Geotechnical Instrumentation News

John Dunnicliff

Introduction

This is the twenty-sixth episode of GIN. Five articles this time, including three on the hot topic of in-place inclinometers.

In-place Inclinometers (IPIs)

In-place inclinometers (IPIs) are being used in large numbers in many countries to monitor subsurface horizontal deformation, for example around excavations, and to define landslide movements. An IPI typically consists of a series of interconnected hinged rods, installed in a vertical borehole, with a tilt sensor mounted on each rod. In this way the lengths of the rods and the output of the tilt sensors can be used to provide horizontal deformation data throughout the depth of the borehole. The most commonly used tilt sensors are electrolytic levels (electrolevels), but vibrating wire, accelerometer and magneto-resistive tilt sensors are also used. Often the tilt sensors are connected to a datalogger, with pre-set warning levels and alarm features. Performance has been very mixed, with temperature sensitivity, zero drift and ground-borne electrical noise as significant problems. There have been competing claims from manufacturers, such that some users have found it difficult to decide which commercial version to use (I am one of them!).

In an effort to shed some light on this important and rather murky subject, here are three articles. The first two, one by Jeremy Sweetman and Stephane Carayol of Sol Data, and the other by Chris Rasmussen of Instrumentation Testing and Monitoring are case histories, with conclusions that can help the rest of us.

The third article, by Jean-Ghislain La Fonta of Sol Data and myself, tells about a comprehensive test program on eight different commercial versions of IPIs, which we expect to be well underway by the time that you read this. Does this kind of thing get you excited? Me?— yes!

Instrumentation of Driven Piles

The fourth article, by Bengt Fellenius, explains how to determine load in an instrumented pile, from measurements of strain. A straightforward issue of knowing the modulus, you may think, but not so.

When I've been involved with instrumenting driven piles with strain gages, for determination of load transfer relationships during a subsequent loading test or during the onset of dragload, I've worried about the consequences of zero drift of the gages during pile driving. It's obvious that if we rely on gage zero readings taken before driving, and the gage readings DO drift during driving, we will have a problem. In discussing this with Bengt, he wrote:

"The zero drift can occur in the gage itself, for example by a slip of the fixed ends of a vibrating wire. However, zero drift can also be due to factors unrelated to the gage itself. It is, for example, of particular concern in driven H-piles, but also in other steel piles. The manufacturing process of these piles involves an unequal rate of cooling, resulting in the parts that cooled first being under compression, while the parts that cooled last are in tension. The driving of the pile causes a stress/strain equalization, manifested as a change of strain gage readings which, depending on whether the gages are in a compressed or tensioned location, can be positive or negative. A similar effect occurs in a driven concrete pile. Bored piles (drilled shafts) are not spared this problem, because the zero value will be affected by unequal response of the concrete to change in temperature and water absorption during the curing. It will also be affected by unintentional pull or push forces imposed on a gage when placing it in the pile, such as when lowering an instrumented reinforcing cage into fresh concrete. Simply stated, we may not know the zero value with anywhere near the accuracy that we believe.

"Also, we must not ignore the fact that residual stresses always exist in a pile before the test, be the pile driven or bored. If we ignore these, we will draw incorrect conclusions about the load transfer relationships. Residual stresses, or locked in loads, are consequences of penetration-rebound occurring during the driving of a pile and/or reconsolidation of the soil around the pile after driving or after construction and curing of a bored pile. Unless the pile is in a soil undergoing swell, the pile is always in compression before the start of the test".

Pungent stuff!

In the midst of my puzzling about these things, Bengt said to me that he had a method for offsetting the uncertainty of both zero drift and residual stresses. The method is briefly mentioned in the paper (Fellenius et al, 2000) that Bengt refers to in his article. To me, this seemed like getting something for nothing, some kind of magic, but Bengt insists that it is merely common sense and simple mathematics. So, having twisted his arm to create the article in this episode, I'm working on the other arm in the hope of an explanation of the magic for a future episode of GIN. Watch this space!

On a relevant point of interest, Fellenius and Haagen (1969) describe a vibrating wire strain gage, arranged in a load cell, which is capable of withstanding pile-driving forces and maintaining the true zero. The ends of the vibrating wire are in recessed lathed-out small cylinders at the end of the large steel cylinder, such that the dynamic load is not transmitted directly through the wire clamping points during the large number of impacts during pile driving. The full reference is: Fellenius, B.H. and Haagen, T. (1969), "New Pile Force Gauge for Accurate Measurements of Pile Behaviour During and Following Driving", *Can. Geotech. J.,* Vol. 6, No. 3, Aug., pp. 356-362. **To the manufacturers of vibrating wire strain gages: would it be possible to incorporate a similar feature in special versions of arc-weldable strain gages?**

More on Strain Gages and Temperature

In the September 2000 episode of GIN Storer Boone and Adrian Crawford guided us with an article on "The Effects of Temperature and Use of Vibrating Wire Strain Gauges for Braced Excavations", and Dave Druss has a discussion of that article in the December episode. In my September 'column' I said, "Does anyone have any idea on how to cope with the temperature problem when struts are exposed to changing sun and shade?" Storer Boone and Hossein Bidhendi have responded with the last of the five contributions to this episode of GIN.

Corporate Changes

Until recently Boart Longyear (a division of Anglo American Industrial Corporation) was the parent company of Slope Indicator Company (in USA), Interfels (in Germany) and Instrumentation Testing and Monitoring (ITM, in England). In September 1999 ITM was purchased by Jon Scott and Chris Rasmussen of East Sussex, England. In November 2000 Slope Indicator Company was purchased by Durham Geo-Enterprises of Stone Mountain, Georgia. Interfels remains owned by Boart Longyear.

Installation of Inclinometer Casing — Again

The last two episodes of GIN have included recommendations for overcoming buoyancy during installation of inclinometer casing, the first one focusing on the typical North American practice of using ABS casing, the second one focusing on use of PVC casing. In response to these suggestions, I've recently received a very interesting and useful draft article from Kevin Nelson, a geologist with St. Paul District Corps of Engineers, telling about his experience with using barite-bentonite weighted mud inside the casing to counteract buoyancy. The article will be in the next episode of GIN (June 2001). In the meantime, if anyone would like to have more information before then, Kevin's contact information is:
tel. (651) 290-5844,
email:Kevin.S.Nelson
@mvp02.usace.army.mil.

Cricket

For those of you who have always wondered about this fascinating game, here's what it's all about:

- You have two sides, one out in the field and one in.
- Each man that's in the side that's in goes out and when he's out he comes in and the next man goes out.
- When they are all out, the side that's out comes in and the side that's been in goes out and tries to get those coming in out.
- When both sides have been in and out, including the not outs, that's the end of the game.
- Howzat!

So now you know! Just like baseball, you see. If you want to know about "Howzat," ask an Australian, Englishman, Indian, New Zealander, Pakistani,. South African or West Indian.

Instrumentation for Pain

That's a pretty eye-catching subheading, isn't it?

I've said several times in this column that my primary purpose in spending all this time on GIN is to share with others some of the things that I've learned and that may perhaps be useful. In that context, here's a report on a recent discovery of instrumentation (not geotechnical at all, although some people have bracketed the art of geotechnical engineering with the art of medical practice, because both have naturally occurring materials as their subjects) for the relief of pain. TENS — "transcutaneous electrical nerve stimulation", if you want it in full. A small box that fits in a pocket or on a belt, connected to two pads attached (by capillary forces, I think) on either side of a place where it hurts. A low-intensity electrical charge passes between the two pads and, bingo, the pain is significantly reduced! Something to do with blocking the pain messages from reaching the brain, or increasing the number of pain-relieving endorphins, or ..who cares, as long as it stops hurting?! And it's drug free, and therefore non-intrusive. Of course no geotechnical engineer would claim that a single remedy works for all conditions, but I can vouch for its help after structural damage resulted from falling off a ladder. Sorry to go on a bit about this, but if you try it, I think you'll like it. If you do, thank my wife Irene for the discovery. For more, search the net, using "TENS Unit".

Closure

Please send contributions to this column, or an article for GIN, to me as an email attachment in ms-word to *johndunnicliff@attglobal.net, or by fax or mail: Little Leat, Whisselwell, Bovey Tracey, Devon TQ13 9LA, England. Tel. +44-1626-836161, fax +44-1626-832919.*

We had Christmas cards addressed to "Little Peat" (was this from a boring contractor?), "Little Feat" (was this from someone who wants to belittle my GIN efforts?) and "Little Feet" (no, we don't bind feet in Devon).

Here's looking at you! (England)

Vibrating Wire In-place Inclinometers - a Case History

Jeremy Sweetman
Stephane Carayol

Description of the Project

In 1998 the Mass Transit Railway Corporation (MTRC) in Hong Kong, in conjunction with Ove Arup & Partners Hong Kong Ltd. (Arup) were finalizing their design for the Tsueng Kwan O underground station and public transport interchange, a part of the Tsueng Kwan O Extension of the existing Kwun Tung Line towards Junk Bay.

A large cut-and-cover station box was to be built on reclaimed land - consisting of approximately 15m of coarse fill placed on top of 15m of marine clay, underlain by weathered granite with rock head at 50 to 60m depth. It was uncertain whether a diaphragm wall trench excavation through the marine clay would stand open long enough to provide adequate time for chiseling of the rock, and more than sufficient time for the installation of steel reinforcement and placement of concrete.

MTRC and Arup put out to tender a trial to study the stability of the marine clay and methods of ground improvement. This trial was carried out by Bachy Soletanche Group and involved the construction of a number of single (2.7m long) and double (6.6m long) diaphragm wall panels in ground of varying quality i.e. best and worst ground. Jet grouting was carried out prior to excavation of some panels. As a subsidiary of Bachy Soletanche Group, Sol Data (Asia) Ltd was awarded the instrumentation works package.

The trial procedure adopted was as follows:

1. Excavate first bite, using a 2.7m long grab, of wall trench to base of marine clay and circulate fresh bentonite to displace that contaminated by this work.
2. Excavate down to rock head and again exchange bentonite.
3. Chisel at rock head level for several hours - to see whether this would cause the trench to collapse.
4. Leave the trench standing over night.
5. Koden testing was carried out at each stage to measure the trench wall profile. This consists of measuring the width the diaphragm wall trench with an ultrasonic sonde suspended on a wireline logging cable (this equipment is manufactured by the Koden Electronics Company Ltd, Tokyo, Japan (www.koden-electronics.co.jp).
6. Repeat the procedure for the second bite of a double panel.

Vibrating Wire In-place Inclinometers

After considering products from various manufacturers Geokon's vibrating wire in-place inclinometers were selected for monitoring lateral movement of the clay, as their frequency output is independent of resistance effects and many types of electrical interference, which are often encountered in a site environment. This permitted maximum flexibility in the routing of cables to the dataloggers and the siting of the power supply generators. They were also keenly priced and available at short notice.

Having previous experience using the electrolevel type of in-place inclinometer which we found sensitive to temperature and which used much heavier connecting rods, we were very interested to try this alternative technology in the field.

The Geokon Model 6300 vibrating wire in-place inclinometer consists of a string of vibrating wire tilt sensors connected by lengths of stainless steel tubing which are linked together by universal joints (Figure 1). A spring-loaded wheel assembly designed to engage the grooves of conventional inclinometer casing is located at each joint. The inclinometer casing is installed in the normal way, either vertical or horizontal depending on the application. The string of sensors is installed inside the casing with all the sensor cables passing to the surface where they are connected to terminal boxes or data loggers. Movements of the ground deflect the casing, causing one or more of the inclinometer segments to undergo changes of inclination. Summations of all these changes in tilt are plotted to give profiles of lateral deflection.

Details of the Tilt Sensor

The vibrating wire tilt sensor is shown in Figure 2.

The pendulous mass is supported by a hinge, and is held in an off-center position by the vibrating wire. Gravity tends to pull the center of mass below the hinge, creating a tensile force in the wire, which varies with the angle of tilt.

The fact that the vibrating wire tilt sensor is a force sensor means that temperature changes, which cause the wire to change length, have only a very small effect on the sensor output. Temperature effects are predictable (usually less than 10 arc seconds/^{o}C) and can be corrected for. This is a marked improvement over the electrolytic level types that, in the experience of the authors are susceptible to temperature effects.

The precision of the vibrating wire tilt sensor is of the order of 10 arc seconds, which is somewhat less precise than the electrolytic level type (commonly stated as 1 arc second) but the vibrating wire tiltmeter has a correspondingly greater range of approximately 10 degrees. This increased range makes installation much easier, particularly in boreholes which are off-vertical.

The vibrating wire tiltmeter has very simple electronics consisting only of a coil. This makes it less susceptible than sensors containing more sophisticated circuitry, such as piezoelectric models, to damage from over-voltages. Also, the vibrating wire types can be used with long cables and the frequency output is not changed by moisture infiltration or by changes in contact resistance. Vibrating wire technology, providing good quality shielded cables and properly grounded dataloggers are used, is also resistant to electrical noise.

Instrument Configuration

One inclinometer string was installed on either side of each bite of the trial panels, keyed 2m into rock, and nominally 1m from the trench.

The instruments in each pair of holes were connected to a Campbell Scientific CR10X datalogger i.e. for the double panels one CR10X data logger was connected to the two inclinometer installations on one side and another CR10X to the two installations on the other side.

These inclinometers were not the daisy-chain version, which uses one or more pairs of cables (theoretically one pair is enough but often two pairs or more are used to provide a back-up parallel circuit in case of cable failure) to retrieve values from addressable sensors. For this project, each tilt sensor was connected with its own individual two-pair cable for vibrating wire (tilt) and thermistor (temperature) measurements.

Installation of the Equipment

Inclinometer casing of 86mm external diameter was installed, socketed 2m into the underlying rock, prior to the casting of the guide walls for the diaphragm wall panels, allowing 5 or 6 days for grout hardening prior to the installation of the in-place sensors. The casing profile was measured with a manual biaxial inclinometer system.

The in-place inclinometers were installed in the casing with 3m length rods through the fill and 1.5m length rods through the marine clay. As the main

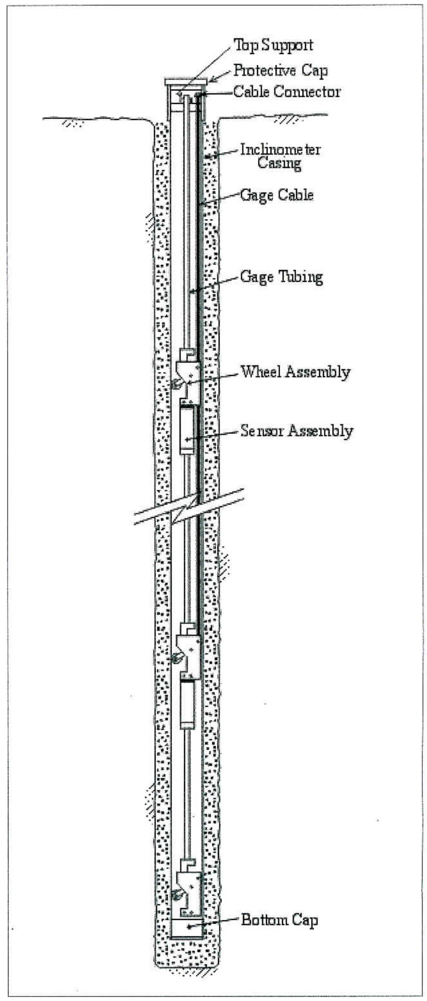

Figure 1. Typical VW in-place inclinometer installation

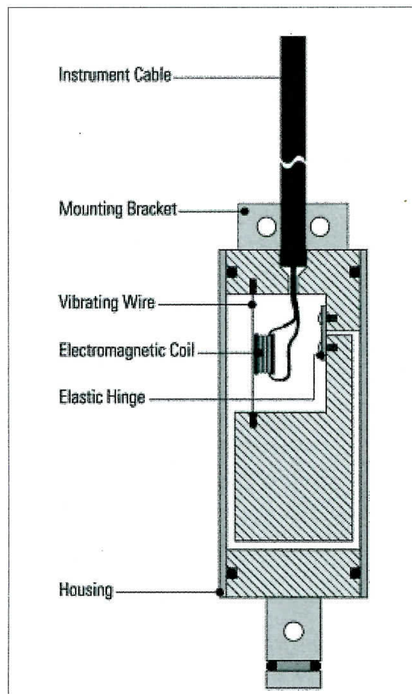

Figure 2. Schematic of VW tilt sensor

concern was relative displacement of the marine clay, the inclinometer string extended only 1.5m below the clay. Typical components for installation of the string are shown in Figure 1.

The supplied sensors were uniaxial, with fittings to allow them to be mounted as orthogonal pairs one above the other, the uppermost attached to the guide wheel assembly. Basic acceptance tests were carried out prior to joining the sensors, which were then clearly labelled with their intended orientation/position in the string assembly.

Using a supporting strain wire attached to the bottom, the first sensor pair was lowered into the casing until the connecting rod could be mounted above. The two sensor cables were taped to the rod as the assembly was lowered into the casing sufficiently far so that the next rod could be attached. This process was repeated until all 16 sensors had been joined to the string and the final support rod with its top support rested on top of the casing. As the number of cables increased more care was taken to ensure they were securely taped to the connecting rods and would not interfere with the guide wheels. The initial installation required four hours but with familiarity this dropped to a little under two hours.

Removal and Relocation of the Equipment

For the diaphragm wall trial the sensor strings were removed and re-installed at new locations twice. The removal procedure was the reverse of that for installation; with considerable care being needed to manage the cables on the ground as they were brought up. For later installations the string length was easily increased by one pair of inclinometers to accommodate increased fill thickness.

Monitoring Configuration and Software

The CR10X dataloggers were installed in large back-to-back cabinets to one side of the trial panel, with the inclinometer cables protected by steel ducts. Each cabinet also contained two multiplexer boards, a vibrating wire interface, an RS232/485 converter and a 12V battery. The dataloggers were programmed with the instrument coefficients and constants to give an output directly in millimetres per metre. Using an RS485 data cable the dataloggers were in turn connected directly to a PC computer running Sol Data's SMACS (a French acronym approximately translated as Soil Monitoring And Control System) monitoring and presentation software in a nearby monitoring container. The dataloggers were programmed to take readings at five-minute intervals, and the software was set to retrieve the latest data every five minutes, for storage in the database

As the dataloggers ran on battery power, data could be recovered from their own internal memory in the case of power outages to the monitoring PC computer (not an infrequent overnight event).

Data Management

The SMACS software was configured to display displacement profiles in both axes for each inclinometer string. This allowed the owner's representative on the site to follow critical operations in near real-time.

Using this system it was seen that the marine clay began to exhibit lateral movement when fresh bentonite was tremied into the trench upon completion of the excavation to bedrock. The development of more than 300mm of lateral movement was followed over the course of one day and confirmed by repeating the Koden test.

Data were regularly transferred to the Sol Data GeoScope data management package to print representative displacement graphs of the key events on site.

Results

The trial demonstrated that it was practicable to construct a diaphragm wall through the marine clays. The in-place inclinometers performed well and revealed some interesting effects from the softest areas of the clay, where considerable lateral movement into the trench was measured. The data obtained from the inclinometers correlated well with that from the Koden testing. A graph of representative data is shown in Figure 3.

We found the vibrating wire inclinometers to be robust enough for three consecutive installations on this project. The same equipment was subsequently shipped to Copenhagen in Denmark where it was used again. Obviously the duration of these trials is too short to determine long-term stability but over the each trial period of 10 to 14 days we did not find any drifting effects or any temperature effects.

Lessons Learned

At the time of ordering, the sensors had a reputation for fragility and therefore 12 spares were ordered. *(Geokon has since modified the design, incorporating additional shock absorbent bumpers and by strengthening the hinge, to make the sensor less delicate and susceptible to damage that might be brought about by shock or rough handling.)* Good care was taken not to drop them and no sensor faults were identified during the trial, excepting one bracket that had been welded out of alignment. The same equipment was subsequently shipped to Denmark for use on a project there and only one sensor was damaged in transit

The first inclinometer casings were installed with 250mm telescopic couplings in the fill and clay because of the perceived risk of plant causing settlement and casing deformation. Unfortunately these couplings did not include a key-way over the telescopic length and this caused a problem with the guide wheels, which are free to rotate to accommodate casing twist. To overcome this, electrical tape was used to restrict the rotational movement, and a radius curve was cut on the leading edge of the wheel assembly to avoid it jamming on the coupling step. Subsequent installations were made without telescopic couplings. The manufacturer has since modified the profile of the wheel assemblies and the authors recommend that only telescopic couplings incorporating key-ways should be used in future.

Subsequent questions of absolute movement were raised by engineers who were interested in overall trench

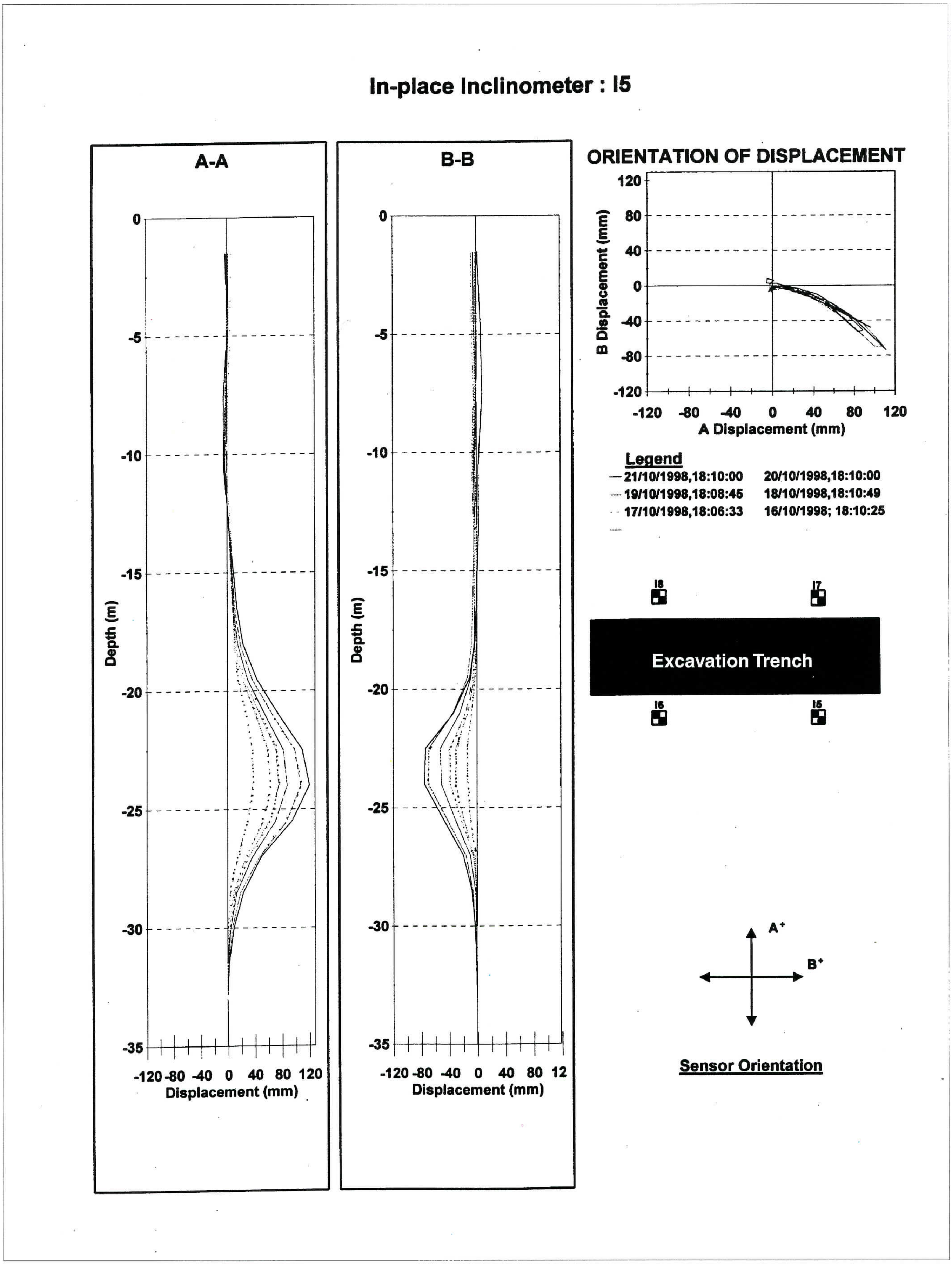

Figure 3. Typical graph showing displacements through the marine clay over a six day period

deflection as this information is used in the design of the steel reinforcement for diaphragm walls. A manual survey to check the overall deflection of the casing was performed after removal of the sensor string from the casing, indicating slight displacement below the clay of small magnitude compared with that in the clay. However, had budgetary considerations permitted it, a full string down to bedrock would have given absolute displacement data in real time rather than the relative displacement of the marine clay.

As expected, readings taken during chiseling or grabbing were unstable, as these caused vibration in the ground. Readings taken at other times were stable and no obvious temperature effects were detected despite the introduction of fresh bentonite into the trench several times during each trial excavation. The sensors were not damaged in any way by the vibrations caused by the excavation plant.

Summary

The performance of the equipment on this project was satisfactory and no significant problems were experienced. For repeated installations like this there may be some benefits from using a daisy-chain version because of its simplified cabling. However, this must be weighed carefully against the possible drawbacks, such as not easily being able to substitute for a damaged sensor, inability to adjust the configuration on site, and the longer lead time required to manufacture such systems to special order.

Acknowledgements

We are grateful for the assistance of Mr Tony Simmonds of Geokon Inc, 48 Spencer Street, Lebanon, NH 03766, USA, Tel: (603) 448-1562, Fax: (603) 448-3216, e-mail: tony@geokon.com, who provided the schematic diagrams and technical details of the vibrating wire tilt sensors.

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Electrolevels — A European View

Chris Rasmussen

Introduction

In his article on pages 26-30 of the December 2000 issue of *Geotechnical News,* "Geotechnical Field Instrumentation — Whats New in 2000", Gordon Green refers to several concerns about use of electrolytic levels (electrolevels), including temperature sensitivity and long-term stability.

From a European perspective, it often seems that opinions from 'across the pond' over-emphasise the above concerns. In Europe and in particular in the UK, electrolevels are widely accepted and used. For example, all of the recent major UK tunnelling projects in or around London have specified comprehensive and extensive instrumentation systems which use electrolevels as their sensing elements.

I believe that this is due to two factors. First, there is more hands-on experience of electrolevels in the UK, and to a lesser degree in central Europe, than in North America. Second, there have recently been major improvements in both electrolevel vial and completed sensor design, driven from the wide use of electrolevels in the UK and Germany.

For clarification, by 'vial' I mean the electrolevel itself as delivered from the two main manufacturers, Spectron Inc. and The Fredericks Company, both based in the USA. By 'sensor' I mean the vial incorporated in a mount and installed in a beam, tiltmeter or in-place inclinometer (IPI), as manufactured by various instrumentation companies world-wide

The concerns voiced tend to be generic, but in reality should be split into two product groups, surface and sub-surface electrolevels.

Surface Systems

Since 1993 electrolevel sensors have been available that do not suffer from temperature or drift effects to any great degree. Due to their limited range and the need to zero during installation, they are suitable only for use where they are easily adjusted to their zero position.

When installed on a structure, either as a tiltmeter or in a beam and set to zero, the effects of temperature on the structure to which the sensor is fixed are typically much greater that the effects on the electrolevel vial and its mounting arrangements (if properly designed). This can lead to some confusion, because reading changes that result from temperature changes can often be interpreted as temperature sensitivity of the sensor, when in fact the real cause is structure movement. Forbes et al (1994) and Schuyler and Gularte (2000) describe this in detail (see references).

In our experience of installing many thousands of surface-mounted electrolevels on both structures and in tunnels, they perform extremely reliably and with high accuracy. Many papers have been presented on such systems, some of which are listed in references at the end of this article.

Sub-surface Systems

The situation with sub-surface sensors is somewhat different in that borehole temperatures tend to be relatively stable and of course there is no direct sunlight to warm the system. The sensors for borehole use must have a significantly larger range than those used at the surface, due to the inability to zero them easily in use.

The "observational and progressive modification" type of construction techniques which have come into use in the UK and Europe often require sub-surface monitoring in boreholes, most often to determine and vertical and/or lateral deformation of the ground. The low cost/high sensitivity demonstrated by surface systems based on electrolevels makes them particularly attractive for this task, if the range can be extended to that required for borehole use.

Until recently none of the manufacturers whose sensors we have used in the UK (mostly USA and German based), have been able to produce a wide-range electrolevel-based IPI which demonstrates sufficient stability for reliable use.

An Example of Recent Experience with Electrolevels

A recent project in central London involved a deep excavation that was immediately adjacent to many historic buildings, at least two foreign embassies and several multi-million pound private residential properties. Automated monitoring (both surface and sub-surface) of the diaphragm walls and the existing structures was required. The surface electrolevel beam and electrolevel tiltmeter arrays performed extremely well, but the sub-surface arrays of IPIs performed so poorly that they had to be replaced by 24-hour manual readings via traversing inclinometers whilst an alternative was found.

A detailed investigation was undertaken to ascertain why two types of instrument, based on essentially similar technology, worked so differently. It is relevant to note that the one that performed poorly was in the most favourable (i.e. temperature stable) environment.

Representatives from the sensor manufacturers travelled to the UK and determined that there was a high level of ground-borne electrical noise. This was being rectified (converted from and alternating current signal to a direct current voltage) by the fluid within the electrolevel vials, which was then causing an offset to the output. Various grounding and noise-reducing wiring/shielding techniques were tried, but without success, and the sensors were removed from site.

A trial was initiated to test various sensors in an attempt to resurrect the automated monitoring. As a result of this work the problem with the electrolevels was identified as being the type and the excitation of the vial. Electrolevels require a precise AC (alternating current) excitation signal at very low current to be passed through them in order to prevent a breakdown of the electrolytic fluid with consequent changes in sensitivity. They are extremely intolerant to a DC (direct current), which causes an almost instant electrochemical reaction, leading to the formation of salts on the electrodes of the vial. The investigation at the site and subsequent independent work in Germany and the US determined that in the case of in-ground sensors, for optimum performance, the excitation of the sensor must be immediately adjacent to the vial, and not remotely at the data acquisition system

After the trial both Slope Indicator Company and Interfels embarked on separate programs to re-develop their IPIs using ceramic sensors and improved excitation circuits in order to overcome the ground noise problem.

This separate development, albeit by two companies which at the time shared a common owner (Boart Longyear) lead to two distinctly different sensors. Eventually Interfels IPIs with electrolevel sensors were installed, and they performed very well. Experience with other projects has shown that this level of performance has been maintained. Both Interfels and Slope Indicator have continued their work with excitation boards located adjacent to the sensor, and both companies now offer a completed and tested product sharing a common ceramic electrolevel vial, but with different excitation circuits.

Interestingly, the problems with the IPIs on this project became quite well known in Europe, yet there is still a general acceptance of IPIs, and even the contractor on the delayed project in London continues to use the new versions. In a separate article by Sweetman and Carayol in this issue of GIN similar problems with electrolevel sensors in Hong Kong are mentioned. In that case the solution was to use vibrating wire-based in-place sensors.

Conclusion

All of the usual criteria (as defined in John Dunnicliff's 'red book') for selection and installation of instruments apply to instruments with electrolevel sensors, as with any other type. With this caveat, electrolevels have become a mainstay of the instrumentation mix in Europe, with the level of usage (and indeed acceptance) growing year on year.

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In-Place Inclinometers - A Significant Test Program

John Dunnicliff
Jean-Ghislain La Fonta

Introduction

In-place inclinometers (IPIs) are being used in large numbers in many countries to monitor subsurface horizontal deformation, for example around excavations, and to define landslide movements. An IPI typically consists of a series of interconnected hinged rods, installed in a vertical borehole, with a tilt sensor mounted on each rod. In this way the lengths of the rods and the output of the tilt sensors can be used to provide horizontal deformation data throughout the depth of the borehole. The most commonly used tilt sensors are electrolytic levels (electrolevels), but vibrating wire, accelerometer and magneto-resistive tilt sensors are also used.

Often the tilt sensors are connected to a datalogger, with pre-set warning levels and alarm features. Performance has been very mixed, with temperature sensitivity, zero drift and ground-borne electrical noise as significant problems. There have been competing claims from manufacturers, such that some users have found it difficult to decide which commercial version to use (we are two of them!).

We have therefore initiated a comprehensive test program on eight different commercial versions of IPIs, which we expect to be well underway by the time this article is published. This article is written after receipt of all the hardware, but before the start of the test program.

Testing Laboratory

The tests will be performed at the French National Testing Laboratory (LNE, Laboratoire National d'Essais) in Paris. LNE is one of the major independent testing houses in Europe for testing and reporting on the quality and technical conformity of measuring equipment. This state company has 600 staff and more than 100 years of experience in this field.

Procedure Before Start of Testing

We have followed the customary procedure of LNE, and also of other organizations that make comparative tests among commercial products, by obtaining the IPI hardware without informing the suppliers about the test program.

A letter was sent to eight suppliers, requesting a quotation and full specifications for an in-place inclinometer. The letter was sent from a separate company, with which Sol Data has working relationships, so that there was no evidence of a link to LNE or to the authors of this article. Requirements included one string of three sensors with uniaxial sensors, for a vertical installation with 3 m spacing between adjacent sensors. Sensors were required to have a range of +/-10°, and a built-in system of temperature correction. However, the suppliers were invited to recommend a different product if they thought it would be more appropriate. It was not necessary to order dataloggers and related software, because the second author's company already owned these. The separate company's address was given for shipping purposes.

All eight quotations were accepted. After receipt in France of all eight IPIs, a letter was sent to each supplier to inform them of the planned test program, and enclosing a detailed step-by-step test procedure. Each was encouraged to review and comment on the procedure "so that we can do our best to make sure that our tests meet with your approval", and one week was available for this review. We also stated

"If we receive any questions/comments/requests from you, we reserve the right to inform each of the other seven suppliers about these, including our answers".

After receipt of the reviews, the test program will be started.

Outline of Test Program

Prior to the testing, a general description of each sensor will be made, and the supplier's specifications will be reviewed. Table 1 gives a brief outline of the test program.

After completion of the testing, a team of four people, including a mechanical engineer and an electronics engineer, will make a critical visual inspection of each sensor. This will include all the rods and other ancillary equipment that have not been subjected to testing. One sensor from each supplier will be opened for visual inspection.

Type of Test	Purpose of Test
Dielectric and resistance tests	To indicate the quality of the electrical isolation
Determination of calibration curve, together with numerical values for repeatability, linearity, hysteresis error and sensitivity	To characterize the sensor
Study of cross-axis sensitivity	To determine to what extent the sensor responds to changing inclinations in a plane perpendicular to its sensitive axis
Study of zero stability	To evaluate any zero drift
Temperature tests	To determine the sensor performance at different temperatures, and to evaluate the influence of temperature on the sensor reading
Power supply test	To evaluate the effect of changes in power supply voltage
Noise test	To examine the effect of ground-borne electrical and magnetical noise on the sensor
Vibration and shock tests	To determine the ability of the sensor to resist vibration and shock during transportation and handling
Water pressure test	To test the sensor seal and cable seal
Cable wrenching force test	To determine the required force to strip the cable from the sensor

Table 1. Outline of test program

Outline of Reporting Procedure

The data will be disseminated within the instrumentation community by several means including this magazine, the internet and a detailed hard copy report, which will be copyrighted. We expect that the reporting in this magazine and via the internet will be brief only. We expect to recover part of the cost of the test program by selling the detailed report. **If you believe that you'll be interested in having a copy of this, will you please tell the second author? The more of you who respond, the lesser will be the price!**

Listing of Suppliers

IPIs were purchased from the eight suppliers listed in Table 2.

Supplier	Type of Sensor
Applied Geomechanics, Inc., Santa Cruz, CA, USA	Electrolevel
Geokon, Inc., Lebanon, NH, USA	Vibrating wire
Glotzl GmbH, Rheinstetten, Germany	Electrolevel
Interfels GmbH, Bad Bentheim, Germany	Electrolevel
Roctest Ltd., St.-Lambert, Quebec, Canada	Accelerometer
RST Instruments Ltd., Coquitlam, BC, Canada	Electrolevel
Sisgeo S.r.l., Segrate, MI, Italy	Magneto-resistive
Slope Indicator Company, Bothell, WA USA	Electrolevel

Table 2. Suppliers and types of sensor

Budget and Funding

The cost of the test program, including buying the instruments, is approximately US$ 13,000 for each set of three IPIs, for a total cost of a little over US$ 100,000. A part of the funding is provided by the French Government, the remaining being supported by Sol Data. It is hoped that a significant part of this support will be recovered by selling the detailed report.

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From Strain Measurements to Load in an Instrumented Pile

Bengt H. Fellenius

Introduction

More and more, our profession is realizing that a conventional static loading test on a pile provides limited information. While the load-movement measured at the pile head does establish the capacity of the pile (per the user's preferred definition), it gives no quantitative information on the load-transfer mechanism (magnitude of the toe resistance and the distribution of shaft resistance). Yet, this information is what the designer often needs in order to complete a safe and economical design. Therefore, more and more frequently, the conventional test arrangement is expanded to include instrumentation to obtain the required information. Normally, the instrumentation consists of strain gages placed at selected levels to determine the load at that location for each load applied to the pile head. The gages are used to measure strain and this article provides guidelines for how to convert the strain to load.

Aspects to Consider

In arranging for instrumentation of a pile, several aspects must be considered. The gages must be placed in the correct location in the pile cross section to eliminate influence of bending moment. If the gages are installed in a concrete pile, a key point is how to ensure that the gauges survive their installation — a gage finds encountering a vibrator a most traumatic experience, for example. We need the assistance of specialists for this work. The survival of gages and cables during the installation of the pile is no less important and this requires the knowledge and interested participation and collaboration of the piling contractor, or, more precisely, his field crew.

Once the gages have survived the pile manufacture and installation — or most of the gages, a certain redundancy is advised — the test can proceed and all should be well. That is, provided we have ensured the participation of a specialist having experience in arranging the data acquisition system and the recording of the readings. Then, however, the geotechnical engineer often relaxes in the false security of having all these knowledgeable friends to rely on. He fails to realize that the reason why the friends do not interfere with the testing programme and testing method is not that they trust the geotechnical engineer's superior knowledge, but because advising on the programme and method is not their mandate.

The information obtained from a static loading test on an instrumented pile can easily be distorted by unloading events, uneven load-level durations, and/or uneven magnitude of load increments. Therefore, a static test for determining load transfer should be carried through in one continuous direction of movement and load followed by unloading without disruptions.

So, once all the thoughts, know-how, planning, and hands-on have gone into the testing and the test data are secured, the rest is straightforward, is it not? No, this is where the fun starts. These notes will address how to turn strain into load, a detail that often is overlooked in the data reduction and evaluation of the test results.

Converting to Load Using the Elastic Modulus

Strain gages are usually vibrating wire gages. The gages provide values of strain, not load, which difference many think is trivial. Load is just strain multiplied by the cross sectional area of the pile and the elastic modulus, right?

The modulus of a steel is known quite accurately, but the modulus of concrete is not. The latter can vary within a wide range, and common relations for its calculation, such as the relation between the modulus and the cylinder strength, are not reliable enough. A steel pile is only an all-steel pile in driving — during the test it is often a concrete-filled steel pipe. The modulus to use in determining the load is the combined value of the steel and concrete moduli. By the way, in calculating the concrete modulus in a concrete-filled steel pipe, would you choose the unconfined or the confined?

Well, the question of what modulus value to use is simple, one would think. Just place a gage level near the pile head where the load in the pile is the same as the load applied to the pile head, and let the data calibrate themselves, as it were, to find the concrete modulus. However, in contrast to the elastic modulus of steel, the elastic modulus of concrete is not a constant, but a function of the imposed load, or better, the imposed strain. Over the large stress range imposed during a static loading test, the difference between the initial and the final moduli for the pile material can be substantial. This is because the load-movement relationship (stress-strain, rather) of the tested pile, taken as a free-standing column, is not a straight line. Approximating the curve to a straight line may introduce significant error in the load evaluation from the strain measurement. However, the stress/strain curve can with sufficient accuracy be assumed to follow a second-degree line: $y = ax^2 + bx + c$, where y is stress, and x is strain (Fellenius, 1989). The trick is to determine the constants a and b (the constant c is zero).

The approach builds on the fact that the stress, y, can be taken as equal to the secant modulus multiplied by the strain. This is achieved by way of first determining the tangent modulus, and then using it to determine the secant modulus. The following presents the mathematics of the method.

Mathematics of the Method

For a pile taken as a free-standing column (case of no shaft resistance), the tangent modulus of the composite material is a straight line sloping from a larger tangent modulus to a smaller. Every measured strain value can be converted to stress via its corresponding strain-dependent secant modulus.

The equation for the tangent modulus line is:

(1)

$$M = \frac{d\sigma}{d\varepsilon} = A\varepsilon + B$$

which can be integrated to:

(2)

$$\sigma = A\varepsilon^2 + B\varepsilon$$

However,

(3)

$$\sigma = E_s\varepsilon$$

Therefore,

(4)

$$E_s = 0.5A\varepsilon + B$$

where

M = tangent modulus of composite pile material

E_s = secant modulus of composite pile material

σ = stress (load divided by cross section area)

$d\sigma$ = $(\sigma_{n+1} - \sigma_1)$ = change of stress from one load increment to the next

A = slope of the tangent modulus line

ε = measured strain

$d\varepsilon$ = $(\varepsilon_{n+1} - \varepsilon_1)$ = change of strain from one load increment to the next

B = y-intercept of the tangent modulus line (i.e., initial tangent modulus)

With knowledge of the strain-dependent, composite, secant modulus relation, the measured strain values are converted to the stress in the pile at the gage location. The load at the gage is then obtained by multiplying the stress by the pile cross sectional area.

Procedure

When data reduction is completed, the evaluation of the test data starts by plotting the tangent modulus versus strain for each load increment (the values of change of stress divided by change of strain are plotted versus the measured strain). For a gage located near the pile head (in particular, if above the ground surface, the modulus calculated for each increment is unaffected by shaft resistance and the calculated tangent modulus is the actual modulus. For gages located further down the pile, the first load increments are substantially reduced by shaft resistance along the pile above the gage location. Therefore the load change at the gage is smaller than the increment of load. Initially, therefore, the tangent modulus values calculated from the full load increment divided by the measured strain will be large. However, as the shaft resistance is being mobilized down the pile, the strain increments become larger and the calculated modulus values become smaller. When all shaft resistance above a gage location is mobilized, the calculated modulus values for the subsequent increases in load at that gage location are the composite tangent modulus values of the pile cross section.

For a gage located down the pile, shaft resistance above the gage will make the tangent modulus line plot below the modulus line for an equivalent free-standing column — giving the line a translation to the left. The larger the shaft resistance, the lower the line. However, the slope of the line is unaffected by the amount of shaft resistance above the gage location. The lowering of the line is not normally significant. For a pile affected by residual load, strains will exist in the pile before the start of the test. Such strains will result in a raising of the line — a translation to the right — offsetting the shaft resistance effect.

It is a good rule, therefore, always to determine the tangent modulus line by placing one or two gage levels near the pile head where the strain is unaffected by shaft resistance. An additional reason for having a reference gage level located at or above the ground surface is that such a placement will also eliminate any influence from strain-softening of the shaft resistance. If the shaft resistance exhibits strain-softening, the calculated modulus values will become smaller, and infer a steeper slope than the true slope of the modulus line. If the softening is not gradual, but suddenly reducing to a more or less constant post-peak value, a kink or a spike will appear in the diagram.

Example

To illustrate the approach, the results of a static loading test on a 20 m long Monotube pile will be used. The pile is a thin-wall steel pipe pile, tapered over the lowest 7.6 m length. (For complete information on the test, see Fellenius et al., 2000).

The soil consisted of compact sand. Vibrating wire strain gages were placed at seven levels, with Gage

Level 1 at the ground surface. Gage Levels 2 through 5 were placed at depths of about 2, 4, 9, and 12 m. Gage Level 6 was placed in the middle of the tapered portion of the pile, and Gage Level 7 was placed at the pile toe.

The loads and associated measured strains are presented in Figure 1. Because the load-strain curves of gages 1, 2, and 3 are very similar, it is obvious that not much shaft resistance developed above the Gage Level 3.

Figure 2 shows that tangent modulus values for the five gages placed in the straight upper length of the pile, Gages Levels 1 through 5. The values converge to a straight line represented by the "Best Fit Line".

Linear regression of the slope of the tangent-modulus line indicates that the initial tangent modulus is 44.8 GPa (the constant "B" in Eqs. 1 through 4).

The slope of the line (coefficient "A" in Eqs. 1 through 4) is -0.021 GPa per microstrain ($\mu\varepsilon$). The resulting secant moduli are 40.5 GPa, 36.3 GPa, 32.0 GPa, and 27.7 GPa at strain values of 200 $\mu\varepsilon$, 400 $\mu\varepsilon$, 600 $\mu\varepsilon$, and 800 $\mu\varepsilon$, respectively.

To illustrate the importance of establishing the strain dependency of the modulus: at the applied load of 2,400 KN, Gage Level 3 located at a depth of 5 m registered a strain of 625 $\mu\varepsilon$. At the same load, Gage Level 5 at a depth of 12 m registered a strain of 217 $\mu\varepsilon$. The strain values correspond to stress levels of 21.9 MPa and 9.6 MPa, respectively. If the 36 GPa average constant modulus had been used, the stress levels would have become 22.5 MPa and 7.8 MPa and the shaft resistance acting between the two levels would have been determined with an about 10 percent to 20 percent error.

The pile cross sectional area as well as the proportion of concrete and steel change in the tapered length of the pile. The load-strain relation must be corrected for the changes before the loads can be calculated from the measured strains. This is simple to do when realizing that the tangent modulus relation (the "Best Fit Line") is composed of the

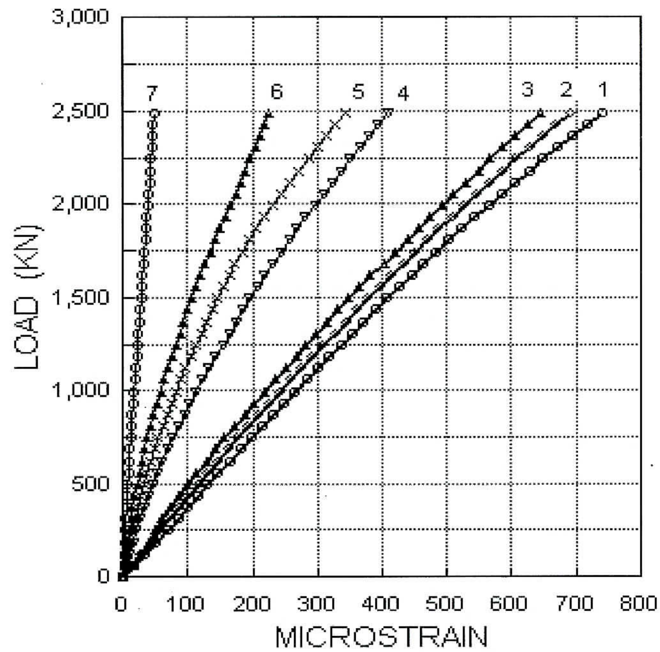

Figure 1. Strain measured at gage levels 1 through 7

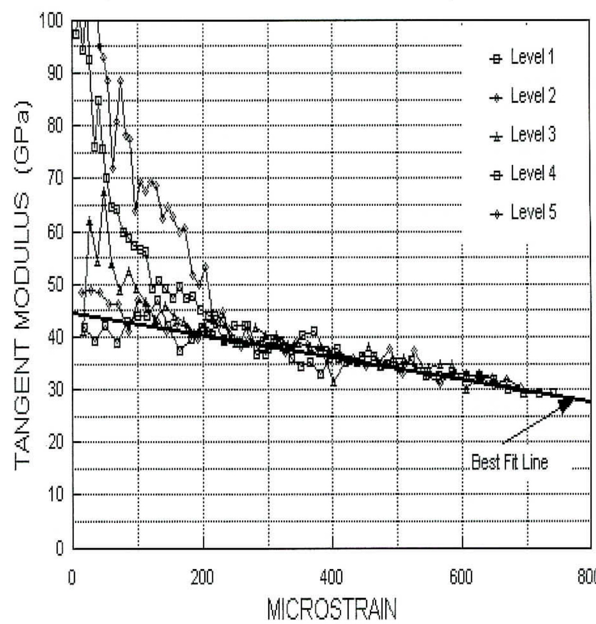

Figure 2. Tangent modulus diagram

area-weighted steel and concrete moduli. Conventional calculation using the known steel modulus provides the value of the concrete tangent modulus. The so-determined concrete modulus is then used as input to a calculation of the combined modulus for the composite cross sections at the locations of Gage Levels 6 and 7, respectively, in the tapered pile portion.

Figure 3 presents the strain gage readings converted to load, and plotted against depth to show the load distribution as evaluated from the measurements of strain used with Eq. 4. The figure presents the distribution of the

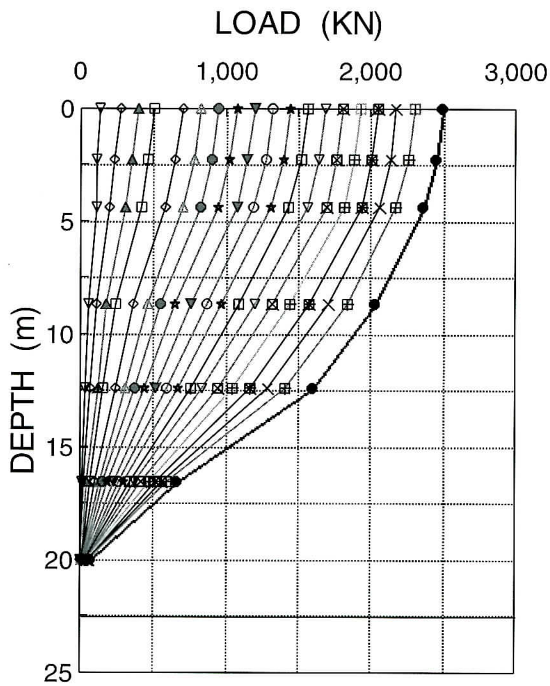

Figure 3. Load distribution for each load applied to the pile head

loads actually applied to the pile in the test. Note, however, that the strain values measured in the static loading test do not include the strain in the pile that existed before the start of the test due to residual load. Where residual loads exist, the values of applied load must be adjusted for the residual loads before the true load distribution is established.

Summary

When determining the load distribution in an instrumented pile subjected to a static loading test, engineers often assume that the loads are linearly proportional to the measured strains, and multiply the strains by a constant — the elastic modulus. However, only the modulus of steel is constant. The modulus of concrete can vary within a wide range and is also a function of the imposed load. Over the large stress range imposed during a static loading test, the difference between the initial and the final tangent moduli for the pile material can be substantial. While the secant modulus follows a curved line in the load range, in contrast, the tangent modulus of the composite material is a straight line. The line can be determined and used to establish the expression for the secant elastic modulus curve. Every measured strain value can therefore be converted to stress and load via its corresponding strain-dependent secant modulus.

For a gage located near the pile head (in particular, if above the ground surface, the tangent modulus calculated for each increment is unaffected by shaft resistance and it is the true modulus (the load increment divided by the measured strain). For gages located further down the pile, the first load increments are substantially reduced by shaft resistance along the pile above the gage location. Initially, therefore, the tangent modulus values will be large. However, as the shaft resistance is being mobilized down the pile, the strain increments become larger and the calculated modulus values become smaller. When all shaft resistance above a gage level is mobilized, the calculated modulus values for the subsequent increases in load at that gage location are the tangent modulus values of the pile cross section.

Results are presented from a static loading test on a pile equipped with vibrating wire strain gages at seven levels. The measured strains were used to plot the tangent modulus values for the gages. The modulus values converged to a straight line showing the secant moduli to reduce from about 40 GPa at the initial loads to about 28 GPa toward the end of the test. Neglecting the strain-dependency of the modulus and using a constant (an average) modulus value would have introduced errors of about 10 percent to 20 percent in shaft resistance determined from the measurements.

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(The paper by Fellenius et al., 2000, presents details on the continued analysis. A copy of the paper is available for downloading from *www.unisoftltd.com*).

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Strain Gauges, Struts and Sunshine

Storer J. Boone
Hossein Bidhendi

The September 2000 episode of GIN included an article on the effects of temperature and use of vibrating wire strain gauges for braced excavations (Boone and Crawford, 2000) and a discussion of the article followed (Druss, 2000). In his December 2000 GIN column (page 22) John Dunnicliff asked the question "Does anyone have any ideas on how to cope with the temperature problem when struts are exposed to changing sun and shade?" We have a few comments, considerations, and suggestions.

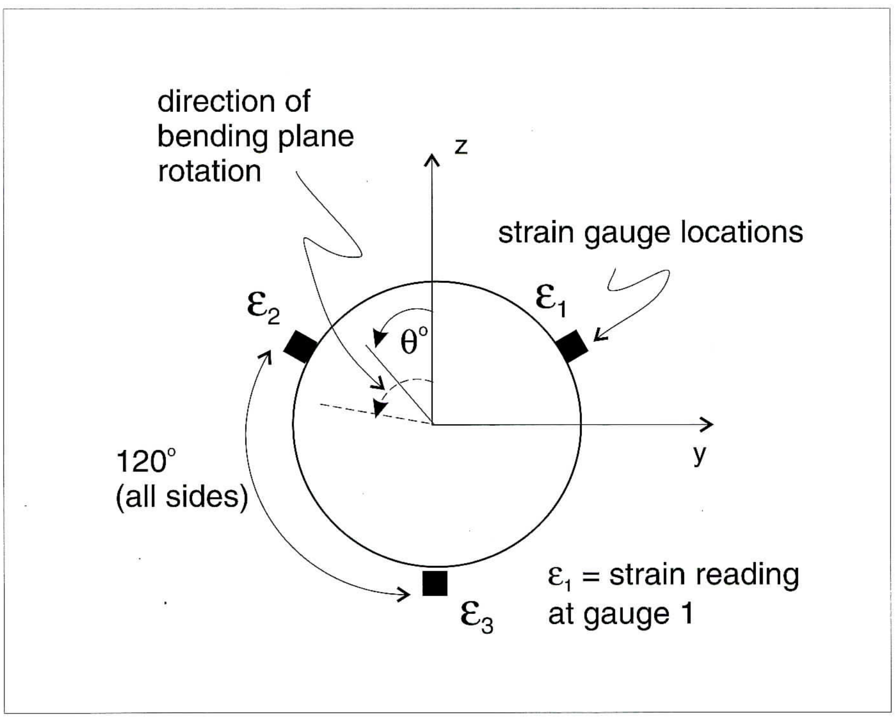

Figure 1. Cross-section through pipe strut showing strain gauge positions and direction of bending plane through strut.

$$\theta = \cos^{-1}\left(\frac{\varepsilon_1 + \varepsilon_2 - 2\varepsilon_3}{\pm 2\sqrt{(\varepsilon_1^2 + \varepsilon_2^2 + \varepsilon_3^2) - (\varepsilon_1\varepsilon_2 + \varepsilon_2\varepsilon_3 + \varepsilon_3\varepsilon_1)}}\right)$$

Strain Gauge Positions and Bending Moments

On several recent projects sets of three strain gauges were used at 120° intervals around pipe struts, with one of these located at the bottom center. During construction, it was argued that averaging the gauge readings did not give reasonable readings of the compressive loads in the struts since the average would be skewed to the two gauges nearer the top. With self-weight bending, compression load could be over-estimated by about 7% in favor of the top gauges by virtue of their positions. However, with the end of the struts fixed against rotation (welded directly to the piles) the magnitude and direction of bending also changed, depending on the wall loading conditions and exposure to sunlight (as discussed below).

By knowing the strains measured by individual gauges, however, the true plane of bending for the three-gauge measurements can be determined by the equation and Figure 1.

Based on the positions of the gauges relative to the bending plane, the compression component can be determined accordingly. Similar correction formulae could also be derived for other gauge and strut configurations. However, for complex wide flange sections such as used in Druss's case (trio of W36x393 sections bundled together flange-to-flange) the calculations could be messy (maybe impossibly so) and the use of simple averaging, more gauges, and careful selection of gauge locations would be preferable. In addition, gauges should always be mounted sufficiently distant from the connection points (3 to 4 times the maximum cross-sectional dimension) to minimize potential local end-effects.

Sunlight & Shade

When struts are exposed to sunlight, thermal, stress, and strain gradients will develop within the steel, possibly inducing bending. This plane of bending will move depending on the direction of the sunlight. For example, a strut fully exposed in an open excavation and oriented north to south could experience a rotation of the thermally-induced bending plane by 120^{o} or more between sunrise and sunset. When the sun is directly overhead compression stresses will increase in the top and exacerbate stresses induced by self-weight. Since strain is directly proportional to temperature (when no external loads change) the thermal plane of bending could also be deduced by substituting the change in temperature for strain in the equation above (for the geometry shown). For struts that are fully exposed to sunlight, directly using the strain readings in the equation above would be inclusive of external, self-weight, and thermal loads, and the resultant plane of bending would be derived. It was observed for the exposed struts in a number of recent projects that the temperature readings and plane of bending "followed the sun".

In many situations struts may not be either fully in sun or shade, and thermal gradients will exist through the strut from top-to-bottom *and* end-to-end. Consider that the steel temperature will be influenced by:

- radiation heating;
- convection heating and cooling;
- thermal conductivity and time - e.g. if a piece of steel exists part in shade and part in sun for an infinitely long period (forget about that inevitable problem of night and day) the steel temperature will eventually come to equilibrium with known temperature gradients;
- the ambient temperature in the ground, the contact between the ground and wall, and the contact between the wall and strut, and the thermal conductivity of each.

These conditions result in theoretical solutions that are complex. However, for practical purposes:

- vibrating wire strain gauges located in the shaded part of a strut will fully measure the compression and tension changes as stresses will be transferred through the entire strut (yet readings could be interpreted as non-thermally induced load changes);
- temperature gauges located in the shade will not identify the thermal effects (at the other end) that contribute to stress changes;
- gauges located in the sunlight will represent only a point measurement of the steel temperature and will indicate stresses that are not accurate for the entire strut (as the wire will respond directly to the local temperature).

With measurements of the steel temperature in the exposed area (average temperature) and measurements of load in the shaded area (based on average strains), the change in temperature and load could be monitored and the empirical relationship between the two derived (*m* as in Boone and Crawford, 2000). For design purposes, though not thoroughly rigorous, the theoretical changes in stresses could be estimated considering a direct proportioning of the length (and thus stress and strain) to the sun and shade areas. The net temperature changes of each part of the length (considering top-to-bottom average temperature) and the methods suggested by Boone and Crawford (2000) could be used to estimate the stress changes based on estimated end-restraint conditions.

Gauge Protection

For a number of recent projects that we were involved in, the strain gages were protected by a small piece of angle steel that was spot-welded to the strut such that it provided a "shade" over the gage but allowed access, air flow, and protected the gauge from damage. The stainless steel gauge surface also had different solar radiation absorption/reflection properties than the struts (they had a rusted surface appearance). It was thought, however, that with the protection angle "shade", the thermistor gave a reasonably reliable indication of the strut steel temperature through thermal conductivity at the gauge mounting points. As stated in the "Red Book" (page 316), such protection should be designed and constructed such that it does not influence the stiffness of the strut.

Other Measurements

Druss (2000) suggests that an equivalent strut stiffness could be derived essentially by:

- measuring strain, based directly on precise measurements of length change (e.g. DEMEC gauges) - note that both Druss and we consider that thermally-matched vibrating wire strain gauges do not really measure change in length except in a constant temperature environment;
- measuring/calculating load based on use of vibrating wire strain gauges;
- when combined with temperature, the full load-deformation-temperature relationship could be deduced.

We also highly recommend taking precise measurements of actual deformations of the steel sections, especially where the sections are complex in shape, the end connections are complex, or where beam-column effects might be significant in the strut performance. In the cases used as the basis for this discussion and in Boone and Crawford (2000), we did not have the luxury of taking such measurements due to budget and access limitations.

Gauge Readings & Lessons Learned

When using thermally-matched vibrating wire strain gauges, we suggest that the following "lessons learned" be considered:

1. If the goal of using strain gauges is to measure total load in the strut (safety and ultimate loads), then the gauge readings should be used directly, as these will included all thermal and external loads;
2. Maximum compressive loads (combined thermal and external) will occur during the hottest parts of the day and season, and these times should be included in any monitoring regimen;
3. If the goal of using strain gauges is to determine the external earth

loads, for confirmation of design loading assumptions or research, then thermal effects will need to be considered, as discussed above, in Druss (2000), and in Boone and Crawford (2000);

4. If the thermal behavior of the strut is of interest it would be preferable to use a datalogger, capable of taking at least four readings per day to track daily thermal and stress cycles;
5. If struts will be in partial sunshine and shade, place the vibrating wire gauges on the shaded end and independently measure temperatures in the area of sunshine (if thermal behavior is of interest);
6. "Zero" readings should be taken *after* the strut is placed in the excavation and is supported only at the ends but *prior* to full end connection/welding (echoing David Druss's comment that a strut resting on intermittently spaced blocks may not represent the true "zero" compressive stress condition, as sagging from self-weight will influence readings);
7. When interpreting gauge readings, pay particular attention to temperature and the conditions along the strut;.
8. When preloading a strut, measurements should be taken prior to loading, at the peak jack load, and once again after the jack is removed and the end fully connected. Losses in preload can occur (see also Boone et al, 1999) and these will assist in providing confirmation of gauge readings, as Druss (2000) suggests.

Theory, Practicality and the Bottom Line

Some of the issues associated with temperature fluctuations, design of struts, and monitoring with strain gauges might appear to be excessive naval-gazing since, as in David Druss's case, designs using conservative criteria or deformation limits might easily accommodate the effects of temperature. Indeed, conventional apparent earth pressure diagrams include some effect of temperature, as they are based on measurements independent of the cause of the load or end-restraint considerations. How much of this load is due to temperature has not been well understood. Unrealistic and overly conservative criteria, however, can also result in excessive cost. With the desire to limit costs and to analyze complex problems, the use of and reliance on numerical models is becoming more common. With ever-more sophisticated instrumentation systems, more data can be collected and the detail of interpretation can be far more complicated. During subsequent debates on braced excavation design, construction, instrumentation, and performance, confusion can result when "actual loads," "earth pressures," "temperature stresses," and "apparent pressures" are all thrown into the mix. The bottom line: design of safe and economical braced excavations and interpretation of their performance must consider and weigh *all* fundamental influences on loads to form a reasoned basis for engineering judgment.

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