Geotechnical Instrumentation News

John Dunnicliff

Introduction

This is the thirty-first episode of GIN, with two articles.

Determining the Resistance Distribution in Piles

Here's another article in the series by Bengt Fellenius on his favorite subject of piles. There were three others in this magazine during last year:

- *From Strain Measurements to Load in an Instrumented Pile.* March 2001, pp 35-38.
- *Where to Plot Average Loads from Telltale Measurements in Piles*. June 2001, pp 32-33.
- *The O-Cell – An Innovative Engineering Tool.* December 2001, pp 55-59.

This one is in response to my plea for guidance on how to determine strain gage values that correspond to the "no-load" condition, and how to interpret strain gage data while recognizing that there are almost certain to be residual loads in the pile.

Several colleagues have, in the past, expressed concerns to me that strain gages might be subject to a shift in the reading, caused by dynamic forces during pile driving, for example by slippage of the wire connections of a vibrating wire gage. In such a case the gage data would be questionable. You may note that there only one minor reference to this concern in the article, in the second paragraph under the subheading 'The Reading of No-Load'. Having 'asked around', I believe that there is no likelihood of slippage of vibrating wire connections during driving, and that therefore this is a non-problem. If others have different views, I'd like to hear them.

The article is divided into two parts, with Part 1 in this episode.

Part 1 discusses the issue of what strain gage readings correspond to the 'no-load' condition in a pile loading test. It also demonstrates that if residual load is not accounted for in the analysis of data, the interpretation of the instrumentation risks being in error and the instrumentation has added very little to the value of the test. On the other hand, if residual load is accounted for, the analysis procedure (this will be described in Part 2) not only provides a correct distribution of soil resistance, but also provides the spin-off benefit of increasing the understanding of pile-soil interaction.

Part 2 will be in the next episode of GIN, will present how to make the analysis and will include examples.

Another In-place Inclinometer Case History

The article by Brian Johnson provides a good case history of use of a conventional inclinometer together with a single-sensor in-place inclinometer (IPI). The 5-foot gage length IPI was placed across a shear zone to provide near real-time data, and was removed and then replaced whenever a full set of inclinometer data was required. When selecting the gage length of the IPI for similar applications, it should be remembered that if the IPI spans a depth band significantly larger than the thickness of the shear zone, it is likely to provide false data if the gage tubing contacts the distorted casing as shear progresses.

Search Function for Articles in GIN

I now have a Microsoft Access database file that allows searching for any article in GIN since the first episode in September 1994. This has been prepared at the suggestion of Elmo DiBiagio from Norway. Articles can be searched either by author or by any word or combination of words in the titles, and there is also a report with a complete chronological list. It is planned to include the database in an instrumentation web site now under construction, but in the meantime if anyone wants a copy of the database file, please let me know.

I'll say more about the instrumentation web site in a later episode of GIN.

Research on Fiber Optic Sensors for Monitoring Deformation of Tunnels

An extensive research program is just underway in England to develop fiber optic sensors for monitoring deformation in tunnels. The two basic types of fiber optic sensors that are of interest to geotechnical and structural engineers are Fabry-Perot and Bragg Grating. Fabry-Perot sensors are available commercially for monitoring strain, temperature and pressure, and each incorporates an individual fiber optic cable. Bragg Grating systems incorporate a series of sensors on the **same** cable, and have the capability of monitoring deformation and temperature at each sensor point.

Three articles on this subject have been in previous episodes of GIN:

- Tsang C.M. and England G.L. *Potential of Fibre Optic Sensing in Geotechnical Applications.* Dec. 1995, pp 36-39.
- Idriss R.L., Kersey A.D. and Davis M. *Highway Bridge Monitoring Using Optical Fiber Sensors.* June 1997, pp 43-45.
- Choquet P., Quirion M. and Juneau F. *Advances in Fabry-Perot Optic Sensors and Instruments for Geotechnical Monitoring.* March 200, pp 35-40.

Significant efforts are underway to develop Bragg Grating systems for monitoring deformation of embankment dams (Sweden) and highway bridges (USA).

The new research project in England is directed at tunnels, under the name 'OFSTUNN' (Optical Fibre System for Tunnelling). The specific objectives are to design and manufacture an array of fiber optic sensors that can be fixed at discrete points to tunnel linings and that are able to measure accurately, reliably and economically tunnel strains and displacements associated with settlement, rotation and distortion. The research program is planned for three years. Participants are The University of Birmingham, Smart Fibres Ltd., London Underground Ltd. and SolData. If anyone is interested in further information, please contact Chris Rogers at *c.d.f.rogers@bham.ac.uk.*

I plan to include occasional progress reports in future episodes of GIN.

FMGM-2003

In the last episode of GIN I announced the plans for the next FMGM Symposium (Field Instrumentation in Geomechanics), which was to have been held in Germany in September 2003. These plans have now changed.

The symposium, which is devoted specifically to instrumentation, will be held in 2003 in Norway. It will be organized by the Norwegian Geotechnical Society, the Norwegian Geotechnical Institute and the Norwegian Public Roads Administration. Check the web site *www.fmgm.no* for details.

Watch this space for more on this symposium.

Symposium on Deformation Measurements

The 11th International Symposium on Deformation Measurements, organized by the International Federation of Surveyors, will be held in Greece on 25-28 May 2003. The main topics will include instrumentation and case studies relating to geotechnical, mining and structural engineering. Although focussing on surveying methods, the symposium is expected to be of interest to those of us involved with geotechnical instrumentation.

Visit *www.heliotopos.net/conf/11fig/.*

Closure

Please send contributions to this column, or an article for GIN, to me as an email attachment in MSWord 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.*

Sláinte Mhath ('SLANJEE VA') (Scotland)! Thanks to Irene Dunnicliff for this, and also to a waitress in Portugal for help with the spelling – figure that one out!

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Determining the Resistance Distribution in Piles

Part I. Notes on Shift of No-Load Reading and Residual Load

Bengt H. Fellenius

Introduction

A pile loading test carried out just to confirm that the pile has a certain at-least capacity, that is, a proof-test, needs no special instrumentation. However, when the purpose of the test is to provide data for design of a piled foundation, for example where the test results will be applied to piles that can be longer or shorter or have different size, for resolving a downdrag problem, or for determining the distribution of the soil resistance, then, the pile must be instrumented so that the load transfer (resistance distribution) can be determined.

With few exceptions, pile instrumentation consists of strain gages, i.e. the measurement is strain, not load. The load in the pile at a gage location is determined from the change of strain (induced when load is applied to the pile head in the test) by multiplying the strain value with the modulus of the pile material and cross sectional area. The change of strain is the strain reading minus the "zero reading", or the "initial reading" of the gage, assuming – somewhat optimistically or naively – that the reading represents the "no-load" condition (i.e., when no external load acts at the gage location). However, calling a reading "the zero reading" does not mean that its value is null – that it would represent the no-load reading. One must recognize that, at the time of the start of the loading test, loads exist in the pile and they can be large. Such loads are due to locked-in strain, i.e., strains that are present in the pile at the start of the test. Locked-in strains are the cause of loads called "residual loads". And, if residual loads are not considered in the evaluation of the measurements, the conclusions drawn from the test will be suspect. It might seem that the problem would be eliminated by relying on the gage calibration that determines the "no-load" reading of the gage. The gage reading during the test would then indicate the true load in the pile at the gage location. However, the gage may be influenced by a shift in the no-load reading resulting in a false indication of load in the pile for a no-load condition. The conditions for shift of no-load reading and the residual load will be addressed in this article. A second article will present a method for analysis and determination of residual load and true resistance in an instrumented pile.

The Reading for No-Load

A strain gage can be subjected to direct damage, such as overstressing when extracting or pushing down a rebar cage, which can cause a gage attached to the cage to be pushed or pulled beyond its safe limit. Overstressing will not only shift the gage reading for the no-load condition, it can also disturb the calibration for a change of strain, severely impairing the gage and making the data unusable for analysis. It is important to ensure that such damage be avoided, and if it yet occurs, that it be discovered, e.g., by that the gage response conflicts with values from other gages (obviously, a redundancy is necessary when planning what number of gages to place in the pile). Damage due to overstressing is usually fatal for a gage, and data from such a gage must be discarded.

Other potential occurrences are more subtle as they can occur without gage damage and only result in a change of the no-load reading of the gage, leaving the linear response calibration intact. Such occurrences are slippage of the fixed end of a vibrating wire, bending of a pile (resulting in increase of strain on one side and release of strain on the other), strain transfer between materials in the pile, and temperature change.

The influence of bending is offset by having a gage level in the pile consist of a pair of gages placed diametrically opposed at equal distance from the pile center. Of course, should one gage become damaged, the surviving gage of the pair will be affected by bending and become less "truthful". Therefore, where information from a certain gage level is important, good practice is to place four gages – two pairs – at that level to achieve redundancy. Placing three levels in a triangular orientation is not a good idea. The loss of one gage will impair the usefulness of the other two.

A transfer of strain within the pile material without a corresponding change of load in the pile can, for example, be caused by a change of net prestress in a prestressed pile, changes during the curing of the concrete in a bored pile, and relaxation of strain induced by unequal cooling during the manufacture of a steel pile. Moreover, for gages attached to the pile before it is installed, even if the gages are insensitive to temperature change, the pile material is not, and the cooler environment in the ground will have some effect on the strain in the pile across the gage length.

There is not much information available on the magnitude of the shift of the no-load reading due to such strain transfer. Although the common thought is that the effect is insignificant, it is desirable that the magnitude of such shifts be investigated (by manufacturers or other interested parties) so that the potential influence can be quantified. (For example, no-load condition strain transfer between materials due to temperature change, shrinkage, and aging can be studied by placing a sister bar in a steel pipe and attaching resistance gages to the side of the pipe, taking frequent readings before, during, and some time after filling the pipe with concrete).

To find the gage reading that represents no load in the pile, the gages need to be read several times before the start of the test. All of these readings need to be considered (and included in the report of the factual test results) to enable the engineer charged with the analysis of the test data to find the true no-load value of the gages. For example, in case of a sister bar gage used in a driven prestressed concrete pile, the first reading is always the "factory zero reading", the reading for no-load established in the gage calibration. A second reading is the reading taken immediately before placing the gages in the casting forms. Third is the reading after the release of the strands and removal of the piles from the forms. Fourth is the reading before placing the pile in the leads to start driving. Fifth is the reading immediately after completion of driving. Sixth is the reading immediately before starting the test. Similarly, in case of a sister bar in a bored pile, the second reading is taken immediately before placing the gages (attached to the rebar cage) in the shaft hole, third is when the gages have adjusted to the temperature in the ground, fourth is immediately after placing the concrete, fifth the readings (***note, plural***) taken during the curing of the concrete. Sixth, again, is the reading immediately before starting the test. The principle is that readings should be taken immediately before (and after) every event of the piling work and not just during the actual loading test. A similar sequence of readings applies to other types of piles and gages. These readings will tell what happened to the gage before the start of the test and will be helpful in assessing the possibility of a shift in the reading value representing the no-load condition.

Instrumentation cases do exist, where readings one through six are more or less identical (but for the influence of the weight of the pile, of course). However, for the majority of tests, this is not the case. The reason is that between the pile installation and the start of the test, residual load will build up in the pile. For a driven pile, this is obvious. However, residual load will also develop in a bored pile.

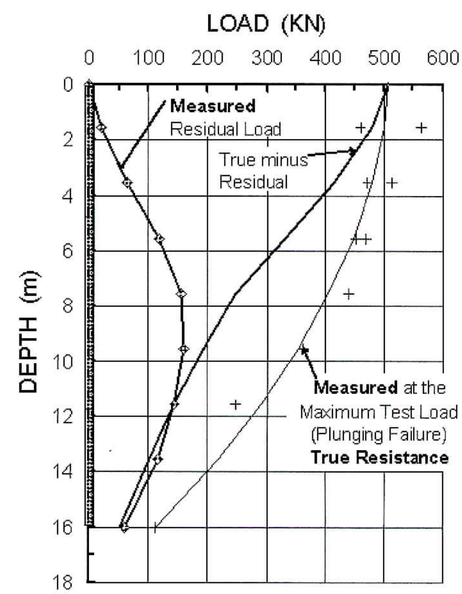

Figure 1. Measured distributions of residual load and true resistance with difference between the two (from Fellenius, 2002; data from Gregersen at al., 1973)

Residual Load

The residual load in a pile is, for example, caused by recovery of the soil after the disturbance of the installation ("set-up"), such as dissipation of induced excess pore water pressures (called "reconsolidation") be the pile driven or bored. Residual load (as well as capacity) may continue to increase after the excess pore water pressures have dissipated as the soil continues to recover from the construction disturbance. In driven piles, residual load also results from shear stress developed between the pile and the soil during the driving ("locked-in load"). Residual load is characterized by negative skin friction in the upper part of the pile, which is resisted by positive shaft resistance in the lower part of the pile and some toe resistance. (The mechanism is analogous to the build-up of dragload in a pile. The difference between residual load and dragload is merely one of preference of terms for the specific situation: "Residual load" is used when analyzing the results of a loading test and "dragload" is used when considering long-term response of a pile supporting a structure).

Residual load is associated with movement of the soil relative to the pile and the difference in stiffness between the pile and the soil. Such differences are not unique in civil engineering composite materials. For example, a reinforcing bar placed in concrete will experience noticeable compressive strain, as the concrete cures, ages, and shrinks. The stiffness ratio for steel and concrete is about 10. The stiffness ratio for pile and soil is a hundred to thousand times larger than that for steel and concrete and its effect is correspondingly more important.

The main error resulting from not recognizing the residual load in the evaluation of results from a pile loading test is that the shaft resistance appears larger than the true value, while the toe resistance appears correspondingly smaller than the true resistance. If the residual load is not considered, then, in a homogeneous soil, the results will typically show a load-transfer distribution that gets progressively steeper below approximately a third to half of the pile length. That is, the load-transfer curve denotes a unit shaft resistance that gets smaller with depth, as opposed to the resistance represented by a more realistic curve, one that becomes less steep with depth in keeping with a progressively increasing unit shaft resistance. Therefore, where residual load is present in a pile at the start of a loading test, if ignored, the measured load distribution is a false distribution of the soil resistance.

The existence of residual load in piles has been known for a long time. Nordlund (1963) is probably the first to point out its importance for evaluating load distribution from the results of an instrumented static pile loading test. However, it is not easy to demonstrate that test data are influenced by residual load. To quantify their effect is even more difficult. Regrettably, common practice is to consider the residual load to be small and not significant to the analysis and to proceed with an evaluation based on "zeroing" all gages immediately before the start of the test – solving a problem by declaring it not to exist, as it were. This is why the soil mechanics literature includes fallacies such as "critical depth" and the erroneous conclusions that unit shaft resistance would be essentially constant with depth in a homogeneous soil.

That residual load does exist and is significant is demonstrated in numerous tests on driven and bored piles (Hunter and Davisson 1969; Hanna and Tan 1973; Holloway et al. 1978; Fellenius 2002). However, most conventional static loading tests on instrumented piles do not provide the distribution of residual load in the pile immediately before the start of a test, only the load introduced in the pile during the test. An exception is presented

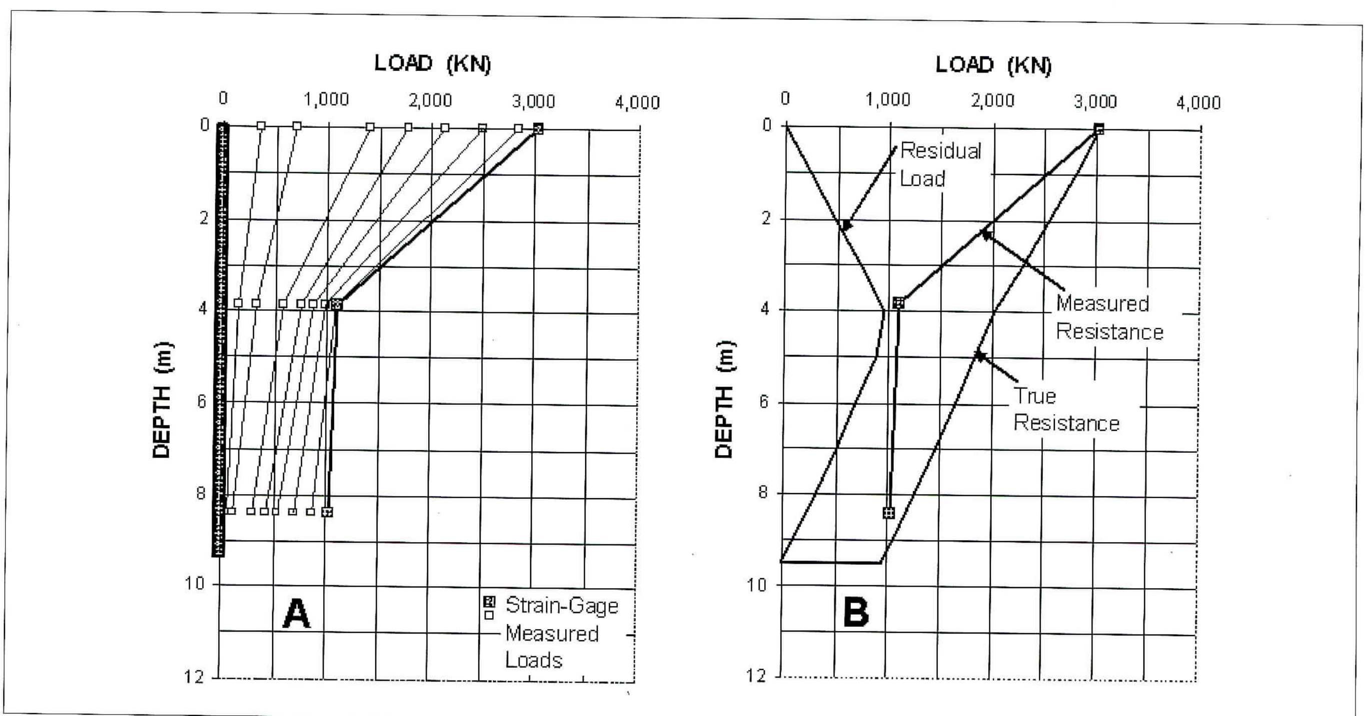

Figure 2. Load distribution in a 0.9 m diameter, 9.5 m long bored pile (from Fellenius, 2002; data from Baker et al., 1990)
2A. Measured load distributions
2B. Distributions of measured load, residual load, and true resistance (loads corrected for residual load)

by Gregersen et al. (1973) who reported tests on instrumented, 16 m long, 280 mm diameter, precast concrete piles driven into a very loose sand. The pile experienced plunging failure in the test and Fig. 1 presents the distributions of residual load (diamond symbols) and the load in the pile at the maximum test load (plus symbols). Fig. 1 shows also a curve determined by subtracting the residual values from the values measured for the maximum load. Had this test been performed without measuring the residual loads and with "zeroing" of the gages before the start of the test, the latter curve would have shown a "false" resistance that might have been taken as representative of the actual resistance distribution along the pile.

Most of the time, a test on an instrumented pile includes no measurements of the distribution of load in the pile at the start of the test. That is, whether or not and to what extent the pile is subjected to residual load is not directly known. However, on the condition that the soil profile is reasonably uniform, the measured load in the pile during the test – the "false" distribution – can still be used to determine the distributions of true load and residual load in the pile. To illustrate, Fig. 2 presents the results of a static loading test to plunging failure on a 0.9 m diameter, 9.5 m long bored pile in clay. The pile was instrumented with two levels of strain gages placed at depths of 3.8 m and 8.3 m. The strain gage values represent the load increase due to the load applied to the pile head. A series of load distribution curves are obtained by connecting the load at the pile head with the load measured at the strain gage levels.

As shown in Fig. 2A, the loads measured at the two strain-gage levels are about equal, implying that no shaft resistance exists below the depth of 3.8 m. It would appear that either one or both gages are malfunctioning. But this they are not. The distribution shown is typical of a pile affected by residual load and both gages are working well. Fig. 2B compares the measured distribution at the maximum load to the results of an analysis of the distributions of residual load and true resistance.

Fig. 3 presents results from a static loading test on a driven pile, a 21 m long Monotube pile in a loose to dense sand.

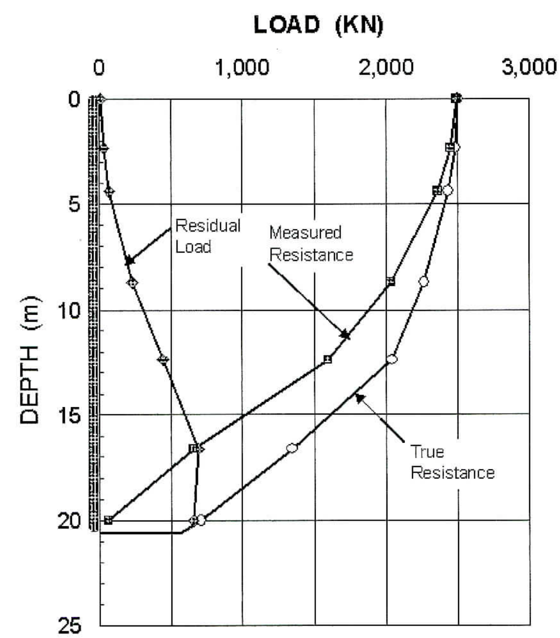

Figure 3. Distributions of measured load, residual load, and true resistance (from Fellenius et al., 2000)

(The Monotube pile is a 450-mm diameter steel pipe with a 7.6-m bottom section that tapers down to a 200-mm diameter at the pile toe). The measured distribution is shown together with the distribution of residual load and the resulting true resistance distribution. Notice that a residual load is indicated at the pile toe.

It is obvious from the results of the analysis that ignoring the residual load would have resulted in very different conclusions. For the tests shown in Figs. 2 and 3, the distributions of residual load and true resistance were not measured directly, but determined from the measured increase of strain in the gages due to the load applied to the pile head. The method of analysis presupposes that one understands and accepts that significant shear forces and corresponding strain in the pile will have developed before the start of the test, that the shear forces along the pile have different directions, and that the magnitude and distribution of these forces follow certain rules. The analysis process establishes the soil response to the loading of the pile and the soil parameters to use when subsequently applying the results of the test to the design of the piled foundation.

The method of analysis used for the two example cases will be presented in a second article scheduled for the next episode of GIN. It applies to loading tests where instrumentation or other methods have been used to determine the resistance distribution in the pile The method is independent of strain-gage shift of no-load reading, and, indeed, for where the gages were installed after all or some of the residual load already had developed in the pile.

Closing Words

When analyzing data from a loading test on an instrumented pile, one must ascertain whether or not all gages have operated correctly and whether or not residual loads were present in the pile before the start of the test. It is easy to jump to conclusions, as the appearance of residual load can be deceiving and might be due to erroneous gage readings (e.g., gage damage and calibration changes caused by mishaps during the construction of the pile). However, unless residual load is accounted for in the analysis of the test data, instrumentation adds very little of value to a pile test. On a positive note, when the residual load is accounted for, the procedure increases the understanding of the pile-soil interaction for the specific project beyond the correct separation of shaft and toe resistances for the tested pile.

Acknowledgment

Considerable thanks are due to the GIN Editor, John Dunnicliff, for his patience with the arduous task of developing the Author's understanding of how to express his thoughts on the topic of the paper.

References

Baker, C. N., Park, G., Braid, J-L, Drumright, E. E., and Mensah, F., 1993. *Drilled shafts for bridge foundations*. Federal Highway Administration, Washington, Report No. FHWA-RD-92-004, 335 p.

Fellenius, B.H., Brusey W. G., and Pepe, F., 2000. *Soil set-up, variable concrete modulus, and residual load for tapered instrumented piles in sand*. ASCE, Specialty Conference on Performance Confirmation of Constructed Geotechnical Facilities, University of Massachusetts, Amherst, April 9 - 12, 2000, GSP94, pp. 98 - 114.

Fellenius, B.H., 2002. *Determining the true distribution of load in piles*. Proceedings of International Deep Foundation Congress, An international Perspective on Theory, Design, Construction, and Performance, ASCE, O'Neill, M.W. and Townsend, F.C., Editors, Orlando, Florida, February 14 - 16, 2002, GSP116, Vol. 2, pp. 1455 - 1470.

Gregersen, O.S., Aas, G., and DiBiagio, E., 1973. I. Proceedings of the 8th ICSMFE, Moscow, August 1973, Vol. 2, pp. 109 – 117.

Hanna, T.H. and Tan R.H.S. 1973. I. Canadian Geotechnical Journal, Vol. 10, No. 2, pp. 311 - 340.

Holloway, D.M., Clough, G.W., and Vesic A.S., 1978. *The effects of residual driving stresses on pile performance under axial load*. Proceedings of the 10th Offshore Technology Conference, Houston, Tex., Vol. 4, pp. 2225-2236.

Hunter A.H and Davisson M.T., 1969. *Measurements of pile load transfer*. Proceedings of Symposium on Performance of Deep Foundations, San Francisco, ASTM, STP 444, pp. 106 - 117.

Nordlund, R.L., 1963. *Bearing capacity of piles in cohesionless soils*. ASCE, Journal of Soil Mechanics and Foundation Engineering, Vol. 89, SM3, pp. 1 - 35.

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An Application for a Single-Sensor In-Place Inclinometer

Brian K. Johnson

Project Description

The St. Paul District, Corps of Engineers, has approximately 40 inclinometer casings at eight project sites. Most are used for long-term performance monitoring of dams, levees, and abutments. Others, such as those detailed below, are used to monitor existing river banks. For those applications, inclinometer data in conjunction with borings, soil strength testing, and stability analyses are used to determine stable locations for levees.

The Red River of the North creates the border between North Dakota and Minnesota, and eventually flows north into Canada. Prior to the disastrous flood of 1997 in Grand Forks, ND and East Grand Forks, MN, a 100-year flood control project was being prepared for Grand Forks.

In early 1997, the City of Grand Forks retained a geotechnical consulting firm to provide an engineering opinion regarding slope stability of the bank of the Red River of the North, located behind the city's water tank and sludge treatment facility. The firm installed three inclinometers, shown on Figure 1. Our focus in this article is on SI-3.

In 1997, a 125-year flood occurred which overtopped the permanent and emergency levees with a discharge of 114,000 cubic feet per second (780 cfs is normal) at a record crest of 54.4 feet (flood stage is 28 feet). Other consequences included evacuation of 46,000 people in Grand Forks (90% of population); almost two billion dollars in damage; 13 days without running water; and 23 days without drinkable water. Fortunately, no lives were lost to the flood.

After the flood, Congress authorized the design and construction of a flood control project for both Grand Forks and East Grand Forks. To improve understanding of the slope stability aspects of the deep lacustrine clay foundation, the St. Paul District installed additional inclinometers and piezometers in other sensitive bank areas. The instrumentation plan included the automation of instruments at the water reclamation facility, including a single-sensor in-place inclinometer at SI-3.

Conventional Inclinometer Readings at SI-3

The amount of downslope movement from an initial reading on 3/21/97 is shown on Figure 2. The shear in the Upper Brenna soil unit is quite evident. Several questions were developed for the instrumentation plan, including: "*How does the rising and falling of the river affect the slide movements?*"

Obtaining river elevation data was easy. The USGS has an automated gage on the East Grand Forks side of the river downstream of the instrumented site. The St. Paul District's Water Control Section has access to the data. Obtaining frequent deflection readings in the Upper Brenna with a conventional inclinometer would be difficult and expensive. The decision was made to install a single-sensor in-place inclinometer within the Upper Brenna. The data collection, reduction, and reporting would be with an automated data acquisition system (ADAS).

In-Place Inclinometer

A Geokon single-sensor vibrating wire in-place inclinometer, Model 6300, was installed in the inclinometer casing at SI-3. The main inclinometer components are upper wheel assembly, vibrating wire sensor, gage tubing (4 feet long), and a lower wheel assembly. The completed in-place inclinometer had a gage length of 5 feet (i.e. the distance between the upper and lower wheels). It was suspended by a 1/8 inch diameter stainless steel cable from a top support piece.

The assembled in-place inclinometer was suspended with the lower wheel assembly at elevation 771 and the upper wheel assembly at elevation 776, as shown on Figure 2, on November 25, 1998. The sensor was read with a portable vibrating wire readout unit during installation, and then connected to the ADAS, manufactured by Geomation, Inc.

The in-place inclinometer was read every 12 hours, and the data were reported daily to the District Office.

Conventional Data

Inclinometer SI-3 was read conventionally prior to the installation of the in-place inclinometer. For subsequent conventional readings, the in-place inclinometer was simply pulled from the casing and stored vertically during the reading. When the conventional reading was completed, the in-place inclinometer was lowered back into position. During the conventional reading, it was not required to gather any data from the in-place inclinometer. The ADAS automatically read it on its programmed schedule.

Data and Verification

Weekly data points from the in-place inclinometer and the USGS automated river elevation gage are shown on Figure 3. Generally during high river stages, the rate of slide movement abated. When the river returned to lower levels, the rate of movement continued in a linear fashion.

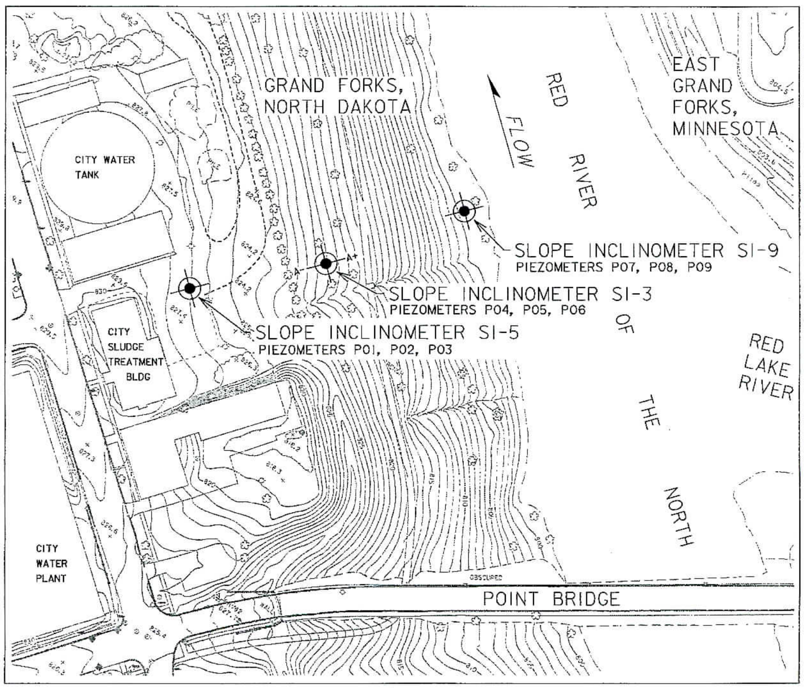

Figure 1. River bank instrumentation in Grand Forks, ND

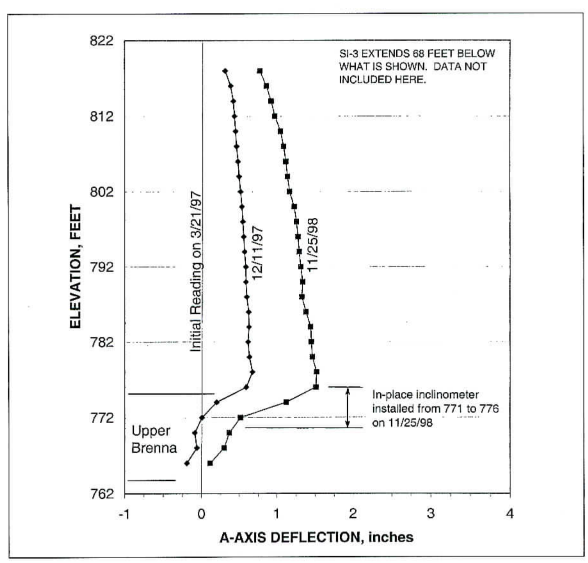

Figure 2. Conventional inclinometer data at SI-3

The conventionally-read data provided an independent means of verification. Since the in-place inclinometer was placed at the location where manual readings were also obtained, the manual readings can be used for comparison with the in-place inclinometer data.

The conventionally-read data were recorded in 2-feet increments, the length of the conventional inclinometer probe. The upper wheel assembly of the in-place inclinometer was at the same location as for the conventional reading (45 feet depth). The lower wheel assembly was at 50 feet depth because of the five feet gage length, i.e. between the 49 to 51 feet depth for the conventional inclinometer reading. In order to compare the two measured deflections, a reading was interpolated for the conventional data for the 50 feet depth. The change in inclination as determined from the conventional readings between November 25, 1998 and March 24, 1999 was 0.1235 degrees. The in-place inclinometer read 0.1302 degrees. Subsequent comparisons also closely matched, as shown on Figure 3.

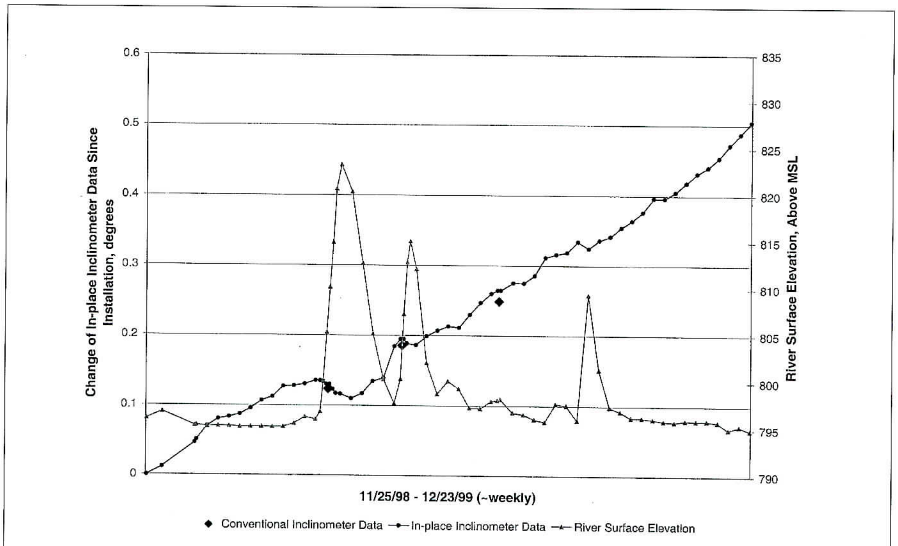

Figure 3. Relationship between river elevation and inclinometer data

Additional Application Further Downstream

In an instrumented riverbank location further downstream, the same methods were used, but the inclinometer casing and in-place inclinometer were installed at the same time. With no prior conventional readings, the location of shear zone was not known. Using the boring log and judgment, the depth of the in-place inclinometer was selected. By taking conventional readings, it became apparent some months later that the in-place inclinometer was too low. It was raised to the location of the shear zone.

Lessons Learned

Using a single-sensor in-place inclinometer in an isolated zone (as defined from the conventional readings) was an economical method to obtain more frequent data. The cost was essentially the cost of the in-place inclinometer equipment, since the ADAS and cabling were already on site for reading piezometers installed in the vicinity of SI-3. Benefits included near real-time reporting of the movements in an unstable soil unit, at a depth of more than 40 feet below ground.

The installation of the in-place inclinometer did not interfere with the ability to obtain conventional readings. During conventional readings, the in-place inclinometer and ADAS remained connected. The 12-hourly readings were timed such that none was taken during the time that the in-place inclinometer was not in place.

It would have been preferable if the gage length of the in-place inclinometer had been a multiple of 2-feet, to avoid the need for interpolation when making comparisons with conventional inclinometer data. For example, if a 3-foot length of gage tubing had been used, the gage length of the in-place inclinometer would have been 4 feet.

The reading on the in-place inclinometer was different prior to and after the disruption for the conventional reading. The reading on the in-place inclinometer on the day prior to the conventional reading was subtracted from the reading on the day after. This offset was used on all subsequent in-place inclinometer readings until the time for the next conventional reading. Since the change in inclination as computed from the conventional readings closely match the in-place inclinometer readings, this method of correction appears valid.

Conventional and in-place inclinometer readings on SI-3 are being maintained, however, data only through 1999 is presented on Figure 3. The main reason is good fortune. The two high water events in the spring of 1999 provided the correlation sought by this phase of the instrumentation. Project design decisions for this area were made and emphasis switched to more critical areas of the project.

Conclusion

A single sensor in-place inclinometer was installed in the failure plane of a landslide on the banks of the Red River of the North at Grand Forks, ND. The automated readings on the in-place inclinometer provided the rate of movement and were correlated to river elevation. The data from the in-place inclinometer compared favorably with the data obtained from conventional inclinometer equipment.

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