, and bentonite. Additional bentonite was prepared in anticipation

Table 1. Properties of grout constituents			
Mix Component	**Brand**	**Specific Gravity**	**Moisture Content (%)**
Portland Cement Type I	LaFarge	3.15	—
Bentonite Quickgel (Mixes 1-4)	Baroid	2.41 to 2.45	11
Aquagel Gold Seal Bentonite (Mixes 5 and 6)	Baroid	2.4	10

of adjusting the mix viscosity. The properties of the individual mix components used in the laboratory testing are listed in Table 1.

To begin, the cement was added to the water slowly while mixing. The benefit of adding the cement first in the mixing process is that it ensures the correct water:cement ratio before adding the bentonite.

Table 2. Summary of cement-bentonite grout mixes used in the study			
Mix	**Water : Cement : Bentonite by Weight**	**Marsh Funnel Viscosity (sec)**	**Bentonite Type**
1	2.5 : 1: 0.35	50	Quickgel
2	6.55 : 1: 0.40	54	Quickgel
3	3.99 : 1: 0.67	60	Quickgel
4	2.0 : 1: 0.36	360	Quickgel
5	2.49 : 1: 0.41	56	Aquagel Gold Seal
6	6.64 : 1: 1.19	60	Aquagel Gold Seal

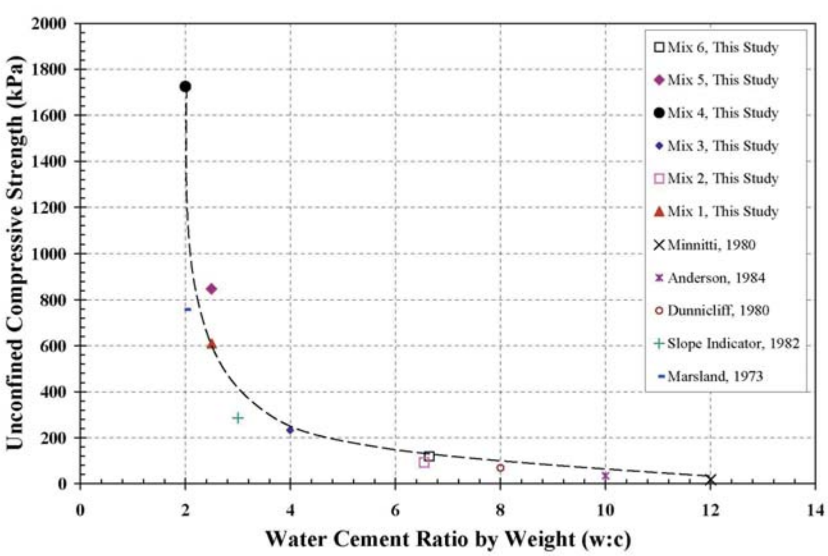

Figure 4. Variation of unconfined compressive strength versus water:cement ratio.

After the cement and water were mixed and the water-cement paste appeared uniform, which generally took five minutes, bentonite was slowly added to the bucket. The cement-bentonite grout was then mixed for approximately five additional minutes until it appeared uniform and did not contain lumps. Viscosity was measured at various times during mixing to evaluate the condition of the mix. Samples of the final mix were taken using plastic molds and the density was measured.

After a short cure period, the samples were carefully extruded out of the plastic molds and stored until the test date. For the Unconfined Compressive Strength testing (UCS), a set of two specimens were tested at 7, 14, and 28 days. Permeability testing was completed on specimens from each mix at 7 and 28 days under three different confining stresses. In addition to strength tests, basic index properties, such as moisture content and dry density of the samples, were also measured.

Laboratory Test Results

Table 2 summarizes the final cement-bentonite grout proportions used in this study. The results of the laboratory testing are presented in a series of figures.

Figure 4 summarizes test results as the average UCS at 28 days versus the water:cement ratio by weight. It shows that the UCS decreases with increasing water:cement ratios. In fact, the UCS at 28 days is approximately 1700 kPa at a water:cement ratio of 2:1; it then decreases to approximately 90 kPa with increasing water:cement ratio. Also included on Figure 4 are data presented by Mikkelsen (2002), which show a relatively strong correlation with the data of this study.

The void ratios of the samples were computed based on the measured water content of the specimens and the specific gravity of the grout-mix constituents. The computed void ratios of the mixes are relatively high, in fact, these are higher than soils with similar strength and permeability. However, the data show that the amount of cement controls the strength characteristics of the grout mix. Bentonite appears to in-

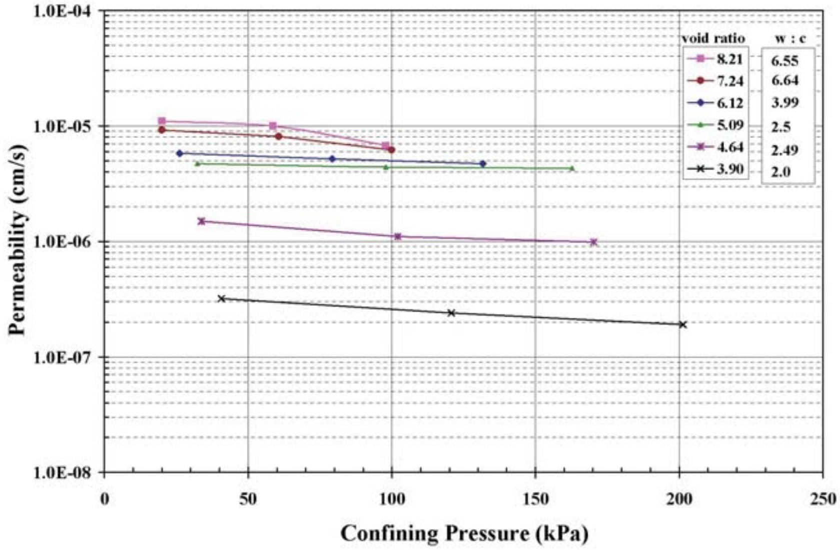

Figure 5. Variation of permeability versus confining pressure at 7 days.

fluence the amount of bleed water and volume change of the specimen during curing. Additional information on the strength and deformation properties of cement-bentonite mixes can be found in Contreras, et al. (2007).

Figure 5 summarizes the test results in terms of the permeability of the specimens at seven days for various confining pressures. The data show that samples with a higher water:cement ratio or void ratio have higher permeability than those with lower water:cement ratios.

Figure 6 shows the permeability in the same format for specimens at 28 days. Data are very similar, showing that the permeability is relatively constant or decreases slightly with confining pressure. One important result is that, from seven to 28 days, the permeability continues to decrease. For example, mixes with 2.49 water:cement ratio indicate a permeability greater than 1.0×10^{-6} cm/sec at 7 days and less than 1.0×10^{-6} cm/sec at 28 days. The data indicate that, as hydration of the cement occurs, the permeability of the mix decreases. The high void ratio and low permeability are two reasons the fully-grouted method works; it allows transmission of a low volume of water over a short distance yet maintains overall low permeability in the vertical direction.

Figure 7 shows the variation in permeability data with respect to void ratio. The data indicate that specimens with lower void ratios typically exhibit lower permeability, while those with higher void ratios exhibit higher permeability. With grout mixes, the cement has a greater influence on the void ratio than the bentonite and is considered the controlling factor in the permeability of the grout. The difference between the seven and 28-day permeability is relatively small, as shown on Figure 7.

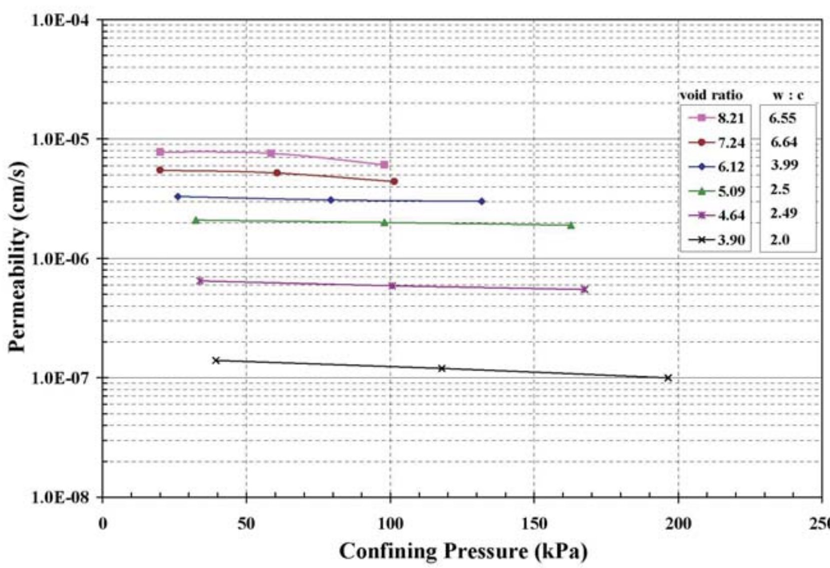

Figure 6. Variation of permeability versus confining pressure at 28 days.

Field Examples

This section describes three field examples in which the fully-grouted method was successfully applied. The first example compares pressure readings between one installation using the fully-grouted method in a nested configuration and the traditional approach with individual piezometer installations in separate boreholes. The second example describes use of the fully-grouted method with the installation of nested piezometers in an upward-flow condition. The third example is for a nested, fully-grouted method installation in a downward-flow condition.

Example 1. Comparison Between Nested and Individual Installations

This field example compares two methods of installation:

- Three vibrating-wire piezometers in a single borehole using the fully-grouted method.
- Four individual pneumatic piezometers in separate boreholes using the traditional sand pack around the piezometer tips.

The two installations were within 7.5 m of each other. As a result, some differences in the pressure readings were expected. Figure 8 shows a comparison of the pore-water pressure profile with elevation for both installations. The figure illustrates a fairly similar response considering the distance between the two sets. Similar data have been presented

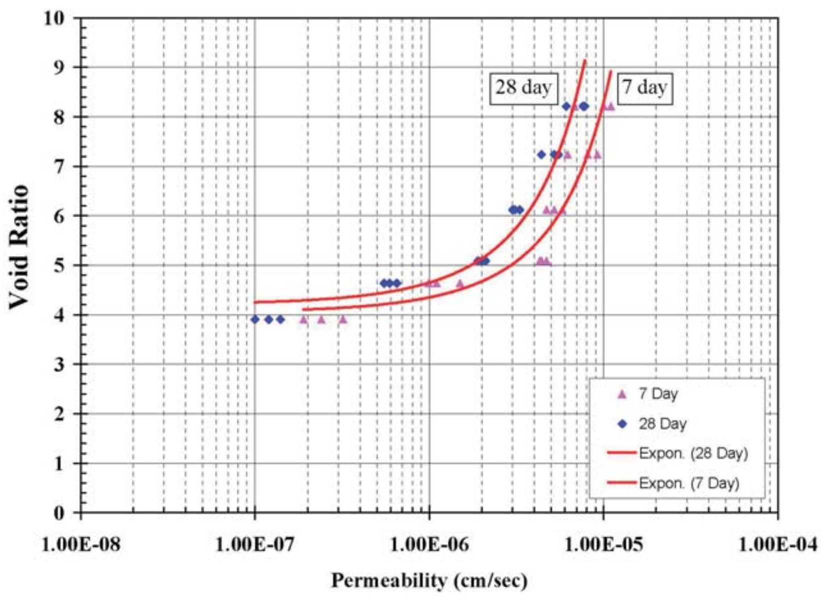

Figure 7. Void ratio versus permeability.

by McKenna (1995), further confirming the validity of the fully-grouted method.

Example 2. Upward-Flow Conditions

This field example illustrates the use of nested piezometers using the fully-grouted method in upward-flow conditions. The site is in an area where three distinct stratigraphy units are found (alluvial deposits, Huot Clay Formation, and Red Lake Falls Formation). The upward-flow conditions play a major role in the slope instability of the area (Contreras and Solseng, 2006).

Figure 9 shows the pore-water pressure and total-head profiles at the site, illustrating the upward-flow conditions. Two vibrating-wire piezometer tips were installed in the Huot Formation and one in the Red Lake Falls Formation. The Huot Formation is fairly uniform and has a permeability in the range of $1.2x10^{-8}$ to $1.9x10^{-8}$ cm/s. The cement-bentonite grout mix used in the nested installation had a 2.66:1:0.27 water:cement:bentonite ratio with a permeability of approximately $2.0x10^{-6}$ cm/s. This example presents the results of the fully-grouted method in a low-permeability unit.

Example 3. Downward-Flow Conditions

Finally, this field example demonstrates the use of nested piezometers with the fully-grouted method in downward-flow conditions. A total of four piezometer tips were installed in three units, with permeability ranging from $1.0x10^{-3}$ cm/s to $9.49x10^{-7}$ cm/s. Where there is a wide range of permeability, the least permeable unit controls the cement-bentonite grout permeability. As a general rule, the less permeable the cement-bentonite grout, the better, and as shown by the computer model, for most soil, a cement-bentonite grout with a permeability of $1.0x10^{-6}$ cm/s will be adequate. Figure 10 shows the pore-water pressure and total-head profiles at the site, illustrating the downward-flow conditions. This example presents the results of an installation of nested piezometers with up to four piezometer tips in a single borehole.

Summary and Conclusions

This two-part article presents a detailed discussion of the fully-grouted method for piezometer installation, including the procedure and theoretical background. It also discusses the results of a laboratory testing program on six cement-bentonite grout mixes, along with an evaluation of a computer model to determine the impact of the difference in permeabilities between the cement-bentonite grout backfill and the surrounding ground. The following summarizes the article's main issues and findings:

- The practice of installing diaphragm piezometers in a sand pack with an overlying seal of bentonite chips or pellets could be discontinued.
- The fully-grouted method is a fairly simple, economical, and accurate procedure that can be used to measure pore-water pressures in soils and fractured rock. It allows easy installation of a nested piezometer configuration, resulting in drilling cost savings. It can also be used in combination with other instrumentation (e.g., inclinometers) to measure deformation and pore-water pressures, provided the inclinometer joints remain sealed.
- The permeability of the cement-bentonite grout mix can be up to three orders of magnitude greater than the permeability of the surrounding ground without a significant error in the pore-water pressure measurement. This finding differs from previous assessments.
- Laboratory test results show that the permeability of the cement-bentonite grout mixes is a function of the water:cement ratio. As the water:cement ratio (void ratio) decreases, the permeability decreases.
- Bentonite has little influence on the permeability of the mix, but rather appears to stabilize the mix, keeping the cement in suspension and reducing the amount of "bleed water."

Acknowledgments

The support provided by the Innovation Committee of Barr Engineering Company is gratefully acknowledged. The careful performance of the laboratory testing by Soil Engineering Testing of Bloomington, Minnesota, is greatly appreciated. The continual assistance from Erik Mikkelsen and his thoughtful insight and contributions from the beginning of the research program have been extremely helpful in pursuing the research and use of the fully-grouted method. John Dunnicliff's thorough review and comments on this manuscript are also greatly appreciated.

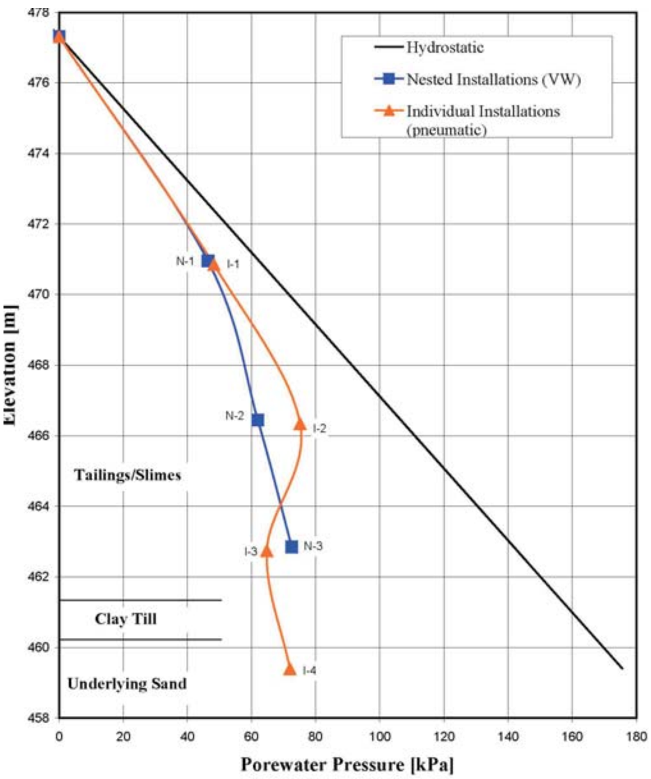

Figure 8. Comparison between a nested fully-grouted installation and individual separate installations.

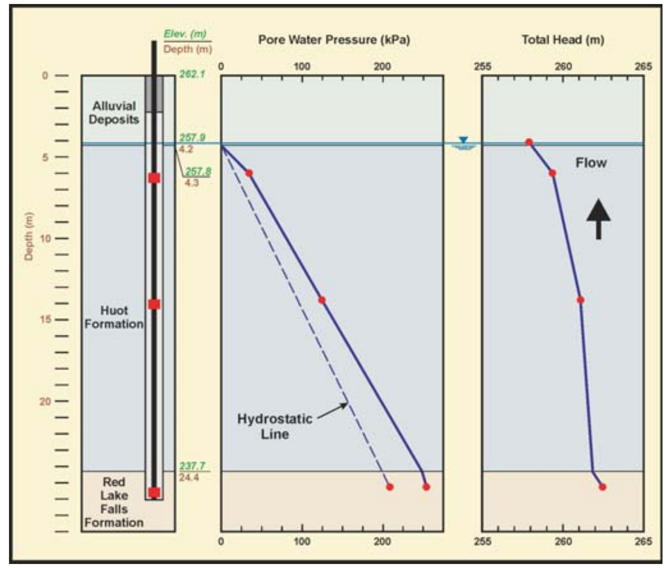

Figure 9. Field example of fully-grouted method in upward flow condition.

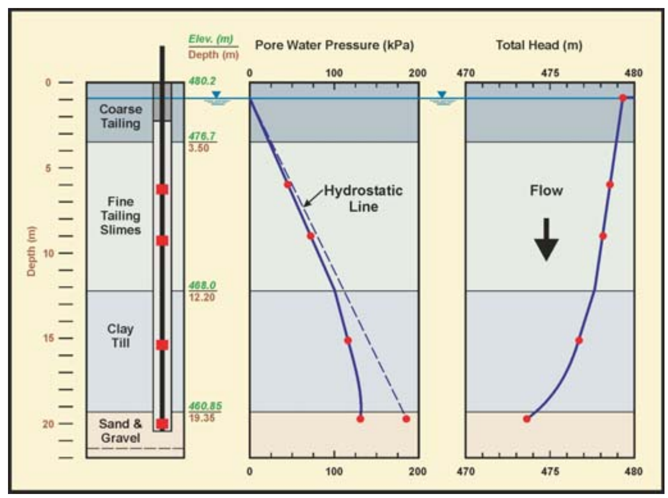

Figure 10. Field example of fully-grouted method in downward flow condition.

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Discussion of "The Use of the Fully-grouted Method for Piezometer Installation"

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Richard H. Ver Strate

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John Dunnicliff

Thank you to all three authors for their excellent practical contribution. I've been waiting for this for 39 years—see the last reference citation in Part 2 of your article, Vaughan (1969)!

Other Experiences with the Fully-Grouted Method

In my view, the rationale for accepting the fully-grouted method is very convincing. Despite that view, owners and their consultants may tend to be wary of what they consider to be a 'new and radical' method. As you'll see below, there's plenty of positive experience 'out there', and if we're to convince owners and their consultants, we need as much supportive information as possible. Among other experiences are:

- The engineers at Applied GeoKinetics, located in Irvine, California (www.appliedgeokinetics.com) have used the fully-grouted method successfully on approximately 400 installations since 1990. Several of these installations have been to depths of approximately 500 feet, with up to ten piezometers tips installed within a single borehole. For more information, please contact Glenn Tofani at glenn@geokinetics.org.
- Colleagues in Australia, Geotechnical Systems Australia Pty Ltd., (www.geotechsystems.com.au) have used the fully-grouted method with very good results on about 15 sites since 2001. Installations have been up to 500m deep. For more information, please contact Matt Crawford, matt@geotechsystems.com.au.
- Geometron of Seattle (Bellevue) and Kleinfelder of Denver installed about 40 fully-grouted vibrating wire piezometers on a multiple-dam project in Southern Oregon over the last three years, with real-time-monitoring. It includes several in unsaturated embankments, some reacting to rainfall recharge. For more information please contact Erik Mikkelsen, mikkelsen.pe@comcast.net.
- Many firms in Washington State regularly specify and use the method, including Camp Dresser McKee (CDM), CH2M Hill, and Jacobs Associates. It is in current use on major transportation and tunnel projects.
- The US Army Corps of Engineers in Omaha started using fully-grouted piezometers on Oahe Dam on the Missouri River, SD in 2000, particularly for piezometers installed in Pierre Shale. A pilot relief well program showed that the vibrating wire piezometers responded better than conventional open standpipes.
- Syncrude Canada Ltd., in Fort McMurray, Alberta have used fully-grouted piezometers successfully in 83 vibrating wire piezometer tip installations since 2003 in stiff in situ soils using Syncrude installation procedures. Syncrude has also grouted in about 65 vibrating wire tips when installing in compressible fills, but in those cases used a bentonite seal within 3m above the tip to protect against the potential of the grout cracking due to the settlement.
- Strata Control Technology, a mining consulting firm in Australia that also specializes in geotechnical instrumentation, have used the method and conclude, "Fully-grouted vibrating wire piezometers are proving an excellent tool for investigating the impact of coal mining on groundwater systems". For more information, please contact Ken Mills, kmills@sctaust.com.
- DeJong et al (2004) describe comparative tests between a single vibrating wire piezometer installed in a soft varved clay deposit with a sand pack, and a fully-grouted piezometer. They conclude, "The performance of the fully-grouted piezometer was shown to be nearly identical to that of the sand packed instrument". This paper also describes the use of pre-manufactured, expandable piezometer assemblies, to which the authors refer in Part 1 of their article, in the context of reducing the distance between the piezometer filter and the wall of the borehole.

If any reader has other data, pro or con, about the fully-grouted method, I'd very much welcome hearing about it, and will consider it for publication in a later episode of GIN.

Use of Bentonite Pellets and Chips over Sand Packs for Diaphragm Piezometers

The authors conclude, "The practice of installing diaphragm piezometers in a sand pack with an overlying seal of bentonite chips or pellets can be discontinued". As I wrote at the beginning of this discussion, owners and their consultants may tend to be wary of the fully-grouted method. In these cases I suggest that we ask them how confident they are that bentonite pellets or chips arrive at the depths shown so neatly in your Figure 1(b). I've had numerous experiences of these infuriating things arching part way down the borehole, and have little confidence that Figure 1(b) represents reality. As an aside here, I've tried to retard the onset of swelling so that they don't become sticky for a few minutes, including freezing, coating with hydraulic oil, and spraying with hair spray—forget it!

If owners and their consultants fully appreciate these uncertainties, perhaps they may be more willing to accept the fully-grouted method.

Use of Bentonite Pellets and Chips over Sand Packs for Open Standpipe Piezometers

Figure 1(a) shows a bentonite seal above the sand pack for an open standpipe piezometer. For the reasons given above, I believe that a more reliable installation method is to omit the bentonite seal and to place the grout directly over the sand pack. To prevent contaminating the sand pack with grout, a bottom plug should be used on the grout pipe, side-discharge holes drilled near the bottom of the pipe, and the grout pumped very slowly.

Use of the Method in Soft Ground below Future Embankments

When there is a need to monitor pore water pressure in highly-compressible ground as an overlying embankment is constructed, it is usual to do so with piezometers at various elevations. This has been done where vertical compression in the soft ground has been up to 35% (e.g. Handfelt et al, 1987). Because it allows several piezometers to be installed in a single borehole, the fully-grouted method is very attractive in this application, but I don't know if this has been done. Some of the installation procedure that is described by the authors would have to be changed for this application.

First, a sacrificial grout pipe couldn't be used, because it would impede conformance as vertical compression progresses, and I believe that a telescoping grout pipe would introduce too many problems. Perhaps the piezometers could be attached to 'aircraft cable' (stranded flexible and thin stainless steel cable), which would readily cope with the vertical compression. A flush-coupled (inside and outside) PVC grout pipe would be used, and withdrawn after grouting. If drill casing has been used, care would need to be taken to maintain piezometer depths when pulling the casing.

Second, arrangements would have to be made to ensure that the piezometer tubing or cable doesn't fail in compression. I know that this can happen. For pneumatic piezometers the tubing can be pre-spiraled as shown in Figure 9.31 in the red book—this was for a project in Hong Kong, described by Handfelt et al (1987), where the pneumatic tubes were wrapped around a 3" steel pipe, placed in very hot water for a few minutes, removed and allowed to cool. But for a future project I'd prefer to use vibrating wire piezometers, and wouldn't want to trust that the cable would move around in the grout and survive. Various unhappy experiences have taught me a golden rule about installation of instruments: "if you can think of something that might go wrong, deal with it by changing the planned procedure". Apparently it isn't possible to pre-spiral the types of electrical cable that are used for field instrumentation. One possible way could be to coil the piezometer cables loosely around the grout pipe as all components are lowered together, but would that run the risk of cable damage and possible lifting of piezometers when removing the grout pipe? Another possible way could be for the manufacturer to insert the cable in a plastic tube and coil the tube as was done for the Hong Kong project, but would that run the risk of tubing damage and creation of a bleed path for pore water pressure?

Third, the compressibility of the grout must not be less than that of the surrounding ground—this would need to be taken care of during design of the grout mix.

For this application it is necessary to keep track of the elevation of the piezometers as they settle, because the measurements of pressure need to be converted to piezometric elevations. This is done by monitoring settlement nearby, usually with probe extensometers, and ensuring that the probe extensometers can also cope with the vertical compression.

I'd very much welcome the authors' comments on these suggestions.

Borehole Diameter

In Part 1 the authors refer to "typical geotechnical boreholes (i.e., 140 mm)". In my experience many piezometers are installed in smaller diameter boreholes, often as small as 76 mm. Do the authors have any recommendations if we do this?

Use of a Single Borehole for Fully-grouted Piezometers and Inclinometer Casing

In Part 1 of the article the authors say that a single borehole can be used to install both piezometers and inclinometer casing, resulting in reduced drilling costs. They add the caveat, "However, the inclinometer casing joints must be sealed". I want to emphasize that "sealed" must be taken very seriously—any lack of sealing will create a path for dissipation of pore water pressure, therefore false readings.

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Authors' Reply

The authors appreciate the opportunity of discussing the details and concerns of the fully-grouted method with other colleagues.

Other Experience with the Fully-Grouted Method

The authors have successfully installed over 100 piezometers using the fully-grouted method to depths greater than 100 feet since 2003. Some installations have had up to four piezometer tips in a single borehole.

Use of Bentonite Chips and Sand Packs

The best way to obtain confidence in the use of the fully-grouted method is to construct a trial application using both the traditional sand pack method and fully-grouted method in adjacent boreholes. This comparison test will quickly reveal the benefits of the fully-grouted method regarding the ease of installation, time, and cost reduction. This is the way the authors became convinced the method works. Arching of sand pack and bentonite chips can be a very frustrating and costly problem during construction and the fully-grouted method eliminates this problem.

Use of the Method in Soft Ground below Future Embankments

The authors have successfully used the fully-grouted method in soft ground with sacrificial grout pipes; however, in their experience, the magnitude of settlement has generally been much less than 35 percent. It is recognized that in ground with high compressibility, the grout pipe would impede conformance as vertical compression progresses.

The application of the "aircraft cable" installation sounds reasonable and will be considered for future installations in the authors' practice. It appears to be the easiest and most sound solution to the problem.

The installation using traditional methods and fully-grouted method appear to have the same concerns regarding the compressibility of the grout and the formation.

Borehole Diameter

Typical hollow-stem auger drill casing used in the Midwest region of the United States has inner diameters of 82 mm to 108 mm, which allows the use of a 25 mm grout pipe and multiple piezometer tips and cable bundles. Casing diameters less than 108 mm may cause casing removal problems as the piezometers and cables may catch on the casing and damage the installations. For multiple-device installations, the larger casing is preferred. Some manufacturers also offer protective casing for the piezometers that helps reduce damage to the devices. For a single-tip installation, the small-diameter casing should be adequate.

Use of a Single Borehole for Fully-grouted Piezometers and Inclinometer Casing

The authors' experience of installing inclinometer casing and piezometers has been successful. Care has been taken to separate the tips from the joints. Additionally, care has been taken to ensure the joints are as watertight as possible. A verification test can be performed by adding and removing water inside the casing and measuring readings at the piezometers to identify any impact on the readings from leaks. A stable reading may indicate a successful installation. However, the readings should be evaluated during monitoring in the event casing movement has caused a joint to leak.

Geotechnical Alarm Systems Based on TDR Technology

Kevin M. O'Connor

Abstract

Geotechnical applications of time domain reflectometry (TDR) are continuing to evolve and usage is increasing, particularly for monitoring deformation over large lateral distances when a priori knowledge of movement locations is limited. Applications include monitoring subsidence along major roadways over active and abandoned mines and monitoring movement along the toe of dams. It has evolved into real time monitoring with alarms and a variety of notification schemes. Fully automated systems have been installed which detect when measured deformation on the order of millimeters has exceeded specified magnitudes and/or rates, and initiate phone calls to responsible parties. The diversity of project-specific details (e.g., cables installed in trenches, in horizontal directionally drilled holes, in angled holes, notification via telephone or radio, etc.) is a reflection of the range of site conditions and owner requirements. This article is based on the paper, "Geotechnical Alarm Systems Based on TDR Technology," which was presented at FMGM 2007 and is published in GIN with permission from ASCE.

Representative Projects

The following projects illustrate alarm systems utilizing TDR technology. Common features of the hardware and software include:

- 22 mm diameter solid aluminum coaxial cable
- robust datalogger and TDR unit

Figure 1. TDR data acquisition system. Four coaxial cables are connected to the multiplexer inside the smaller cabinet. The datalogger, TDR unit, external storage module, phone modem, and auto-dialer are also in the smaller cabinet.

Figure 2. Holes being drilled for grout injection into abandoned mine along I-77 in Summit County, Ohio.

- cable lengths up to 610 m
- maximized number of interrogation points per cable
- baseline reading, or specified reference values, stored on datalogger
- difference between baseline reading, or specified reference value, and current reading computed by datalogger
- datalogger initiates alarm sequence when difference exceeds specified threshold value(s), and
- external data storage module

While there are common features, project-specific installation details, data requirements, and interrogation details reflect the flexibility of TDR technology in these applications.

There are also project-specific rationale and objectives that reflect the flexibility of TDR technology.

Table 1. Installation Details

Site	Orientation	Cables
I-77 Ohio	HDD holes and trench	8 cables 317 to 374 m (1040 to 1227 ft)
SR 91 Virginia	Angled holes and trench	18 cables 36 to 447 m (118 to 1466 ft)
Tuttle Creek Dam Kansas	Trench	4 cables 610 m (2000 ft)
McMicken Dam Arizona	Trench	6 cables 416 to 497 m (1366 to 1630 ft)

I-77 Summit County, Ohio

This project was motivated by previous experience of the Ohio Department of Transportation during stabilization of a section of interstate highway impacted by abandoned mine subsidence in Guernsey County. At that location, the highway was closed as the mine was backfilled with grout. As grout was injected, water within the mine was displaced and subsidence sinkholes developed under the roadway.

For the project in Summit County, it was specified that the highway remain open for traffic during grout backfill injection and it was necessary to provide an early warning system to detect if subsidence was occurring beneath the highway as water was displaced. Holes were horizontal directionally drilled (HDD) beneath the centerline of each lane of the highway. Coaxial cables were pulled back through the holes and then connected to a remote data acquisition system. Cables were also installed in a trench along the road. When the system detected an alarm condition along any cable, it communicated via phone modem with on-duty GeoTDR personnel.

State Route 91 Plasterco, Virginia

When the United States Gypsum Company was decommissioning its facility in Plasterco, it was known that subsidence would occur when pumps were turned off and water levels rose within the mine.

One component of the decommissioning plan was construction of a new alignment for SR91 outside the projected influence of subsidence. Construction of the new alignment involved blasting for rock excavation and company personnel were concerned that blast-induced vibrations would accelerate subsidence of the soft overburden beneath the existing highway. Coaxial cables were installed in angled holes drilled under the road and also in a trench along the road.

In addition, mine personnel were concerned that subsidence would occur in the former plant area where exca-

Table 2. Interrogation Details

Site	Data Points	Interval
I-77 Ohio	1 point/ m	3 hrs
SR 91 Virginia	3 points/m	3 hrs
Tuttle Creek Dam Kansas	2 points/m	10 minutes
McMicken Dam Arizona	20 points/m	24 hour

Figure 3. Installing coaxial cable in the trench along SR91 in Plasterco, Virginia.

Figure 4. Installing coaxial cable in the downstream trench at Tuttle Creek Dam in Manhattan, Kansas.

vated rock was being stockpiled. Cables were also installed in trenches in this area. As each of the four remote systems detected an alarm condition along a cable, a datalogger would activate an auto-dialer to call on-duty U.S. Gypsum personnel.

Tuttle Creek Dam, Manhattan, Kansas

Under contract with the U.S. Army Corps of Engineers, URS Corporation installed a warning system at Tuttle Creek Dam. The concern was motivated by a possible seismic event within the New Madrid Fault Zone that could initiate movement of the downstream slope of the dam. URS installed a multi-faceted instrumentation system to monitor surface and subsurface movement in real time. When multiple sensors detect changes that exceed specified threshold values, a sequence is initiated that can mobilize evacuation of downstream residents.

If slope movement should occur, modeling by URS has indicated bulging of the toe would intersect the downstream trench in which cables are installed. Two adjacent cables extend west from the data acquisition system (DAS) and two adjacent cables extend east from the DAS. When deformation exceeds a threshold value on adjacent cables simultaneously, the datalogger activates one channel of a control measurement unit. The CMU polls several different sensors and communicates with the base station via radio.

Figure 5. Data acquisition system at McMicken Dam in Maricopa County, Arizona.

McMicken Dam, Maricopa County, Arizona

Based on the subsidence history of the McMicken Dam embankment crest and other information, AMEC and the Maricoupa County Flood Control District have determined that ground strains and fissuring are developing due to consolidation of the underlying alluvial aquifer caused by ground water withdrawal. Based on further studies, it was determined that there exists a high probability of earth fissures being present in the soils underlying McMicken Dam.

As a component of the Fissure Risk Zone Remediation Project, two adjacent coaxial cables were installed in a trench downstream of the dam to detect development of earth fissures. When deformation exceeds a threshold value on adjacent cables simultaneously, the datalogger initiates a call via radio to the ALERT-protocol control center.

Alarm Activity

When calls are received from a remote data acquisition system, information transmitted includes the cable identification number and location along the cable where the alarm condition exists. Algorithms, which have been programmed into the dataloggers, do not distinguish the cause of the alarm condition. They only alert responsible personnel to the fact that a condition exists in which the difference between the current reflection magnitude and baseline value is greater than the specified alarm level threshold. These alarms can be triggered by causes other than cable deformation and the alarms must be filtered.

A context for the performance of the alarm systems is provided by some historical data for police alarms and debris flow alarms. The false alarm rates for three systems listed in Table 3 ranged from 70% to 90%:

Monthly alarm activity for the TDR based systems in Ohio and Virginia is summarized in Table 4. These projects

Table 3. Public Safety Alarm Activity

Location	Alarms	Reference
Bellevue, Washington	75% - 90% filtered out	AIREF, 2002
Columbus, Ohio	90% false alarms	Andes, 2005
Taiwan debris flow	70% false alarms	Wu, 2005

used single cables in each borehole or trench without redundant measurements, and false alarms ranged from 0 to 100% with averages of 52% and 76%.

Average rates are not really meaningful since weekly and monthly rates provide a more realistic assessment of the impact on the response of personnel.

Table 4. TDR Alarm Activity

Period	Total Calls	Ground Moving	Other Cause	
Summit County, Ohio				
8/01	10	4	6	60%
9/01	82	40	42	51%
Total	**92**	**44**	**48**	**52%**
Plasterco, Virginia				
6/02	5	0	5	100 %
7/02	17	4	13	76%
8/02	14	2	12	86%
9/02	7	0	7	100 %
10/02	2	2	0	0
11/02	0	--	--	--
12/02	52	0	52	100 %
1/03	6	0	6	100 %
2/03	0	--	--	--
3/03	108	49	59	55%
4/03	42	3	39	93%
Total	**253**	**60**	**193**	**76%**

Alarm Response

Various techniques have been used to filter alarm calls. Personnel assigned responsibility for responding to alarms have developed operational procedures to filter calls as they gained experience. Subsequent system designs have been modified to incorporate redundancy to improve the reliability of alarm calls.

Time identifier - each cable is assigned a time when it is interrogated so the cable is identified simply by the time at which an alarm call is received. The datalogger is programmed to stop calling after a specified number of retries, and this information has been utilized to filter calls. If a call is not received from the same cable at the next assigned time for that cable, the alarm condition is typically intermittent and not associated with ground movement. This technique that has been used to respond to alarm calls when the cause of the alarm was determined previously and the alarm condition is being addressed.

Adjustment of alarm levels – this is a relatively straightforward measure in which the specified threshold value is increased either temporarily or permanently,

Specific portions of cable are interrogated – it is possible to specify if the entire cable is interrogated or specific segments of the cable are interrogated,

Simultaneous deformation of adjacent cables – two cables can be placed in one trench and the alarm condition is not verified unless deformation has occurred on both cables simultaneously.

Action Plan

Consider the following action plan that was implemented for the U.S. Gypsum project in Plasterco.

Action Level 1:

- Receive call from remote datalogger
- Down load data, identify cause of alarm condition

Action Level 2:

- Visual inspection of alarm location
- Increased frequency of data acquisition
- Confirmed movement (based on visual inspection and/or redundant measurements)
- Notify management

Action Level 3:

- Accelerating movement
- Confirmation with visual evidence or redundant data
- Shut down highway

This type of action plan is based on reaction to an alarm call from the remote monitoring system. It inherently involves: decision making within a compressed time frame, and personnel on call 24/7.

During the USG project, the tasks of monitoring and response were assigned to in-house personnel to control costs and to expedite the decision making process. For the other projects, calls were handled by an in-house central control center or out-sourced personnel.

Monitoring a phone 24/7 is reactive and can lead to "burnout" when alarm levels are being exceeded frequently. This operational issue has been addressed by the call-filtering techniques and system design features mentioned above. Equally significant are periods during which there is no alarm activity (e.g., Oct-Nov 2002 in Table 4). For each project, dummy cables are in place that are used to create an artificial alarm condition, verify system operation, and verify personnel response during periods when there is no alarm activity.

Closure

TDR technology is capable of monitoring movement over large lateral extents and to great depths with a high density of monitoring points. It is being used to monitor deformation over active and abandoned mines, deformation along dams and slopes, movement in landslide areas, and sinkhole movement in karst areas.

TDR-based systems are similar to other geotechnical measurement alarm systems. The rationale for these measurements is significantly different from the rationale for performance monitoring where measurements are made to compare actual and anticipated behavior. Alarm calls are received that may not be associated with actual deformation, but an action plan must be in place to respond to each call and determine the cause. Unless this is accomplished, the alarm calls will continue.

Proactive, scheduled data acquisition and display has been the most effective monitoring plan to observe movement before alarm levels were exceeded.

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