Geotechnical Instrumentation NEWS

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John Dunnicliff

Introduction

This is the thirty-fifth episode of GIN. Two articles this time, and some discussions.

Measurement of Pore Water Pressures (as Opposed to Pore Gas Pressures)

Four discussions of Arthur Penman's article, published recently in *Geotechnical News (December 2002, pp 43-49)* are published immediately after this 'column', together with the author's reply.

My discussion includes two quotations from Jim Sherard's definitive 1981 paper, "*Piezometers in Earth Dam Impervious Sections*". The paper may not be readily available to some readers of GIN – if anyone wants to read the other 12 of the 14 "Summary of Main Points", please let me know, and I'll e-mail them to you.

Two of the discussions (Martin Beth's and mine) refer to an ongoing project in Europe – a cut slope in Boulder clay. Negative pore water pressures are being measured, using flushable vibrating wire piezometers with high air entry filters, to address concerns about the stability of the slope. I've arranged with the engineers who are responsible for those measurements to send me a case history for GIN when there are enough data.

The following is a summary of GIN articles and discussions on the subject of measurement of pore water pressure as opposed to gas pressure:

- Penman (2002). Measurement of Pore Water Pressures in Embankment Dams. December, pp 43-49.
- Ridley (2003). Recent Developments in the Measurement of Pore Water Pressure and Suction. March, pp 47-50.
- Thomann, Goldberg and Napolitano (2002). Are Those Pore Pressure Readings Correct? March, pp 50-53.
- Sellers (2003). Discussion of Penman (2002). June (this issue).
- Dunnicliff (2003). Discussion of Penman (2002). June.
- Mikkelsen (2003). Discussion of Penman (2002). June.
- Beth (2003). Discussion of Penman (2002). June.
- Penman (2003). Author's Reply to the above four discussions. June.
- Long and Menkiti. Future article describing measured pore water suctions in a cut slope in Boulder clay.

Time Domain Reflectometry

In the previous episode of GIN I said that it was time we had a comprehensive update on time domain reflectometry (TDR). The geotechnical practitioners who I believe know most about this in North America are, in alphabetical order, Chuck Dowding, Bill Kane and Kevin O'Connor. Here are two articles by the trio, together with Matthieu Dussud. The first describes the concept of TDR and includes a case history of deformation of a landfill slope. The second gives a crisp summary of lessons learned, relating both to sensor cable installation and to the instrumentation itself.

The March 2003 Instrumentation Course at Cocoa Beach, Florida

The course was attended by 58 registrants, and I believe that we all learned enough to justify our being there. Provisional plans for the next course are for West Coast of Florida, in the Tampa/St. Petersburg/Clearwater area, in March 2005. If you'd like to be on the mailing

Variable	Year			
	1997	1999	2002	2003
Month	November	November	March	March
Publicity	Old style - primarily paper ads and mailing of brochures		New style - paper ads, brochures plus web sites and e-mails	
Ralph Peck was present	No	No	Yes	Yes
Number of registrants	34	29	59	58

list for this, please let me know.

You know how difficult it is to interpret instrumentation data (to determine the relationship between cause and effect) when there are too many variables! Please study the data in the table on page 41.

Any interpretations? Do you think it was the month?

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.

Past the lips and over the gums, look out stomach, here she comes (USA). Thanks to Charles Daugherty for this.

Discussions of "Measurement of Pore Water Pressures in Embankment Dams"

Arthur D. M. Penman

Geotechnical News, Vol. 20 No. 4, December 2002, pp 43-49

Barrie Sellers

First I would like to thank Arthur Penman for his very interesting and informative article. I certainly learned a lot from it. There are a couple of points with which I would like to take issue. These mainly revolve around the expressed need for high air entry (HAE) filters on diaphragm type piezometers.

I am not convinced, for instance, that the results shown in Figure 7 (the figure that shows that pore pressure readings with a coarse filter were higher than those with a fine HAE filter, at Chelmarsh Dam) could not have been due to a measured suction merely in the pores of the HAE filter. We certainly know that a saturated HAE filter will, on exposure to air, create a vacuum inside a piezometer.

Also I remain unconvinced that, from a stability viewpoint, the measurement of suction pressures is at all important. To my way of thinking, the only thing that suction pressure achieves is an increase in the shear strength of the soil, due to a pulling together of the soil grains. Granted that this is desirable, but still, the main cause of instability is the hydraulic head of the ground water exerted by the totality of the water in the interconnected pores. This pressure, along with seepage pressure, acting internally in the soil mass at the edges of a fill or embankment, is the destabilizing pressure, and is the one that needs to be measured. And the measurement can be made quite adequately using a piezometer equipped with a standard low air entry filter.

If the objection is raised that this will perhaps measure the pore air (or gas) pressure then I contend that the pore air pressure can be only slightly higher than the pore water pressure and will lead to a calculation erring on the side of safety.

I am certain that a conventional diaphragm-type piezometer will never be able to measure suctions in excess of 1 bar on account of the cavitation that will take place in the space between filter and diaphragm. This same water-filled space also, perhaps, makes nonsense of the expressed need to have the filter in intimate contact with the surrounding soil. Whether or not there is a water-filled space on the outside of the piezometer, there is always going to be a water-filled space on the inside through which any capillary suction must be transmitted to the piezometer diaphragm.

I think it highly probable that an HAE filter in a partially saturated soil will, at some point, itself become partially saturated and the resultant air blockage will prevent the accurate transmission of ground water pressures to the piezometer diaphragm – another example of the Jamin, (felicitous name), effect mentioned by Penman.

I am concerned that any unusual or academic requirement for high air entry filter stones may become entrenched in practice with no benefit...

To sum up: I am concerned that any unusual or academic requirement for HAE filter stones may become entrenched in standard practice with little or no tangible benefit to compensate for the greater difficulty in saturating the filter stones and in installation procedures, and for the opportunity for spuri-

ous readings to occur due to menisci effects in the HAE filter stone itself.

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

John Dunnicliff

The pore water/pore gas issue, and the associated use of high air entry (HAE) filters is one that confuses many of us. The article by Arthur Penman helps to clarify the issue, and I thank him very much for this. But I admit to some remaining confusion.

For some applications, the need for HAE filters (together with the associated requirements about saturation and intimate soil/filter contact that Penman points out in his final paragraph) is clear. I think it is useful to paraphrase part of what I wrote in my March 2003 GIN 'column', to illustrate why this is clear, before discussing the special case of embankment dams:

- *The article by Andrew Ridley [Geotechnical News, March 2003, pp 47-50] helps us to understand the basic issues relating to negative pore water pressure (soil suction) and describes recent developments of monitoring instrumentation, including a 'flushable' piezometer, with a HAE filter. The flushable piezometer is designed to minimize the presence of air in the piezometer cavity and to provide a means of removing it if and when it enters the cavity.*
- *The article by Thomas Thomann et al [Geotechnical News, March 2003, pp 50-53] describes measurements in an organic clay layer below a large embankment in Staten Island, New York. Piezometric data were collected and used to estimate consolidation stresses and undrained shear strength, and were then used in stability analyses. The article clearly demonstrates the need to use HAE filters when gas is present in the pores.*
- *A third article will be published later in GIN, giving an ongoing case history of a project in Europe – a cut slope in Boulder clay. Negative pore water pressures are being measured, using flushable vibrating wire piezometers with HAE filters, to address concerns about the stability of the slope.*

Now to the special case of embankment dams. While interacting with Arthur Penman and Barrie Sellers during preparation of Penman's December 2002 article and these discussions, the latter reminded me of Jim Sherard's definitive 1981 paper in which he concludes (changes to bold text are mine):

If we want to measure 'pore pressure' ...should we use LAE or HAE filters? And, of course, why?

- *"In impervious embankments with clayey fines, which have appreciable initial capillary suction, the air pressure is always greater than the water pressure. The difference is generally not great when the water pressure is above atmospheric pressure.* ***There are very few situations where the difference between measured air pressure and true water pressure could have a significant influence on the evaluation of the behavior of a dam."***

and

- *"When vibrating wire or pneumatic piezometers are used for routine instrumentation in the impervious sections of embankment dams,* ***it is considered reasonable current practice to use coarse porous tips (low air entry value)."***

Further, Sherard has also written:

- *"The compacted fill in an embankment dam may remain unsaturated for a prolonged period after the reservoir is filled, and in fact the fill may never become permanently saturated by reservoir water".*

This last quotation is from page 145 in the red book – Jim Sherard and Arthur Penman were my much-needed helpers when writing the chapter on instrumentation of embankment dams.

In summary to Sherard's views, he recommends use of low air entry (LAE) rather than HAE filters on vibrating wire piezometers installed in the cores of embankment dams, even though he accepts long-term unsaturation of the core material.

These views are different from those expressed by Penman in his last paragraph: *"Modern diaphragm piezometers for installation in the cores of embankment dams should be supplied with high air entry filters...".* Therein lies my confusion, particularly as I have always greatly respected the views of both Sherard and Penman.

So I will close with two questions to Arthur Penman (some editorial prerogative here, because discussions are supposed to be discussions, and not questions!):

If we want to monitor 'pore pressure' in the core of an embankment dam, using vibrating wire piezometers, (a) for the purpose of monitoring consolidation and stability during construction and/or (b) for monitoring long-term pore pressures as a health check, should we use LAE or HAE filters? And, of course, why?

Reference

Sherard, J.L., (1981), "Piezometers in Earth Dam Impervious Sections", in Proc. ASCE Symp. on Recent Developments in Geotechnical Engineering for Hydro Projects, F.H. Kulhawy (Ed.), ASCE, New York, pp 125-165.

John Dunnicliff,
email: johndunnicliff@attglobal.net

P. Erik Mikkelsen

The paper by Dr. Penman beautifully describes the physics of pore air and water pressure in unsaturated soil and the need for saturated high air entry filters. The detailed requirements for hydraulic piezometer hardware and field procedures using Bishop tips and deaired water with the Nold DeAerator are extremely well explained. It is quite clear from his discussion that the hydraulic piezometer system needs special care and attention to detail in all aspects of installation, operation and maintenance for it to work properly.

Unfortunately the experience with hydraulic piezometers in North America has been much less than satisfactory. Lack of attention to the details described by Dr. Penman has probably been the main problem. In fact it has been so dismal that no US manufacturer of geotechnical field instrumentation makes the equipment. Instead the trend has been toward the use of pneumatic and vibrating wire pressure (VWP) sensors. High air entry filters are sometimes specified, but there are serious practical problems with their use. They simply dry out over a few months (through a diffusion process) without any facility for re-saturation. Therefore it is reasonable to use standard (low air entry) filters that do not require special saturation in the field. Under such circumstances no pore water suction will be sensed and only the air (gas) pressure will be registered. However, according to Figure 7 in Penman's paper it is reasonable to expect that pore air and water pressures merge at a positive pressure level of 0.5 to 1 atmospheres when air dissolves into water. That is significant because, after all, it is usually high pore water pressures that are of concern, not low suction in the pore water.

Fortunately there is another simple borehole installations method that can be used to improve measurements in compacted embankments. The recent trend towards the installation of VWP sensors in fully-grouted boreholes would also be applicable to measuring pore water pressure in the lower pressure and suction range. Reportedly, suction to – 52kPa below ambient atmospheric pressure has recently (Geotec Co. Ltd./ Interfels, 2002) been measured by fully grouted VWPs in boreholes above the water table under embankment fills. Not only would this method simplify the installation process in embankments, it nearly meets all of the requirements listed by Dr. Penman. The VWP cavity could still not be re-saturated, but the sensor could be protected from drying out by surrounding it with a greatly increased volume of grout.

The cement-bentonite grout to be used was the subject of the article preceding Dr. Penman's in the same issue of GIN (Mikkelsen, 2002). This type of cement-bentonite grout has properties similar to a saturated high air entry filter. It uses no sand pocket and is in intimate contact with the soil. The grout is fully saturated. A large volume of grout would give up its saturation very slowly, much slower than a filter the size of a Bishop tip. The volume of grout surrounding a VWP installed in the core of the dam should be large enough to keep the sensor cavity saturated from the time of installation to the first filling of the reservoir.

...the purpose of this discussion is... to get people moving ahead with grouted installations...

For example, an embankment VWP with a standard filter should be preinstalled inside a cylinder of cement-bentonite grout (10-cm diameter by 25-cm high), kept saturated and cured for a minimum of 7 days. Calibration checks should also be carried out in a lab environment before the field installation to ensure that the VWP responds correctly inside the grout cylinder. The response of the VWP in the grout could, for example, be tested in a 2 to 3 meter tall 15-cm diameter pipe against a known head of water (Tofani, 2000). The installation in the embankment core should be made at the center of an excavated cubical pit. A pit, 60 cm cubed, would hold about 200 liters (50 gal.) of grout and is a volume three orders of magnitude greater than a Bishop tip volume (about 0.2 liters). To improve the saturation situation further, the pit could be filled with water prior to installation and the installation and grouting done "in the wet". In any event, the exposed sides of the pit must be kept moist. Another variation on the installation particularly in silty and sandy soil would be to line the bottom and sides of the pit with polyethylene sheeting so only the top of the "grout cube" would be in direct contact with the embankment soil. This would effectively retard the tendency for draining and drying of the grout. The grout should be allowed to cure for a couple of days before production compaction resumes above the pit.

In conclusion, the purposes of this discussion are (1) to show appreciation for the tremendous contribution Dr. Penman has made to our profession on this subject over the last 50 years and (2) to get people moving ahead with grouted installations, something Dr. Penman did not mention. However, I have little hope that hydraulic piezometers will have a renaissance in the US based on his article. But his knowledge along with the work of other researchers on this subject in the UK helps us chart a revised course towards better understanding and improved measurements of low pore water pressures in North America.

References

Geotec Co. Ltd./ Interfels (2002). Personal communication.

Mikkelsen, P.E., (2002). Cement-Ben-

tonite Grout Backfill for Borehole Instruments. Geotechnical News, Vol. 20, No. 4, December, pp 38-42.
Tofani, G.D., (2000). Grout-in-Place Installation of Slope Inclinometers and Piezometers, in Seminar on Geotechnical Field Instrumentation at University of Washington, Am. Society of Civil Engineers, Seattle Section, Geotechnical Group.

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

Martin Beth

I have read Dr. Penman's December article about pore pressure measurement. I read it with great interest. It manages the incredible thing of making the surface tension story clearly understandable to most, and summarises very well the state-of-the-art in pore pressure measurements. Congratulations!

I have some comments about installation of piezometers, in the hope that they can be useful for the discussion.

Our company is convinced by the theory of the fully-grouted method, and we advocate it...

Our company is convinced by the theory of the fully-grouted method, and we advocate it on sites where the conditions are difficult for the conventional method (for example, multiple installations in one borehole, installations under large water pressure and water flow, etc). For simple sites, we stick to the conventional method, mainly because it is a difficult task to convince the other parties about the fully-grouted method.

I will mention a modest "half" case history from Europe. Half because I am not able to give you the full picture, as we prepared a design for a job but unfortunately lost it at the last minute.

An excavation is to be held in place mainly by the suction pressures in the local clay, during the construction phase. The specifications required installation of vibrating wire piezometers inside Plaster of Paris filters, with two tubes to allow "re-saturation" from the surface. That was to deal with the possibility of the piezometer filter and Plaster of Paris becoming unsaturated. We were worried about two things:

- Installation of Plaster of Paris at the bottom of a borehole - a difficult thing to do.
- The risk of measuring hydrostatic pressure in the re-saturation tubes for a long time after flushing.

So we proposed a fully grouted-in method, and had Geokon devise a flushable piezometer for us, with solenoid valves near the tip, so that there would not be the problem of hydrostatic head. We also based our design on the fact that the high air entry filter would be replaced by the grout surrounding the sensor.

A "cost saving" element during the negotiation with the main contractor was to put more than one piezometer per borehole, which is possible with the fully-grouted method. Because of the extremely low permeability of the clay (from memory, $1x10^{-10}$ to $1x10^{-11}$ m/sec.), we were planning to design a special grout.

Anyway, my part of the story stops here as the main contractor, who was our prospective client, finally gave the contract to another company.

Martin Beth, Operations Manager, Soldata Group, 294 avenue Georges Clemenceau, 92000 Nanterre, France, Tel. +33-1-41-44-85-10, Fax +33-1-41-44-85-11, email: martin.beth@soldata.fr

Author's Reply

Response

I am pleased to see the interest in my article and thank Barrie Sellers, John Dunnicliff, Erik Mikkelsen and Martin Beth for their contribution to the discussion. I am delighted that I have been able to shed light on the behaviour of surface tension and the part it plays in preventing gases from entering saturated fine pored filters. I thank the discussers for their kind remarks about my contribution to our profession during the past 50 years.

Properties

There are a number of things about which we should be clear when considering the measurement of pore pressures in partly saturated soils. We have discussed the surface tension of water and mentioned that it can have a value of 75mN/m, which provides strong resistance against the penetration of a gas into the small diameter holes on the surface of a saturated fine pored filter, that form the entrances into the pore space of the filter. When that surface is pressed into intimate contact with a wet soil, water in the soil can contact the water in the filter, making a continuum with no intermediate surface, so that the pressures of the two waters can reach equilibrium.

Another property of water is its high tensile strength. This has been well described by Andrew Ridley (2003) in his recent article published in the March episode of GIN, who has shown that it has a theoretical value of about 50 MPa (7200 lb/in^2) and that more than 100 years ago a value of about 700 kPa (100 lb/in^2) was measured on carefully de-aired water. All natural water contains air, as those who keep ornamental fish in a goldfish bowl know very well. Their behaviour soon tells you when they have insufficient oxygen. You can extract air from water by reducing pressure, and when your suction approaches a vacuum, the water appears to boil. Because of this it is commonly believed that water cannot be subjected to suctions greater than one atmosphere. But, as Andrew Ridley has pointed out, to form a cavity within water requires breaking the bond between adjacent molecules, which as we have seen from surface tension, is a strong bond, that provides the tensile strength of the water. It is most difficult to enclose water in a cavity such as that formed between the diaphragm and the saturated fine pored filter of a piezometer, without including some gas adhering to the surface or hidden in a corner or crevice. The laboratory piezometer developed at Imperial College (Ridley and Burland 1993, and Ridley, 2003) was designed to have a water cavity of only 3 mm^3 and it was made to an exceptionally high standard by the laboratory workshops so that the surfaces of the stainless steel body of the instrument were very smooth, free from re-entrant corners and crevices. It was fitted with a fine pored filter able to withstand a blow through pressure of 1500 kPa (220 lb/in^2), and was found, perhaps a little surprisingly, to be able to measure suctions of –1200 kPa (-170 lb/in^2).

Principles

Another thing about which we should be clear is the principle of effective stress. It was first stated by Terzaghi in 1923, again in his *Erbaumechanik* in 1925 and in English in 1936 (Terzaghi, 1936). It has been given in most textbooks on Soil Mechanics, and relates to saturated soil. Civil engineers have tended to use it rather blindly for all soils although it is well known that many soils above the water table are not fully saturated. Engineering fills placed and compacted by machinery as in embankment dams are invariably partly saturated. If not very well compacted, they may suffer collapse settlements on wetting, leading to the comments made by Jennings and Burland (1962) that they do not follow the principle of effective stress. The term "effective" is used because this stress is effective in controlling both strength and consolidation. It is the total stress minus the pore pressure. Total stress may be estimated from the weight of soil above the given point. But without knowledge of the pore pressure, the vital effective stress remains unknown. For partly saturated soils Bishop (1959) proposed his *chi* theory, using values of both pore gas and pore water pressures.

Intake Filters

Standpipe piezometers were the first type to be used in embankment dams. They require a considerable volume of water to flow through their intake filters

Engineering fills placed and compacted by machinery as in embankment dams are invariably partly saturated.

to record changes of pressure and it became usual when they were installed in boreholes to surround their filters with sand, converting their filters to quite large cylinders of sand. The two tube hydraulic piezometers required a very much smaller volume of water for their operation, so their intake filters could be small. Vibrating wire piezometers require negligible volumes of water. The effect of the volume of water needed to operate a piezometer and the size of its intake filter on its response time has been considered by Penman (1960).

Sherard (1981) regards vibrating wire piezometers as "no-flow" instruments, i.e. the volume of water needed to operate them is infinitely small so that intake filters of low permeability and relatively small size are suitable. They may be bedded into the soil during installation by coating with a slurry of the soil at about its liquid limit. This ensures intimate contact with the soil to give an air free installation that will immediately measure the existing suctions. The use of sand pockets for standpipe piezometers became so ingrained that specifications have been drawn up showing how the sand should be placed and how a seal should be formed above the pocket in a borehole, using bentonite pellets plus bentonite grout. Such pockets were then provided for all types of piezometer, including the diaphragm types that clearly do not need large intake filters. Sherard gets quite excited about it and says very clearly that these sand pockets should never be used for vibrating wire piezometers.

Fine Pored (High Air Entry) Intake Filters

The need for fine pored filters was brought home to us during the construction of the 41 m high Usk embankment dam in Wales during 1950-54. I installed twin tube hydraulic piezometers in the boulder clay fill in July 1951 at a position that would be at mid height of the first years fill. The dam was being built during three summer seasons: placing the boulder clay during the winter months was not feasible. These early measurements of pore pressure in a British dam aroused considerable interest, most particularly when the measured values exceeded the overburden pressure. Using today's terms, r_μ exceeded 150%. Head of the Geotechnical Section at BRS was Dr Cooling, who had worked with suction plates during studies of building stone, and suggested that the coarse pored filters that we were using, would permit the pressure of the gas (air) in the part saturated fill to affect the readings. An account of the Usk conditions was given by Penman (1979). We began looking for suitable fine pored materials for making the filters, but were concerned that flat discs of the type we were using would not make good contact with the fill during installation. Eventually Bishop, at Imperial Collage persuaded a ceramic manufacturer to make tapered cylindrical filters from material with small enough pores to provide a "blow through" pressure of 200 kN/m^2 (30 lb/in^2) but at the same time have a permeability of 3 x 10^{-6} cm/sec. The resulting unit is shown by Fig.1 of the paper and was first used in Selset dam during 1958/9. News of this development quickly spread throughout the geotechnical field and soon many other countries were using fine pored filters for their piezometers.

The experience of measuring pore pressures in Norwegian embankment dams has been described by DiBiagio and Kjærnsli (1985) and DiBiagio and Myrvoll (1985). For the 93m Hyttejuvet dam seven types of piezometer were used in order to assess the most suitable. This enabled comparisons to be made of the pressures measured by both coarse and fine pored filters. The conical shaped vibrating wire piezometer shown by Fig.9, with fine pored filter shaped to make good contact with the fill was developed for this dam and became the standard instrument for future dams for the next 8 years. But the results of the comparative study of filters with different thickness and fineness showed, somewhat surprisingly, that in this moraine fill no significant differences were observed in the measurements. Despite this result, a new vibrating wire piezometer, shown by Fig. 10, was designed with a special fine pored filter so that it would remain saturated. It was installed in the fill in a hole formed by a mandrel of the same shape as the nose of the piezometer, so that the filter made intimate contact with the fill. It was first installed in the Svartevann dam in 1973 and 120 had been installed by 1985. Its accuracy is claimed to be better than 1%.

In general discussion with British dam engineers it was found that they all assumed that fine-pored intake filters would be used with vibrating piezometers. In his book, Craig (1997)

My friends in Brazil tell me that the necessity to use fine pored intake filters in piezometers has been recognized since the early 1960s.

says, “A high air entry ceramic tip is essential for the measurement of pore water pressure in partially saturated soil, e.g. compacted fills.” “Coarse porous tips can only be used if it is known that the soil is fully saturated.”

My friends in Brazil tell me that the necessity to use fine pored intake filters in piezometers has been recognized since the early 1960s. They say that their experience in the measurement of pore pressures in embankment dams was sent to Dr Sherard who incorporated it in his excellent paper ‘Piezometers in Earth Dam Impervious Sections’ (1981). John Dunnicliff has extracted selected quotations from this paper, which of course is very naughty, because any bits of a paper must be considered in the context of the whole paper. So I, in my turn, will extract some pieces from the same paper. To quote from Sherard: “There are two principal reasons for using high air entry tips with diaphragm piezometers: (1) to allow measurement of the pressure in the water phase of the soil voids (instead of air pressure) and (2) to allow measurement of sub-atmospheric (capillary) water pressure in clayey embankments during construction”, and a second quote: “Since the cost and effort of providing a fine ceramic tip (high air entry value) and saturating the piezometer is not great, the question arises: ‘Why not equip all diaphragm type piezometers used in impervious dam sections with high air entry tips?’

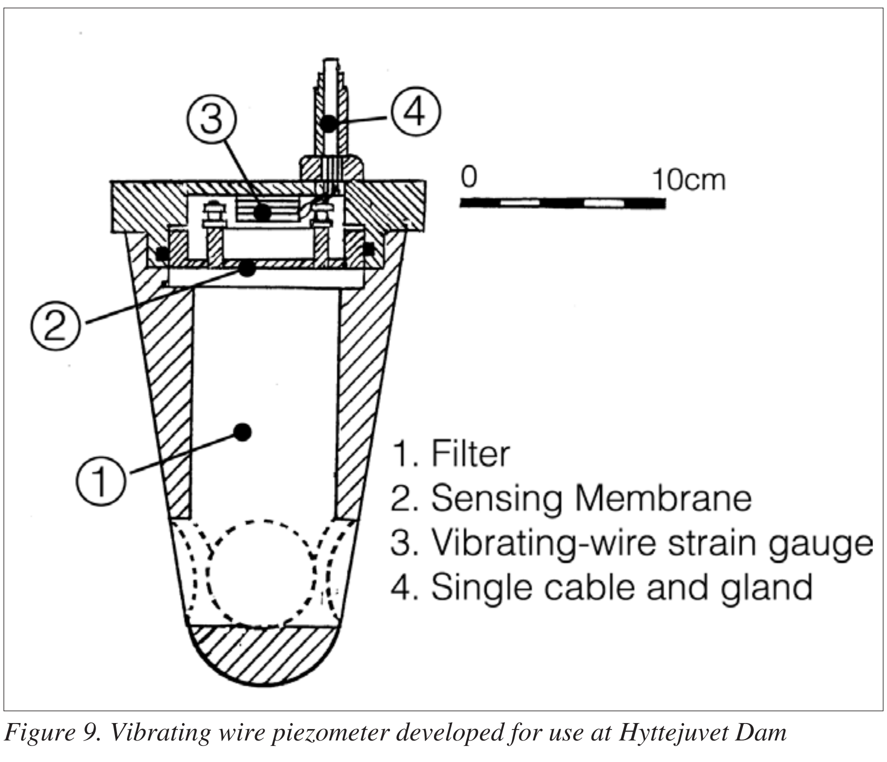

Figure 9. Vibrating wire piezometer developed for use at Hyttejuvet Dam

Points Raised by Discussers

To answer the particular points raised by the discussers, may I first turn to Barrie Sellers. He asks if the results shown by Fig 7 could not have been due to evaporation from the outer surface of the fine pored intake filter. Since it was buried in moist fill while the height of fill above it increased to 14 m over a period of several months, I think that evaporation from the filter would not have been possible. Instability in an embankment dam is not usually due to pressure from the ground water, but construction pore pressures can be of concern. With regard to measuring negative pore pressures (suctions), water has a considerable tensile strength and under suitable conditions can transmit large suctions. The paper by Ridley (2003) throws much more light on this aspect and should be referred to.

Turning to the contribution made by John Dunnicliff himself, he refers to the outstanding paper by the late Jim Sherard, whom we both greatly admire. We have taken extracts from the paper, as discussed above, but have to admit that it is a most authoritative treatment of the use of piezometers in the less pervious sections of dams. In his Acknowledgements Sherard says the greatest debt is owed to Dunnicliff who gave so much of his time and knowledge to assist the writer's effort that he should be more appropriately listed as an author rather than a main contributor. Sherard has drawn his Fig 5 (reproduced here as Fig 11 of this Reply) from Brazilian measurements made in their Tres Marias dam using three types of piezometer, all using coarse pored filters. The average placement water content was only about 2% above P.L. so that the initial suctions would be expected to be quite high. If we extend Sherard's dashed line, we see that this suction might have been –15 m head of water. This could have been measured by a vibrating wire piezometer fitted with a high air entry filter. It is important to know the response of the fill to increase of total pressure at an early stage so that if the pore water pressure is not increasing fast enough, corrections can be made to the fill before too much of it has been placed. If you do not wish to check on such behaviour and are careless about the true origin of the pore pressure to overburden relation and are not worried about air entering the filter during initial installation, then a coarse pored filter may satisfy your limited needs, but since so little effort is required to use the correct high air entry filters, there is really no need not to do so. Unless, of course, the manufacturer is unable or reluctant to supply his vibrating wire piezometers with HAE intake filters. Many manufacturers offer alternative types of intake filter and it should be possible to find suitable vibrating wire piezometers for your particular installation. Clearly both discussers are concerned about what they see as great difficulties caused by using HAE intake filters.

Both Erik Mikkelsen and Martin Beth are advocates of vibrating wire piezometers being installed in fully grouted holes.

A solution lies in the following: … The grout would form the HAE filter for the piezometer and all would be well.

The Solution

Both John Dunnicliff and Barrie Sellers are particularly concerned about the need to use fine-pored [HAE] filters with vibrating piezometers and are looking hard for a reason to use coarse filters [LAE]. John Dunnicliff asks, *"If we want to measure 'pore pressure' … should we use LAE or HAE filters? And, of course, why?*

A solution lies in the following:

Peter Vaughan (1969), faced with the problem of using several piezometers in one borehole, studied the solution of filling the borehole, including the intake filters of small bore standpipe piezometers, with a cement-bentonite grout of permeability comparable with that of the surrounding soil. True to style, he produced theory to predict expected behaviour and showed from the measured response times in the field that his theory gave satisfactory values. He showed that this technique worked even for the small bore standpipes that required some flow through the intake filters. How much better when this system is used for placing vibrating wire piezometers, with their negligible flow requirement. In addition, a grout of low permeability behaves as a fine pored intake filter and will withstand a fairly high blow through pressure. Both Erik Mikkelsen and Martin Beth are advocates of vibrating wire piezometers being installed in fully-grouted holes. This has the double advantage of bedding the intake filter correctly in the soil to ensure continuity of the pore waters of filter

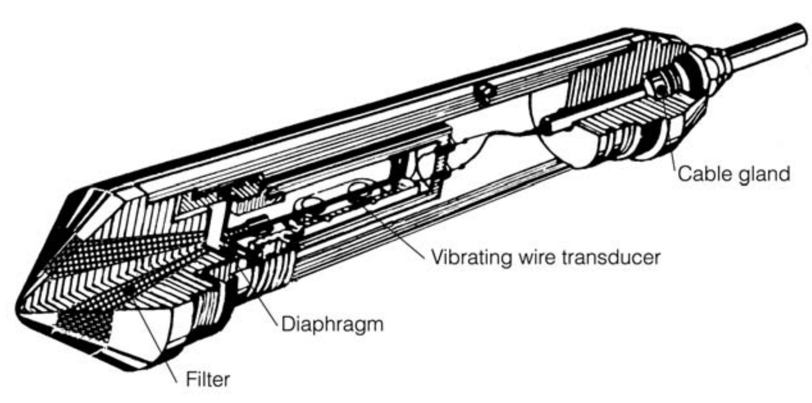

Figure 10. Geonor vibrating wire piezometer, Model S-411

and soil, as well as making the filter a HAE filter. This is excellent news and shows that vibrating wire piezometers could be installed in the fill of an embankment dam during construction by forming a hole deeper than the length of the instrument and filling it with a suitable cement-bentonite grout. The instrument, with its intake filter fully saturated and carried in a container of air free water, could be quickly inserted into the grout and pushed down to the bottom of the shallow hole, its connecting cable laid in trench, with others, to the read-out station. The grout would form the HAE filter for the piezometer and all would be well.

As mentioned above, it is important to know the response of the fill to increase of total pressure at an early stage so that if the pore water pressure is not increasing fast enough, corrections can be made to the fill before too much of it has been placed. With placed and compacted fill, the initial pore water pressures are below atmospheric pressure and it is incorrect to assume that initial pressures will be atmospheric.

I am very pleased to hear that John Dunnicliff, Erik Mikkelsen and Allen Marr are planning to make experimental tests designed to study the properties of grouts to be used to surround piezometers.

Martin Beth has given us a very clear statement that his company is convinced by the theory of the fully-grouted method and has given us a very interesting example where soil suction is to be relied on to maintain the stability of a deep excavation in clay. Because of a fear that even within the grouted borehole, the intake filter might become unsaturated, a special design was made for a vibrating wire piezometer with solenoid valves connected to flushing tubes that would connect the filter to the surface. It is interesting to note that Andrew Ridley (2003) also describes the development of a flushable electric piezometer with hydraulically operated valves, because it was felt essential to have a method of removing air from the piezometer to obtain successful long term in situ measurements of suction. I have difficulty in believing that such arrangements would be necessary for a vibrating wire piezometer, installed as I have suggested above, to measure pore water pressures in an embankment dam fill.

It is not clear to me how suction measurements can be improved by flushing water through the piezometer, when water can be expected to flow into the soil, destroying the suction that will take time to re-develop once flushing is stopped and what happens about leakage through the valves? I will be interested to see the forthcoming paper, Ridley et al (2003) to be published in the Géotechnique Symposium in Print to be discussed on 6th May 2003 in London.

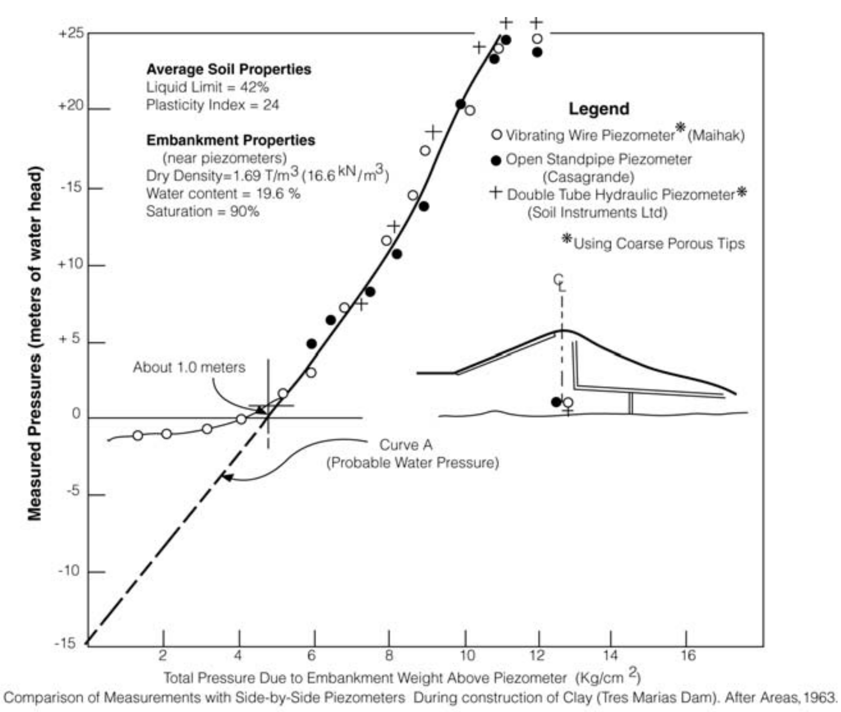

Figure 11. Pore pressures measured in the Tres Marias Dam, Brazil. (Modification of Sherard's Figure 5)

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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

Monitoring Deformation in Rock and Soil with TDR Sensor Cables

Part 1. Concept and Case History

Charles H. Dowding
Matthieu L. Dussud
William F. Kane
Kevin M. O'Connor

Historical Background

Time Domain Reflectometry (TDR) is a remote sensing electrical measurement technique that has been used for many years to determine the spatial location and nature of various cable faults. In the 1950s TDR technology was adapted to locate and identify faults in power and communication cables. As a result, TDR cable testers are considered standard equipment in these industries. In the 1970s TDR technology began to be applied to geomaterials and has been adapted for use by soil scientists, agricultural engineers, geotechnical engineers and environmental scientists. This article concentrates on the geotechnical application of monitoring subsurface deformation in soil. If there is sufficient interest, future articles in GIN could focus on use of TDR for monitoring moisture content and pore water pressure.

TDR Concept and Cable Installation

In concept, TDR is similar to radar along a cable. As shown in Figure 1b, a voltage pulse, produced by a TDR pulser, travels along a two-conductor coaxial metallic cable until it is partially reflected by deformation of the cable. The distance to the deformation can be calculated knowing the propagation velocity of the signal in the cable and the time of travel of the voltage pulse from the disruption to the cable tester. As shown in Figure 1a, a cable is grouted into a borehole, then rock or soil movement shears the grout and deforms the cable, which changes the geometry (thus impedance) between the inner and outer conductors. This change in impedance produces the reflected voltage pulses shown in Figure 1c. The travel time of the reflected pulse determines the location of the shearing zone. The amplitude of the voltage reflection is proportional to the amount of cable deformation that is correlated with the rock or soil movement.

Initially, TDR was geotechnically applied to monitor rock mass deformation, which occurs predominantly along joint interfaces (Dowding et al., 1988). The large stiffness of rock and the high degree of strain localization along rock joints allow installation with stiff cable and standard drilling and grouting procedures. As a result, the technique has been adopted worldwide by the mining industry.

At the opposite end of the spectrum of geomaterials, the low stiffness of soft soil and the relatively small strain localization in the early stages of failure in soft soils, complicate the application of TDR technology. For TDR to be effective in soil, a shear band must occur to produce the localized strain necessary to locally deform the cable. Deformation occurring along a shear band in soil must be transferred to the cable through the grout. Thus, the composite soil-grout-cable must faithfully transfer the relative soil displacement to the cable. Ideally the grout should be no more than 5 to 10 times stronger than the surrounding soil (Blackburn, 2002). A grout that is too strong may not fail with the soil and thus smears or widens the shear band, whereas a grout that is too weak will not kink or distort the cable.

A coaxial cable consists of a solid core (inner conductor) and a cylindrical shield (outer conductor), separated by a dielectric such as foam polyethylene. As shown in Figure 2, two main types of coaxial cables are recommended for TDR application. Bare solid aluminum or copper outer conductor cable are the most common types; however, more compliant copper braid outer conductor cables are also being developed for use in soft soils (Cole, 1999). At this time, the stiffer cables are commercially available while the compliant cable is under development and fabricated manually in short lengths.

Grout for TDR cable installation is typically a lean cement mix with the bentonite and water content adjusted to achieve various compressive strengths. Ideally it's viscosity should be low enough to be pumped with a drill rig water pump, but it is common to use a grout pump. The viscosity can be reduced (fluidity increased) by introducing additives such as Intrusion Aid, which also acts as an expansion agent to reduce shrinkage. Refer to Mikkelsen (2002) for an excellent discussion of grout mixing procedures and strength, as well as field crew errors in using grout mixes with higher water content and bleeding.

For best results, the cable should be installed in its own dedicated borehole and the grout must be strong enough to shear the cable, but weak enough to be failed by the surrounding soil, (Pierce, 1998). For installation in rock, this retain strength stiffness consideration is not important because of the relatively high strength and stiffness of rock. In order to maximize cable/grout composite sensitivity in soil, it has been hypothesized that the shear capacity of the grout should be less than the bearing capacity of the soil just outside the localized shear plane. This may be as high as 5 to 10 times the shear strength of the soil. However, some results of installations in soft natural soils and fills indicate the need to carefully calibrate the stiffness of the grout with the soil. More research is needed in this regard.

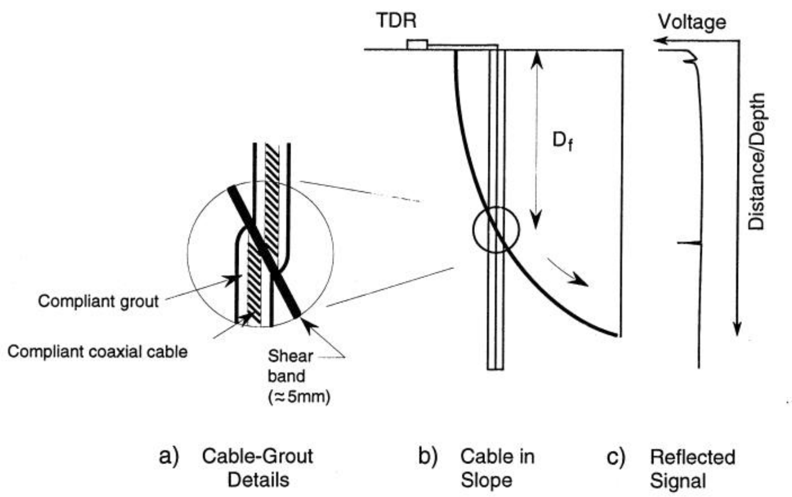

Figure 1. Shearing mechanism and induced reflection on a grouted TDR sensor cable

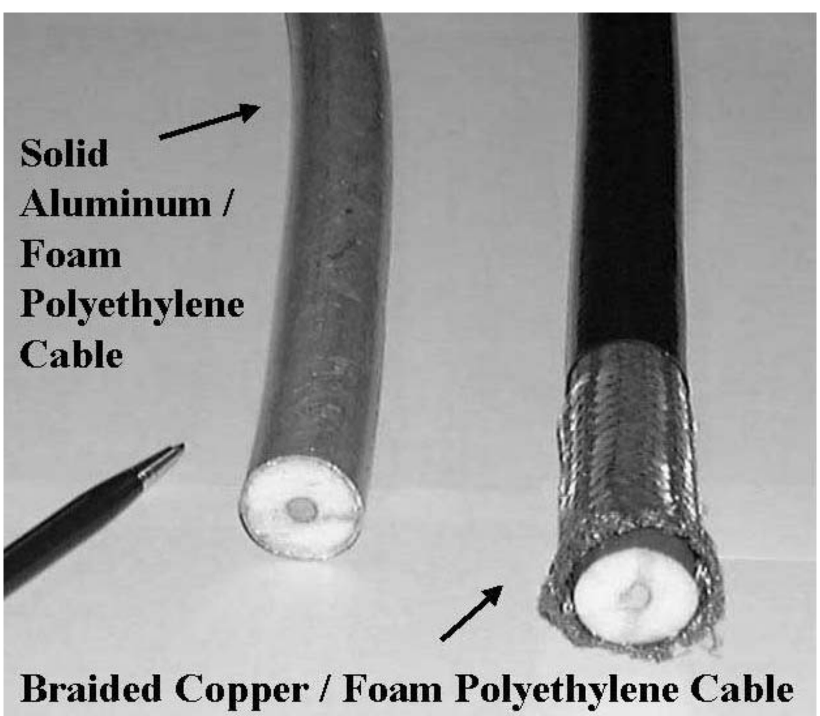

Figure 2. Two most common types of coaxial TDR sensor cables.

Deformation Modes

Crimping and localized shearing of a coaxial cable will produce a distinct TDR reflection spike such as the one in Figure 1c. If the cable is severed by shear, there is a large positive reflection immediately following the negative spike.

If the cable is simply cut off with a saw or severed in tension, there will not be a negative spike preceding the large positive reflection. Consequently, in cases where TDR has been used to monitor strata movement in mines it has been possible to determine if the strata separate in extension or shear at joints or rock mass discontinuities. It has also been possible to quantify the tensile deformation by monitoring changes in distance between crimps made in the cable prior to installation in drill holes (O'Connor and Dowding, 1999).

Correlation Between TDR Reflection Magnitude and Inclinometer Displacements in Soil

TDR technology provides a method of

deformation measurement that can be employed as a complement to, and comparison with, inclinometer measurements. The two technologies have different advantages and disadvantages. For brevity, the present discussion concentrates on the issue of localized shearing.

Inclinometers and TDR sensor cables respond differently when subjected to localized shearing. TDR sensor cables are most sensitive to highly localized shear, and have been found especially useful in rock where deformation occurs along thin joints. On the other hand, inclinometers are more sensitive to general shear or gradual changes in inclination. Localized shearing of inclinometer casing causes it to kink so it cannot be profiled with an inclinometer probe. Thus in situations involving both general shear and localized shear, the two technologies respond differently. These differences have been documented for four cases in "Comparison of TDR and Inclinometers for Slope Monitoring" (Dowding and O'Connor, 2000).

There are two alternative methods of evaluating inclinometer response: 1) total displacement or deformation profile of the casing, and 2) incremental displacement or slope of the deformation profile. Dowding and O'Connor (2000) compared inclinometer incremental displacement (IID) with TDR reflection magnitude. IID is also the inclination of the inclinometer probe, and therefore a measure of the local shear strain.

The difference in response of these two approaches results from the span over which relative displacement is measured. IID is the change in angular displacement every 60 cm (2 ft) which is the wheel-base of the standard inclinometer probe. Thus a IID of 1 mm over 60 cm (0.04 in. over 24 in.) is a slope or shear strain of 0.0017. However, this shear strain is averaged over a distance of 60 cm (24 in), which is a fairly large gage length when measuring localized shear within a discrete plane or shear band.

Conversely, the sensitivity of TDR sensor cables decreases as the shear zone increases from a thin band to a large mass undergoing general shear. O'Connor et al. (1995) reported that reflections decline by a factor of 2 when the thickness of a shear zone in the laboratory was increased from 1 mm to 40 mm, and declined by a factor of 20 when the shear zone thickness was increased to 80 mm. Thus these data could be interpreted to imply that the TDR sensor responded optimally to localized shear zones with thickness of 1/100 to 1/10 times the gage length of an inclinometer.

Example Comparison: Landfill Slope Deformation

A case history involving slope movement that occurred in an industrial landfill provides a useful comparison between inclinometer and TDR response in soil. The slide mass was some several hundred meters long and tens of meters high. As shown by the soil profile in Figure 3, the landfill rests on a very thin layer of silt and sand which is underlain by 9 to 12 m of soft, glacial lake clay, and a lower stiffer clay.

In accordance with standard geotechnical practice, inclinometers and piezometers were installed to define the extent of the slide mass and assess the effective stress within the failure "plane." As a field trial of TDR technology to detect and quantify shear within soft clays, an aluminum outer conductor coaxial cable was installed in a separate borehole 35 m from an inclinometer.

The lower bulge in the IID profile at the right of Figure 3 indicates 3 mm of incremental subsurface deformation within a shear zone at a depth of approximately 30 m within the soft to medium stiff clay layer. This depth corresponds to the zone of maximum total displacement adjacent to the IID. As shown by the 07/10/98 TDR record, there is a 5 mrho reflection spike just below the distance-calibration crimp at a depth of 22 m. This is the interface between the fill material and the underlying soft clay layer. A year later, TDR reflection spikes appeared at depths of 28 m and 31 m. These reflections correspond with the portion of the IID profile between depths 27 m and 37 m. The large IID at 8 m depth may correspond with a sliding block boundary that did not intersect the TDR cable as the inclinometer and TDR cable are separated by 35 meters.

These field measurements indicate that abrupt changes in shear strains at

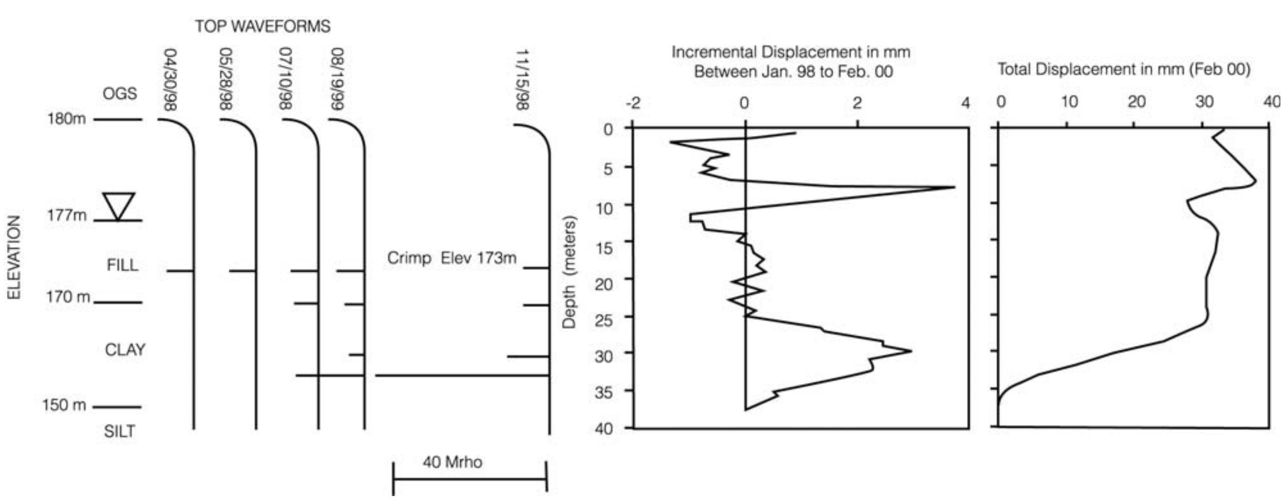

Figure 3. Comparison of TDR sensor cable and Inclinometer Response in Soft to Medium Stiff Glacial Lake Clay

the boundaries of thick shear bands in soft to medium clay with large relative displacements will produce TDR sensor cable response. TDR sensor cable response and subsequent computer modeling (Blackburn, 2002) indicate that shearing is sufficiently large at this boundary to cause a TDR reflection spike at each boundary of the localized shear. The responses at 28 m and 31 m in Figure 3 may define the thickness of the failure zone at the bottom of the sliding mass. This observation is not inconsistent with that of O'Connor et al. (1995) whose laboratory data were obtained with no confinement of the grout between the laboratory shear rings. In the field the grout is confined by the soil in the shear zone, which would change the deformation regime considerably.

Summary

Both inclinometers and TDR sensor cables will indicate the location and magnitude of subsurface shear strain. TDR sensor cables are especially sensitive to shear in rock, or in soil at locations of highly localized shear strains. On the other hand, inclinometers are especially sensitive to gradual, general shear and respond to early stages of plastic deformation in soils undergoing general shear. TDR sensor cables may also respond at abrupt changes in shear strain at the boundaries of thick localized shear zones.

The case presented here illustrates that TDR sensor cable can be used to locate and quantify localized shearing in soft soil, at least when the deformations are large. Other cases (Dowding and O'Connor, 2000) demonstrate that TDR sensor cables have detected deformation at locations where inclinometers did not detect deformation and vice versa. These differences do not imply that either method is more correct, but the two methods respond optimally to different degrees of shear localization. The real challenge is to explain these different responses more precisely.

TDR sensor cables provide another instrument to supplement and/or verify subsurface deformation measured by inclinometers. One approach that has been adopted, combines the technologies by installing TDR cables and inclinometers in separate holes and remotely interrogating TDR cables using an automated data acquisition system connected to a phone or radio modem. When the TDR cable indicates that movement has occurred, an independent measurement is then made by profiling the inclinometer casing.

Monitoring Deformation in Rock and Soil with TDR Sensor Cables

Part 2. Lessons Learned Using Time Domain Reflectometry

Charles H Dowding
Matthieu L. Dussud
William F. Kane
Kevin M. O'Connor

Introduction

Listed below are the top TDR sensor cable installation and communication lessons learned from installations by Northwestern University, KANE Geotech Inc., and GeoTDR Inc. Installations involved a wide range of situations that called for TDR monitoring of the deformation of:

- Bridge piers and abutments
- Landfills & embankments
- Rock/soil masses (sinkhole and mining-induced deformation)
- Excavations in soft soils

Top 11 "TDR Sensor Cable Installation" Lessons

1. Monitoring Over Large Surface Areas

Long (> 300m) TDR sensor cables can be installed horizontally beneath/beside highways, above mines, near landslides, etc. to monitor more surface area with fewer cables. Installation has been

Figure 1. Installation of horizontal TDR sensor cable in a grouted trench over a stabilized sinkhole

accomplished both by trenching as well as horizontal boring. Figure 1 shows the installation of a horizontal 36 m (120 ft) long TDR sensor cable in a shallow grouted trench parallel to a road subjected to sinkhole subsidence. Detailed information about this project can be found at:
http://www.iti.northwestern.edu/tdr/operational/florida.

2. *Monitoring at Great Depth*

As shown in Figure 2, deep (> 500m) vertical TDR sensor cables are being installed to monitor mine-induced deformation at great depths (O'Connor and Wade, 1994).

3. *Solid Aluminum Outer Conductor Coaxial Cable*

The current preferred cable for installation in rock and stiff to medium stiff soil is the 75 Ohm, 22 mm diameter, bare aluminum outer conductor, foam polyethylene dielectric cable (CommScope Parameter III 875 or equivalent). In order to investigate the sensitivity of a more flexible cable in soft soil a compliant cable was made by Cole (1999) by stripping the solid aluminum outer conductor from a cable. The exposed polyethylene foam was fitted with a flexible copper braided outer conductor. Studies are continuing to assess the relative strength and stiffness of similar, more flexible cables.

4. *Cables Installed in Dedicated Boreholes*

TDR sensor cables must be installed in their own hole especially in soil. **Strapping flexible cables to inclinometer casing degrades TDR sensitivity for monitoring soil deformation.** Localized shearing response is reduced by the stiffening provided by the grouted inclinometer casing. Figure 3 compares three installation geometries. The leftmost two geometries are too often chosen to save the cost of two holes (one for the inclinometer and another for the TDR sensor cable) and are not recommended.

While **not recommended**, some results may be obtained in rock by strapping TDR sensor cables outside a inclinometer casing. Figure 4 compares

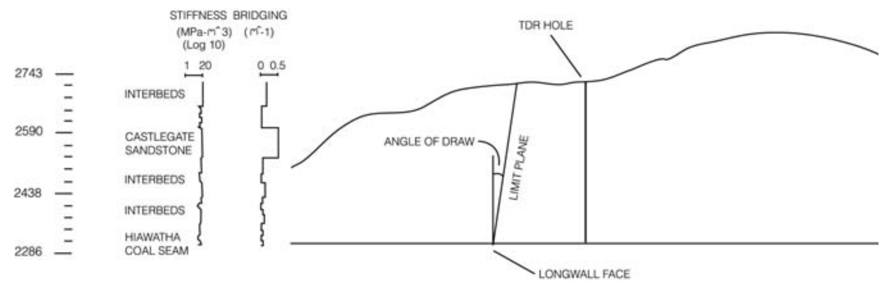

Figure 2. Cross section of the installation of a deep TDR sensor to monitor mine-induced deformation above a long wall coal mine

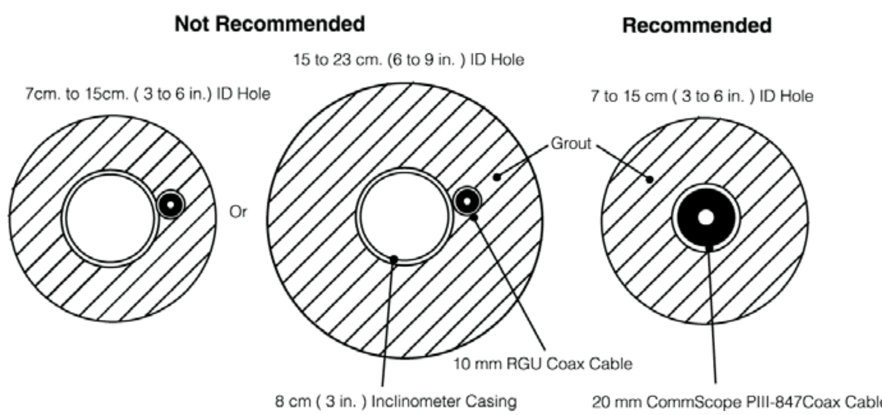

Figure 3. Comparison of geometrics of a TDR sensor cable in its own hole (recommended) and TDR sensor cable strapped around an inclinometer casing (not recommended).

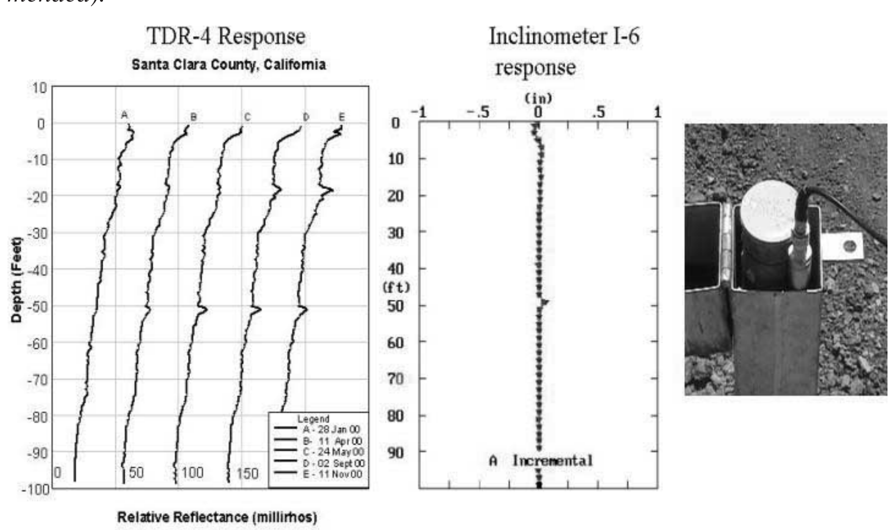

Figure 4. Comparison between response of inclinometer and strapped TDR sensor cable

Figure 5. Installation of TDR sensor cable in sheared inclinometer casing extends the life of the monitoring borehole

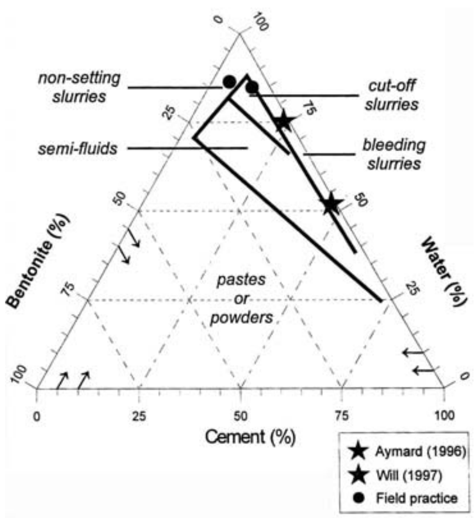

Figure 6. Principal mix proportions of cement-bentonite-water system (Aymard, 1996 and Will, 1997)

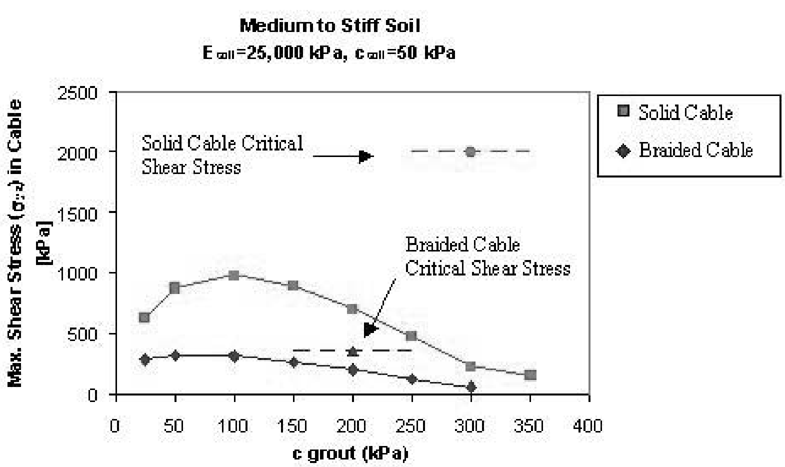

Figure 7. Plot of model shearing sensitivity of stiff and special braided flexible cable that shows there is an optimal grout strength (Blackburn, 2002)

the response of such an installation in a landslide that occurred in the California Coast Range in heavily sheared and broken Franciscan sandstone. It was particularly fast-moving and the inclinometer casing became kinked at 15.8 m (52 ft). The inclinometer probe could not be lowered below this depth after February 10, 2000. The TDR sensor cable (RG50/U), however, remained usable for some months afterward. The TDR sensor did not show a reflection until at least April 11, 2000 after a significant amount of movement occurred in the inclinometer casing. Because the TDR extended the usable life of the hole, it was able to detect an additional shear displacement at a depth of 5.5 m (18 ft) seven months after the inclinometer casing had been abandoned.

5. Retrofit Kinked Inclinometer Casing

Assessment of the response of cables installed in kinked inclinometer casing in rock indicates that TDR sensor cables can also extend the useful life of existing inclinometer instrumentation holes. Such retrofitting shown in Figure 5, allows continued monitoring deformation of critical structures without the need to drill additional holes. Solid aluminum outer conductor cable must be used and, in rapidly moving rock or soil, the cable must be installed relatively soon after the inclinometer casing has been kinked to ensure that the cable can be inserted past the kink in the casing. Pushing the cable past the kink has been a problem when using flexible coaxial cable.

6. Pumping Grout

Specialized low strength cement bentonite grout mixtures shown in Figure 6 have been employed for TDR installation in soil (Aymard, 1996 and Will, 1997). For least installation cost, they should be able to be tremmie pumped with the drilling rig's water pump. At first these mixtures appear to be more viscous than the higher strength cement only mixtures. But low viscosities can be produced by the addition of fluidizing/expansion agents such as Intrusion Aid R. The fluidity achieved will have to be demonstrated to the drill-

Figure 8. View of the sealed end tip of a flexible TDR sensor cable also fitted with a plastic cone to catch the PVC grout tube for insertion

Figure 9. Insertion of TDR sensor cable in CPT rods after attaching special disposable tip

ing crew, who may not wish to pump it for fear of blocking their pump. A separate grout pump can also be used for cable installation. Refer to Mikkelsen (2002) for an excellent discussion of grout mixing procedures and strength, as well as field crews errors in using grout mixes with high water content and bleeding.

7. Installation Using Hollow Stem Augers

Installation with hollow stem augers may lead to degradation of response through two mechanisms. First, grout slumps into the large void left as the auger is extracted. Unless a sufficient head of grout is maintained in the auger as it is extracted, voids will exist between the cable and hole walls. Extra grout at a higher head should be available to fill the large annulus created as the auger is extracted. Secondly, extraction of the auger will disturb the soil around the TDR sensor cable (Dussud, 2002).

8. Soil-Grout-Cable Interaction

Use of TDR sensor cables in soft soil requires special cement-bentonite grout mixes with prehydrated bentonite and fluidizing agent which should be care-

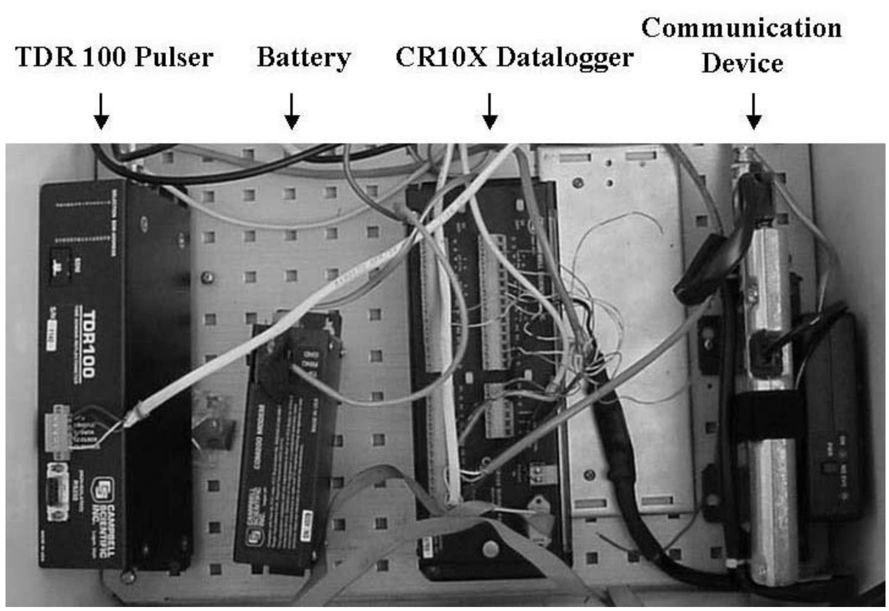

Figure 10. View of an integrated DAS comprising (from the left) a TDR 100 pulser, a 12V battery, a CR10X datalogger and communication equipment (phone modem and cellular phone)

fully designed to match the soil properties. The grout must be stiff enough to kink the cable, but not so stiff (strong) that it resists localized soil shearing. Special low loss, flexible cables will allow use of low strength grouts in soft soils. Model results in Figure 7 show that shearing sensitivity of stiff and special flexible grouted coaxial cable is optimal at a ratio of grout to soil strength of 1 to 5 (Blackburn, 2002). Shear stresses in the more compliant braided cable are closer to the critical value, which is the model shear stress associated with the first appearance of a TDR voltage reflection. More research will be needed to determine optimal grout mixtures.

9. Sealing and Insertion of Cables

Appropriate techniques for cable insertion are dependent upon cable stiffness. Flexible cables have been inserted by attaching to the cable tip a plastic cone as shown in Figure 8 in order to catch the stiff flush coupled PVC grout pipe as it is pushed in the hole. Alternatively the bottom of stiff solid aluminum coaxial cables can be fitted with a meter-long section of PVC or steel pipe (acting as a stiffener/strengthener) and then pushed down the hole. Before insertion, the bottom end of the TDR coaxial cable must be sealed to prevent intrusion of water between the inner and outer conductor.

10. Installation using CPT Rig

As shown in Figure 9, 12.5 mm diameter FLC12-50J cables have been inserted in soft soil with cone penetrometer equipment (CPT). After determination of stratigraphy the CPT rods are reinserted and the cable placed inside. A special tip is machined for the cable and left in place as the rods are withdrawn. The hole is grouted while extracting the rods. Such technique was used in a landslide in Orange County, California to install a 25 meter deep cable. There are many situations in soft soils, such as investigation of levee stability, where the CPT method works well.

11. Crimping and Connectors Details

Miscellaneous details include: 1) making distance-calibration crimps while lowering the cable to avoid accidental kinking at the crimp during installation and 2) ensuring top-of-hole connectors are moisture-proofed and placed in a locked protective cover (Dussud, 2002).

TOP 8 "TDR Instrumentation" Lessons

1. Integration of Data Acquisition Components

PC based data acquisition systems (DAS) with off-the-shelf components should be avoided because of integration and reliability problems. Reliable, rugged systems should be employed such as those offered by Campbell Scientific Inc. combining a TDR 100 pulser and a CR10X datalogger. These instruments, shown in Figure 10, also have relatively low power consumption, which is an advantage for remote site monitoring.

2. Alarm Call Capability

Automated surveillance of remote sites from a central polling computer (passive monitoring) as well as callback alarm notification from remote sites (active monitoring) has been successfully implemented with TDR (O'Connor et al, 2002). Figure 11 shows a typical DAS equipped with an alarm autodialer.

3. Web-based data Display

Autonomous posting of TDR waveforms over the internet on a daily basis has been successfully implemented for monitoring of deformation of multiple cables at multiple locations. Examples can be seen at http://www.iti.northwestern.edu/tdr (Kosnik and Kotowski, 2002).

4. Telemetry

Hard-wired phone and power lines are preferable at sites that involve real time monitoring and callback alarms. However, several truly remote operations are being operated using cell phone, radio communication and solar power (Dussud, 2002).

5. Low-loss Lead Cable

Long lead cables should be of the low loss, 75 Ohm F11 variety. The often employed, standard, 50 Ohm, RGU connecting cables should be kept as short as possible (<50 m) to minimize attenuation and noise. Such problems have

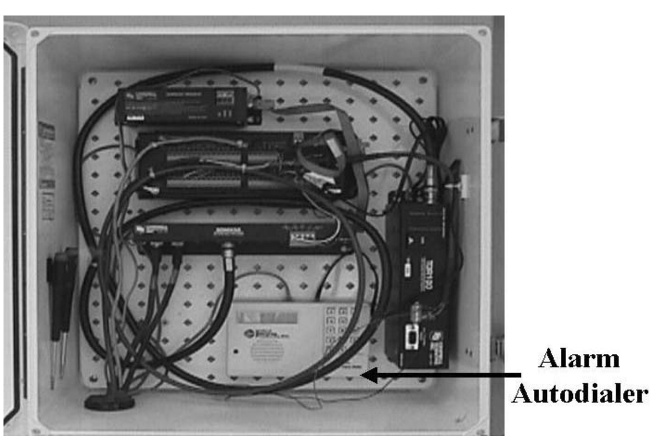

Figure 11. Remote TDR monitoring system with alarm autodialer

Figure 12. Protective enclosure for connection between a sensor and a connecting cable

arisen with RG58 and RG59 lead cables.

6. Multisensor Monitoring Systems

Integrated multiparameter monitoring systems have been implemented with tiltmeters and TDR sensor cables at remote datalogger-controlled installations. These have involved monitoring of bridge pier deformation from scour and from mining induced subsidence along highways.

7. Connector Accessibility

Connections between different cables (i.e. transmission and transducer cables) are a weak link and should be made as robust and water proof as possible. N-type connectors are recommended, but F-type have also been used. They should also be accessible for maintenance as shown in Figure 12.

8. Digital Data Format

If cables are interrogated manually with a Tektronix 1502 cable tester, it should be equipped with a SP232 module to acquire digital records for display, analysis and quantification of TDR reflections.

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Charles H. Dowding and Matthieu L. Dussud, Professor and Graduate Student: Department of Civil & Environmental Engineering, Northwestern University, Evanston, IL 60208,
Tel: 847-491-4338,
Fax: 847-491-4338,
email: c-dowding@northwestern.edu; m-dussud@northwestern.edu

William F. Kane, President, Kane Geotech, Inc., PO Box 7526, Stockton, CA 95267-0526, Tel: 209-472-1822, Fax: 209-472-0802,
email wkane@kanegeotech.com

Kevin M. O'Connor, President, GeoTDR, Inc. 720 Greencrest Drive, Westerville, OH 43081-4902,
Tel: 614-895-1400,
Fax: 614-895-1171,
email: kevin@geotdr.com