Geotechnical Instrumentation News

John Dunnicliff

Introduction

This is the thirty-third episode of GIN (toidy–toid, if you live in Brooklyn). There are two articles in this episode.

Cement-Bentonite Grout Backfill for Borehole Instruments

In the previous GIN column I said, "Erik Mikkelsen is preparing an article for GIN to help the rest of us to understand the why and how of using cement-bentonite grouts as backfill for borehole instruments". Here it is. As Erik says in his introduction, the backfill for a borehole instrument is often an item that receives a disproportionate lack of attention, yet its behavior is critical for obtaining correct measurements.

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The two associated topics that I mentioned in the previous GIN column – two test programs, one on cement-bentonite grouts and one on bentonite chips and pellets, will be started in the near future, and we'll report results in GIN when they are available.

Measurement of Pore Water Pressures in Embankment Dams

The article by Arthur Penman sheds significant light on both the history and the dos-and-don'ts of pore water pressure measurement in embankment dams. When discussing this subject with Arthur I asked him to comment on the reasons why designers of embankment dams might opt for monitoring long-term performance by measuring pore water pressure in the cores. He replied:

> *"In the cores of dams, strength is not of importance, hence there is no reason to measure pore water pressures for the purpose of estimating strength. Soft core material should be used to avoid "arching", to maintain full overburden pressure on all cross sections in order to avoid hydraulic fracture. The purpose of the shoulders is to hold up the core, and the purpose of the core is to prevent the reservoir water from flowing into the downstream shoulder.*
>
> *The permeability of core fill is dependent on effective pressure: the larger the effective stress, the lower the permeability. A typical core of a dam in operation, supported by a downstream filter, has high pore water pressures, equal to reservoir level on its upstream side, and because this reduces the potential effective stress, the fill is more permeable and the higher pore water pressure is found well into the core, maybe even three-quarters of the way through. The behaviour of the core can be checked by placing lines of piezometers across the core at various levels. The number of piezometers in each line depends on the width of the core. Only where pore water pressures are reduced by drainage into the downstream filter do the effective stresses increase sufficiently to reduce permeability and prevent excessive flow through the core. This effect, when first observed, was thought to be due to faults in the core, with cracks allowing reservoir water to get into the middle of the core, and thereby increasing the pore water pressures."*

Manufacturers' Product Data Sheets

Many manufacturers' product data sheets (sometimes called 'specifications') contain information that can readily be misunderstood, such that the user is led to expect unrealistically high performance. Two examples follow. First, terminology is often confusing to the user, a frequent confusion being to quote a figure for resolution (defined as *the smallest division on the readout scale*) but nothing for accuracy (*close-*

ness to truth), so that the user believes that the accuracy is higher than it is. Second, figures quoted for accuracy are often based on calibrations in the laboratory, whereas accuracy in the field may be much less. An obvious example is embedment earth pressure cells, for which product data sheets usually indicate an accuracy based on calibration tests in air or water, whereas in the field accuracy is greatly reduced by inclusion effects.

DiBiagio et al (1999) identify the problem of misleading product data sheets and recommended establishing appropriate contents and uniform use of terminology. These ideas are currently being pursued on the discussions page of the FMGM web site (*www.fmgm.no*), under the heading 'Specifications'. Please visit the site and contribute to the discussions.

Reference: DiBiagio, E., Pezzetti, G., and B. Bruzzi, (1999). *Classification, Certification and Specification of Instruments for Field Measurements.* Proc. Symp. on Field Measurements in Geomechanics, Singapore, pp. 125-134.

become part of construction contract documents. The consequences of horizontal deformation beyond a certain small threshold were unacceptable, and maximum data quality was essential.

One option was to write a specification that named commercial sources that are believed to be appropriate with, being a public agency project, the ubiquitous words 'or acceptable equivalent'. However, how to determine which commercial versions to name? Rely on past personal experience? Read the literature on performance? Ask around? Trust the performance numbers on manufacturers' product data sheets, even though required characteristics are often not there?

A second option was to write a performance specification. But how to be sure at the time of writing the specification that at least one of the available commercial versions would satisfy this specification? And how to verify, after the contractor submitted the chosen version 'to the Engineer for review', that it was satisfactory?

Neither of those two options was very attractive, but because we had a copy of the French report on sensor testing, we could choose a third way. We began by defining the required performance characteristics, specific to the project, and then studied the report to see which of these characteristics were realistic. The specification states that the sensors shall have been proven, by laboratory testing that is independent of the manufacturer and acceptable to the Engineer, to have the following performance characteristics, and numbers follow. A submittal is required to demonstrate acceptability.

Several colleagues have since expressed concerns about this approach, and have sent me forthright e-mails. I've therefore posted the topic, as a new thread with subject "Independent Testing of Sensors" on the discussions page of www.fmgm.no, in the hope that we can share views, and point the best direction for the future. If you have opinions on this, I hope that you'll post them. Of course, this issue is closely related to the one discussed above – the content and correctness of manufacturers' product data sheets. If we knew they were complete and correct, we'd have it made, wouldn't we?

Now that the French report of independent testing of in-place inclinometers is available, questions such as "what can/should users do with it?" are being asked

Independent Testing of Sensors, and Use of the Results – What is Your Opinion?

Now that the French report of independent testing of in-place inclinometers is available (*www.soldatagroup.com*), questions such as "what can/should users do with it?" are being asked. One possibility is illustrated by the following.

I was recently asked by a client to help with writing a specification for in-place inclinometers, which was to

Guidelines for Articles in GIN

Guidelines for articles are now on the BiTech website, *www.bitech.ca.* If you have a practical topic that you think would help the rest of us, please look at the guidelines and send me an abstract if you want to go ahead.

Some Reminders

10-13 March 2003. Next instrumentation course in Florida. See page 50 and visit *www.doce-conferences.ufl.edu/geotech/.*

28 May 2003. An instrumentation seminar and discussion on Santorini Island in Greece.
Visit *www.heliotopos.net/conf/11fig/.*

September 15-18 2003. Next FMGM (Field Measurements in Geomechanics) International Symposium, in Oslo, Norway. Visit *www.fmgm.no.*

Closure

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

Yah-suh! (Greece) – "to your honor". Thanks to Vahan Tanal for this.

Cement-Bentonite Grout Backfill for Borehole Instruments

P. Erik Mikkelsen

The backfill for a borehole instrument is often an item that receives a disproportionate lack of attention. The behavior of the backfill, the material that is in the most intimate contact with both the formation and the instrument, is critical for obtaining correct measurements. In many situations, instrument observations may just reflect unstable backfill, lack of backfill or backfill that is too stiff or too soft. Sand, gravel and various bentonite products have proven to be both too difficult to place and often entirely inappropriate. Experience has shown the author that cement-bentonite grout is the most universally applicable material for successfully backfilling a borehole instrument. Single-component bentonite grouts have been used in related industries a long time, and have been adopted for borehole instrumentation with mixed success. Their uses are more involved and, as explained below, should be avoided. The use of fly ash as a substitute for cement promises to be a good way for reducing grout stiffness when required.

Current Use of Bentonite

Materials and Technology

Although this article advocates the use of cement-bentonite grout it is appropriate to review current bentonite backfill and sealing products to illustrate why their use should be limited. To the uninitiated, there is a confusing array of various pellets, chips, granulated and powder-forms of sodium bentonite commercially available in North America and elsewhere. Calcium type montmorillonite and opalite are also used. The proper use of these products is a mature and complex technology employed by the environmental, water-well and petroleum industries. Basic research has been done and their properties are known. For example, Baroid Industrial Products (1994), a manufacturer of about two-dozen drilling-products gives a five-day workshop in Houston, Texas including a hands-on laboratory day. Here, the user learns about the difference between drilling muds and bentonite used for sealing (single-component grouts) and how to place the materials. However, drillers and geo-professionals who install piezometers and other borehole instruments for the geotechnical industry generally do not have this background and often have limited understanding of bentonite products outside of powder used to make mud and dropping chips down the hole. Going "high-tech" requires more knowledge, better equipment, and a higher level of quality control, which appears to be unrealistic for borehole instrumentation.

Solid Bentonite Seals

Installation of bentonite balls, pellets or chips as seals above a sand pocket have dominated piezometer installation procedures over the last 50 years. It is classic procedure for open standpipe piezometers. Installation is usually very time consuming, particularly on deeper boreholes or when caving occurs. When you manage to get such seals installed without bridging the hole, there is usually no question about their permeability being adequately low. These seals have a very low permeability, often lower than many in-situ clays. Establishment of proper procedures for placement of such materials has been important not only because sealing is important, but also because the installation conditions are often difficult and the procedures cumbersome. Many installations end up less than satisfactory. Over the last decade, experience has shown that bentonite chips (as opposed to balls and pellets) are the easiest to place. These chips look like crushed gravel and hydrate very slowly. However, the fine clay-dust unavoidably mixed in with the chips can make conditions increasingly sticky as filling proceeds, leading to bridging and blockage higher in the borehole. For relatively simple installations where the seal heights and volumes are not too extensive, this material usually does not bridge and is often the seal of choice for environmental observation wells and open standpipe piezometers.

Pellets, chips and polymer-suspended granules can also be tremied to the desired location in the boring. The potential for clogging is always a hazard, but at least a clogged tremie-pipe can be withdrawn and discarded. If clogging or bridging occurs while dropping the materials directly into the borehole, there is no recourse except to start over.

Bentonite Grouts

These products are the least desirable for sealing or backfilling. They are made from water and powdered bentonite mixed into slurry-like drilling mud, but to a higher density with the aid of additives and specialized grout mixing units. The higher the bentonite solids-content is, the lower the permeability is. The water-content of such slurry is extremely high and it never really sets up to anything more than thick paste, not a solid like the chip-seals. A number of bentonite sealing grouts are available, but none appear to set up to a solid form. They are sensitive to over-mixing (leading to a flash set) and can be difficult to pump down the small diameter grout pipes (3/4 inch) often used for piezometers and other geotechnical borehole instruments where space is at a premium. Their working time tends to be too short, and mix dilution to circumvent mixing and pumping problems will lead to a permanently soupy backfill.

Cement-Bentonite Grouts

Basics

A bentonite grout backfill consisting of just bentonite and water may not be volumetrically stable and introduces uncertainty about locally introduced pore water pressures caused by the hydration process. Introducing cement, even a small amount, reduces the expansive properties of the bentonite component once the cement-bentonite grout takes an initial set. The strength of the set grout can be designed to be similar to the surrounding ground by controlling the cement content and adjusting the mix proportions. Controlling the compressibility (modulus) and the permeability is not so easy. Weaker cementitious grouts tend to remain much stiffer than normally consolidated clays of similar strengths. The bentonite solids content has the greatest influence on the permeability of cement-bentonite grout, not the cement content.

Cement-bentonite grouts are easier to use than bentonite grouts, provide a long working time before set and are more forgiving should the user deviate from the design recipe or mixing equipment and method. It is easier to adjust the grout mix for variations in temperature, pH and cleanliness of the water. Pure bentonite grouts must be mixed and deployed by strictly following measured quantities and procedures that are not common practice among drillers doing test borings.

Strength and Deformation

The general rule for grouting any kind of instrument in a borehole is to mimic the strength and deformation characteristics of the surrounding soil rather than the permeability. However, while it is feasible to match strengths, it is unfeasible with the same mix design to match the deformation modulus of cement-bentonite to that of a clay for example. The practical thing to do is to approximate the strength and minimize the area of the grouted annulus. In this way the grout column would only contribute a weak force in the situation where it might be an issue.

Strength data collected informally from various sources by the author over the years are summarized in Figure 1. A trend line drawn through the data points illustrates the decrease in strength with increasing water-cement ratio. The water-cement ratio controls the strength of the set grout (Marsland, 1973). Marsland's rule-of-thumb is to make the 7-day strength of the grout to match one quarter that of the surrounding soil.

Water and cement in proportions greater than about 0.7 to1.0 by weight will segregate without the addition of bentonite or some other type of filler material (clay or lime) to suspend the cement uniformly. In all cases sufficient filler is added to suspend the cement and to provide a thick-creamy-but-pumpable grout consistency. The bentonite does not add significant strength to the grout. The background data for Figure 1 also suggests the amount and type of bentonite or hydrated lime does not influence strength as long as the grout is non-bleeding and pumpable. If the grout bleeds the water-cement ratio decreases and strength increases. If fly ash were to be used as a substitute for cement the strength and modulus would be expected to drop. Fly ash contains less cementing agents (calcium and gypsum).

Grout Permeability

Permeability of the grout is mainly an issue that is limited to piezometer installations, and is the subject of a paper to be published in the near future. It is general practice to grout the borehole above a bentonite seal placed above the piezometer "intake zone" (sand that surrounds the piezometer), but the pa-

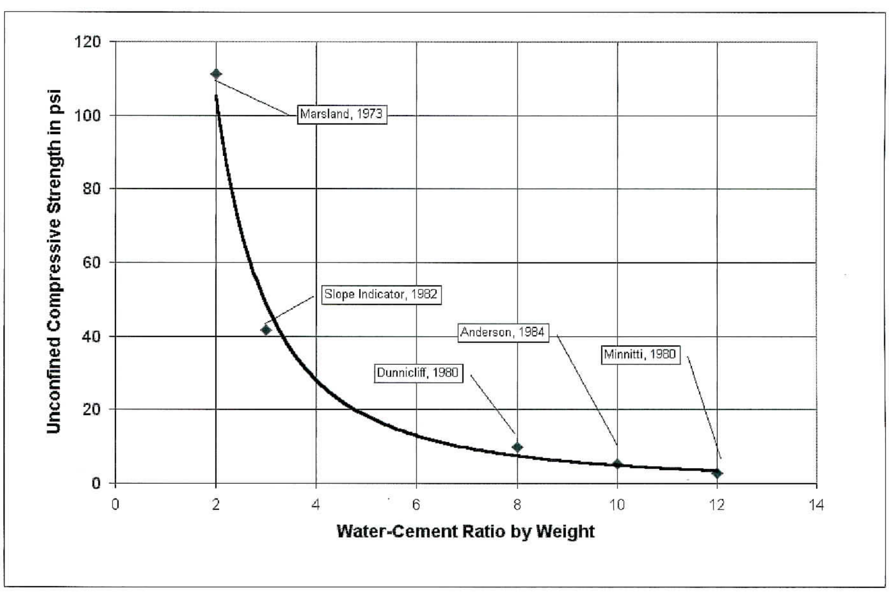

Figure 1. 28-day cement-bentonite grout strength vs. water-cement ratio. Data from author's personal files.

Table 1. Permeability, k, of some grouts			
Grout Type	**Characteristics**	**k (cm/sec)**	**Source**
Neat cement	w/c ratio = 0.89 to 0.53	10^{-5} to 10^{-7}	Baroid
Bentonite chips	hydrated	10^{-8}	Baroid
Bentonite slurry	6 % solids	10^{-5}	Baroid
Bentonite slurry	20 % solids	10^{-8}	Baroid
Cement-bentonite	water/solids = 4 to 1	10^{-6}	Vaughan, 1969
Cement-bentonite	w : c : b = 4 : 1 : 1	5×10^{-8}	Vaughan, 1973

per argues that boreholes may be fully-grouted for diaphragm piezometers, omitting both the sand and the bentonite seal. Two articles by Vaughan (1969 and 1973) make the point clear in theory and practice. This procedure not only simplifies difficult installation situations, but also improves the quality of the installation. It is really not a question of whether or not diaphragm piezometers work when fully surrounded with grout, but rather it is a question of making a grout with a suitable permeability. Cement-bentonite grout is generally well suited to accomplish this task.

The cement-bentonite fabric when set is an irregular honeycomb structure held together with both cured cement and colloidal bonds. It is a highly porous solid with a low permeability that lies somewhere in the cement and bentonite range, from 1×10^{-5} to 1×10^{-9} cm/sec. Typical published values of permeability are listed in Table 1. Vaughan (1973) quotes a coefficient of permeability for a pumpable cement-bentonite grout mix on the order of 5×10^{-8} cm/sec. For low bentonite solid contents the permeability can be expected to be close to 1×10^{-6} cm/sec and for higher bentonite solids content it would be close to 1×10^{-8} cm/sec. This is an area for further testing and research where the water-cement ratio, bentonite solids content and permeability should be established.

Typically used Bentonite

Drilling contractors in the US who do test borings in soil usually carry a high yielding sodium bentonite such as Supergel or Quik-Gel brands on their rigs. This is a finely ground, powered form of bentonite that yields as much drilling mud per sack as possible without additives. Other bentonite powder products have additives to enhance certain mud characteristics such as viscosity, density and filtration. Such additives are probably not detrimental to making a suitable cement-bentonite grout, but are not really relevant or cost effective in such a context. What seems not to be well understood is the feasibility of using coarser grains of bentonite to increase solids content for lowering the permeability in cement-bentonite grout. Would polymer additives be needed for mixing stability, for example?

As a side-note, sodium bentonite absorbs more water than calcium bentonite. According to clay mineralogists (Papp, 1996) the presence of sodium as the dominant exchangeable ion facilitates many interlamellar water layers to be absorbed into the crystalline structure, a phenomenon which does not occur with calcium or magnesium as dominant ions. Sodium bentonite is characterized as capable of absorbing at least five times its weight in water and expands when fully saturated with water to a volume 12 to 15 times its original dry size.

Sodium bentonite powder appears to be the most practicable and efficient to use in this context, but this does not mean that other fillers of different grain sizes and composition could not be used. It is matter of availability and convenience of a good product for this application. For example, in a 50-gallon batch of cement-lime grout, 150 pounds of hydrated lime can be replaced by using about 25 pounds of bentonite powder.

Mix Design Rules

In order to keep field procedures simple the emphasis should be on controlling the water-cement ratio. This is accomplished by mixing the cement with the water first. This is contrary to procedures used at more sophisticated grout plants for compaction grouting and sealing purposes. When water and cement are mixed first, the water-cement ratio stays fixed and the strength/modulus of the set grout is more predictable. If bentonite slurry is mixed first, the water-cement ratio cannot be controlled because the addition of cement must stop when the slurry thickens to a consistency that is still pumpable.

Making cement-bentonite grout in the field is a straightforward process. The most effective mixing is done in a barrel or tub with the drill-rig pump, cir-

Table 2. Cement-bentonite grout mixes				
Application	**Grout for Medium to Hard Soils**		**Grout for Soft Soils**	
Materials	Weight	Ratio by Weight	Weight	Ratio by Weight
Water	30 gallons	2.5	75 gallons	6.6
Portland Cement	94 lbs. (1 sack)	1	94 lbs. (1sack)	1
Bentonite	25 lbs. (as required)	0.3	39 lbs. (as required)	0.4
Notes	The 28-day compressive strength of this mix is about 50 psi, similar to very stiff to hard clay. The modulus is about 10,000 psi.		The 28-day strength of this mix is about 4 psi, similar to very soft clay.	

culating the batch through the pump in 50 to 200 gallon quantities. The rig pump provides the kind of jet-mixing required for getting the job done quickly. Any kind of bentonite powder used to make drilling mud combined with Type 1 Portland cement and water can be used, but the appropriate quantity of bentonite will vary somewhat depending on grade of bentonite, mixing sequence, mixing effort (agitation), water pH and temperature.

Grout mixes should be controlled by weight and proportioned to give the desired strength of the set grout. The conversion factors contained in Appendix H.10. in Dunnicliff (1988, 1993) are very helpful in mix design. Two mixes are given in Table 2 that varies in 28-day strength from 50 psi to 4 psi for water-cement ratios of 2.5 to 6.6 respectively.

The amount of bentonite that is required for the above mixing procedure would vary due to factors mentioned earlier. The amount of bentonite shown in Table 2 should only be used as a guide, but is also handy for estimating material quantities to be shipped to the site. With this method **more** bentonite is required than if water and bentonite were mixed first. This is an advantage from the standpoint of wanting a low permeability. When the bentonite solids content increases, the density increases and the permeability is lowered. A lower permeability is generally preferred since cement-bentonite grouts have a higher permeability than high-density bentonite grout or chip seals. Thus, another good reason for mixing water and cement before adding bentonite.

Old habits die hard, so that some users will insist on mixing water and bentonite powder first. This is normally the way drilling mud is mixed and it yields more slurry per sack of bentonite than the above method. Also, use of hydrated bentonite with cement added last is common practice in grouting technology for ground improvement. Such mixes are highly thixotropic and rely on industrial type mixing plants and methods. The cement content is difficult to control under ordinary borehole installation circumstances.

Mixing Procedure

A rig pump with one suction hose and a return hose fitted with a jet nozzle and a 50-gallon barrel, shown in Figure 2, are the minimum requirements for circulation batch mixing of grout. Paddle or high shear mixers can also be used. A measured quantity of clean water goes into the mixing tub/barrel first and the pumping and circulation starts. Then the cement is gradually added to the water and mixed thoroughly. At this stage the mix is like gray water. Next, bentonite powder is slowly added into the jetting area of the barrel, slowly enough so clumps of bentonite do not form. This should be constantly checked by scraping the bottom with a shovel. When clumps form, slow down and do not add any more powder until they are dissolved. Keep adding bentonite until the watery mix transitions to an oily/slimy consistency. Observe the consistency while mixing and let the grout thicken for another five to ten minutes. Generally, the mix thickens some more with added mixing time. Add more bentonite as required. When it is smooth and like thick cream or pancake batter, it is as heavy as is it feasible to pump. Drips of the grout should then barely come off a dipped finger and should form "craters" in the fluid surface. That is the correct consistency for pumping the grout batch down the tremie-pipe. When possible, withdraw the tremie after each batch an amount corresponding to the grout level in the boring to keep the pumping pressures as low as possible. When mixing grout 100 feet or more from the borehole, thinner consistency would be required, but at the risk of some bleeding.

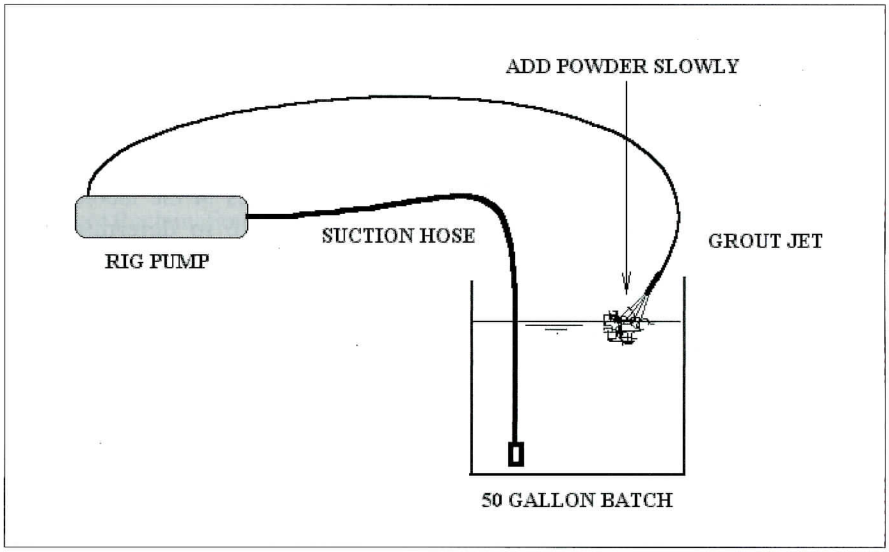

Figure 2. Circulation batch mixing of grout (minimum configuration). After mixing grout is tremied to bottom of the borehole.

Additional Considerations

Strength is often used to characterize a grout for deformation-type instruments, but modulus of deformation should ideally be the basis for judging compatibility with ground conditions. The grout column in a borehole will carry a total axial force smaller or greater than the material it replaced, according to its stiffness. When there is too much stiffness or force, displacements will be diminished and axial measurements can be less than displacements of the surrounding ground. Thus extra grout stiffness for extensometers is much more undesirable than for inclinometers, for example. More care should be taken in making a grout for axial borehole deformation measurements than for lateral deformation measurements.

Instrumentation installations often encounter the combination of both soft and hard ground in the same borehole. Obviously, staged grouting to match the required properties would be the ideal procedure, but this is seldom warranted or practicable. Most of the design and installation challenge lies with deformation measurements in the axial direction of the borehole where large volumes of grout backfill must be placed. So, for extensometers, it is better to err on the softer side of the spectrum.

Lateral displacements of an inclinometer casing are generally unaffected by added grout stiffness. Where the grout column is too stiff the displacements will be distributed over a greater depth interval, but not be diminished in overall magnitude. The same is probably true if the grout is too soft, but there is the additional concern for lack of lateral confinement. Since inclinometer casings generally are under compression, lack of backfill or confinement can produce localized shifts in the borehole, masking smaller actual displacements. So, for inclinometers, it is better to err on the stiffer side of the spectrum.

It may not be possible to achieve a suitable grout with cement for softer clays. Fly ash promises to be a good substitute for reducing the modulus, but more testing is needed in this regard.

Conclusions

Grout backfill should ideally be selected according to the field instrument type being used and the given ground conditions. The reality however is that grouting practices will remain relatively crude and, at best, with only marginal control over the grout properties. Drillers, geologists and engineers alike still have a lot to learn about what is appropriate. We cannot just borrow ideas from drilling mud and grouting technology that have no relevance to what is needed for instrumentation functionality.

1. Avoid using a bentonite alone for a borehole grout. It is not a volumetrically stable material and can influence both piezometer and displacement measurements when it keeps hydrating or desiccating. It is often very difficult to place successfully.
2. A stable grout can be made using cement or fly ash with bentonite. Relatively small amounts of cement or fly ash are used as compared to grouts used for other geotechnical purposes such as compaction grouting and sealing of seepage. Grouting for instrumentation has different property priorities.
3. Grouting for field instrumentation should remain a relatively simple endeavor, using materials commonly available to drillers. However, when working in soils like normally consolidated clay, more attention should be paid to the mix design. Since little information is available on softer grouts, particularly those mixed with fly ash, a few trial batches in the laboratory are appropriate to determine basic characteristics for use on such projects.
4. Grout mixing should start with water and cement (or fly ash) first. Strength and modulus are more predictable that way. Also, and just as important, is that more bentonite solids can be added to the mix to lower permeability where required for sealing.
5. Grout permeability is an issue for piezometers installed in clay. The grout should have permeability no greater than one (possibly two) orders of magnitude above the clay to get representative readings.
6. Grout stability is very important during both the liquid and set conditions. The liquid grout consistency should be as thick as possible, yet liquid enough to be pumpable. This is a property that requires field experience. Field crews tend to err to the more liquid end of the spectrum, resulting in bleeding and possibly cracking when set.
7. Strength is often used to characterize a grout for deformation-type instruments, but modulus of deformation should ideally be the basis for judging compatibility with ground conditions. There is very little in the literature to help us select a grout mix for sealing piezometers in boreholes. Further testing and research is needed (see editor's note below).

Acknowledgements

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P. Erik Mikkelsen, Consulting Engineer, Geometron, 16483 SE 57th Place, Bellevue, WA 98006
Tel: (425) 746-9577
e-mail: mikkelsen.pe@attbi.com

Editor's Note:
John Dunnicliff, Erik Mikkelsen, and Allen Marr have decided to plan and conduct a test program to mix various proportions of cement and bentonite, also fly ash and bentonite, and test for strength, permeability, compressibility and volume stability. The results will be published in GIN as soon as they are available.

Measurement of Pore Water Pressures in Embankment Dams

Arthur D.M. Penman

A Brief History

Developments and application of the art and science of soil mechanics since the publication of Terzaghi's *Erdbaumechanik* in 1925 have enabled the rational design of embankment dams. Terzaghi's description of effective stress drew attention to the need of measurement of pore water pressures, although the position of the phreatic surface had been of concern to dam engineers before that time. In 1907, British engineers installed standpipe piezometers in the Waghad dam in the Nasik Collectorate, India, to check on the position of the phreatic surface under the downstream slope of the dam, following a slip. Subsequently Nagarkar et al. (1981) used the values measured by those piezometers, particularly for the condition of full reservoir, to make a modern analysis of the 1907 slip.

In North America engineers were equally interested in the position of the phreatic surface in embankment dams. Early in the history of the United States Bureau of Reclamation, a need was seen for an apparatus that could delineate the phreatic surface and help determine flow patterns. Bartholomew et al. (1987) have described the history of the instrumentation used to measure pore water pressures. An early example was the Cold Springs 31m high zoned earth fill dam that was completed in 1908. It was fitted with 31 standpipe piezometers of a type called by the Bureau 'porous tube piezometers'. They had a porous cylinder of alundum attached to a standpipe and were very similar to the type now known as the Casagrande standpipe piezometer. The Bureau also developed slotted pipe piezometers that provided a longer intake area and these two types were used extensively in many of their future dams from 1908 onwards. During the 1920s six embankments dams with height from 19 to 97m were fitted with the slotted pipe and the porous tube standpipe piezometers, primarily to check on the position of the phreatic surface when the reservoir was filled. None of these installations could be said to have been designed to measure pore water pressures *per se*: they were intended to measure the positions of the phreatic surfaces.

The Bureau recognised the slow response time of open holes and standpipes, but had been surprised to find in some newly constructed dams that water rose in the standpipes to above the level of the fill before any wa-

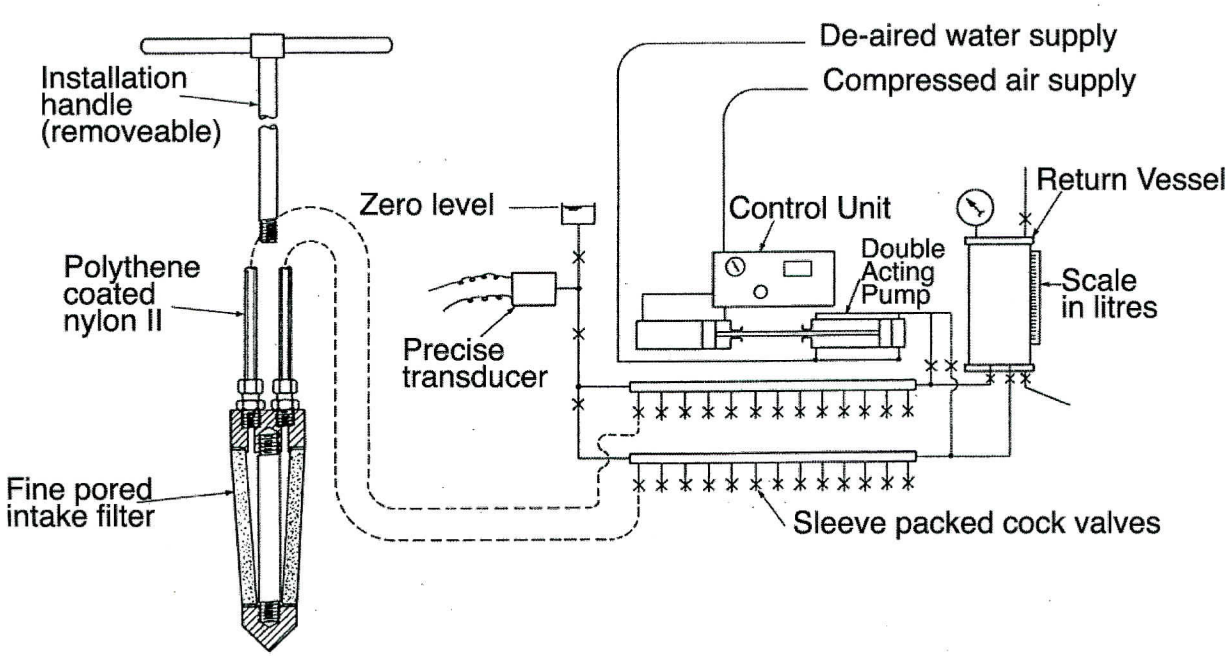

Figure 1. The twin-tube hydraulic piezometer.

ter had been put in the reservoirs. It was in this way that construction pore water pressures were revealed. A saturated fill weighs about twice the weight of water, so there is the potential for the pore water pressure, expressed as a head of water, to rise in a standpipe to twice the height of the fill. If the water rises to only the surface of the fill, the pore water pressure is only about 50% of the total pressure caused by the weight of the overlying fill, ie $r_u = 0.5$. The symbol r_u is used to express the ratio of pore water pressure to applied overburden pressure.

In the 1930s a systematic study of the behaviour of embankment dams was started by the Bureau. Equipment better than the open observation wells or open standpipes was developed. An example was the hydraulic pressure indicator, based on the principle of the Goldbeck earth pressure cell that was described by Goldbeck and Smith (1916). It contained a diaphragm, and water pressure, passing through a porous intake filter, pressed the diaphragm against an electrical contact. Readings were taken by increasing air pressure behind the diaphragm until an electric light showed that the contacts had separated. No attempt was made to use clean, dry air and unfortunately condensed water could bridge the gap at the contacts and keep the light on, so that the operator continued to increase the air pressure. The use of dissimilar metals caused electrolytic action and corrosion, so that eventually a diaphragm fractured. The piezometer was left with the air tube connected to the pressure gauge in the instrument house and after a few days it was found that the pressure gauge was showing a pressure that, with allowance for level difference, was pretty well the previously measured pore water pressure. In this way it was realised that there was no need for the diaphragm and the intake filter could be connected directly to a pressure gauge. It was apparent that the connecting tube had to be completely full of water so that the head difference between the intake filter and the instrument house could be correctly allowed for. It was also clear that a more rapid response time was obtained when the system was completely water filled. To achieve this, a second tube was provided so that water could be circulated round the tubes and intake filter unit. In this way the well-known two-tube hydraulic piezometer was born.

Walker and Daehn (1948) describe ten years of pore pressure measurements that had been made in dams in the USA, while Peters and Long (1981) discuss measurements made by the Canadian Prairie Farm Rehabilitation Administration on more than 50 dams and hydraulic structures since the mid 1930s. The apparatus was described by Walker and Daehn (1948) and in the Earth Manual published by U.S. Dept. Interior (1960).

The Twin-tube Hydraulic Piezometer

This accurate, simple, reliable and long-lasting piezometer was further developed at the Building Research Station and Imperial College in England during the 1950s, installed in dams in Britain and overseas and described by several publications including Penman (1956), Bishop, Kennard and Penman (1960), and Penman (2001). A typical apparatus is shown in line diagramme form by Fig.1. The early ones used polyethylene connecting tubes and coarse carborundum intake filters, of larger area than those used by the USBR at that time to give a better response time. But both tube and filter allowed of the ingress of air, and the need to circulate water to clear air bubbles became a regular chore and gave the apparatus a bad name, from which it has had difficult in recovering. The problem was overcome by the use of fine pored intake filters and connecting tubes made from nylon 11 coated with polyethylene. The modern equipment using these features, has electronic control equipment in the instrument houses that measures the pore water pressures which can be stored for later downloading and/or transmitted to a central station for study and analysis.

An advantage of this type of piezometer is that it is normally installed in trenches cut into the surface of the fill during dam construction, and when buried is no further hindrance to fill placement and will not be damaged by the placing and compacting machinery. Its measuring unit is not buried in the fill and is accessible for checking, re-calibration or replacement if it is found to have the wrong range or is in some way malfunctioning. It can be used for special tests such as measuring the in situ permeability of the surrounding fill, and hydraulic fracture tests, both of which can be of particular use in the core of a dam.

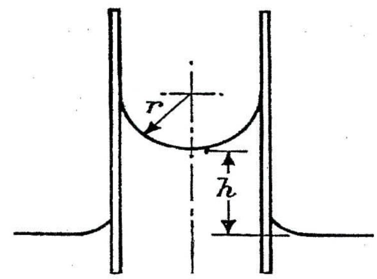

Figure 2. The rise of water in a glass tube.

Surface Tension

Water molecules are very tightly packed together, making water very incompressible and ensuring a strong attraction between molecules. Within a body of water each molecule has equal pull from the attraction of all the surrounding molecules, but at the surface a molecule is attracted equally on all sides and downwards, but not upwards in the direction away from the body of water. All the surface molecules, pulling together sideways, form in effect a thin skin, although there is nothing there except this

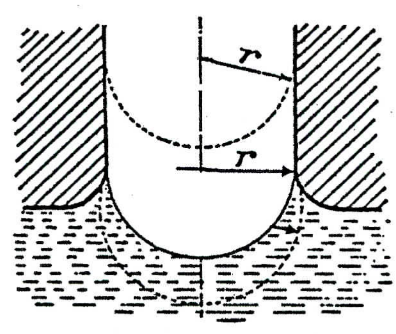

Figure 3. Formation of a bubble at the end of the glass tube.

surface layer of molecules.

Glass and most soil particles are readily wetted by water and this apparent "skin" clings to the object and can exert enough force to draw the surface up to it. If a glass tube is dipped into water, the surface will rise up the sides of the tube, and inside the strength of the "skin" will draw the water into the tube, as shown by Fig. 2. The pull exerted by the "skin" is 2 ã/r, where ã represents the surface tension of the water and r represents the radius of the tube. The value of this pull can be measured by the height h by which the water is raised inside the tube. In a similar way, the "skin" produces a semi-spherical bubble if air pressure inside the tube is increased so as to push the surface down below the end of the tube into the water, as shown by Fig. 3. The smaller the value of r the greater the pressure difference produced, as can readily be demonstrated by several capillary tubes of decreasing diameter. The smaller the tube, the greater the height of water that will be drawn into it. A rough value of surface tension for water at room temperature is 75mN/m causing water in a 1mm diameter glass tube to rise 30mm and with a smaller capillary of 0.1mm diameter, the rise would be 300mm. From this it can be seen that the pores of a soil will draw in water and that gas bubbles trapped within the soil will be at a higher pressure than the water.

The other important effect is that pores exposed at the surface of a soil can act like a series of fine capillary tubes that will remain full of water with the "skin" preventing the ingress of air. An air pressure p = 2ã/r would be required to force the water down into the soil and permit the entry of air. The pore sizes of soil vary from point to point, but a man-made ceramic made from uniform sizes of very small particles can present a surface containing pores all of the same size. If these are sufficiently small, the ceramic when saturated, will withstand an air pressure inversely proportional to the radius of the pores. In this way intake filters for piezometers can be made to withstand considerable air pressures once they have been saturated, without any air getting into or through them directly.

The Jamin Effect and Reducing Permeability

Another aspect of surface tension known as the Jamin effect, is the resistance to flow caused by air bubbles in a tube. At each air/water interface, surface tension causes the air bubble to be at a higher pressure than the water. When a pressure is applied to the end of the tube to cause flow, the curved "skin" is deformed at both ends of the bubble, offering a slight resistance. Sufficient bubbles can cause a significant resistance. Work we did on a length of 3mm inside diameter connecting tube showed that 1000 air bubbles would resist a pressure of one atmosphere, so that use of suction could not clear the tube of the air – an important aspect when wishing to circulate water through tubes.

Gas bubbles in a soil affect its properties. They cause reduction of permeability by blocking voids between the mineral particles though which water would be free to flow if the soil was saturated. This property is being used to form barriers in soil to limit the spread of undesirable toxins from waste dumps and other concentrations of toxic materials. Air bubbles are introduced from pipes put into the soil as though to form a grout curtain. The system is referred to as air sparging.

Bubbles put into sand beds have been used to prevent seismic liquefaction of the sands. Shunta Shiraishi (2000) described the behaviour of the Bandai Bridge over the Shinano River in Niigata, Japan during the earthquake of 16 June 1964. While the river banks moved towards the river by typically 8m, there was little damage to the bridge arches supported by caissons that had been built in 1927 using compressed air. Air bubbles forced into the sand then, 37 years earlier, still remained in the positions where they had been first held in the clusters of sand grains, preventing liquefaction of that sand. Tests made of blowing air into sands had shown that the effect of the bubbles remained the same during five years of testing, and the method had been patented both in Japan and in the United States.

Unsaturated Fill and the Gas Bubbles

Bubbles in soil are commonly of air, but methane and other gasses may be present, and where this is likely, the term 'gas' is used in this text.

When soil particles are transported to the construction site by water in pipes or flumes as were used for the construction of hydraulic fill dams in the old days, and for the deposition of tailings today, the resulting fill may be nearly

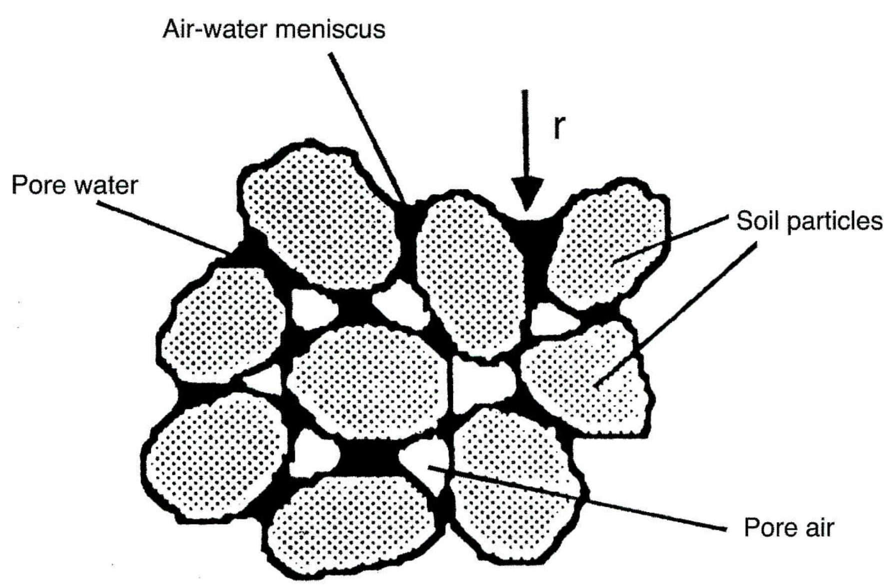

Figure 4. Diagrammatic representation of air within compacted fill.

saturated, but the placing of fill by machine ensures that air will be included. The stronger the borrowed fill the greater will be the amount of air trapped when the lumps of the borrowed fill are pressed together by the compacting machines. Bubbles included within the fill are surrounded by particles with water between them and the "skin" will ensure that the air is at a higher pressure than the pore water. This is illustrated diagrammatically by Fig. 4. If a sufficient total pressure is applied, i.e. if the position is low down in a dam and the overburden is considerable, the air or gas may be driven into solution in the water. Should there be any flow through the fill, this water may carry the gas in solution away, and leave a fill that is truly saturated. But usually this does not occur and the presence of air bubbles can maintain pore water pressures when the total pressure on the fill is reduced.

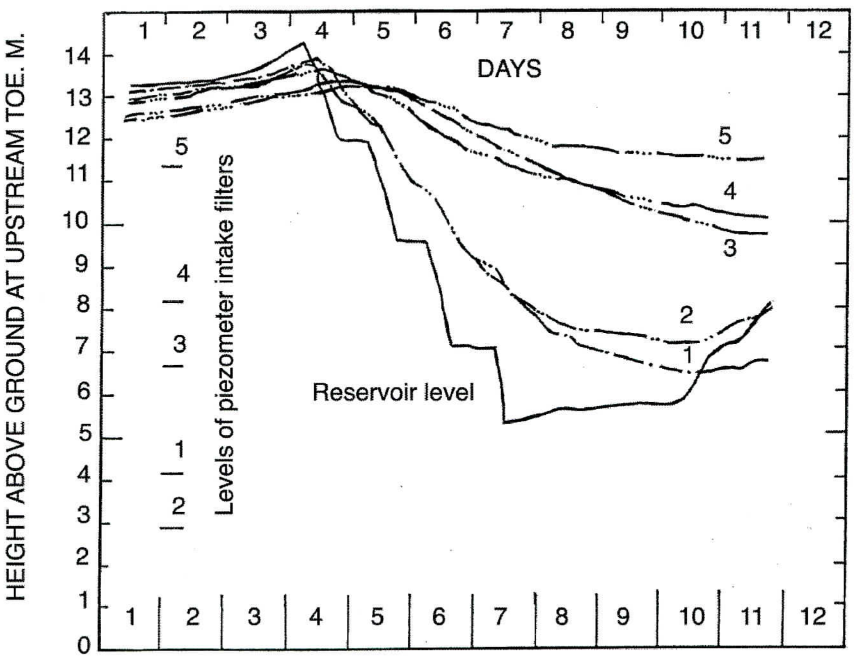

Figure 5. Glen Shira – first drawdown test.

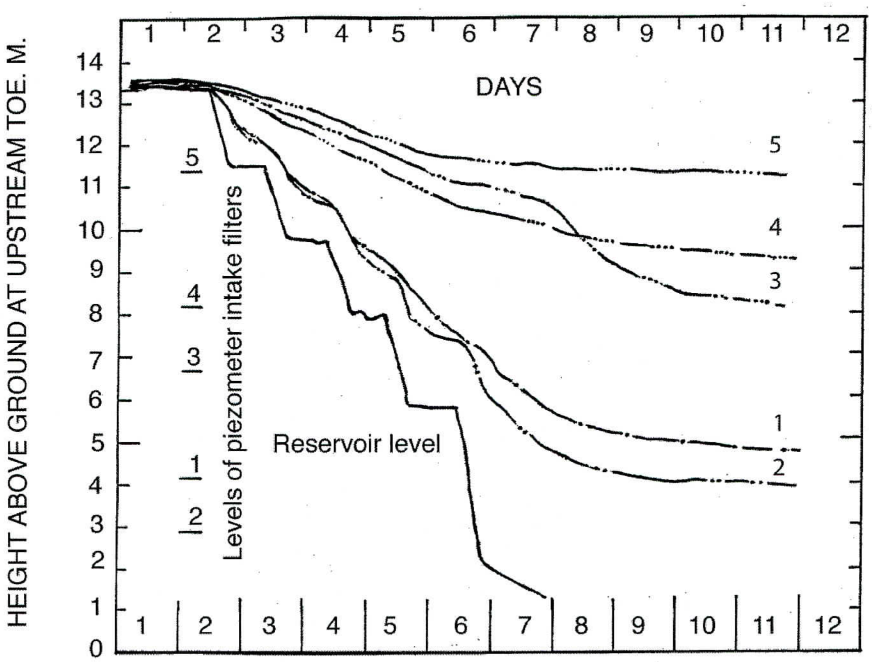

Figure 6. Glen Shira – second drawdown test five years later.

Sudden Drawdown

The question used to be asked about conditions in an upstream shoulder, *'Will the pore water pressure after drawdown remain the same, or will it be reduced by the reduction in total stress caused by the removal of pressure from the reservoir water?'* If the former, then the factor of safety must fall to a very low level. It seemed more comfortable to assume that the pore water pressure would fall by the amount of reduction of total pressure. Tests that we made on the Glen Shira dam showed that the pore water pressures remained remarkably high after drawdown and the explanation given was that the pressures were being maintained by expansion of the trapped gas bubbles in this new fill. After a few years, it was expected that because the upstream shoulder was under water, it would become saturated, so when an opportunity arose some years later to repeat the drawdown test, some surprise was expressed when it was found that the pattern of pore water pressures was very similar to that found during the first test. An account of this was given by Penman (1995) and the measured values are shown by Figs 5 and 6.

Measuring Suctions

Compacted fill must initially have a pore water suction to provide the effective stress needed to give the fill the strength to support the weight of the placing and compacting machinery. This aspect is all too readily ignored, and should not be, because the changes in pore water pressure that occur during subsequent construction and operation of a dam are changes from the initial value. It is all too common to assume that the initial pore water pressure is atmospheric, which is wrong.

Fill won from a borrow pit, transported to site, spread and compacted must be only part saturated, even if the original material was approaching saturation. At best it is placed as lumps of the original, but these cannot be pressed together by the machines without trapping air. The result is a partly saturated fill, with the pore water pressure exhibiting a suction, i.e. it will be below ambient atmospheric pressure. A porous ceramic that used to be made by Aerox had a porosity of 46%, a permeability of 2.9 x 10^{-6}cm/s and it required an air pressure of 200 kN/m^2 to blow water out of its pores. The pressure required to force air through a saturated ceramic can be a measure of the pore sizes. The carborundum intake filters had a blow through pressure of only about 5 kN/m^2, limiting their use to saturated soils below the water table, or for the measurement of pore gas pressure. The fine pored filters are often described as having a *high air entry value*, meaning that it requires a high pressure to force air into them when saturated, whereas conventional coarse filters are referred to as *low air entry filters*. A comparison of the use of fine pored and coarse pored intake filters was given at Chelmarsh dam where the two types were placed side by side in the fill during construction. The results shown by Fig. 7 reveal that the pore water pressure measured by the fine pored intake filter was initially a suction of nearly –7m water head, whereas the coarse filter, measuring the pore gas pressure, gave an initial pressure of only about –0.3m head of water. As the height of the fill above the piezometers increased, both pressures increased, and had the piezometers been installed at a lower level, the two would have become the same when all the gas had been driven into solution when both filters would have measured the pore water pressure.

It is clear that to measure the pressure or suction in the pore water, the ceramic of the intake filter must be in close contact with the fill. The cylindrical filter used by Bishop, shown by Fig.1, was made tapered so that it could be pressed into the fill to make close contact along its sides. The unit itself must be fully saturated with its connecting tubes also full of air-free water, so that when it is pressed into the fill, the pore water will connect with the water in the pores of the ceramic and the suction can be measured by the equipment in the instrument house. The fine pored ceramic is best saturated initially by allowing air-free water to be sucked into its pores from one side only to avoid having air trapped within the thickness of the ceramic itself. This can be done by attaching short lengths of connecting tubing to the unit and standing it in air-free water with the top ends of the tubes above the water to allow of the escape of air. It should be left for some time, suitably overnight. The unit would be taken on to the fill while still in the water, and attached to its connecting tubes, previously completely filled with air-free water, with water flowing from them, to ensure an air free connection. Then, at time zero, the unit should be pressed into the fill, possibly using a small tapered hole made by driving in a special steel former, while pressures are measured at the instrument house. In this way a correct zero reading will be obtained. On no account should any sand be placed in the hole or used to surround the intake filter. It was the custom to place cylindrical intake filters in boreholes surrounded by sand to obtain a larger intake area, but if pore water suctions are to be measured, it is essential that the fine pored saturated ceramic is pressed tightly against the fill so that water in the pores can make contact with the fill and exclude gas.

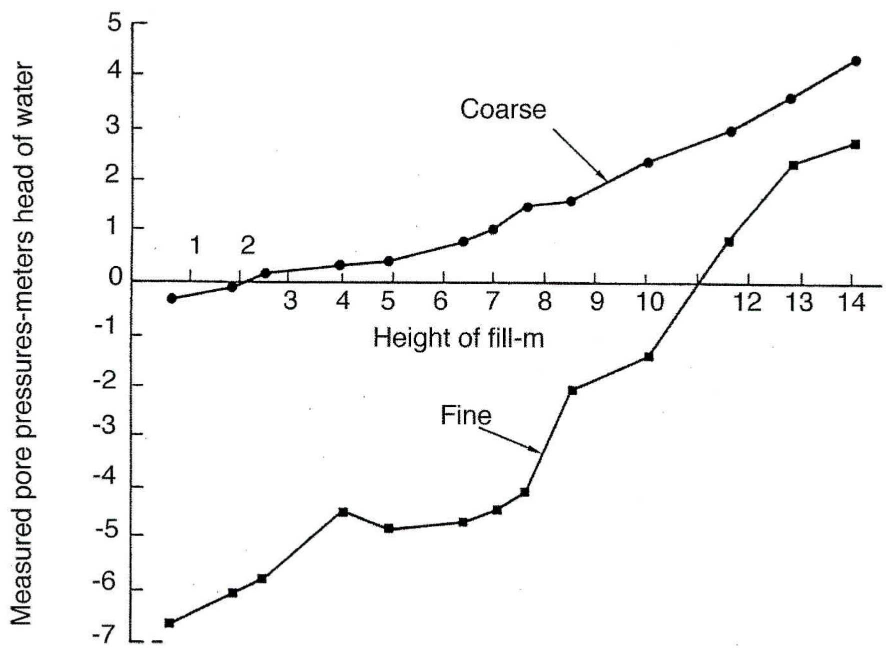

Figure 7. Pore pressures at Chelmarsh Dam.

It might be noted here that air-free water should not be produced by boiling. Several commercial suppliers used to offer large water boilers so that clean tap water could be brought to the boil to drive off dissolved air. A difficulty is that boiling usually produces chemical precipitate, which pollutes the water and when it is used in a pore pressure apparatus, the chemical precipitate can form blockages in the small diameter tubing, which cannot be readily removed. It is better to remove the air by reducing the pressure over the water until it boils at room temperature. It can then be used immediatly without having to allow it to cool, and it contains no impurities that were not in the water before treatment. A high standard of de-airing can be obtained with the Nold DeAerator which agitates the water while it is under reduced pressure, giving a very rapid rate of de-airing, allowing of an ample supply of air-free water for use in the field.

Pore Water Pressures in "Clay" Cores

Dam cores made from fine-grained material, often referred to as clay cores, although the percentage of clay sized particles may be fairly small, should be placed wet enough to develop fairly high pore water pressures during construction. There is no need to expect any shear strength from a wet core. It is supported by the shoulders and its purpose is to act as a water stop and prevent excessive amounts of reservoir water from reaching the downstream shoulder. It should have a low enough strength to prevent vertical support from the shoulders, to ensure that its weight acts at all levels. An ideal situation would be for the developed pore water pressures, in terms of head of water, to be at dam crest level at the end of construction, then during reservoir filling, the water level of the full reservoir would be balanced by the pore water pressure, making the core completely water tight. With time flow lines would be expected to form as the core pore water drained towards the downstream shoulder. At one time it was expected that a fairly straight phreatic surface would develop, sloping down from reservoir level to the level of the downstream stilling basin. In fact this does not happen, because the permeability of the core material is controlled by its effective stress, which presses the grains together, causing consolidation. As the downstream side of the core drains into the downstream filter of the dam, the downstream side of the core consolidates, decreasing its permeability. In this way, the zone of lowest permeability is on the downstream side, and much head loss through the core from upstream to downstream occurs through this zone, leaving the pore water pressures on the upstream side of the zone still at reservoir level. This produced a steeply curved phreatic surface, with the pressures at reservoir level extending though perhaps ¾ of the core width. This used to be regarded as a sign of faults in the core, allowing reservoir water to pass unrestrained to beyond the half way point, but it is now understood to be simply due to the variation of permeability caused by the variation of effective stress due to drainage from the downstream side.

A theoretical study of this phenomenon has been published by LeBihan and Leroueil (2002) who found a similar distribution of pore water pressures through the core.

Diaphragm Piezometers

These instruments containing a diaphragm use it as part of the measurement system and because they become buried with the instrument, are inaccessible for repair, replacement or general maintenance. The pneumatic piezometer has become popular because of its low cost and the use of a dry gas in the connecting tubes avoids risk of freezing, less elaborate equipment is required for taking readings, and no height allowance has to be made. But faults with the diaphragm can cause malfunction, which cannot be corrected. Misuse in the application of gas pressure, and the introduction of water or dirt into the system can prevent correct values from being measured, and in general they cannot be used to measure suctions.

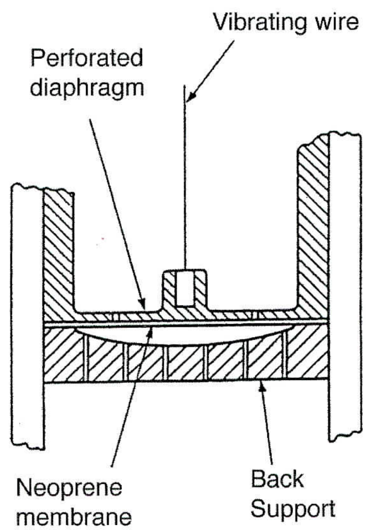

Figure 8. The DiBiagio arrangement enabling a check of zero reading.

The vibrating wire strain gauge is a very robust, accurate apparatus and when used in a piezometer produces a very satisfactory instrument. Manufacturing techniques have been developed to the stage when the piezometer remains stable and it has the advantage that it can accept almost any length of connecting cable without its calibration being affected. It can be upset by electrical storms, although methods have been developed to minimise this hazard.

During the early days of development creep in the diaphragm, wire or supporting frame could affect the zero reading and this used to be regarded as a drawback. It is useful to remember that Dr. DiBiagio, who has done a very great deal of the development of the vibrating wire gauge and of the vibrating wire piezometer, devised a way of checking both zero reading and the calibration of the instrument in situ. He used two tubes in the connecting cable so that the inside of the piezometer could be ventilated, kept at a known pressure and filled with an inert gas such as dry nitrogen to both keep the vibrating wire and other working parts in a non-oxidising environment and remove traces of moisture or other gases that might get into the buried gauge. This enabled the calibration to be checked by applying known backpressures and recording the change of readings. But to check on the zero reading, he devised the arrangement shown by Fig. 8, which he described in 1974. Small holes are drilled through the metal diaphragm which is made waterproof against the pore water by a thin neoprene membrane. To check on the zero reading, after calibrating the diaphragm, the applied gas pressure was increased carefully until it exceeded the pore water pressure. When readings revealed no further change, indicating that the membrane has been pushed away from the metal diaphragm, it was then in its zero position. Although this ingenious development gave the advantages of correct calibration and zero reading, it has not been widely used because modern vibrating wire piezometers that are sealed during manufacture cause less trouble than the units with the gas tubes fitted inside the connecting cable. Slight damage to the cable in the field can allow the ingress of dirt and water, whereas the tough armoured electrical cable without tubes appears to have proved to be more reliable. This reflects favourably on the stability of the modern instrument, which maintains its reliability for prolonged periods.

Modern diaphragm piezometers for installation in the cores of embankment dams should be supplied with high air entry filters, which must be fully saturated before use, with the cavity between filter and diaphragm completely filled with deaired water, and there must be intimate contact between filter and soil to ensure hydraulic continuity. The piezometer must be pressed into the compacted core material for this purpose. In cases where a flat disc filter is used at the end of piezometer, great difficulty can arise in obtaining corect contact between the flat surface of the filter and the base of the hole into which the piezometer is inserted. The filter should therefore daylight on the conical nose or on the cylindrical side of the piezometer. Ceramic filters are now available with such fine pores that they require a pressure of 1,500kN/m² (220 psi) to force air through them, once fully saturated. A special piezometer has been developed by Ridley and Burland (1993 and 1995) that has been used to measure pore water pressures as low as -1,200 kN/m² (- 170 psi).

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Arthur D. M. Penman,
Chartered Engineer, Sladeleye,
Chamberlaines, Harpenden,
Herts AL5 3PW
Tel and fax: +44-1582-715479
e-mail:
admp@sladeleye.freeserve.co.uk

Correction

In the article by Bengt Fellenius in the September 2002 issue of Geotechnical News, pages 25-29, we incorrectly printed Figure 7 twice (on pages 26 & 27). We neglected to print the correct Figure 6 on page 26. It appears below.

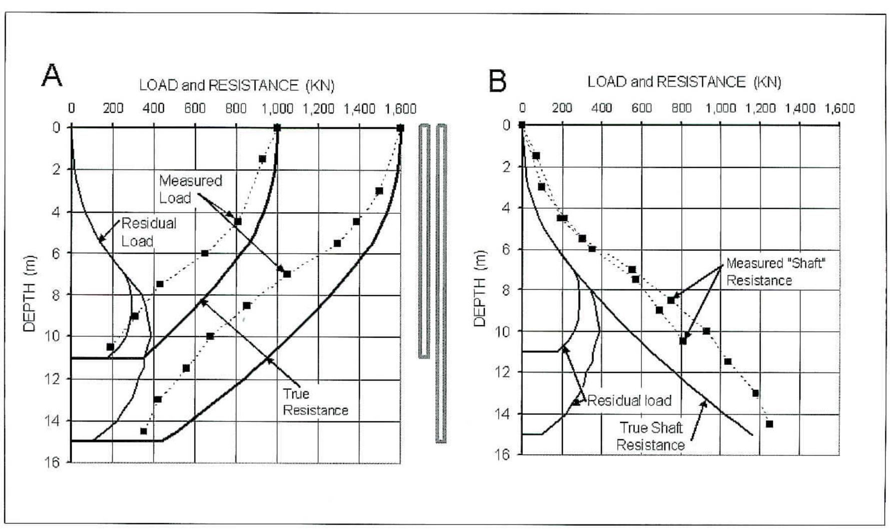

Fig. 6 Results of the Analysis of Both Piles (Data from Altaee et al, 1992)