Geotechnical Instrumentation News

John Dunnicliff

Introduction

This is the twenty-fourth episode of GIN. Several things to tell you about this time, and five separate articles.

Overcoming Buoyancy During Installation of Inclinometer Casing

The article immediately following this column is one of those nuts-and-boltsy attempts to improve installation practices. As always, I'll welcome any additional suggestions from you.

Profiling the Width of Slurry-Supported Excavations

The article by Peter Deming and David Good describes a caliper for profiling the width of slurry-supported excavations, and is a good example of conveying practical information to the rest of us. The authors refer to other equipment used to make measurements the most common method of which I'm aware is use of the "Koden Drilling Monitor DM-682/684": http://www.koden-electronics.co.jp. This uses ultrasonic wave technology to measure the complete profile of an excavation.

Next Instrumentation Course: "On the Beach" in Florida, with Ralph Peck

The next course is scheduled for March 12-15, 2001. The format will be similar to the 1999 course, with three days of pre-arranged topics, and a fourth day for discussion of topics that will be selected by attendees.

Ralph Peck has agreed to give a lecture, provisionally titled "Observation, Instrumentation, Action - Chicago in the 30s to San Francisco in the 90s". He will also join in a discussion on "People Issues with Observation and Instrumentation".

As in 1999, I am expecting that seven instrumentation manufacturers will give technical lectures and display their instruments. In addition to the 1999 program there will be lectures on time domain reflectometry, global positioning systems, and embankment dam case histories.

Please visit the University of Florida's website, at http://www.doce-conferences.ufl.edu/geotech/geotechn.htm for information as the course is developed, and mark your calendar. It would be a good idea to register early, as space is limited. More details on page 33. Please contact me if you need information that isn't on the website.

Determination of Loading on Internally Braced Excavations

Storer Boone and Adrian Crawford describe a method for determining loading on internally braced excavations that takes into account the effect of temperature change on the bracing and associated vibrating wire strain gages. This is a much more complex issue than might appear at first sight, and is particularly significant when end restraint is high, such as for excavations in dense soils or rock. In the case history described by the authors, the top of the excavation was fully decked, and therefore temperature variations along a strut were minimized. Does anyone have any ideas on how to cope with the temperature problem when struts are exposed to changing sun and shade?

I once wrote in a specification for a certain multi-billion dollar project in a historical East coast US city, that the gage readings should be taken near dawn or dusk, but that was ridiculed with: "you are calling for a Native ceremony, with drums a-beating, fires ablaze, and a sacrificial goat in waiting"!

Lessons Learned

In the last GIN I referred to a seminar in Seattle in April titled *Geotechnical Field Instrumentation, Applications for Engineers and Geologists,* and indicated that I hoped to work with some of the authors to summarize the sharing of "lessons learned". Two are included here, the first by Greg Monley and Andrew Soderborg, the second by John Paxton.

Words of Wisdom

A report on the opening of the Ralph B. Peck Library at NGI is on page 34, and the "words of wisdom" that are referred to in that report are on page 35.

On his way to Norway, Ralph Peck gave a lecture at the Institution of Civil Engineers in London, during which he told us about early use of the observational method during construction of the Chicago Subway. I tell about this only as a lead-in to some more words of wisdom that were spoken on that occasion - "the observations are the heart of the observational method".

Judgment

A High Court action in England, brought by Mrs Eileen Burnett (formerly a director of Monitoring Systems Limited) against Boart Longyear, Chris Rasmussen and Slope Indicator Company has been dismissed on the third day of the trial before Mr Justice Jacob, by consent of the parties. The dismissed claim included allegations of misuse of confidential information and infringement of design rights in analogue electrolevel systems.

Symposium on Time Domain Reflectometry, September 5-7, 2001

An announcement and call for papers has been issued for a symposium "TDR2001 - Innovative Applications of TDR Technology".

Similar to the symposium held in 1994, it will convene at Northwestern University in Evanston, Illinois. Sessions will cover: rock and soil deformation, soil moisture, subsurface chemical transport, leak detection, groundwater level changes, optical fiber applications, structural cracking and deformation, instrumentation and telemetry, and basic physics. If you wish to present a paper, demonstrate equipment, or be a sponsor, please see the website for guidance and information - http://www.iti.northwestern.edu/tdr/tdr2001.html

Resolutely Imprecise Inaccuracy

A flippant heading, but I want to complain about something. On their data sheets for instruments, some manufacturers give a number for resolution, but nothing for accuracy or repeatability. I don't find this useful, and it can be misleading if a user is unsure of the meaning of the terms, and believes that this number refers to accuracy or precision. *Resolution* is the smallest division on the instrument readout scale. *Accuracy* is the closeness of approach of a measurement to the true value of the quantity measured. *Precision* (a synonym for *repeatability*) is the closeness of approach of each of a number of similar measurements to the arithmetic mean. As a user, I need to know either accuracy or precision. I need to know about accuracy when I'm interested in an absolute measurement, such as when measuring pore water pressure or total stress. I need to know about precision when I'm interested in a measurement of change, such as when measuring deformation (I don't care about actual position in this case, only about change of actual position). Do I care about resolution? I don't think so - it seems to me that this is one of the issues that a manufacturer needs to address in system design, in order to arrive at the stated accuracy or precision. If this is correct, please tell us about accuracy or precision, and keep the resolution issue to yourselves. If you have a different view, please share it with me.

On a similar subject, I recently found that an instrument didn't live up to the manufacturers stated accuracy. We discussed it. The manufacturer offered, in subsequent publicity, to "reset user expectations". I thought that was a masterpiece of gobbledegook.

Closure

Please send contributions to this column, or an article for GIN, to me as an e-mail 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.*

Genatse! (Armenia)

Overcoming Buoyancy During Installation of Inclinometer Casing

John Dunnicliff
P. Erik Mikkelsen

Introduction

We have become aware that some installers of inclinometer casing overcome buoyancy during installation by using the drill head or a weight to restrain the top from moving upward. This is not an acceptable procedure.

Statement of the Problem

When grout is used to fill the annular space between the casing and the wall of the borehole there will be a buoyancy force, acting on the bottom cap, to push the casing upward. This will happen even when the casing is filled with water, because the density of grout is significantly greater than water. If restraint is provided at the top of the casing, the buoyancy force will set up a compressive force in the casing, and cause the casing to support itself from side-to-side (snake) within the borehole. This problem is particularly severe with deep installations, where the buoyancy force is largest and where portions of the borehole diameter may be enlarged. The combination of compressive force and eccentric loading may produce excessive bending moments in the casing. This problem is aggravated if snap-together joints are used rather than cemented or riveted joints or couplings. Kinking may occur at snap-together joints, and at a recent 350-foot deep installation with snap-together joints the inclinometer probe could be made to stick in the 3.34-inch casing at 120-foot depth.

Another drawback with 'snaked' casing is the potential for reading errors due to variation in depth control of the probe. Any change or error in the positioning of the probe will produce reading errors - the larger the curvature, the larger the error. For example, if the change of inclination between adjacent reading increments is two degrees, and the probe is positioned one inch from the correct depth, the resulting error in measured displacement would be 0.04 inches.

The density of typical grout used for this purpose is between 75 and 90 pcf, so that the buoyancy force on a water-filled 2.75-inch o.d. casing is between 0.5 and 1.1 pounds per foot of depth. For a 3.34-inch o.d. casing it is between 0.8 and 1.7 pounds per foot. It can be seen from these figures that the buoyancy force can be large.

Acceptable Methods for Overcoming Buoyancy

In our view, the following methods are acceptable (all with water-filled casing):

1. ***Insert steel pipe inside the casing until the grout has set.***

This can be done when an external tremie pipe is used for grouting. Flush to be sure there is no grout inside.

It can also be done in conjunction with the gasket-type, one-way grout valve, in which case the grout pipe is temporarily raised off the gasket, the top of the casing temporarily held down, the casing flushed clean with water (with this method some grout will inevitably be released into the casing when the grout rods are pulled, hence the flush is essential), and the grout pipe lowered to rest on the bottom cap to act as a counterweight.

The quick-connect type grout valve is more time consuming and risky to use, because the grout pipe and associated female part of the quick-connect must be removed from the casing (again temporarily holding the top of the casing down), and replaced with a pipe for weighting purposes, which may trip the check valve at the bottom.

Lowering steel pipe into the casing without reinforcing the bottom cap is risky, as the plastic bottom cap is usually only 1/8-inch thick and can easily break or be punched through by the pipe.

The main disadvantage of this general method is that if the weight of the pipe is larger than can easily be lifted by two people, the drill rig needs to return to the location to pull it out, at least 12 hours after grouting.

2. ***Insert PVC pipe inside the casing until the grout has set.***

This is essentially the same as the first method, when an external tremie pipe is used for grouting, except that an adequate weight must be placed on the top of the PVC pipe until the grout has set. If Schedule 80 pipe with threaded couplings is selected, it can be re-used. If Schedule 40 is used, with cemented couplings (threaded couplings should not be used on Schedule 80 pipe, as this weakens the pipe too much), it can be re-used by sawing off the couplings. The pipe should be as large a diameter as possible, remembering of course that it must fit inside the inclinometer casing, and that threaded Schedule 80 couplings have a large outside diameter.

The drill head can be used for weighting. Alternatively, arrangements can be made to hold down the top of the PVC pipe by tying it with rope to weights, such as drill casing, on the ground at either side. Calculate the buoyancy force before doing this (see the fourth method)! Holes can be drilled near the bottom of the PVC pipe to allow for flushing without lifting the pipe off the bottom cap.

3. ***Install a special anchor at the bottom of the casing.***

Simple prong anchors or packer types have been used. The main disadvantage is that no standard parts are available for this purpose, although some manufacturers will supply them to special order. Some users have made anchors to suit their situations. The design of the anchor must depend, of course, on the properties of the material in which the casing is to be anchored.

4. ***Attach a weight to the bottom of the casing***

If this method is used the density of the grout must be predetermined, so that the necessary weight can be calculated. Remember that the calculation must recognize that the weight is submerged in grout. There needs to be an adequate and special method for attaching the weight to the casing, which should of course be tested ahead of time. If connected to the bottom cap, this needs to be reinforced. Use of this method is limited to relatively shallow installations, because of the magnitude of the weight. The borehole will need to be drilled deep enough to accommodate the weight. A disposable tag line, e.g. 6-mm manila rope, is usually required to lower the assembly.

5. ***Two-stage grouting***

The first stage isolates the bottom cap from the potential buoyancy force. Note that the benefit of this method is hydraulic isolation, not the restraint against uplift caused by bonding between grout and the outside of the casing.

When planning for this method it is important to make sure that the sequence is compatible with the sequence of withdrawing any drill casing. A quick-set grout should not be used for the first stage because the heat of hydration could melt and deform the plastic. No more than five to ten feet need to be grouted in the first stage. If the normal bentonite/cement grout is used it needs to set for at least 12 hours before second-stage grouting.

The two stages can be placed via an outside tremie pipe. Alternatively the first stage can be placed before lowering the casing, provided that all is done efficiently so that there is no chance of the grout setting prematurely.

If either grout valve method is used, first stage grouting can be done through the valve, and then the valve is abandoned. An outside tremie pipe for the second stage is lowered with the casing, with its bottom at the level planned for the top of the first stage.After first stage grouting, the excess is flushed out via the tremie pipe, and then this is raised until the first stage has set.

6. ***Installations with telescoping couplings***

When using telescoping couplings it is important to minimize compression and/or tension in the inclinometer casing string. For installations less than about 100-ft deep, the fourth of the above methods is preferable. For deeper installations, the first or second methods should be used, using an external tremie pipe for grouting.

Suggestions to Manufacturers

We suggest to the manufacturers of inclinometer casing that they may wish to consider some of the above methods for inclusion in their installation manuals. Also that they consider making available to users appropriately strong bottom caps and perhaps a selection of special anchors.

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Caliper for Profiling the Width of Slurry-Supported Excavations

Peter W. Deming
David R. Good

Introduction

A mechanical caliper device was developed to profile the width of a slurry trench excavation. Width measurements were needed to determine if the side walls of the trench, in a cohesionless crushed stone fill deposit were stable below the slurry line. The device constructed by the authors performed well, determining trench width to depths of 20 m (65 ft). This article describes the design, calibration, and field use of the caliper for measuring the width of slurry-supported trenches or large diameter drilled holes.

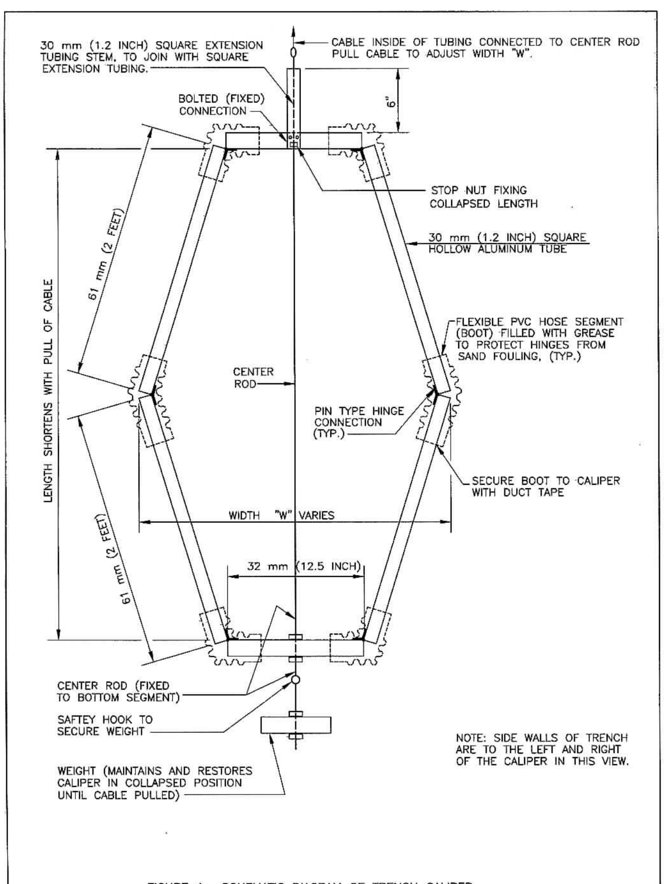

Figure 1. Schematic profile of trench width caliper

Design and Construction of Caliper

A schematic diagram of the trench caliper is provided in Figure 1. The caliper is a hexagon-shaped frame made of square aluminum tubing segments hinged at the vertices. A weight suspended from the bottom maintains the device in its collapsed (calibrated) position and provides stability to the caliper when measurements are taken. 1.5 meter (five foot) lengths of extension tubing are added sequentially to the extension tubing stem at the top of the caliper as the caliper is lowered into the slurry-supported excavation. The extension tubing is the same hollow square aluminum tubing as used for the caliper. The lightweight aluminum tubing construction facilitates field handling and operation.

By holding the extension tubing in place, the top of the caliper remains at a fixed depth. A thin plastic coated "aircraft" pull-cable is connected to a center rod, which is fixed to the bottom segment of the caliper. Pulling the cable, while holding the extension tubing in place, forces the bottom segment of the caliper upward, and widens the hinged polygon shape until contact is made with the side walls of the excavation. The amount of cable extension required to contact the side walls is related to the width of the caliper, as shown in Figure 2.

Our field staff designed and assembled the caliper. The extension segments were fabricated from 30 mm (1.2 inch) square hollow aluminum tubing. The extension segments have a drawn-in end to fit inside the adjoining segment. The square shape of the tubing allowed the operator to orient the caliper perpendicular to the trench wall. The tubing was supplied cut to size by an awning manufacturer. The pull cable was threaded through the center of the extension tubing as the instrument was lowered. Adjacent lengths of extension tubing were pinned together with a coarse threaded lag screw. Lag screws, hand-turned into position, allowed a convenient connection of the extension tubing when wearing gloves and with sand and slurry coating the equipment.

Simple pin hinges were bolted to the square aluminum tubing frame at each of the hexagonal caliper vertices. We found that sand in the slurry fouled the hinges, preventing free collapse of the hexagon. To prevent fouling, we covered each hinge with a "boot" of flexible PVC construction hose and secured the hose segments to the caliper frame with duct tape. The boot was packed with grease to keep the hinge lubricated and to exclude slurry.

Calibration of Caliper

The caliper was calibrated by measuring its width at various cable extensions. This was done with the caliper hanging with the weight in place, just as the caliper was operated in the slurry trench. The theoretical calibration curve for the hexagonal shaped caliper is a parabola. For an accurate width measurement, the collapsed caliper width must start at the same collapsed width as the calibration. We set the collapsed width to have a slight bend in the side hinges. A stop nut on the center rod above the top segment of the caliper fixed the maximum amount of caliper collapse, providing a consistent initial geometry. The measured field calibration, performed at one inch cable-pull intervals, is shown on Figure 2.

The dimensions of the caliper device shown in Figure 1 were developed to work in a one meter (3 ft) wide trench. This caliper could measure widths up to 1.4 m (4.8 ft). The accuracy of the width measurement was suitable for quality assurance of slurry trench construction. By changing the lengths of the caliper segments, a similar device could be constructed to measure wider or narrower excavations. Because the width of the caliper changes rapidly in the first few inches of cable pull, the caliper should be constructed to be somewhat narrower than the excavation, so that measured widths are in the range where the instrument readings are more accurate.

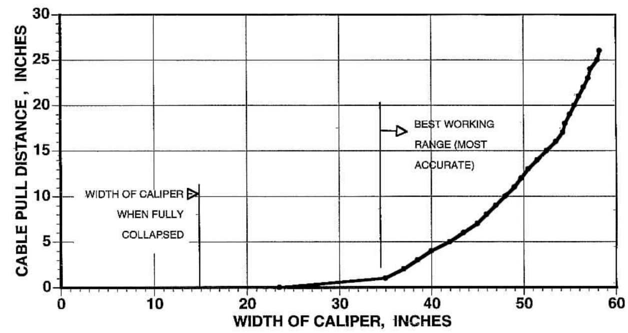

FIGURE 2 - TRENCH CALIPER CALIBRATION

Figure 2. Measured calibration of trench width caliper

Use of Caliper

The contractor provided a lightweight portable bridge that spanned over the trench excavation to provide safe access to the center of the trench for caliper use. The bridge deck was often used as the depth reference, and a slot in the bridge deck was used to support the weight of the caliper when the readings were taken. A two-person crew operated the caliper. Having a two-person crew added safety at the edge of the slurry trench, and was needed to move the bridge.

Our procedure for measuring a trench width profile was:

- The width of the trench at the ground surface of the profiling station was measured directly.
- The end of the pull-cable was secured to the top of the center rod of the caliper with a safety hook. A section of extension tubing with the pull cable threaded through its hollow core was secured with a lag screw to the caliper. A second section of extension tubing was similarly secured to the first section. A weight was then attached with a safety hook to the bottom of the caliper. The caliper was lowered until the lag screw between the first and second sections of extension tubing rested on the inspection bridge and the caliper was oriented perpendicular to the side walls of the trench. The weight helped to lower the caliper through the slurry, and maintained the caliper in its calibrated collapsed position. After cable slack was removed, the pull-cable was then grasped where it exited the extension tubing and pulled upward until resistance indicated that the sides of the caliper were snug against the side walls of the excavation. After removing the slack in the cable, the cable extension was measured and recorded.
- Additional sections of extension tubing were added and the measurement procedure was repeated. Readings were performed at regular depth intervals in order to obtain a profile of the excavation width. The measured cable extension was converted to trench width using the calibration shown in Figure 2.
- Disassembly was performed in reverse order as the caliper was withdrawn. When no other profiling was to be performed in a given day, the caliper, rods and cable were washed clean of slurry.

If the trench profile was symmetrical, the extension tubing remained vertical in the center of the trench when the cable was pulled. When the rods tilted away from one side and the measured width was large, this indicated that the side wall from which the rods tilted away had caved. This information was recorded along with the cable extension. However, as the depth of measurement increased, tilt became less visible, and translation of the caliper was taken in bending of the extension tubing. Few sidewall irregularities were noted.

On the project for which the caliper was developed we typically observed the stable trench to be from 1.4 to 1.5 m (4.5 to 5 feet) wide at the ground surface. The side walls of the trench sloped uniformly from the top to a constant width of 0.9 m (3 feet) below 7.5 m (25 feet) of trench depth. The narrowest width of the trench measured was approximately equal to the width of the widest dimension of the bucket. Widening of the top portion was attributed to the multiple entry and exit passes of the bucket.

Subsurface currents in the slurry caused by the backhoe motion within the trench were found to be far greater than expected from observation of the slurry surface. Dynamic movement of the slurry at depth made reliable operation of the caliper impossible even at a distance greater than 100 feet from the operating backhoe. Trench width measurements were taken when the backhoe was not operating.

Conclusions

The mechanical caliper described above was reliable and easy to operate, and gave an accurate measurement of trench width below the slurry surface. By changing the dimensions of the hexagonal sides, a similar device could be constructed to measure the width of any large diameter drilled hole or excavation. A trench width profile was useful to confirm trench wall stability, provided a documentation of the barrier width for quality assurance, and enabled investigation of areas where collapse of the side wall below the slurry surface was suspected. We found it possible to repeat excavation width readings at one location over time. This provides a means to determine excavation stability and raveling potential.

The authors are aware of other equipment used to measure the width of slurry supported excavations. The distinguishing features of the caliper described are inexpensive fabrication, ease of use, and portability along the rough mud covered terrain which typically accompanies slurry trenches. Dunnicliff discusses the use of in inclinometers installed adjacent to the excavation. This method has its use, but the caliper provides a direct measurement of trench width and is able to measure width at any location.

Reference

Dunnicliff, J. “Geotechnical instrumentation for monitoring field performance” John Wiley and Sons, New York, 1988, 1993.

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The Effects of Temperature and Use of Vibrating Wire Strain Gauges for Braced Excavations

Storer J. Boone
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Introduction

Strut loads in braced excavations have routinely been monitored with strain gauges using devices ranging from simple mechanical measurements to vibrating wire transducers. Because of varying end restraint arising from the stiffness of the ground, thermal load changes are not readily accounted for with simple temperature "corrections." In soft soils the effect of temperature changes on strut loads may not be great. For stiff and dense soils or rock, however, thermal loading may be significant. Without consideration of thermal effects, measured loads may be misinterpreted solely as earth loads, or design loads that are based only on computer analyses may not be representative of potential field loading.

Background

Until vibrating wire strain gauges were widely available, stresses in excavation struts were often indirectly measured using mechanical gauges that measured the true strain experienced by the strut. Mathematical correction for temperature was made by using the measured values of total strain, ε, and the temperature change in comparison to measurements made on a free piece of strut steel. Using the coefficient of thermal expansion of steel (α_S), applied external stress ($\Delta\sigma s$), steel elastic modulus (E_S), and the cross-sectional steel area (A_S), the stress-strain equations for thermal and static loading of steel were used to define the external load (P) in terms of total strain and temperature strain:

$$P = \varepsilon E_S A_S - \alpha_{S(\Delta T)} E_S A_S \quad (1)$$

Eq. (1), however, is only representative of earth loads if the strut is perfectly fixed. As described by Dunnicliff (1988, 1993), when using thermally matched (i.e. $\alpha_G = \alpha_S$) and calibrated vibrating wire strain gauges, a strut with perfectly fixed ends exposed to a temperature increases ($\Delta T = +$) will not experience any change in length, L. However, the temperature change will cause expansion and therefore relaxation of the vibrating wire, thus indicating compression loading. The compression (strain) indicated by the gauge will, when combined with the appropriate steel end area and elastic modulus, correctly indicate the increase in thermally-induced compressive stress. The opposite conditions hold true for temperature decreases ($\Delta T = -$), only strut expansion would be indicated, resulting in a correctly calculated increase in tensile stress. For a perfectly free piece of steel, a temperature increase will result in expansion of both the strut and the gauge wire; i.e. the expansion of the strain gauge wire that produces a relaxation of the wire tension will compensate for the actual lengthening of the strut. Output from the vibrating wire strain gauge will indicate that there was no change in length (although strictly incorrect) and that the strut will have experienced no change in stress. The opposite conditions would apply to a decrease in temperature.

In spite of the mathematical simplicity of accounting for temperature outlined above, struts in braced excavations are neither perfectly free nor perfectly fixed. Separation of these indeterminate loading conditions has been problematic. To resolve this issue, Endo and Kawasaki (1963) proposed an expression relating the two through the use of a generalized "spring constant" term. Chapman et al. (1972) combined the equations for the deformation of an elastic-half-space under a uniform rectangular load (the full excavation wall) and those for thermal expansion of steel to deduce the temperature-dependent load from a given temperature change. Their approach was based on empirical measurements of loads and wall movements and a back-calculated deformation modulus, E_d. While this was a useful approximation, comparison of E_d values from back-calculation and field measurements did not compare well (1/2 to 1 orders of magnitude difference). Though not widely adopted in texts or published literature, this expression has been one of the few available tools to separate and evaluate the effects of temperature and earth loads.

A Case History

Shoring for a recent braced excavation project consisted of soldier-piles placed in pre-bored holes with wood lagging between the piles. Soldier-piles were installed with a 3m center-to-center spacing and the vertical strut spacing generally ranged between 2.4 and 5.8 m. Horizontal support was provided by deck-beams, and two to three pipe-struts directly connected to each soldier-pile (i.e. there were no wales). The pipe struts were typically 10 m to 20 m long and varied in diameter from about 600 mm to 900 mm. Since the excavation was made beneath a street, it was fully decked during the majority of construction.

Shoring was instrumented using thermally matched vibrating wire strain gauges. The gauges used were Irad SM-5A units that were welded to the structural steel using end blocks that clamped the gauge assembly in place once welding was completed and the steel cooled (see Figure 1). It was considered that the spot-welded vibrating wire strain gauges would be too fragile for this application. The Irad SM-5A gauges exhibited a differential thermal coefficient of expansion about 0.2 $\mu\varepsilon/C^{\circ}$ as compared to thermal coefficient of structural steel such that they were, for practical purposes, "thermally matched." It should be noted that published differential thermal expansion coefficients for "thermally matched" vibrating wire strain gauges can vary (though most range between 0.2 and 0.7 $\mu\varepsilon/C^{\circ}$) and such differences should be considered depending on the intended gauge use.

Three strain gauges were fixed to each monitored pipe strut and two to four were fixed to the neutral axis of deck beams prior to installation so that the influence of bending stresses could be minimized and accounted for (see Dunnicliff 1988, 1993). All struts and deck beams were monitored at a total of eight vertical sections along the excavation. Strain gauge readings were generally taken daily throughout the project, typically resulting in 100 to 300 temperature-correlated load readings per instrumented strut as some gauges were in place for over a year. Readings were taken more frequently during the preloading stage of strut installation, and somewhat less frequently when excavation was not occurring in the area. Full instrumentation was maintained for 19 of the 21 monitored struts and 7 of the 8 deck beams while one or several of the gauges on the other struts and beams were destroyed during construction.

During the time the struts were in place, they experienced daily and seasonal cycles of warming and cooling. Absolute gauge temperatures ranged from near 30° C to -15° C. Since the excavation was decked, it was relatively free of thermal effects related to sunlight and shadows. Thermal gradients, from top to bottom of the excavation, rarely exceeded 1° C. During monitoring a consistent pattern was observed where increments of temperature change induced incremental changes in strut loads.

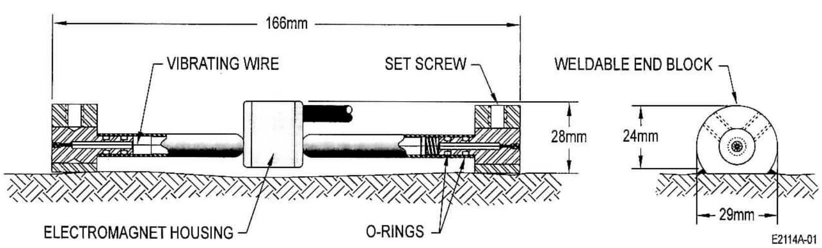

Figure 1. Schematic diagram of vibrating wire strain gauge (provided courtesy of Roctest Ltd.)

Figure 2 presents measured strut loads and changes in relative temperatures (compared to initial temperature) for one upper-level strut (deck beam above and two struts below) as examples of the strut load data. It is clear from Fig. 2 that temperature and load fluctuations are synchronous and that seasonal decreases in temperature does not result in a prolonged decrease in strut loads.

Theoretical and Practical Implications of Observations

When the temperature change relative to the original gauge/strut temperature, ΔT, or the incremental difference between consecutive gauge temperatures, ΔT_i, are positive measured loads will exceed the loads from true earth pressure (see Fig. 3 for terms). When ΔT or ΔT_i are negative, the struts are essentially free to contract, allowing rebound to occur. Provided that the retained earth is continuously applying force to the wall, and that temperature contraction of the strut does not exceed the inward deflection of the wall due to earth loads, then the relationship between ΔT_i and ΔP_i should remain reasonably elastic for small movements. When the incremental load and temperature changes are plotted against one another (ΔP_i v. ΔT_i in lower plots of Fig. 2) this trend is clear. This sequential increase and decrease of temperature and load are directly proportional indicating that both the support system and the soil are behaving linearly and elastically. Those points that do not lie within the cluster of linear data points (bottom plot of Fig. 2) represent both load changes due to external earth loading and errors in measurement (note that the line must pass through the origin by definition). Data from another array (vertical section) of strain gauges is illustrated by Fig. 4 where the loads in the top, middle, and lowest strut are shown, relative to both time and the incremental load-temperature relationship. From a plot of ΔP and ΔT_i the linear portion of the plot, *m*, can be readily determined (as in Fig. 2). The value of m accounts for both the mobilized elastic modulus of the soil, $E_{s(m)}$, and the elastic properties, end area, and length of the strut. It should also be noted, however, that in order to adequately asses the load-temperature relationship, the number of data points becomes important as illustrated in Fig. 4 where the bottom strut data illustrates less convincing results than those of the upper two struts. The amount of data is especially important when other construction activities are occurring within the excavation, such as removing earth in the strut vicinity or adding or removing struts below as these activities can obscure any trends in the data. Note also that, because of seasonal temperature changes and continuous earth loading, comparison of absolute temperature and load readings for extended periods of

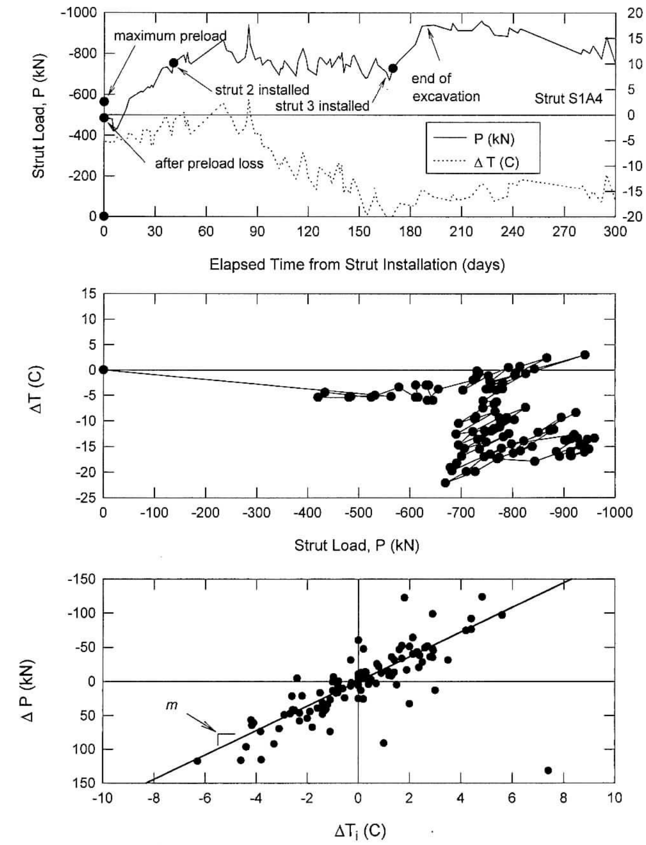

Figure 2. Top: strut load and temperature changes for one example strut. Middle: data from top graph plotted to illustrate linear and cyclical strut loads relative to prolonged seasonal temperature decrease. Bottom: changes in load plotted relative to incremental temperature changes.

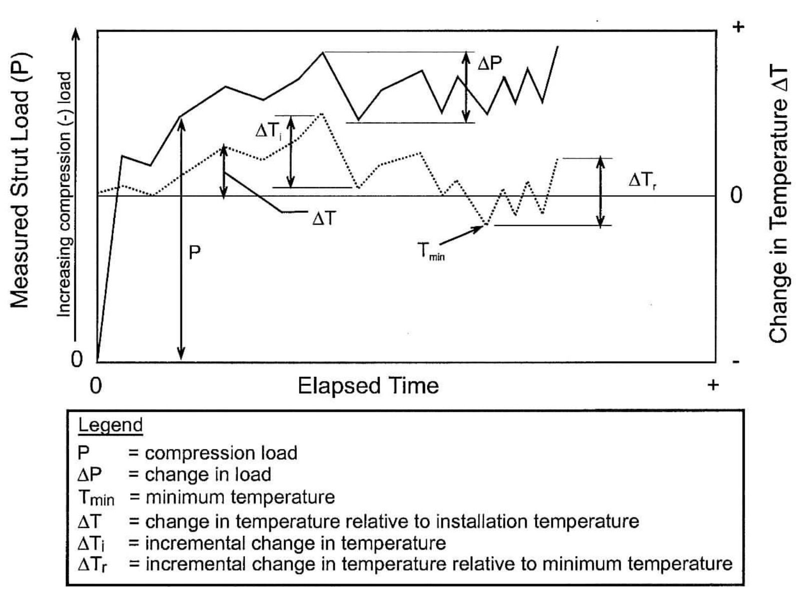

Figure 3. Notation.

time could be misleading if the incremental differences are not compared as above.

To estimate the earth load component of the total load using strain gauge data, it is necessary to "reset" the reference temperature value to which subsequent temperatures are compared. The "reset" reference temperature, ΔT_r, is thus the chronological minimum strut temperature recorded for a given time; i.e. $\Delta T_r = T - T_{min}$. The true temperature dependence of strut load can be graphically derived by the slope, m, of $\Delta P / \Delta T_i$, as shown in Figs. 2 and 4. The earth loads at any particular time can then estimated by:

$$P_E = P - m\,\Delta T_r \qquad (2)$$

The strut force due to earth loads for the example strut shown in Fig. 2, as estimated using the above approach, are illustrated in Fig. 5.

The mobilized resistance of the soil behind the braced retaining system and the load ratio, LR, defined as the temperature induced load for a partially restrained strut divided by the temperature induced load for a perfectly fixed strut ($\alpha_S \Delta T E_S A_S$), were derived for the case project using several simplifying assumptions. For the soldier pile walls with struts at each pile (no wale), the temperature-induced load was assumed to be distributed over an area equal to the horizontal pile width by a length equidistant from the subject strut to the struts above and below (s as used in Table 1), as used for apparent earth pressure diagrams. The deformation of the wall induced by the strut reaction to temperature changes was derived using a simple expression for elastic settlement and an influence value, I, related to the shape of the loaded area (e.g. D'Appolonia et al. 1968, 1970). It was also assumed that the total changes in strut length could be equally distributed to each side of the symmetrical excavation support system. Using these assumptions, the equations provided in Table 1 were derived for further analysis of the data. Comparison of the proposed method and prior approaches as well as the mathematical background of the

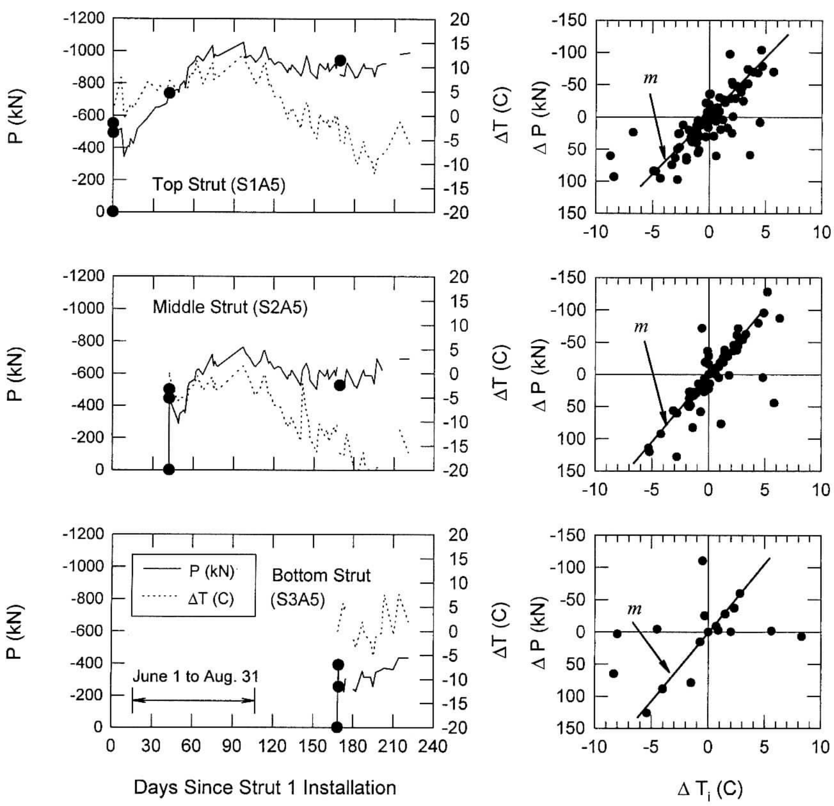

Figure 4. Example load and temperature change data for a vertical section of the excavation support system.

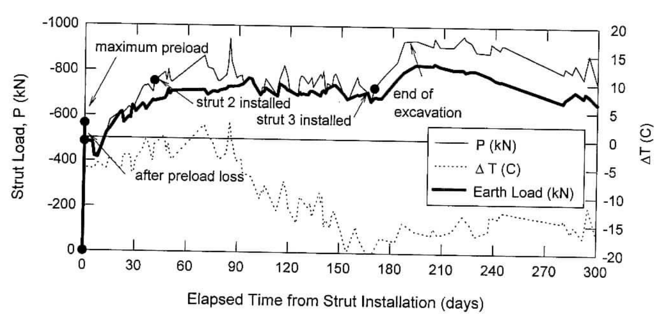

Figure 5. Separated total loads and earth loads.

Table 1. Equations for estimating termperature-induced load, load ratio, and mobilized elastic modulus for a soldier-pile and lagging braced excavation

Temperature-Induced Load, P_T =

$$\frac{\alpha_s \Delta TL}{(2I)/(sE_{s(m)}) + L/(A_s E_s)}$$

Load Ratio, LR, =

$$\frac{L}{(2A_s E_s)/(sE_{s(m)}) + L}$$

Mobilized Elastic Modulus, $Es_{(m)}$, =

$$\frac{-2IP_T}{s[\alpha_s \Delta TL + P_T L/(AsEs)]}$$

equations shown in Table 1 is provided by Boone and Crawford (in press). In general, the proposed approach to estimating thermal effects on strut loads was demonstrated to be consistent with measurements made on struts supporting both relatively weak and competent ground.

For design of future braced excavations it would be necessary to estimate the appropriate range of soil elastic modulus and the potential length, spacing, and size of the struts in order to estimate thermal loading conditions. Figure 6 illustrates the broad range of LR that could occur for a range of strut sizes and ground conditions. The bottom curve of this figure represents strut sizes and spacing that might be appropriate for support of a narrow excavation in soft clay, while the upper curve might be more appropriate for a wide excavation in hard clay and dense sand. Although these curves represent a range of conditions, it should also be expected that as the elastic modulus increases, the responses illustrated by the bottom curve would trend toward those of the top curve since the ground conditions would likely permit wider strut spacing for equivalent strut and excavation sizes. Nonetheless, Figure 6 illustrates the important contribution that thermal loading can have on maximum strut loads depending on the characteristics of the supported ground.

Lessons Learned

From the instrumentation used on this project, greater insight was gained into the coupled thermal behavior of the strut, wall, and soil systems. Specifically, careful observation and analysis of the strain gauge readings provided the following lessons:

- if strain-compatible measurements of the in-situ elastic deformation modulus are obtained (e.g. pressuremeter or plate load tests with intermediate unload-reload cycles), the thermal loads within braced excavation struts can be estimated;
- with frequent measurements of load and temperatures, the mobilized elastic deformation modulus of the ground can be estimated using the strut and strain gauge system as a

thermal loading and measurement tool;

- daily readings of loads and temperatures should be taken (as a minimum)in order to obtain useful temperature load-deformation data; and
- separating thermal loading conditions from total load measurements could lend additional insight into load-redistribution patterns for braced excavations in stiff and dense ground.

The data from this project were collected for a braced excavation that was decked and represents an ideal case for the theoretical examination of indeterminate load conditions induced by temperature changes and the imperfect end-restraint of struts. Thermal gradients within steel struts are more extreme and localized when the steel is exposed to sunlight. Under such conditions, the localized variation of temperature within the steel and gauges must be given careful consideration when evaluating temperature-induced loads for a particular case.

Although space limits a full discussion of other problems and successes on this project, two other valuable lessons learned regarding strain gauge use included:

- the strain gauges used for this project proved to be reliable, sensitive, and sturdy – on other projects the small spot-welded strain gauges were destroyed more easily; and
- the experience of the technician/engineer directing the installation of the strain gauges is critical to avoid inadvertent damage during welding or loss of zero-load data.

Conclusions

By comparing the incremental changes of strut load and temperature from vibrating wire strain gauge data the temperature-dependent loads, relative fixity of the strut end, and the earth loads, and mobilized ground stiffness can be deduced. The proposed approach provides a transparent and strut-specific means of evaluating the effects of temperature on struts within braced excavations and is supported by both empirical data and practical application of elastic theory. The ability to separate components of strut load could aid in a better understanding of all the mechanisms to be considered for future designs and interpretation of observed strut load changes during construction.

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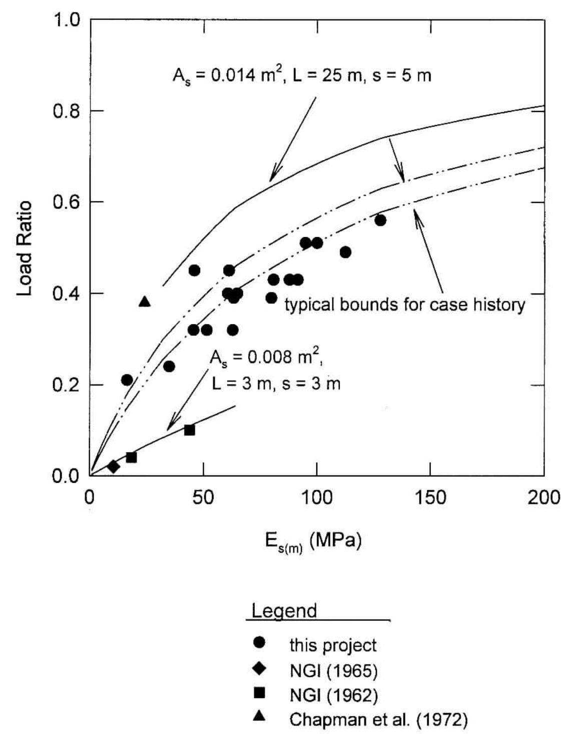

Figure 6. Load ratio for various ground and excavation support system parameters.

I-15 Instrumentation Monitoring Program - Lessons Learned

Greg J. Monley
Andrew H. Soderborg

The I-15 Corridor Reconstruction is a $1.6 billion design/build project that involves demolishing and rebuilding 27 kilometers of interstate highway through Salt Lake City, Utah in a four-year period and prior to the 2002 Winter Olympics. Embankments up to 15 m high and mechanically stabilized earth (MSE) walls up to 9 m high are currently being constructed over approximately 22 m of soft foundation soils. Over 675 settlement devices (riser-type settlement plates, hydraulic leveling devices, and survey points), 20 magnetic probe extensometers, 117 telescoping inclinometer casings, and 140 piezometers (push-in-type with vibrating-wire transducers, and Casagrande piezometers) have been installed. These instruments have been used to measure the actual performance of the soft foundation soils, assess embankment and wall stability during staged-fill construction, and monitor the substantial completion of primary consolidation under surcharge loading.
Lessons learned from the project to date are presented below.

Lesson 1
Consider all Factors when Choosing Between a Magnetic Probe Extensometer and a Sondex Device

Several difficulties were experienced with the magnetic probe extensometers, which were installed early in the project to measure settlement occurring within individual foundation layers in comparison with total foundation settlement measured using the surface settlement devices.

During and shortly after initial embankment filling, obstructions formed in the 25 mm Schedule 40 access pipe of 14 of the 18 magnetic extensometers, and this kept the reed switch probe from reaching the bottom of the access pipe. In general, obstructions appeared to form initially near the bottom of the access pipe, and gradually worked their way up the pipe over time, preventing a complete profile of foundation settlement from being obtained. Several possible causes were investigated, but none could be fully substantiated. The most likely cause was that buckling was occurring near the locations of the thin-walled access pipe and the much stiffer telescoping couplings before the couplings could compress in response to foundation settlement.

The instrument supplier, Geokon, has recently indicated that similar problems had occurred on another more recent project where Schedule 40 access pipe had been used, and that better performance was experienced when a stiffer Schedule 80 access pipe was used. They now recommend using Schedule 80 pipe for all magnetic probe extensometers.

Another difficulty was discovered when the option of switching to a Schedule 80 access pipe was considered. The relatively tight-fitting reed switch probe would not fit inside the smaller inside diameter of a 25-mm diameter Schedule 80 access pipe, and the already purchased magnets would not fit around the outside diameter of the next larger sized, 38-mm pipe. The cost of purchasing several new magnets was considerable, and the project delays that would result from waiting for the new magnets to be manufactured were unacceptable. Also, there were no assurances that stiffer pipe would solve the problem. Instead, a smaller 16-mm diameter reed switch probe was used to try to slip by the obstructions, but with only limited success.

The Sondex device was initially considered instead of the magnetic probe extensometer, based on the positive experience of one team member on a previous soft foundation project in which reliable measurements were obtained for vertical strains ranging to about 20 percent. The inner access pipe of this instrument provides considerably more wiggle room for the probe than the magnetic probe extensometer pipe.

Also, compression can occur along the entire length of the outer corrugated tubing of the Sondex in response to vertical deformations, whereas compression along the access pipe of the magnetic extensometer has to be transferred to telescoping couplings which may be spaced several meters apart.

It is also possible to pull out and replace the inner access pipe of the Sondex if it is hit or damaged, whereas this is not possible with the magnetic extensometer pipe. The magnetic probe extensometer was chosen based on the perceived advantage that the spider magnets would be attached directly to the foundation soil and would therefore be more responsive to foundation settlement. Correct data from the Sondex instrument assumes that the outer corrugated tubing and the grout between this and the wall of the borehole compress in a way that matches the compression of the soil. In hindsight, we believe that this advantage was not worth the mechanical difficulties experienced with this instrument, and that the Sondex device would likely have performed better for this project.

Lesson 2
Anticipate Difficulties in Monitoring Inclinometer Casings Fitted with Telescoping Couplings

Difficulties were experienced in interpreting several inclinometer readings as a result of check-sum errors caused by the telescoping couplings. The inside diameter of the inclinometer casing widens at the couplings, causing one of the two inclinometer wheels to displace outward with respect to the other wheel. As a result, significant check-sum differences were obtained from readings taken at the zero (0) position compared to the 180 degree position, often resulting in unexplainable displacement readings at and near the telescoping couplings.

Because the coupling locations continually changed in response to large foundation settlements, it was not possible to avoid taking the 0.6-m (2-ft) spaced readings at one or several couplings throughout the performance life of the instrument. We also verified that the probe wheels would often slip out of the casing grooves at the couplings, causing further reading errors.

Difficulties in interpreting the resulting data could be lessened by taking more frequent readings and interpreting around the telescoping couplings. Also, special efforts were made to avoid placing the couplings in the middle of the soft foundation layers to avoid introducing the potential for errors at the locations where the largest lateral displacements were expected.

Although these difficulties have been surmountable, the cost in additional time and energy required to collect and interpret the readings has been significant.

Lesson 3
On Large Earthworks Projects, Anticipate a Large Percentage of Damaged Instruments, and Provide Redundant Instruments for Monitoring Critical Structures

Approximately 15 percent of the instruments installed on the I-15 Project have been damaged or completely destroyed by earth moving equipment to date. In one instance, an inclinometer casing was flattened only 15 minutes after installation, barely allowing the field engineer and drill crew to run for safety. Fortunately, a 15 percent contingency for instrument procurement and installation was included in the initial cost estimate in anticipation of these losses. To limit gaps in the monitoring data caused by instrument damage, the project specifications required that the contractor stop filling within 100 m of a damaged instrument until the instrument could be repaired or replaced. Unfortunately, given that this was a design/build project in which the contractor is boss, this specification was difficult to enforce.

For several critical embankments and walls where an uninterrupted record of settlement and/or stability performance was critical, redundant instruments were included so that readings in one instrument could continue if the other was damaged. Significant measures were also taken to protect the instruments.

The most effective protection proved to be large 4- to 5-foot square concrete blocks (referred to as "mongo blocks") that were made available from the many bridges that were demolished on the project. The blocks were placed near several critical instruments and earned the respect of equipment operators who recognized the serious consequences of impacting the blocks.

Lesson 4
Minimize Survey-Related Errors by Using Crews Dedicated to Instrumentation, and by including a Small Percentage of Settlement Devices that Don't Rely on Surveys

Several survey busts occurred in reading the riser-type settlement plates and survey points in the first year of the project, primarily because some "benchmarks" intended for use as fixed reference elevations with respect to the settlement devices were actually on nearby fills or structures which were also settling. The problems became apparent after noticing that some readings indicated that the foundations underlying some embankments were actually rebounding instead of settling.

A primary goal of the instrumentation program was to extrapolate the end of primary consolidation settlement of the embankments and walls under surcharge loading so that the contractor could effectively schedule the surcharge removal. A continuous record of reliable settlement data was required to do this. Uncertainties in interpreting the settlement data caused by the survey busts often resulted in significant construction delays in removing surcharges.

Due to a shortage of qualified surveyors in the Salt Lake market (caused partly by the huge labor demands of the

I-15 Project), the survey crews appeared to be severely stretched in their dual responsibility to complete surveys for both instrumentation monitoring and the structures under construction. As a result, they sometimes used "temporary" benchmarks located within the zone of settlement influence to speed up the surveys.

Following the first year, the contractor began to use survey crews dedicated only to instrumentation, and the quality of settlement surveys has significantly improved as a result. The dedicated crew is more aware of the accuracy required to interpret the settlement data. Delays in removing surcharges have also been reduced, which we believe has more than compensated for the cost of a dedicated survey crew.

Placement of a small number of self-contained settlement gages was considered for the project based on the successful performance experienced on previous projects. The gages are designed to measure settlement above a "fixed datum" below which no settlement is assumed. They generally include a smaller (i.e., 25-mm) diameter inner steel pipe founded in the datum layer using a Borros anchor, that is installed inside of a larger (i.e., 50-mm) steel pipe that is welded to a settlement plate placed at the bottom of the fill.

Foundation settlement occurring above the fixed datum is obtained by measuring down from the top of the inner pipe to the top of the outer pipe with a hand ruler. Foundation settlement, if any, occurring below the fixed datum can be surveyed periodically if required. The self-contained settlement gages were not used on the I-15 project primarily because of the cost associated with installing the inner pipe in the datum layer located about 22 meters below existing grade.

In hindsight, data collected from a small number of gages could have provided us with a continuous settlement record for critical structures, and would have provided a means for detecting survey busts much earlier, resulting in less project delays.

Lesson 5
Hydraulic Leveling Devices can be Less Predictable than the Stock Market

Hydraulic leveling devices, or overflow gages, were used to monitor foundation settlement beneath embankments for one of the three project segments at the contractor's request. The contractor in this segment preferred the overflow gage over the riser-type settlement plates, which were viewed as obstacles to earth moving equipment. Unfortunately, the overflow gages generally performed poorly, fluctuating up and down several millimeters between readings. Whereas some showed an encouraging downward trend, other fluctuated unpredictably. Despite considerable efforts, the causes of this poor performance could not be determined.

Attempts were also made to correlate the reading fluctuations to changes in barometric pressure, but no correlation was apparent. Some members of the project team believe that poor instrument design may have been the cause. The liquid and vent lines of the manometers consisted of very pliable 16 mm diameter vinyl tubing, which were placed inside a black corrugated pipe to protect the tubing from pinching. Despite the protective pipe, some of the tubing may have become pinched, stretched or twisted such that liquid or vent lines were restricted.

After struggling to understand and correct these problems with the overflow gages to no avail, the contractor agreed to switch to the riser-type settlement plates.

An important lesson learned was to know when to stop studying a problem instrument. If the instrument doesn't work, and reasonable efforts have been made to fix the problem with no success, it's time to drop the instrument, and quickly.

A possible alternative to the overflow gage that would also keep the instrument out of the contractor's way, as well as reduce the potential for significant errors caused by barometric pressure changes, would be to use a vibrating wire settlement cell. A description of this device and its performance are given in an article by John McRae of Geokon on page 40 of the March 2000 issue of Geotechnical News.

Given the large number of settlement devices required, and the much larger cost and more complicated installation requirements for this device compared to the riser-type settlement plates, the vibrating wire settlement devise was not considered for the I-15 project.

Lesson 6
Cooperate and Communicate

A close working relationship with the contractor has been essential to the success the I-15 instrumentation program.

The contractor has been continually informed of the instrument readings, in weekly reports and weekly meetings. The contractor's surveyors typically review the weekly instrumentation survey schedule to ensure that the schedule provides as efficient a route as practicable, so that the large volume of settlement readings can be completed each day.

Continually educating and convincing key people of the importance of the program to the overall success of the project has also been beneficial. For example, once key contractor personnel recognized that having a reliable settlement record would expedite the release of surcharge embankments, more pressure was placed on the field personnel to protect the settlement devices from damage.

A top-down education of all the team members of the importance of the program also proved to be more effective than any other method of protecting the instruments.

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Some Lessons Learned from Designing and Installing Instrumentation for Dams

John F. Paxton

The following lessons were learned by John Paxton while designing and installing instrumentation for zoned earth and rockfill dams. He will be describing one of the projects in a future article. One of my goals for GIN is to share practical and constructive information, so we decided to publish the lessons learned without the project description. They apply to your and my projects too.

John Dunnicliff

Details

Any details that are left to the contractor's discretion during construction are at risk of either being done at the contractor's direction or not at all. Shortcutting the details in the plans risks an unsatisfactory implementation of the intended function. This is more critical for instrumentation and automatic data acquisition systems (ADASs) than for many other typical civil drawing details.

Shop Drawings

Full shop drawing sets should be required prior to installation of any instrumentation sub-system. This includes even the most minor or simple detail. Never assume that details will be taken care of by a contractor unless he shows you what he will do beforehand and agrees to do it.

Stainless, Stainless, Stainless

Always specify full stainless hardware on all metal parts and assemblies. Avoid contact between dissimilar metals anywhere in the system. The meaning of the term "stainless" can vary widely depending on the source of the material. Always rely on industry standard types and insist that suppliers document the material source.

The "Or Equal" Trap

Even when we do our best to specify a set of details that will produce a good result, we can be caught in the "or equal" trap. Unless the specifications give us the tools to differentiate between potential alternatives, we may not have a leg to stand on when we need to just say no to substitutions.

Design for Flexibility

Always design instrumentation systems and ADASs with long-term flexibility in mind. Provide for extra conduits, wire pairs, cabinet space, etc. without overbuilding the entire system. Design systems as if you will have to work on them with your own hands. Think through assembly and disassembly of all the component parts.

Grounding

Grounding is one of the key elements in protecting field equipment from transient surges. The most elaborate transient arrestor system will not function without an effective ground. Grounding in rockfill shells and rock foundations can be surprisingly difficult. Installing more grounding rods in a poor location does not give a better ground. Probably the best solution is to utilize a specialty grounding subcontractor who is familiar with telecommunications or radar systems.

Vandalism

Always assume that the facilities will be vandalized at some point. Vandalism, while it is not strictly a design issue, is one of the most common and bothersome events for owners in the United States. Design the instrumentation so that it is difficult to break into or damage without great effort. Include locks or securely bolted closures on everything. Always route cables or wires through conduit of some kind. PVC-coated flex conduit is very flexible and enhances security. Providing entry alarms with an ADAS is relatively simple and cost-effective if sufficient channel capacity is provided.

Vibrating Wire Instruments

Transient Protection – Transient (surge) protection is of paramount importance for vibrating wire (vw) instruments. Next to mishandling the instrument, transients are the most common reason vibrating wire instruments fail or leave calibration in the long-run. Protecting vibrating wire instruments from transients is just as important during construction as it is in the finished installation. Some of the basics of transient protection have been detailed in other recent articles in GIN.

Venting – Whenever an application requires venting a vibrating wire instrument (generally <u>not</u> recommended, but necessary at times), always vent the instrument into the same physical space the instrument occupies if at all possible. Pressure differentials induced by temperature and wind when the vent is terminated in another enclosed space can cause unexpected variations in readings.

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John Dunnicliff, Course Director,
Geotechnical Instrumentation Consultant, England.

- Systematic Approach to Planning Monitoring Programs.
- Overview of Hardware.
- Contractual Arrangements.
- Instrumentation of Slopes, Embankments on Soft Ground, Deep Foundations, Earth Retaining Structures.
- Discussion on People Issues with Observation and Instrumentation. Co-moderator with Ralph Peck.

Ralph B. Peck,
Civil Engineer: Geotechnics.

- Observation, Instrumentation, Action – Chicago in the 30s to San Francisco in the 90s.
- Discussion on People Issues with Observation and Instrumentation. Co-moderator with John Dunnicliff.

Jeffrey A. Behr, Orion Monitoring Systems, Inc. Global Positioning Systems.

Helmut Bock, Geotechnical Consultant, Germany. Instrumentation of Tunnels.

Boyd Bringhurst, Campbell Scientific, Inc. Automated Data Acquisition Systems.

Pierre Choquet, Roctest Ltd., St. Lambert, Quebec. Fiber Optic Sensors.

Gary R. Holzhausen, Applied Geomechanics Inc. Tilt Measurements.

William F. "Bubba" Knight, Florida DOT. Case Histories. Instrumentation of Geogrid Reinforced Embankment Over Soft Soils. Instrumentation of Deep Foundations for Static Load Testing.

Jean-Ghislain La Fonta, Sol Data, France. Case histories. Real-Time Monitoring of Railway Tracks and a Dam, Including Automatic Surveying. Amsterdam Metro.

Kevin O'Connor, GeoTDR, Inc. Time Domain Reflectometry.

Tony Simmonds, Geokon, Inc. Vibrating Wire Instruments for Unique and Custom Applications.

Robert M. Taylor, RST Instruments. Measurement of Negative Pore Water Pressure in Unsaturated Soils.

Hai-Tien Yu, Slope Indicator Co. Electrolevel Sensors and Automatic Data Acquisition Systems.

Instructor to be Determined.
Instrumentation of Embankment Dams. Criteria and Case Histories.

Optional Fourth Day, March 15, 2001:

Discussion Among Attendees and Instructors of Various Topics, to be selected by Attendees. **Attendees are encouraged to send requested discussion topics to John Dunnicliff well before the course date.**

Textbook Included:

Geotechnical Instrumentation for Monitoring Field Performance, by John Dunnicliff, published by Wiley in 1988 & 1993, is a part of the course materials.

Enrollment, Fees, and Registration:

The three-day registration fee (March 12-15, 2001) received by Feb. 16, 2001 is \$1,075. Late registration (after Feb. 16, 2001) is \$1,150. Including the optional fourth day, the fees are: by Feb. 16, 2001 \$1,225; after Feb. 16, 2001 \$1,300.
All the above fees include the textbook and break refreshments. If you have, and bring, the text, each fee is reduced by \$50.

Accommodations:

The course will be held at the Holiday Inn Oceanfront Hotel, Cocoa Beach, Florida. Rates are \$82+tax single/double. To make reservations, call (800) 206-2747, or (321) 783-2271 from outside the US, or fax (321) 407-8878. To ensure a room at this rate, make reservations by Feb. 11, 2001, and mention the Geotechnical Instrumentation for Field Measurements course.

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