Geotechnical Instrumentation NEWS

Geotechnical Instrumentation News

John Dunnicliff

Introduction

This is the forty-seventh episode of GIN. The episode includes discussions of an article on embankment dam instrumentation, which was in the previous episode of GIN, together with the author's reply. These are followed by three articles by regular contributors to GIN: design of pile foundations, by Bengt Fellenius; cost of geotechnical instrumentation, by Gord McKenna; and guidelines on selecting electrical cables, by Barrie Sellers. Thank you to all three for your willingness to help keep GIN alive.

Some Disappointing Total Stress and Pore Water Pressure Data

The previous episode of GIN included an article by Ali Mirghasemi, describing measurements of total stress and pore water pressure at Karkheh Dam in Iran. He told us about his experiences with a large number of earth pressure cells installed in the core of the embankment dam, and concluded that "no consistent data were achieved". He also described pore water pressure measurements with both open standpipe and vibrating wire piezometers which were inconsistent, and said, **"The author will welcome any comments and discussion that may help to explain the differences".**

I was both puzzled and fascinated by the contradictions, so sent Ali's article to several colleagues prior to its publication, asking them whether they could shed any light on the issues. I've received discussions from Elmo DiBiagio, Erik Mikkelsen, Arthur Penman and Barrie Sellers, and one from Louis Marcil of Roctest, the manufacturer of the instruments that were installed at Karkheh Dam. I've also received an unsolicited discussion from Donald Babbitt. All six discussions are published here, together with the author's reply. This has been, for me, an interesting and rewarding experience, as all the discussions are relevant and helpful. Thank you to all the discussers for making the effort to help.

I'm left with two 'umbrella' questions:

1. **What is the best way for manufacturers to help users select the most appropriate instruments for their application?** Manufacturers of geotechnical instruments are a resource that should not be overlooked when selecting appropriate instruments for any particular project. However, users can't expect that manufacturers will put themselves in the same position as geotechnical project designers. It seems to me that if users have any uncertainties about which instruments are appropriate, there needs to be interaction with manufacturers so that the decision can be made mutually. Is this practicable?
2. **How can manufacturers help users to learn about the most appropriate installation methods?** For example, in the case of Karkheh Dam, earth pressure cells and piezometers. I've always contended that manufacturers can provide explicit details about such installations as strain gages on steel, because there are no geotechnical variables. However, manufacturers cannot provide explicit details on how to install their instruments in geotechnical surroundings, because they can't possibly know all the details that are required to do so—this is the job of the user. But of course manufacturers can provide general guidelines. In the experience of manufacturers and users, is there a need to improve interaction to ensure the most appropriate installation methods? If yes, how?

I'm not intending to close the opportunity for discussions of Ali Mirghasemi's article with this episode of GIN. If readers have more input that might help to explain the contradictions, or if any feel like putting forward some answers to the above two 'umbrella' questions, I very much hope that you'll do so. Watch this space!

Design of Pile Foundations

Bengt Fellenius, a regular contributor to GIN, has sent me an article that resolves lack of understanding relating to the design of piles subjected to drag loads.

Cost of Geotechnical Instrumentation

Another regular contributor, Gord McKenna, gives us some pragmatic rules of thumb that are applicable to the cost of geotechnical instrumentation.

Help with Selecting Electrical Cables

The article by Barrie Sellers, yet another regular contributor to GIN, gives nuts-and-bolts guidelines to help us select which electrical cable might be most suitable for each particular application.

Two Reminders for Your Date Book

The next instrumentation course in Florida will be on March 18-20, 2007 at St. Petersburg Hilton *(www.stpetehilton.com).* Details of the course will be on *www.doce-conferences.ufl.edu/geotech* as soon as they are available.

The next international symposium, *Field Measurements in Geomechanics (FMGM),* will be held in Boston, Massachusetts on September 24-28, 2007, at the Park Plaza Hotel. W. Allen Marr of Geocomp Corporation *(wam@geocomp.com)* and Prof. Charles Dowding of Northwestern University *(c-dowding@nwu.edu)* are serving as co-chairs of the organizing committee. More information will soon be available on *www.geoinstitute.org,* under the "Upcoming Events" tab. Anyone interested in helping to organize and carry out this event is encouraged to contact Allen or Chuck directly.

For more information about FMGM symposia, please visit *www.fmgm.no.*

'Meaningful' Books

This has nothing at all to do with geotechnical instrumentation (but neither has cricket!). I'll give my excuse for including it later.

The most recent topic for discussion at a local 'discussion group' was, "The three most meaningful books that we've read" - the word "meaningful" to be interpreted in whatever way a person wished. My books were:

- *A Fortune-Teller Told Me*, by Tiziano Terzani. Non-fiction. Terzani, a middle-aged Italian, was the Middle East correspondent for a German publication, and had been so for many years. His life was plane-taxi-hotel-taxi-plane. On a whim, he went to a Chinese fortune-teller in Hong Kong who, having established credibility by recounting a strange and unique experience that Terzani had had previously, told him that under no circumstance should he fly during a whole year 16 years hence – "not even once". Despite the huge impact on his normal life, Terzani eventually accepted the prediction of doom, and spent that year traveling and reporting throughout the Middle East, without flying. His tales about people and places are extraordinary. And he visited fortune-tellers whenever he could! I won't spoil it for you by telling about any air accidents during that later year! Now to my excuse – it was Giorgio Pezzetti, who took the lead role in organizing FMGM-1995 in Bergamo, Italy, who recommended the book to me, so there **is** an 'instrumentation connection'!
- *A Prayer for Owen Meany*, by John Irving, who is probably best known for "The World According to Garp" and "The Cider House Rules", both of which were made into films. Fiction. Impossible in a few words to say why I think this is so very meaningful. One reviewer wrote, "What better entertainment is there than a serious book which makes you laugh?" Religion, precognition, and an unforgettable and endearing lead character. John Irving at his best.
- *Ishmael*, by Daniel Quinn. Fiction. A gorilla is released by his owner, and sets himself up in rented space as a 'teacher' – in essence a psychologist. Perhaps it can be belittled as a bit of pop-psychology, but a meaningful reversal of human/animal relationships nevertheless.

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.*

Here's mud in your eye! (USA). Thanks to Charles Daugherty for this. But it sounds a bit Irish to me!

Discussions of "Karkheh Dam Instrumentation System – Some Experiences"

Ali Asghar Mirghasemi

Geotechnical News, Vol. 24 No. 1, March 2006, pp 32-36

Donald H. Babbitt

In the mid 1960s, 27 - 18-inch diameter Carlson type electrical soil-stress meters were installed in core and transition zones of 770-foot high Oroville Dam and 15 - 30-inch diameter custom built static/dynamic electrical stress meters were installed in the downstream shell zone (O'Rourke 1974, Cal DWR 1974). The installation procedures were similar to those used in Karkheh Dam and the stress readings obtained were also questionable.

Kulhawy and Duncan (1972) performed a finite element analysis of the embankment and compared the results with the instrument readings. They found good correlations with the measured movements, but not the stresses. The extremes in their Table 2 are a major principle stress in the downstream match within 10 percent and a factor of 7.5 variance in minor principal stress in the upstream transition zone. They provide compelling arguments of why the stress measurements could not be correct.

Two of Mirghasemi's three issues for not being able measure stresses in Karkheh Dam, the inclusion effect and different compaction, likely apply to Oroville Dam. His third issue, possible rotation of the stress meters, is unlikely, because Oroville is a coarse, stiff embankment. The core contains cobbles up to 3 inches in diameter and cobbles in the transition and shell zones are greater than 6 inches in diameter. This coarseness led to a third possible source of error; the stress meters were surrounded by 2 inches of selected fine material to protect them and to preclude coarse particles producing unrepresentative stress concentrations at the faces of the meters.

The questionable stress readings that ultimately developed were a concern during the very thorough design process, that included discussions with Dr. Carlson; and meticulous care was used in installing all the instruments in the dam.

References

Cal DWR (1974). California State Water Project, Bulletin 200, Volume III, State of California, Department of Water Resources, Sacramento, November

Kulhawy, F.H. and Duncan, J.M. (1972). Stresses and Movements in Oroville Dam, Journal of the Soil Mechanics and Foundation Division, ASCE, July

O'Rourke J.E.(1974). Performance Instrumentation Installed at Oroville Dam, Journal of the Soil Mechanics and Foundation Division, ASCE, February

Donald H. Babbit, Consulting Civil Engineer, 3860 West Land Park Drive, Sacramento, CA 95822 USA.
Tel. (916) 442-0990,
email: donbabbitt@sbcglobal.net

Elmo DiBiagio

The article by Ali Mirghasemi is a vivid reminder that implementation of geotechnical instrumentation projects is not always an easy task. This is especially true for what I consider to be one of the most difficult of all measurements, i.e., the measurement of total stress in an embankment dam. My comments are, therefore, restricted primarily to the earth pressure measurements reported by the author.

The article does not contain sufficient measurement data to evaluate the correctness of the stress measurements or to pin-point the reasons for the reported inconsistent data. It particular, it would be helpful to study the performance of the pressure cells in the lower part of the dam during placement of, say,10 to 20 meters of fill over the instruments. In the lower portion of an embankment dam, the stress field during construction is essentially 2-dimensional if the valley is wide and not predominately V-shaped. In this case, it is possible to calculate from the density and thickness of the fill what the measured stresses should be. This procedure is commonly used to check the performance of pressure cells installed in high embankments.

The author lists three reasons for the inconsistent earth pressure data, disturbance of the stress field caused by the instrument itself, problems associated with compaction of the surrounding fill and rotation of the instruments during subsequent compaction of layers over the instruments. I agree with these conclusions. However, in my opinion the principal reason is the manner in which the instruments were placed in the embankment. My conclusion is based solely on the photograph, Figure 4, which shows the pressure cells being installed in small rectangular holes or slots with near vertical sides excavated in the embankment. Differences in compaction (and compressibility) of backfill placed over the instruments relative to the surrounding soil will lead to arching in the overlying fill resulting in either measured pressures that are too high or too low. A better installation procedure would be to form a wide bottomed excavation with flat sloping sides in all directions and then install the instruments in the bottom of the excavation, not in small holes as shown in Figure 4. The size of the excavation depends on the number of instruments to be installed and a significant amount of earthwork may be required. For example, for installation of five pressure cells, an excavation of the order of 10 m x 10 m and 1 m deep would, in my opinion, be adequate.

The potential for rotation of the pressure cells in the core of Karkheh Dam is very likely because the core material (a mixture of 60% high plastic clay and 40% gravel) is indeed plastic and would be subject to large shear deformations during compaction or as a result of the kneading action in the fill caused by heavy construction traffic. On one NGI project we fitted miniature inclinometers to earth pressure cells installed in the core of a dam with a moraine core. In one zone where the moraine had a high water content because of a rain shower during installation of the instruments one of the pressure cells rotated 20^{o}.

In summary, measurement of total stress in an embankment is extremely difficult and should not be done unless absolutely necessary and, if necessary, particular care must be given to installation details. For a large project like Karkheh Dam (510 earth pressure cells installed) the cost of a full scale trial embankment to study installation details would be justified.

I wonder if the author misunderstood the drawing in Figure 3 when installing the instruments. It appears that he may have had a large excavation with sloping sides but dug holes in the bottom for installation of the instruments. The drawing can be interpreted in two ways, i.e., instruments placed on the floor of the excavation or set in cubical sockets beneath the floor.

I would like to make one brief comment on the reported discrepancy between pore pressures measured by open standpipe piezometers and vibrating wire piezometers. Again, I don't feel competent to comment in depth on this without seeing all the measurement data, in particular the initial measurements prior to filling of the reservoir. However, I would like to make one important observation. The vibrating wire piezometer used at Karkheh Dam, Model PWS, has a pressure sensor with a circular membrane mounted inside a cylindrical housing. The housing is relatively thin-walled and in the manufacturer's catalog the housing is described as a "slim housing" model. In a high embankment the earth pressure acting on embedded instruments can be very large, so large in fact that deformation of the housing of an instrument like the Model PWS piezometer may actually deform the pressure sensing element inside the instrument as well. The end result could be a shift in zero-point and/or a change in the calibration characteristics for the sensor. I illustrated this point once at an instrumentation course. I placed a thin-walled vibrating wire piezometer on the floor and put my foot on it. The zero point changed by a very significant amount. The lesson to learn from this is simply: Install robust instruments in high embankments.

I wish to express my thanks to Ali Mirghasemi for reminding us that geotechnical instrumentation is not always straightforward.

Elmo DiBiagio, Norwegian Geotechnical Institute (NGI), P.O. Box 3930 Ullevaal Station, N-0806 Oslo, Norway. Tel.: (+47) 22 02 30 00,
Fax: (+47) 22 23 04 48,
email: Elmo.DiBiagio@ngi.no

Louis Marcil

In his article, Mr. Mirghasemi reports discrepancies in results between two pairs of neighbouring piezometers installed at the Karkheh dam. At the request of Mr. Dunnicliff, we submit hereafter a few comments derived from our instrumentation experience. It is hoped that they will help evaluate possible reasons for these discrepancies.

In Figures 5 and 6 of his article, Mr. Mirghasemi presents readings obtained from two pairs of neighbouring piezometers, each comprising an open standpipe and a vibrating wire piezometer. In both cases, readings from the vibrating wire piezometers are of lesser value by approximately five metres than the ones from the standpipe piezometers. This difference is constant over a period of four years. Assuming that the standpipe piezometers provide accurate readings, one would be tempted to conclude that the divergence is due to a problem of defective functioning of the vibrating wire instruments. Our experience shows that this is most improbable. From a point of view of design, vibrating wire sensors cannot produce such a response with a constant negative offset over such a long period of time. This mode of instrumentation provides either very good results, readings that are unstable, or no readings at all. In fact, the reliability and long-term durability of the vibrating wire principle are the reasons for the widespread use of this mode of instrumentation.

The accuracy of the piezometers at Karkheh dam is ± 0.5 % for a full scale of 1700 kPa, which means that this factor could contribute to no more than ± 0.85 metre of the actual difference of 5 metres.

Amongst causes of error in readings occasionally mentioned to us, such as voltage surge due to lightning, water infiltration, accidental overpressure after installation of the piezometer, and zero referencing, only the last of these can have a significant effect on the actual difference in readings. It must be remembered here that vibrating wire piezometers are instruments that provide relative measurements - not absolute - so that each reading must be referenced to an initial reading considered as the zero reading. This zero reading must be taken after complete dissipation of the overpressure momentarily induced by the installation of the high air entry filter on the piezometer. This may take over 24 hours. In the present case, we recommend referring to the initial readings - the readings taken before and after installation of the filter - and to quantify accurately the undissipated overpressure in order to determine the importance of this cause of error.

Other causes of error should also be considered. For instance, measurements done in standpipe piezometers, though generally considered reliable, must be verified. This should include measuring the tape of the water level indicator as well as the stickup of the standpipe, and making sure that the sealing around the standpipe is efficient in order to make sure that the standpipe piezometers measure the pore pressure in a given point, as the vibrating wire piezometers do, and not the level of the free water surface. The peculiar fact that the difference in readings between both pairs of piezometers are equivalent raises the point that further investigation is required as to what these pairs have specifically in common, e.g. where and how they were installed, and how they are read.

From our experience, it is most improbable that the discrepancies be essentially explained by the vibrating wire piezometers themselves. All manipulations related to both types of piezometers should be considered. Ideally, potential effects of each cause mentioned in this discussion should be verified and quantified.

Louis Marcil, Product Line Manager and Technical Services, Roctest Ltd, 665 Pine Street, Saint-Lambert, QC, Canada, J4P 2P4. Tel: (+1450) 465-1113, Fax: (+1450) 465-1938, email: lmarcil@roctest.com

P. Erik Mikkelsen

I thank the author for sharing his experiences, especially the disappointing and puzzling observations. My experiences with free field total pressure cells have also been disappointing and the cost of their installation in the clay core of a dam is not justified, in my opinion. The results are rarely reliable.

The reliability of piezometer data however is another matter. Piezometer data are generally reliable and I would like to comment on why, however puzzling, open standpipe piezometers (OSP) and vibrating wire piezometers (VWP) or electrical pressure sensors are not measuring the exact same pore pressure. Your article indicates the following:

- Figures 5 and 6 show quite consistently, regardless of the difference in measured pressures by OSP and VWP, that the upstream pore pressures are lower than the downstream pore pressures.
- The measured pressures are also generally higher than the reservoir level.
- The difference between OSP and VWP readings are greater at lower reservoir levels.

From this I conclude that there are **considerable amounts of air in the pores** and that an anticipated seepage gradient (saturated) through the core has not been established at elevation 165 m. The pore pressure is being influenced by three factors:

- Vertical embankment stress and resulting consolidation
- Variable stress from reservoir pool causing consolidation
- Migration of pore water and air due to above stresses and hydraulic gradient

There are several types of moisture or water in the compacted clay in addition to the air, the most significant here being **the adhered water and the free (capillary) water containing air** as shown schematically in Figure 7. The adhered water is a film covering the soil particles, and forms strong menisci forces in the corners where particles meet and contain no air. Most literature on the subject seems to explain this in terms of natural conditions in the capillary zone of low permeability soils where suction is present. I have not seen discussions on this subject for higher pore pressures (45 m), but it seems that in view of the results for Karkheh Dam there still is a significant difference at higher pressure, and that is what is being measured.

Saturated permeability is significantly lowered by the presence of air in the clay. The air has decreased the per-

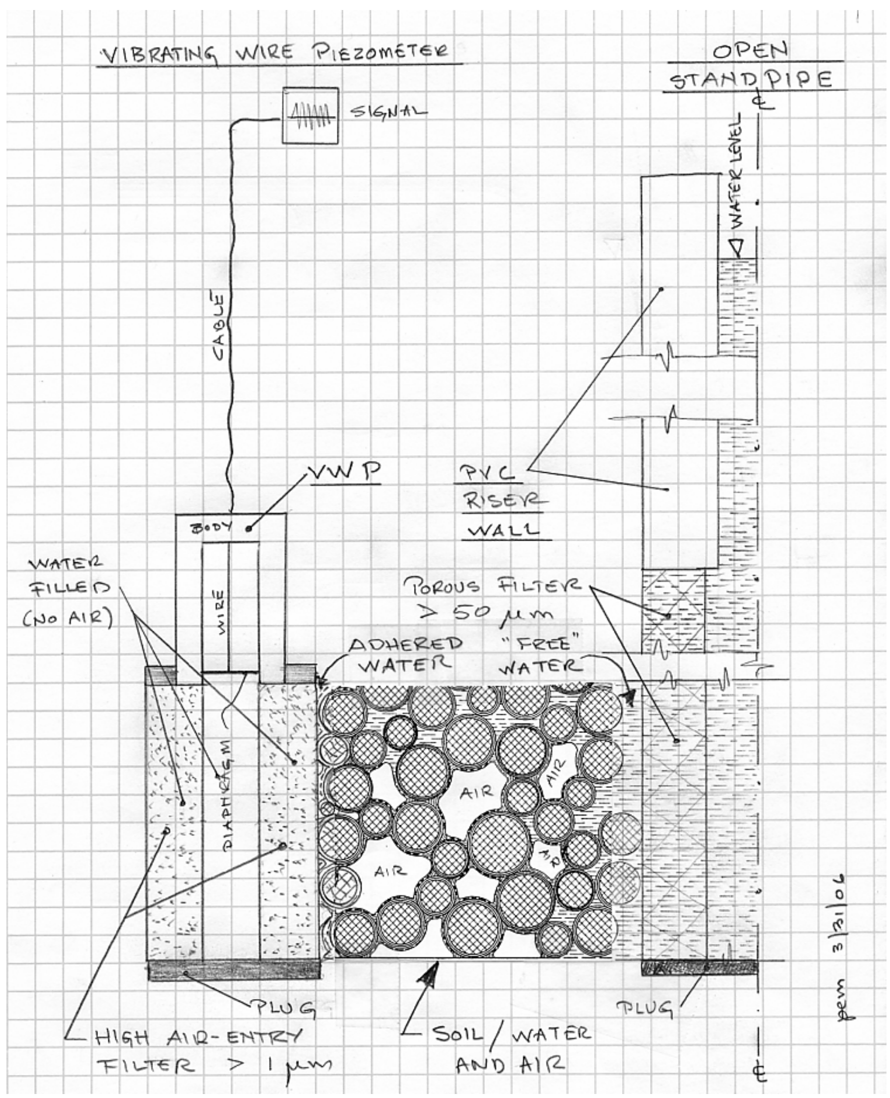

Figure 7. Schematic of pore air/water measurements with open standpipe and vibrating wire piezometer (VWP).

meability and slowed the consolidation process. In this case the standpipe is not effective as a vent for the pore air as one might have assumed. In the past, I have thought of the standpipe as being a natural vent for the pore air, but never thought very much about how the pore air would get there. Based on these measurements the pore air is not readily escaping, at least not yet. The situation will change when saturation eventually occurs, increasing permeability and boosting consolidation rates.

Therefore the difference of 3 to 5 meters of head (30 to 50 kPa) at elevation 165 m may be caused by the difference in the way the two instrument installations include or exclude the pore air pressure as shown in Figure 7. The water in the standpipe is connected to the free water in the clay. The pressure **in the free water is controlled by the pore air pressure** as long as air remains.

The VWP on the other hand was installed with a high air entry filter, carefully under fully saturated conditions. This would set up the potential for being able to measure just adhered water pressure. The adhered water pressure and the tension in the corner menisci between particles support the free pore air/water pressure. This condition continues through the high air entry filter, excludes a portion of the pore air/water pressure and **allows only the adhered water pressure to be measured** by the VWP.

P. Erik Mikkelsen, Consultant in Geoengineering & Instrumentation, Geometron Inc. PS, 16483 SE 57th Place, Bellevue, WA 98006 USA.
Tel: (425) 746-9577,
email: mikkelsen.pe@comcast.net

Arthur D. M. Penman

This large dam in Iran has been well instrumented. It is valuable to hear about such detailed instrumentation in a large dam. Progress in dam design is greatly helped by results from such instrumentation and we are grateful to Ali Asghar Mirghasemi for his article describing the construction and instrumentation of the dam. He has a problem, he says, because a valuable comparison he made between a standpipe piezometer and a vibrating wire instrument has shown the standpipe piezometer reading almost 5m head of water greater than that shown by the vibrating wire piezometers and he seeks an explanation for this unexpected difference.

In comparing piezometers one needs to compare like with like. Compacted fill is a non-saturated material which initially must have negative pore pressures to give it the strength to support the heavy placing and compacting machinery. Pore pressures only become positive when the weight of overburden becomes great enough. The fill usually comes from a borrow pit and lumps of the original material become incorporated in the fill. At Karkheh the core material is a mixture of 60% clay and 40% gravel. The gravel was a crushed conglomerate rock and was added to the clay. This is an important point. The addition of stone to a clay for dam fill is very difficult to accomplish. It might have been passed through a brick maker's type of pug mill, but I think the volumes would be too great for that. The other way would be to spread the stone over the fill during construction and mix it in with agricultural equipment but this would lead to layers of more stony fill amongst the clayey part. That aspect combined with the chance of lumps of original material being incorporated in the fill means that pore pressures at individual points may not be equal everywhere. Vibrating wire piezometers measure the pore pressure that exists around them locally. Standpipe piezometers, on the other hand, measure an average pore pressure over a considerable volume of fill. At Karkheh the piezometers had pockets of unsaturated sand of 15 cm diameter by 150 cm long connected to continuous lengths of 15 cm diameter rigid pipe. They were 'sealed' by bentonite pellets dropped around them followed by the addition of bentonite grout. It is difficult to see how the 15 cm rigid pipe containing a 2 cm pipe rising from the intake filter could be sealed to prevent pore water from getting along the length of the pipes.

Figure 2 indicates that the string of pipes went down to a bottom intake filter, past the mid height position where the comparisons were being made. Were the outer 15 cm rigid pipes common to both piezometer? Exactly how were they installed? Figure 2 suggests that they might have been installed in a borehole made after the fill had reached full height. In that case the measured pore pressures could have originated from any height along the length of the pipes. If the added stone was in layers, then higher pore pressures could be collected by these drainage layers.

It is most interesting that this valuable comparison has been made and we must encourage others to repeat this sort of experiment in a new high dam. The errors indicated above can produce strange results but comparisons between different types of piezometers, with intake filters of high or low air entry, pressed directly into the fill will be very welcomed by the profession. Standpipes of 2.06 cm can be connected directly to the intake filters and sealed by placing in the fill during construction. Careful hand work is needed to place and compact the clayey fill around the pipes and there is always a danger that the hand compaction cannot equal that produced by the heavy machines. But other interesting comparisons can be made between different types of remote reading piezometers, placed close together. My comparison made with high and low air entry intake filters that I made at Chelmarsh dam many years ago has never been repeated. [See Penman, A.D.M., *Measurement of Pore Water Pressures in Embankment Dams*, Geotechnical News, Vol. 20 No. 4, December 2002, pp 43-49]. I should have placed the piezometers much lower in the dam, when I would expect the two measured pressures to become the same when the weight of overburden compressed most of the air out of the fill making an almost saturated material.

May I add support for the detailed instrumentation made at Karkheh, the results from which add greatly to our understanding of dam behaviour both during construction and subsequently?

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

J. Barrie Sellers

It is not unusual for earth pressure cells to give results which do not meet expectations.

According to Oosthuizen et al (2003), Høeg (2000) remarked at the ICOLD meeting in Beijing that:

> "...you should have awfully good reasons for putting in earth and rock pressure cells in any dam of any kind. We spend so much time and money on putting in the cells and we spend much time on not believing the data we get, because, if we do get data, we do not understand them... Forget about pressure cells in general and there is a very good reason for doing so."

Quite commonly the measured earth pressures are less than expected and in most cases the discrepancy can be traced to poor compaction around cells. It seems likely that this was the case here, although, it is not clear from Tables 1 and 2 whether the cells under-registered or over-registered when compared with the expected values.

Oosthuizen et al (2003) devised a method to minimize this problem in earth dams: Installation of the cells begins when the fill has reached a height of 800mm above the instrument level. The instrument location and the cable trenches are excavated 500mm deep. A pocket, with 45° sloping sides, of only a further 300mm depth is required to be excavated at each instrument location. The cells, complete with pinch tubes (see later), are positioned on a thin layer of non-shrink sand-cement grout and nailed in position using the lugs on the cells provided for this purpose. The excavated pocket is then backfilled with a weak concrete (19mm aggregate), in 100mm layers, vibrated with a poker vibrator. The concrete is in all probability stiffer than the surrounding soil so that it acts as a rigid inclusion and attracts additional load from the softer surrounding. This would tend to make the earth pressure cell over-register, possibly by 5%, maybe up to 15% maximum.

The purpose of the pinch tubes is so that the cells can be pressurized to ensure that they are in intimate contact with the concrete. This is standard procedure when installing cells in concrete, because as the concrete cures heat is generated, the cell expands, and then returns to its initial shape after the concrete cools, with the potential of becoming disconnected from the concrete. For this application the pinch tubes may be a bit of overkill, but it is cheap insurance and a good thing to include. After 24 hours the cells are pressurized, by pinching the pinch tubes until the pressure in the cell, displayed on a connected readout box, starts to change. The instrument location, containing the grouted cells and the cable trench, is then backfilled in 100mm layers. Each layer is compacted by a vibratory trench roller. After this, standard construction filling and compaction practices can continue.

Other possible reasons for the unexpected performance could be (a) failure to extract all the air from the cells while filling them with de-aired liquid, and (b) a mismatch between the modulus of the cell and the modulus of the surrounding material.

Regarding Tables 1 and 2, there is no indication, either here or in the text, whether the measured stresses were more or less than the theoretical. It would have been better, I think, if the measured values had been presented as a percentage of the theoretical.

The discrepancy between the vibrating wire (vw) piezometer readings and the open standpipe readings looks like a constant offset—the data curves are pretty much parallel. Maybe due to a faulty base-line zero readings.

Maybe there is a naturally occurring pressure gradient that would cause the piezometers to read consistently lower than the standpipes. (I assume that the standpipes are being read with a dip meter)

In one case there is a two month time lag between reservoir level and vw piezometer and open standpipe readings; in the other case there is no discernable time lag. But the time lag between the vw piezometer and the open standpipe cannot be seen on this time scale. There may have been a time lag amounting to a day or two.

References

Oosthuizen, C., Naude, P.A. & Hattingh, L.C. 2003. Total and Pore Pressure Cells: Method in the Madness. Proc. 6th International Symposium on Field Measurements in Geomechanics, Oslo, Norway, Balkema, pp 273-278.

Høeg, K. 2000. Question 78 Theme B Discussion. Proc. 20th ICOLD Congress 19-22, Sept. Vol 5: 336. Beijing, China.

J. Barrie Sellers, President, Geokon Inc., 48 Spencer Street, Lebanon, NH 03766 USA. Tel. (603) 448-1562, Fax (603) 448-3216, email: barrie@geokon.com

John Dunnicliff

In contrast to the other discussers, I was able to read all these discussions before writing mine!

Because neither Barrie Sellers nor I had a complete copy of Høeg (2000), I asked Kaare Høeg (Norwegian Geotechnical Institute) whether the quotation in Oosthuizen et al (2003) was correct and complete. With his approval to publish his reply, here it is:

> "The quote is valid. I did not write down the oral discussion and did not get a chance to review what was recorded by the secretary at the discussion session. I am not sure I used exactly those words, but I gave that opinion after yet another paper presentation, describing the use of some hundred earth pressure cells, and the author could not figure out what the readings told him. The essence of what I said was:
>
> My experience is, and that of many authors at ICOLD Congresses and other Conferences seems to be, that it is very difficult to interpret and rely on the readings from earth pressure cells installed in embankment dams, especially rockfill dams. Many investigators have spent time and money installing such cells, but have, in general, found the measurements of little value (refer to Hvorslev's classical work). There are examples of pressure cells giving valuable readings when the cells are installed on a structural interface, but not inside the dam body. I would discourage the use of pressure cells in embankment dams, but encourage measurements of seepage/leakage, pore pressures, movements and accelerations in seismic regions. You should provide very good reasons and convincing arguments before going to the installation of earth pressure cells in the dam body."

Again—having had the luxury or reading the other discussions before writing this—I can respond to Elmo DiBiagio's comments about Figure 3 in Mirghasemi's article, which is titled "Layout of embankment earth pressure cells (Dunnicliff, 1993)". In the 1993 reference this same figure was titled "Typical layout of embedment earth pressure cells (courtesy of Soil Instruments Ltd., Uckfield, England)". As Elmo correctly says, "The drawing can be interpreted in two ways, i.e., instruments placed on the floor of the excavation or set in cubical sockets beneath the floor". The second interpretation would certainly lead to large errors. I apologize for perpetuating confusion.

Author's Reply

I thank all discussers Donald H. Babbitt, Elmo DiBiagio, Louis Marcil, P. Erik Mikkelsen, Arthur D. M. Penman, J. Barrie Sellers, and John Dunnicliff for their useful comments to clarify the problems. And a special thank you to John Dunnicliff, who encouraged me to prepare this article, which was first presented at the 2005 instrumentation course in the Netherlands, and was where we met. Also I would like acknowledge him for all of his suggestions and editing made throughout preparation of my article and this reply.

The number and variety of discussions express the interest in my article, and I am very pleased with their contributions. This might persuade others to report not only the consistent data but also the observed uncertainties. Or as John Dunnicliff says, "it's refreshing when someone is willing to publish something that didn't work out well".

Before continuing with my reply, I want to emphasize that in this article the concentration was on instruments where no consistent data were achieved. As I said in the article, valuable and consistent information was also gained from the other instruments, indicating satisfactory performance of the dam. Because of length limitations, this information was not reported in the article.

Uncertainties in Earth Pressure Measurements

Most discussers agree that earth pressure measurement in an embankment dam is not an easy or straightforward task. Therefore, being faced with a set of inconsistent data seems to be normal. However to reply to the discussers, the following explanations are given.

Installation of the pressure cells was carried out according to the manufacturer's instructions:

> "To prevent damage to the cells, the lens and cells are installed in an excavation made to accommodate them. The cells are installed in a lens forming a mound on the base of the excavation or within a lens in a pocket excavated in the base of the excavation. The cell pockets should be excavated with extreme care to avoid disturbance to the soil. The pockets should be located 1 meter away from adjacent pockets or the excavation walls. The width of the pocket should be equal to a minimum of 3 pad diameters to avoid load bridging. The length of the pocket (axis of the pressure transducer) should be 6 pad diameters to accommodate the length of the pressure cell and to provide 1 pad diameter clearance at either extremity of the pressure cell."

As may be noticed, the manufacturer suggests either way of installation (refer to John Dunnicliff and Elmo DiBiagio comments). However, the compaction of surrounding soil when the instrument are placed on the floor, without digging the pockets, is much more difficult (especially for vertical and 45-degree cells). Also, when the cell is not placed in a pocket, the possibility of cell rotation increases during the compaction. If the cells are placed on the excavation floor vertically or 45 degrees inclined, the compaction around them should be performed with great care. In this situation the compacted fill around the cells will not have the same quality as the main embankment, and arching around the cell will occur. The procedure explained by Barrie Sellers (Oosthuizen et al, 2003) is a good idea to reduce the problem of stress arching and rotation of the installed cells. However the selection of concrete specification is a critical issue in this method.

In Tables 1 and 2 in the article, I tried to compare the calculated and measured stresses. As mentioned, the **calculated** stresses acting on a cell are based on the **measured** stresses on the other cells. In other words, using Mohr circles, the stresses measured by the other cells are used to calculate the stresses in a certain cell and then the results are compared with the stress measured by that cell. Thus, no comparison is made between measured and theoretical stresses as pointed out by Barrie Sellers. There is no certain trend for the differences in positive and negative values as reported in revised Tables 1 and 2

The measured and calculated (based on embankment height) stresses as suggested by Elmo DiBiagio, are compared in Figures 8 and 9 for cells PC5-5 and PC6-5 respectively. Again some uncertainties can be found from the figures:

- In Figure 9 the measured pressure by a 45 degree cell is higher than that measured by horizontal cell (vertical pressure).
- Since at the dam the valley is wide and cell PC6-5 is located at the center of the core, the stress field is essentially two-dimensional, as described by Elmo DiBiagio. Also, at the centre of the clay core the maximum principal stress direction is near vertical. Therefore measured stresses by 45 degree downstream and 45 degree upstream cells should be equal. Figure 9 shows a significant difference between two sets of measured stresses.

Based on the experiences described in the article and by the discussers, it could be recommended that if there are "very good reasons and convincing arguments" (as mentioned by Kaare Høeg), at the most only horizontal earth pressure cells may be considered for installation in embankment dams. In this case the installation of the cell at the floor of excavation will not be difficult, and the possibility of rotation of cells during compaction is minimized. The installation of vertical and inclined cells in the embankment may not be justified.

Table 1, Revised. Comparison between computed and measured stresses at 45 degree planes.

Pressure cell number	Differences between measured and computed stresses for the cell oriented 45 degrees downstream (in percent)	Differences between measured and computed stresses for the cell oriented 45 degrees upstream (in percent)
PC5-4	-17	no measurement
PC5-5	+23	no measurement
PC6-4	-21	+11
PC6-5	+17	+115
PC7-3	+48	+70
PC7-4	+23	+22

Table 2, Revised. Comparison between computed and measured horizontal stresses

Pressure cell number	Differences between measured and computed stresses for the vertical cells (in percent)
PC5-4	+266
PC5-5	-33
PC6-4	+120
PC6-5	+41
PC7-3	-70
PC7-4	-64

Comparison of Vibrating Wire and Open Standpipe Piezometers

A wide variety of ideas about the differences observed between vibrating wire and open standpipe piezometer data are presented by the discussers. The variety of the excellent comments on this issue is very helpful to explore the real reasons for these differences. That is one side of the story. From the other point of view, this makes it very difficult to come to a certain conclusion.

To respond to the comments by discussers, I would like to start with the initial reading procedure for the vibrating wire piezometers, which was a main subject among the comments. In the following, the steps carried out for initial reading of Karkheh VW piezometers are presented.

1. At least 48 hours before piezometer installation, the filter was installed on the piezometer and the reading was carried out and then the instrument was immersed in water. The readings were not recorded because the objective of making readings was to make sure that the device was responding.
2. 24 hours after filter installation (at least 24 hours before piezometer installation), readings was made and recorded as L_0 & T_0. After this reading the piezometer was used to measure a known water level in an observation well. In the calculation of water level L_0 & T_0 were used. If the difference between the real

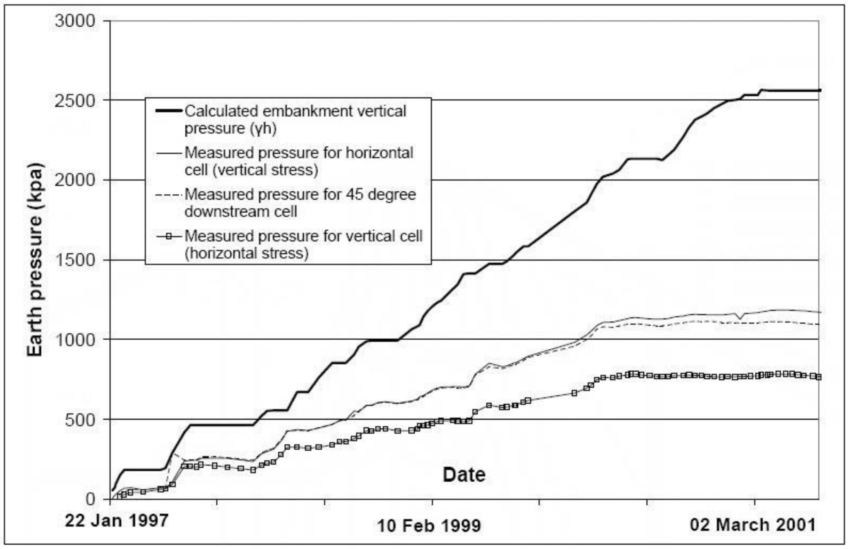

Figure 8. Comparison of calculated vertical pressure based on fill height and measured stresses for PC5-5.

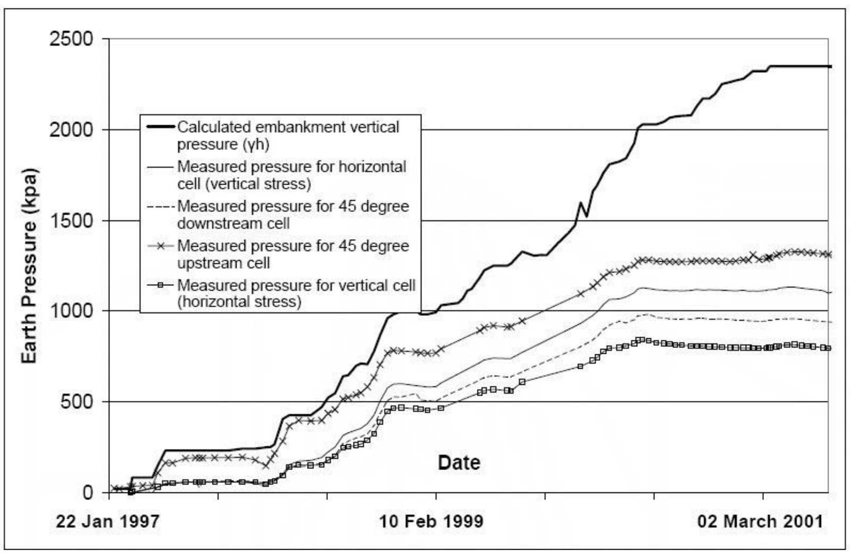

Figure 9. Comparison of calculated vertical pressure based on fill height and measured stresses for PC6-5.

water level and the calculated level was acceptable, the next steps of installation procedure were completed. This step ensured the accuracy of L_0 and T_0.

3. After at least 24 hours of initial reading and piezometer check-up, the piezometer was installed.
4. One and 24 hours after installation, readings were made and recorded as L_1 & T_1 and L_{24} & T_{24}, respectively.

Table 4 presents the recorded values for EP5-15 and EP5-17 VW piezometers. The installation procedure included the four points identified above, together with the following, to ensure correct initial readings:

- The time between filter installation and the initial reading was 24 hours to allow "complete dissipation of the overpressure momentarily induced by the installation of the high air entry filter on the piezometer" as explained by Louis Marcil.
- The L_0 and L_{24} are close to each other; therefore there is no sign of continuation of pore pressure dissipation between a time 24 hours after filter installation (L_0) and at least 48 hours after filter installation (L_{24}). This means that during this time interval the piezometer had came to a steady state condition.

The next subject that I have to explain is the procedure employed for preparation of the mixed clay, pointed out by Arthur D. M. Penman. The Karkheh conglomerate unit at the borrow area is weak to moderately cemented, and it breaks down during excavation to the sand and gravel gradation without any special effort. For the field preparation of the mixed clay, alternate layers of clay and gravel were deposited at their optimum moisture content. These sandwiched layers of clay and gravel were then cut and mixed by appropriate excavating machines. The mixed clay materials were then loaded and carried to the core zone of the dam by special trucks, where they were compacted in layers of limited thickness to achieve the required density. An appropriate test program was implemented to ensure the homogeneity of the fill.

As recommended by Arthur D. M. Penman, the open standpipe piezometers were "connected directly to the intake filters and sealed by placing in the fill during construction". They were not placed in boreholes after completion of the embankment. This method facilitates a better and easier sealing procedure. A single 15-cm (6-in.) PVC pipe was used for the protection of two piezometer riser pipes (each with internal diameter of 20.6 mm

Table 4. Recorded values for EP5-15 and EP5-17 VW piezometers

Piezometer Name	L_0	T_0	L_1	T_1	L_{24}	T_{24}
EP5-15	3409.16	20.3	3405.61	13.3	3408.25	13.3
EP5-17	3097.99	19.9	3093.96	15.2	3095.39	13.7

(0.8 in.)), located at the same stations and distances but different elevations.

Based on above explanations it is not easy to be convinced that the main reason for the difference between SP and VW is either the existence of the more pervious layer in the fill or the breakage of the SP sealing.

In summary, it is not a straightforward and easy job to make a convincing conclusion about this issue. All the noteworthy opinions raised by the discussers such as deformation of the VW housing under high pressure of embankment (Elmo DiBiagio), inaccuracy in zero reading (Louis Marcil & Barrie Sellers), breakage of the SP sealing (Louis Marcil & Arthur D. M. Penman), existence of pervious layer in the fill connected to SP filter (Arthur D. M. Penman) and effect of pore air pressure on measured water pressure (Erik Mikkelsen) may contribute to describe the problem with different levels of influences. But for me it is still very hard to say which one contributes more, and is the key role player.

Ali Asghar Mirghasemi, Associate Professor, School of Civil Engineering, College of Engineering, University of Tehran, Enghelab Ave. P.O. Box 11365-4563, Tehran, Iran. Also senior geotechnical engineer for Karkheh Dam project, Mahab Ghodss Consulting Engineer, Tel: (+9821) 61112273, Fax: (+9821) 66461024, email: aghasemi@ut.ac.ir

Piled Foundation Design – Clarification of a Confusion

Bengt H. Fellenius

Abstract

A frequent confusion and lack of understanding exists with regard to the design of piles subjected to drag loads. Some will lump the drag load in with the dead and live loads when assessing pile bearing capacity. Also common is to disregard the root of the problem: settlement of the piled foundation. **It must be realized that: dead and live loads applies to bearing capacity, dead load and drag load applies to structural strength, and downdrag is settlement.**

A few weeks ago, I was once again asked if the allowable load for a pile should be reduced when considering drag load. Shortly thereafter, when I took a look at the discussions at www.Geoforum.com, I noticed a very similar question. Perhaps I should not be that taken aback by the lack of knowledge displayed by the questions. The persons asking may not have been taught better. The following is a quote from a textbook published in 2001 and assigned to 4th Year Civil Engineering students at several North American Universities:

> *Piles located in settling soil layers are subjected to negative skin friction called downdrag. The settlement of the soil layer causes the friction forces to act in the same direction as the loading on the pile. Rather than providing resistance, the negative skin friction imposes additional loads on the pile. The net effect is that the pile load capacity is reduced and pile settlement increases. The allowable load capacity is given as:*
>
> $$Q_{allow} = \frac{Q_{ult}}{F_s} - Q_{neg}$$
>
> *where* Q_{allow} = *Allowable load capacity*
> Q_{ult} = *Load capacity*
> F_S = *Factor of safety*
> Q_{neg} = *Downdrag*

First, "negative skin friction" is not "downdrag" but defines a downward directed shear force along the pile, while downdrag is the term for settlement of a pile (caused by the settling soil 'dragging a pile along'). Second, the term "load capacity" means different things to different people and "allowable load capacity" is an abominable concoction of words. Third, and very important, the phrasing in the quoted paragraph confuses cause and effect. Drag load is not downdrag, and it does not cause settlement, but is caused by settlement of the surrounding soil and is mobilized when the pile resists this settlement. The worst boo-boo, however, lies in the quoted formula, which does not recognize that the factor of safety and the drag load are interconnected, i.e., changing the factor of safety changes the drag load. As this may not be immediately clear to all, the following example will try to clarify the interaction between the pile, the factor of safety, and the drag load.

Example

Consider the case of a 300 mm diameter pile installed to a depth 25 m through a surficial 2 m thick fill placed on a 20 m thick layer of soft clay deposited on a thick sand layer. The case is from a recent project in the real world. Let's assume that a static loading test has been performed and the evaluation of the test data has established that the pile capacity is 1,400 KN. As is visually presented in Fig. 1, applying a factor of safety of 2.0 results in an allowable load of 700 KN (dead load 600 KN and live load 100 KN). Moreover, due to the fill and a lowering of the groundwater table, an almost 200 mm settlement of the ground surface will develop after the construction. How should the designer assess this case? Incidentally, as several full-scale case histories have shown,

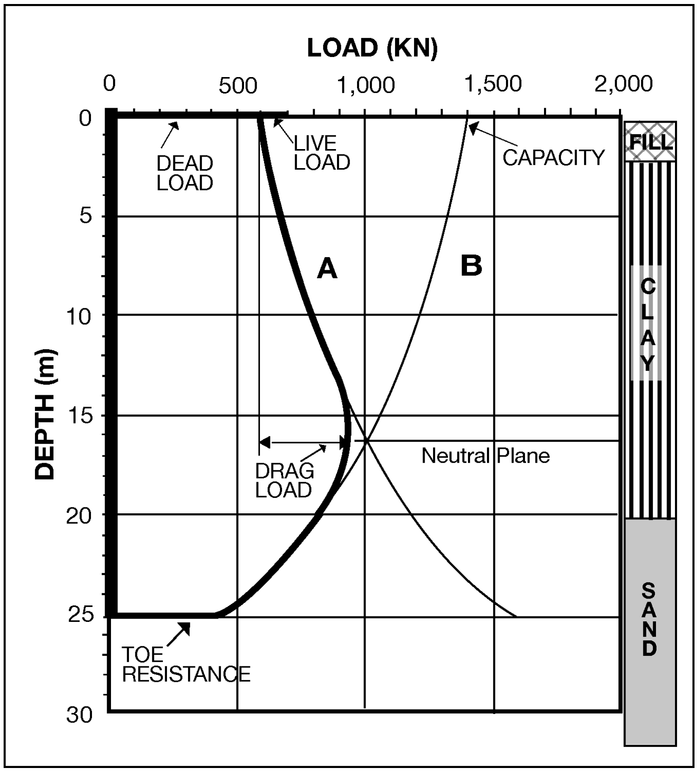

*Figure 1. Distribution of load in the pile. "**A**" is the long-term load-distribution and "**B**" is the resistance distribution measured at ultimate resistance (capacity) in a static loading test*

whether or not the soil at the site settles 200 mm or 2 mm, or for that matter 2,000 mm, the magnitude of the drag load will stay the same.

Comments and Questions

Some practitioners believe that all is well because the foundations include piles with a capacity of twice the desired allowable load. Then, there are those who understand that, for the numbers indicated above, the pile will be affected by a drag load of about 300 KN, acting at a neutral plane located at 17 m depth (Fig. 1). A few of these practitioners will subtract the drag load from the pile capacity before applying the factor of safety and arrive at an amended allowable load of 550 KN (which is a violation of principles as this approach in effect has reduced the drag load by a factor of 2.0). Others will apply the quoted formula, and arrive at an allowable load of 400 KN. Yet others will realize that the last approach means that the drag load is applied without a factor of safety, preferring to apply the formula with the drag load increased by a factor of safety, say 2.0. This results in an allowable load of (1,400/2 - 2x300) = 100 KN — don't laugh, I have seen it done several times and it was proposed for this project! So, which allowable load is right? Is it 100 KN, 400 KN, 550 KN, or 700 KN? (Similar diverging approaches abound in the load-and-resistance-factor-design, LRFD).

Suppose the structure supported on the piled foundation was built before the drag load conditions were recognized (no signs of distress are noticeable). Then, what factor of safety would a back-analysis show the piles to have? Would it be 1,400/700 = 2.0, or (1,400 - 300)/700 = 1.6, or 1,400/(700+300) = 1.4? And, I wonder how the fellows advocating the laughable approach would react when realizing that the piles are supporting seven times more load than the maximum load their approach would allow as safe.

Before answering, consider that the magnitude of the drag load depends on the magnitude of the dead load on the piles. Reduce the dead load and the drag load increases, and vice versa. For example, after reducing the allowable load by 150 KN to arrive at a 550 KN value (made up of a dead load of, say, 475 KN and a live load of 75 KN), the drag load is no longer 300 KN, it is 400 KN! If the allowable load is reduced by an additional 150 KN, say to 400 KN (made up of a dead load of, say, 325 KN and a live load of 75 KN), the drag load increases 500 KN!

Note, for the three values of dead load — 600 KN, 475 KN, and 325 KN — the neutral plane location changes from depths of 17.0 m to 18.0 m to 19.5 m, respectively. To simplify the example, no change of the toe resistance is included. In reality, however, the deeper down the neutral plane lies, the smaller the enforced penetration of the pile toe into the sand and the smaller the mobilized toe resistance, and when the toe resistance is reduced, the location of the neutral plane moves upward and the drag load changes. Altogether, the load at the neutral plane, that is, the maximum load in the pile, is essentially unchanged for the three alternative values of allowable load. In stark and important contrast, for each reduction of allowable load, the project foundation costs increase.

Clarification

Bewildering, ain't it? Many select one of the four approaches as the one to be correct, ignoring the others, thus avoiding having to make the small leap of understanding of what a proper design needs to include, as follows**:**

First, the drag load does not affect the pile bearing capacity — the ultimate resistance. That is, the pile capacity is the same whatever the magnitude of the load from the structure. The factor of safety is applied to ensure that, should the load on the pile be inadvertently larger than intended and should the pile capacity be inadvertently smaller than thought, the pile might be close to failure, but it would not fail. No negative skin friction—no drag load—is present close to failure. Therefore, only the first approach, that with the 700 KN allowable load, is correct.

Second, the drag load has to be considered, of course, but not in the context of bearing capacity. The concern for the drag load only affects the pile structural strength at the location of the maximum load, i.e., at the neutral plane. For the example case, if the structural integrity of the pile is safe considering the sum of dead load and drag load, that is, 900 KN for the example case, the design for drag load is complete.

Third, with (A) the dead load plus live load safe considering the pile capacity, and (B) the dead load plus drag load safe considering pile structural strength, it remains to show that (C) the pile will not settle more than acceptable, that is, that downdrag is kept in check.

In checking the pile for downdrag, it must be realized that there are two different definitions of the neutral plane. Both give the same result, or location, rather. One defines the neutral plane as located at the force equilibrium in the pile, which is where the shaft resistance changes from negative to positive direction and where the sum of the dead load plus drag load is in equilibrium with the positive forces in the pile. The second defines the location to be where the pile and the soil move equally. (Note, the toe resistance is only as large as is needed to establish the equilibrium between forces and movements. Moreover, whatever the factor of safety chosen in the design, if the soil is settling at the neutral plane, the pile will settle too and as much as the soil settles at that location). The influence of varying dead load and toe resistance is illustrated in Fig. 2. The diagram to the left shows the load distributions and locations of the neutral plane for the dead loads associated with the mentioned different allowable loads on the pile (the 100 KN case is excluded). The diagram to the right shows the distribution of soil settlement and location of neutral planes for the three approaches. (More explanation and discussion is available in Fellenius, 2004).

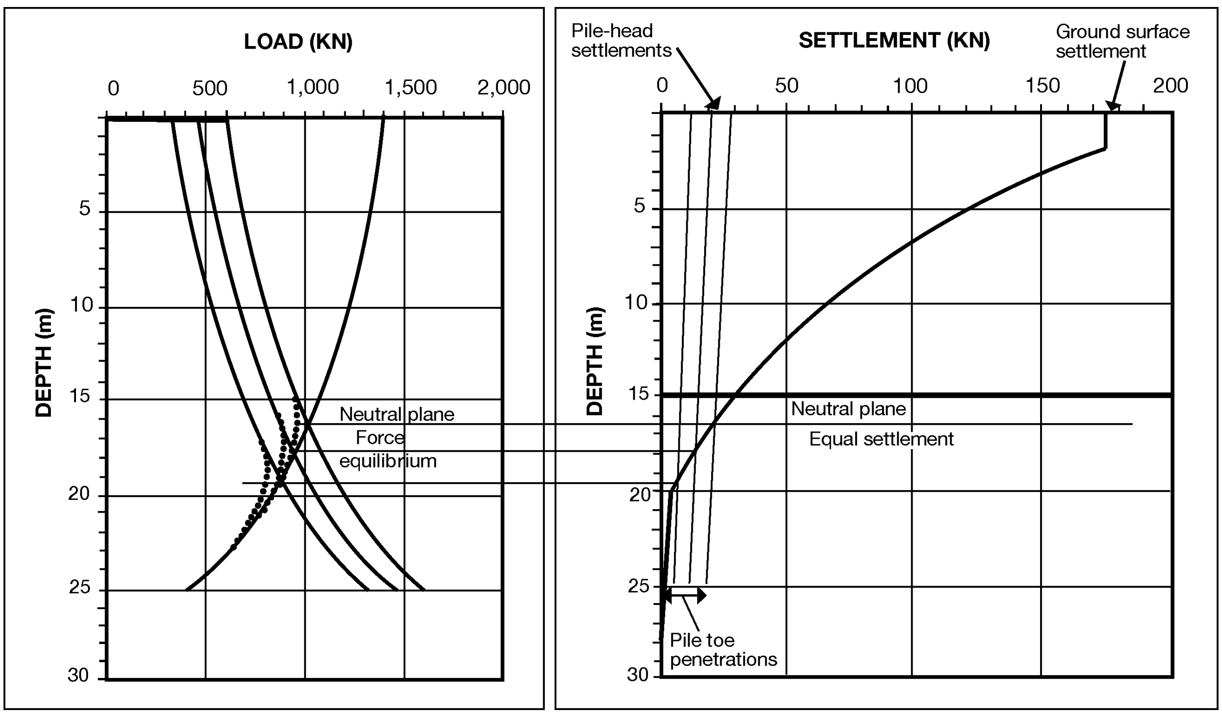

Figure 2. Distribution of load in the pile and interaction with soil settlement. The dashed curves represent the gradual change in a transition zone from negative direction of shaft shear to positive direction.

The Biggest Problem

The foregoing appears to be news to many. It shouldn't. The long-term response of piles in settling soil was made known in several very accessible publications as early as some 40 years ago. How does one get the textbook writers and the teachers of foundation design to become aware of the knowledge and convey it in the teaching of future practitioners? How does one get practitioners to fill the voids in their professional education and to keep abreast with advances in the profession? The latter, unfortunately, may be the biggest problem of all, but discussing it lies outside this contribution.

Reference

Fellenius, B.H., 2004. Unified design of piled foundations with emphasis on settlement analysis. "Honoring George G. Goble—Current Practice and Future Trends in Deep Foundations" Geo-Institute Geo-TRANS Conference, Los Angeles, July 27 - 30, 2004, Edited by J.A. DiMaggio and M.H. Hussein. ASCE Geotechnical Special Publication, GSP 125, pp. 253 - 275.

Bengt H. Fellenius, Bengt Fellenius Consultants Inc., 1905 Alexander Street SE, Calgary, Alberta. T2G 4J3.
Tel.: (403) 920-0752,
email: Bengt@Fellenius.net

Rules of Thumb for Geotechnical Instrumentation Costs

Gord McKenna

Introduction

How much does a piezometer cost? Most of us would guess it equated to the cost of the transducer and a certain amount of signal wire – often about $1000. But the life-cycle cost of many geotechnical instruments is closer to $15,000 when drilling, technician time, future readings, data management, and geotechnical analysis and reporting are factored in. This article provides a few rules of thumb and some suggestions on managing this cost.

In writing this article, I had in mind a hypothetical example of instrumenting a small embankment dam (about a half day away from a major centre) with a few inclinometers and a dozen piezometers to confirm design assumptions by monitoring construction performance. Readers are encouraged to look at their typical projects and adapt these rules of thumb to their own projects. Readers are also directed to a few good references at the end of this article that provide additional information with similar themes.

Instrument Life-cycle

Purchasing and installing an instrument is just the start. It can be useful to think of instrumentation costs in terms of the life-cycle of the instrument – from its initial geotechnical design and procurement, through drilling and installation, including reading and maintenance, and data management and geotechnical analysis back in the office. In addition, we must remember decommissioning.

Rules of Thumb for Geotechnical Instrumentation Costs

Figure 1 shows the costs of a typical geotechnical instrument, in this case an instrument installed in a borehole, read quarterly, with annual data analyses over a ten-year period. Below are some rules of thumb.

- **Rule of thumb #1:** Most geotechnical instruments cost about $1000 to purchase (some more, some less)
- **Rule of thumb #2:** The cost of drilling for an instrument is often about $1000 and the cost of the installation and the first couple of readings is another $1000.
- **Rule of thumb #3:** It costs about $1000 to datalog an instrument (datalogger + cables + installation)
- **Rule of thumb #4:** It costs about $5000 to read each instrument for its life and another $5000 doing geotechnical analysis and reporting on the results.
- **Rule of thumb #5:** A typical geotechnical instrument costs about $15,000 for its life-cycle cost, about 15x the initial instrument hardware cost.
- **Rule of thumb #6:** Expect to replace about 20% of the instruments every 10 years
- **Rule of thumb #7:** Geotechnical instrumentation costs are often 2 to 10% of the total earthwork costs for the structure being monitored (less for large structures)

Some lessons learned from the above rules:

- **Lesson #1:** The best way to save money is to install only the instruments that are **really** needed. Dunnicliff and Powderham (2001) point out "*The purpose of geotechnical instrumentation is to assist with answering specific questions about soil/structure interaction. If there are no questions, there should be no instrumentation.*" Challenge yourself to have the minimum amount of instrumentation that will provide answers to clearly defined questions.
- **Lesson #2:** The actual cost of the instrument is small compared with the costs of reading and analyzing the data. Purchase the instrument that has the lowest life-cycle cost while keeping in mind reliability – having to replace a faulty instrument, or spending

Friesens

Do not print key lines

Please centre the attached original drawing within this frame.

John

The final hand-drawn copy which was delivered to me today, was slightly smaller than the one I received electronically. This is the reason for the entire column not being used.

Figure 1. Rule of thumb costs for the life of a single geotechnical instrument. Sketch by Derrill Shuttleworth, Studio Two, Edmonton, Alberta, Canada

hours trying to salvage "bad" data are two ways to increase the life-cycle costs greatly. American Society of Civil Engineers (1999) states: "*When considering costs, the lowest initial cost of an instrument should never dictate the selection of an instrument. Instead, a comparison of the overall cost of procurement, calibration, installation, life-cycle maintenance, reading, and data process of the available instruments (plus a consideration of the replacement cost if there is some risk the instrument should go bad) should be made... the cost of instruments themselves is usually a minor part of the total cost."* For example, vibrating wire piezometers have a higher initial hardware cost, but generally are cheaper and more reliable to read than pneumatic piezometers.

- **Lesson #3:** Be fanatical about ensuring that every instrument is properly installed and maintained and that all are read properly with good QA/QC. Ensure the technician or engineer who installs the instrument is top notch, well trained, and diligent. Have a formal QA/QC program that involves checking the data in the field when it is read, that it fits historic data when it is uploaded into the database, and that it fits seasonal trends or other patterns on an annual basis. One bad reading can cost as much as 10 good readings.

More Good Practices

- The biggest cost of instrumentation is getting the readings over the years and managing the data. One of the greatest risks is the expense (and sometimes embarrassment) caused by poor data.
- Your instrumentation suppliers are partners in your project – you will be relying of their advice and service. Most geotechnical people have developed long-term relations with preferred suppliers.
- Ensure there are proper procedures in place, good training, and that the equipment (including readout boxes) are kept in good working order and calibrated regularly. Always store the raw data – not just the calculated data.
- Try to use the same person and same equipment for each reading set. The data are often more operator-dependent than you might think.
- Protect your instruments – mark them well in the field, ensure they are well labelled and marked with a contact phone number. Keep wires tucked away where they won't be shot or chewed.
- Where it would be difficult or impossible to replace instruments, consider duplicate installations.
- Consider low-impact access – you don't need a gravel road into every instrument – consider using an ATV or hiking in where practicable (this makes it cheaper when it comes to decommissioning too).
- Look for opportunities for continuous improvement of your instrumentation practices and programs with an eye to reliability and life-cycle costs. Seek out opportunities to gain and share experience with colleagues at conferences and tradeshows, continuing education courses, and chatting with your instrumentation suppliers and drillers.

In Closing

The cost of geotechnical instrumentation is high, so it deserves care in attending to all aspects of design, procurement, installation, reading, maintenance, data management, QA/QC, and analysis. Some fanaticism balanced with practicality helps. Don't skimp on the instrument costs only to pay the piper down the road. Critical issues include the design of the program, the installation of the instruments, ensuring that the data are processed, and properly and efficiently managing the data.

References

American Society of Civil Engineers, 1999. Instrumentation of embankment dams and levees. Technical engineering and design guides as adapted from the US Army Corps of Engineers. no. 26. American Society of Civil Engineers, Reston, VA, 88p *http://www.usace.army.mil/inet/usace-docs/eng-manuals/em1110-2-1908.*

Dunnicliff, J. and Powderham, A.J., 2001. Recommendations for procurement of geotechnical instruments and field instrumentation services. Geotechnical News, Vol. 19, No. 3, September, pp 30-35. Also discussion by Klinger and closure by authors in the same issue.

Gord McKenna, Senior Geotechnical Engineer, Norwest Corporation, #830-1066 West Hastings Street, Vancouver, BC Canada V6E 3X2. Phone 604-602-8992, Fax 604-602-8951, email: gmckenna@norwestcorp.com

Electrical Cables for Geotechnical Instrumentation Applications

J. Barrie Sellers

INTRODUCTION

The choice of cable for geotechnical instrumentation applications must take into consideration many factors, some of them not obvious, others often conflicting. The dictates of environmental factors may need to be reconciled with questions of physical constraints and cost. The purpose of this article is to suggest which cable might be most suitable for each particular application.

CABLE DESIGN

Cables are most often made from individual copper conductors, which may be solid or stranded, and encased in an insulation material. Individual conductors are twisted into pairs or bundled inside a conductive shielding material, and then covered by an outer jacket made from the most suitable material. Cables may, in addition, be water-blocked, armored, or may contain steel or Kevlar® strands for additional strength, or plastic tubes for circulating fluids, or for venting to atmosphere.

The Number, Type and Size of Conductors

The number of conductors in a cable is determined by the number of sensors to be connected to the cable, and the number of conductors required by each sensor. Sometimes economies can be made by 'commoning' one of the leads from each sensor. But to do so runs the risk of losing all the sensors should the common lead fail. (Note: if the leads must be commoned it is better to common the ground leads connected to the negative terminal of the power source rather than the positive power lead). Also, the connection of numerous sensors to one common conductor increases the possibility of electrical noise on that conductor giving rise to unstable readings on all the sensors (this is discussed more fully in a later section under the heading, "Electrical Noise"). Thus, as a general principle it is advisable to avoid using common lead wires, particularly where a datalogger is being used.

Also in this context, there may be a temptation to use the shield as the return or ground conductor. While technically feasible, this is open to the same objections as before, in that it would allow any electrical noise to contaminate the output signal.

The type of conductor normally used is the stranded tinned copper type. Stranded conductors are more flexible then solid conductors, and this makes the cables easier to handle during installation.

The size of the conductors should be chosen to be as small and inexpensive as possible, consistent with the need to avoid line losses and voltage drops. Physical strength is another consideration. For electrical resistance type sensors, 16 or 18 AWG wire is preferred, while for vibrating wire types 22 AWG is sufficient (the larger the AWG number, the smaller is the diameter of the conductor).

Shielding

Shielded cables should always be used where dataloggers are in use. Shielding provides protection from electrostatic radiation coming from nearby electrical equipment, from lightning strikes, and from electromagnetic fields surrounding power lines, transformers, etc. Individually shielded, twisted pairs are the best construction for multi-conductor cables. Drain wires connected electrically to Mylar-type shields provide a simple means of connecting all the shields to a common ground. Individually-shielded, twisted -pairs are preferable for the rejection of common-mode interference. In a Wheatstone bridge, one shielded pair should be the power leads, another shielded pair the output leads, and a third shielded pair the remote sensor leads. For vibrating wire sensors, one shielded pair goes to the vibrating wire sensor, the other goes to the thermistor.

Mylar tape type shielding is usually sufficient, but in severe electrical noise cases cables with braided copper shielding offer slightly better protection. Shielding may also be provided by putting the cable inside a magnetic metallic conduit. Cables should be located as far as possible from the potential source of noise. They should never be placed alongside power cables.

Shields should be grounded only at the readout location. They should never be connected directly to the external sensor housings, Connection at both ends allows the formation of ground loops and defeats the purpose of the shield.

Where additional shielding of the sensor itself is required, it can sometimes be achieved by placing the sensor inside a mild steel (magnetic) enclosure. This is often required where piezometers are located in the same borehole as an electrical pump.

Insulation of Conductors

Individual copper conductors are normally insulated by a layer of rubber or plastic insulation. In general, polyethylene or polypropylene insulation is used at normal temperatures, since PVC has too high a dielectric absorption. For high temperature Teflon is most often used.

Outer Jackets

An outer jacket which is round and firm is easier to grip and seal at the point where it enters the body of the sensor. For this reason it is preferable to use extruded outer jackets.

Several material formulations are available:

1) **Neoprene** – A synthetic rubber compound commonly used for outdoor applications. It has good resis-

tance to gasoline, oils etc. Ordinary rubber should never be used.

2) **PVC** – A common choice for its good electrical properties and for being waterproof. It should not be used at low temperatures, when it becomes brittle.
3) **Polyurethane** – This material is very resistant to cuts and abrasions, making it useful for cables that are subject to repeated rough handling. It is not as water-resistant as PVC, but has better low temperature capabilities.
4) **High Density Polyethylene** – This has many desirable properties. It is the most resistant to environmental attack, and has excellent low temperature characteristics. Unfortunately, like Teflon, the material is so slippery that splicing and potting compounds will not stick to it. This makes it difficult, without the aid of "O" rings, to build waterproof connections to sensors, cable splices and cable connections.
5) **Teflon** – This material is essential wherever sensors and cables are subject to high temperature. It has outstanding resistance to environmental attack and also has excellent low temperature properties. It is expensive, and potting compounds will not adhere to it. Therefore sealing around connections and splices is difficult.
6) Other compounds such as **Kevlar®** or **Silicon Rubber** etc. may be required where there is a need for low smoke emissions, flame retardant, or resistance to nuclear radiation.

Armor

Armor may take the form of a helically laid layer of steel wires for heavy-duty protection, or a corrugated aluminum or steel layer for light duty protection, e.g. for protection against rodents. Unarmored cables may also be confined inside flexible conduit or rigid conduit. Armored cables are very stiff and difficult to work with. Cable splicing becomes a problem and can be expensive if continuity of mechanical strength is required.

Armored cables are most often needed for sensors installed in earth embankments, where large forces are exerted on the cable by compacting equipment and earth moving vehicles, and by settlement, differential settlement, "weaving", and sideways spreading of the embankment as it is built. They should not be connected directly to strain gages, crackmeters, etc., because the stiffness of the cable would allow it to pull on the gage and alter the readings. They are not necessary in concrete.

Fillings

Cables should be of compact construction, using fillers and extruded outer jackets to fill the spaces between the conductors, and to keep the cables round and firm. Water-blocked types for underwater use are available with a gel, powder or grease to prevent the passage of water along the cable.

Cables filled with gel, powder or grease are difficult and messy to deal with and should be considered applicable only to underwater situations, e.g. in earth dams, and in marine offshore environments. They are more necessary when used with sensors outputting low voltages, such as electrical resistance strain gages, where waterlogged cables could affect the sensor output. With vibrating wire sensors, these cable effects are much less pronounced, and filled cables are not required.

I.D. Tags

It is sometimes useful to tag long cables at intervals along their length, so they can be more easily identified at any location.

Strain Relief

Where a cable is required to have extra tensile strength, it should include either a Kevlar® or a braided stainless-steel aircraft cable running down its axis. This construction is typical for inclinometer probe cables, where stretching of the cable is undesirable, and where the cable may have to be used to pull a jammed probe out of a deformed casing.

Another typical application of strain relief is where a sensor hangs vertically inside a deep borehole, and the weight of the suspended cable and sensor exceeds the tensile strength of the cable. In this situation there is an alternative - to tape the cable alongside the aircraft cable, which is then used to take the weight.

Vented Cables

Special cables are available which contain plastic tubes inside them, as well as the usual conductors. These tubes can be used to transport air or other fluids. This kind of cable is required for vented piezometers, where a single vent tube allows the inside of the pressure sensor to be connected to the ambient atmosphere, so as to provide automatic barometric pressure compensation.

Cable Splices

Cable splicing is best done using commercially available splicing kits containing mechanical butt-splice connectors and epoxy potting compounds. These provide a good waterproof and mechanically strong splice. Typical kits are the Scotchcast 3M, Models 82A1 and 72N1. It is advisable also to solder the mechanical butt splices since proper crimping of the butt splices can be difficult to achieve without the proper crimping tools. Armored cables are difficult to splice if the mechanical strength is to be maintained - special mechanical connections need to be fabricated which will grip the armor firmly.

ENVIRONMENTAL FACTORS

Water

Cables should be waterproof, especially where the sensors have a low voltage output. (e.g. electrical resistance types). Vibrating wire types are less susceptible to water effects. Most plastic and neoprene formulations are waterproof. Polyurethane jackets are less waterproof, making them less suitable for prolonged use under water.

Pressure-extruded jackets that fit snugly around the inner conductors and fill the intervening spaces are less likely to act as conduits for water if the cable is cut and water enters. Water-blocked cables, filled with gel, grease or powder are most effective.

Cable splices ought not to be located under water. Whenever possible they should be kept above ground in a dry accessible location. Where underwater splices are unavoidable they should be made using commercially available splice kits, designed specifically for under-water applications, and supplemented where necessary by additional waterproofing.

Water & Sun Resistance

Most plastic jackets and neoprene cable jackets have excellent resistance to weathering. Teflon cables have outstanding resistance.

Chemical Resistance

The resistance of different cable materials to different types of chemicals can be found in the literature of cable manufacturers (e.g. www.belden.com). Teflon cables have outstanding resistance to almost all chemicals.

In very aggressive environments such as landfills (garbage heaps), that contain many aggressive chemicals and leachates, it may be necessary to encase the cable inside stainless steel tubing. This is expensive but effective, and also provides good mechanical protection.

Abrasion, Physical Damage

Polyurethane cables have outstanding resistance to abrasion and cutting. This is important in high-traffic areas, or where cables are being dragged along the ground, or being used repeatedly with portable sensors.

Cables that are buried in the ground should be protected from sharp objects by use of conduits, armor, or by surrounding them with fine-grained material. Particular attention should be paid to zones which will be prone to high shearing forces during the life of the cable. Here it may be advisable to leave enough slack in the cable to accommodate these movements without breakage.

Temperature

The normal operating temperature range of various types of cable is as follows (all temperatures in degrees centigrade):

- Neoprene -20 to +60
- PVC -20 to +80
- Polyethylene -60 to +80
- TFE Teflon -70 to +260
- FEP Teflon -70 to +260
- Polyurethane -40 to +80

For low temperatures, high-density polyethylene or polyurethane is preferable.

For high temperatures, Teflon is preferable.

Electrical Noise

Electrical noise in the ambient environment, at any location along the electrical cable, can affect the output signal. Electrical noise has many sources, for example adjacent power lines and transformers can give rise to powerful magnetic fields, which can induce unwanted alternating current and voltage on nearby sensor cables. Electrostatic radiation from circuit breakers, motor generators, nearby lightning strikes, radio beacons, vehicle ignition systems etc, all can create individual voltage spikes in the sensor cable, and unsteady readings where voltage (electrical resistance gages) or frequency (vibrating wire gages) are being read. Most of these unwanted effects can be filtered out using proper shielding techniques and appropriate electronic circuitry.

Shielded cables are essential where dataloggers are in use and where cables are long.

SUMMARY

It is hoped that the above remarks will be helpful in choosing the right cable for any application. If in doubt always consult the cable manufacturer or the instrument suppliers.

J. Barrie Sellers, President, Geokon Inc., 48 Spencer Street, Lebanon, NH 03766. Tel. (603) 448-1562, Fax. (603) 448-3216, email: barrie@geokon.com

do not print key lines

space is 42 x 20,6