TRANSPORTATION RESEARCH RECORD 1004

Reliability of Geotechnical Instrumentation

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1985

Transportation Research Record 1004
Price $7.80
Editor: Elizabeth W. Kaplan
Compositor: Harlow Bickford
Layout: Theresa L. Johnson

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Printed in the United States of America

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Reliability of geotechnical instrumentation.

(Transportation research record; 1004)
1. Engineering geology—Instruments—Reliability—Addresses, essays, lectures. I. National Research Council (U.S.) Transportation Research Board. II. Series.

TE7.H5	no. 1004	380.5 s	85-13779
[TA705]		[624.1'51'028]	
ISBN 0-309-03814-6		ISSN 0361-1981	

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Contents

Addresses of Authors

Abramson, Lee W., Parsons Brinckerhoff Quade & Douglas, Inc., One Penn Plaza, New York, N.Y. 10119
Bordes, J. L., TELEMAC, 2 Rue Auguste-Thomas B.P. 213, 92600, Asnières, France
Debreuille, P. J., TELEMAC, 2 Rue Auguste-Thomas, B.P. 213, 92600, Asnières, France
Dunnicliff, John, Geotechnical Instrumentation Consultant, 8 Adams Street, Lexington, Mass. 02173
Green, Gordon E., Shannon & Wilson, Inc., 1105 North 38th Street, P.O. Box C-30313, Seattle, Wash. 98103-8067
Hannon, Joseph B., Office of Transportation Laboratory, California Department of Transportation, 5900 Folsom Boulevard, P.O. Box 19128, Sacramento, Calif. 95819
Hansmire, William H., Parsons Brinckerhoff Quade & Douglas, Inc., One Penn Plaza, New York, N.Y. 10119
Horvath, R. G., Department of Civil Engineering and Engineering Mechanics, McMaster University, Hamilton, Ontario L8S 4L7, Canada
Jackura, Kenneth A., Office of Transportation Laboratory, California Department of Transportation, 5900 Folsom Boulevard, P.O. Box 19128, Sacramento, Calif. 95819
McGuffey, Verne C., Soil Mechanics Bureau, New York State Department of Transportation, 1220 Washington Avenue, Building 7, State Campus, Albany, N.Y. 12232
Rawnsley, Robert P., Parsons Brinckerhoff Quade & Douglas, Inc., One Penn Plaza, New York, N.Y. 10119
Russell, Henry A., Parsons Brinckerhoff Quade & Douglas, One Penn Plaza, New York, N.Y. 10119

Introduction to Symposium on Reliability of Geotechnical Instrumentation

WILLIAM H. HANSMIRE

ABSTRACT

Background on the development of concepts of reliability of geotechnical instrumentation is presented. Emphasis is placed on learning from successful experiences as well as unsuccessful experiences or failures. Definitions of reliability are given, but uniform methods of characterizing reliability for geotechnical instrumentation remain to be developed.

Recent efforts of TRB Committee A2K01, Soil and Rock Instrumentation, have included exchanging information on actual instrumentation experience. Often a case history focused on the positive results of a field monitoring program. The negative experiences, the failures, were often not reported. Uncertain liabilities or ongoing litigation kept the facts from being disclosed. Perhaps just as often, unwillingness of the professional worker to share an unpleasant experience kept many failures from being reported. Thus, it was often noted that mistakes were repeated. Neither instrumentation suppliers nor users were learning as much as they should have been from the past experience of others.

Instrumentation failures were for a time the topic of active Committee discussion. Some members believed that practitioners should be able to learn a great deal from the study of failures, in the same way that much has been learned from structure foundation or earth slope failures. Further thinking, however, suggested that a still broader approach should be taken to understanding past instrumentation experience. Why instrumentation did not work, as well as why it did work so well in some cases, was of interest. Reliability of geotechnical instrumentation was then recognized as the broader concept that was appropriate for exploration.

So far there has not been a compact expression to characterize reliability in the context of geotechnical instrumentation. On the basis of Webster's Ninth New Collegiate Dictionary (Merriam-Webster, 1983), the following can be stated:

> Reliability: The quality or state of being reliable (a noun).
> Reliable: Suitable or fit to be relied on (an adjective).
> Rely: To have confidence based on experience (a verb).

The Dictionary of Scientific and Technical Terms (McGraw-Hill, 1974) gives the following:

> reliability: (engineering) The probability that a component part, equipment, or system will satisfactorily perform its intended function under given circumstances, such as environmental conditions, limitations as to operating time, and frequency and thoroughness of maintenance for a specified period of time. (Statistics) 1. The amount of credence placed in a result. 2. The precision of a measurement, as measured by the variance of repeated measurements of the same object.

As can be seen from its definition, reliability can be a broad topic. Perhaps the most telling word is "experience" in the definition of "rely." It means that reliability cannot be created on paper. Instead, reliability of an instrument has to be tested by actual use in the field.

Most practitioners in geotechnical instrumentation agree that there are no mathematical models that characterize reliability. Current work on instrumentation for nuclear waste repositories will no doubt require probabilistic approaches to ensure adequately designed systems. Probabilistic characterization of soil procedures has an active following, but its application to everyday use is beyond the state of the practice of geotechnical instrumentation. Most practitioners in the transportation industry probably do not want to know if something is "90 percent" or "99.9 percent" reliable. Most workers are not able to appreciate something that sounds so much like a technological cliché. Perhaps in the future more rigorous concepts of determining reliability will be used. For now, however, simpler, more subjective tests of reliability must be used.

Subjective evaluations of reliability are typical. In NCHRP Synthesis 89, Geotechnical Instrumentation for Monitoring Field Performance, John Dunnicliff uses the terms "Very Good," "Good," and "Fair." An occasional "Poor" is noted. Often what makes one device good has no application to another. Therefore, it is difficult to make sweeping generalizations about what constitutes reliability.

One of the most difficult aspects of understanding reliability is that it necessarily involves human factors as well as physical factors associated with the instrument hardware and its installed environment. Statistics may be able to characterize reliability in an abstract sense. However, what is of most interest to this Symposium is the "why or why not" physical details behind the reliability of geotechnical instrumentation.

The approach to getting a measure of reliability for this Symposium was to address the following questions:

- How was the correctness of the instrument readings established?
- What was the quality of performance of personnel who installed and maintained the instruments and took the data readings?

- What was the durability of the instrument in the installed environment?
- Did the instrument do the job intended and, if not, why not?
- What were the lessons learned from the instrumentation experience?

This Symposium, then, attempts to address reliability on the basis of the experience of others. Topic reporters gathered information on reliability in the following categories of instrumentation:

- Pore pressure,
- Earth pressure,
- Load and strain in structures, and
- Deformation.

The first three categories are reported at this Symposium. Case histories include all categories. As will be seen from the papers, each reporter's approach to characterizing reliability was somewhat different. This reflects real human considerations and the diverse nature of the topic.

This Symposium is to be a focal point for exchanging information, learning, and improving future work. It is expected that future sessions can be held that will encompass deformation measurements and other geotechnical instrumentation experience. It is hoped that future presentations will report on experiences with well-planned and executed instrumentation programs with well-defined and realistic objectives of reliability.

Reliability of Pore Pressure Measurement

VERNE C. McGUFFEY

ABSTRACT

The importance of reliable pore pressure measurements and their influence on design and construction are discussed. Methods of obtaining high-quality data are related to five major items: (a) system design, (b) instrument design, (c) installation details, (d) operator knowledge, and (e) engineering interpretation methodology. Suggestions for addressing these factors are given. It is concluded that attention to detail in all phases by a responsible engineer is necessary to obtain reliable data.

Engineers have been attempting to determine the state of stress in soil by measuring excess pore water pressure for many years. The results reportedly ranged from good to unacceptable. In an effort to improve results, sophisticated electronic instruments have been developed that measure pressures as small as 1/100 psi. Results have not improved (1).

Improved reliability must, therefore, address two variables: (a) the instrument performing properly and (b) the soil system performing as predicted.

The major items that contribute to successful (or reliable) pore pressure measurements are

- System design,
- Instrument design,
- Installation details,
- Operator knowledge, and
- Engineering interpretation methodology.

Reliable pore pressure measurements can only be obtained by planning equally for all of these factors.

IMPORTANCE OF RELIABILITY

Pore pressure measurements are taken to allow the engineer to accurately predict the state of stress in the soil and to make appropriate engineering decisions. Reliable pore pressure measurements allow the engineer to use specialized cost-saving construction procedures with little risk. Undetected undependable measurements may lead the engineer into taking risks the results of which are costly or disastrous, or both.

The engineer must have a means of evaluating the reliability of all parts of the decision-making system. Some ways of ensuring reliable data for decision making are discussed in this paper.

SYSTEM DESIGN

A high-quality design must be done to allow determination of the type of instrument, location of instrument, frequency of readings, and other key features needed to ensure success of the system.

Design factors that need further discussion are

- Soil profile,
- Geotechnical model chosen for analyses,
- Vertical and horizontal soil parameters,
- Expected loading, and
- Groundwater.

Soil Profile

A pore pressure measuring device in the center of the layer under the center of the loading can be expected to read the maximum pore pressure. However, if the pore pressure measuring device is near a boundary of the compressible layer, the pore pressure will be greatly reduced. In many cases instruments reflect the pore pressures of the free draining adjacent layer because of local variations. A detailed knowledge of the horizontal and vertical variability of the soil profile is, therefore, an essential part of the design of a reliable instrumentation system.

Geotechnical Model

Choosing a suitable geotechnical model for the construction plan is necessary in order to design a reliable instrumentation package. The geotechnical model for sand drains is a relatively straightforward and accepted model. Numerous investigators have installed pore pressure measuring devices near the center of a group of sand drains and have recorded pore pressures that were extremely close to those predicted by the mathematical models for sand drain design (poor construction control of deep drains or piezometers can lead to poor response).

However, the model that is normally used for a simple embankment or abutment loading is not as well understood. The system is highly dependent on the vertical and horizontal drainage boundary conditions at the site. Normal practice is to design for vertical drainage only. This model is unacceptable for most real-life field conditions. All strip-loading situations have a major component of lateral drainage. LaCasse et al. (2) have developed a usable model for including lateral drainage in the normal design process. The New York State approximate method (3) can also be used with reasonable results.

Vertical and Horizontal Soil Parameters

The vertical coefficient of consolidation (c_v) can usually be obtained with a reasonable degree of accuracy by high-quality sampling and laboratory consolidation testing.

The horizontal coefficient of consolidation (c_h) is more difficult to obtain. Earlier work used undisturbed samples with consolidation tests taken across the sample instead of taken vertically. These tests gave reasonable results for the horizontal coefficient of consolidation when c_h was close to the value of c_v and the soil was relatively uniform. However, this approach did not work well on layered systems. A great deal of work was done by various investigators trying to use field percolation tests as a tool for predicting horizontal drainage rates. The reported results were erratic.

Some recent work done in New York State has made use of the "block permeability test" that allows permeability testing to be done in both vertical and horizontal directions on the same sample. This test produces a reliable value of the ratio of horizontal to vertical permeability for the sample. This can then be correlated, through moisture content and plasticity index tests, with the rest of the soil system being studied to arrive at a representative value of the ratio of horizontal to vertical permeability for the design.

New York State experience indicates that the ratio of horizontal to vertical permeability from backfigured field tests (a) has never been less than 1, (b) is usually more than 2, and (c) often will be in the range of 10 or more in even slightly layered systems. Because of the potential for changes within the boundary conditions in nature, it is recommended that a value of c_h over c_v greater than 20 not be used. A ratio greater than this will usually not change the design concepts, but it can give large errors in performance if conditions vary.

Horizontal and vertical coefficients of consolidation in the recompression ranges are appreciably different than those in the normally consolidated ranges. It is common to find the vertical coefficient of consolidation in a precompressed material to be 8 to 10 times the coefficient of consolidation in a normal consolidated soil even though the permeability is less.

A zone of high precompression above normally consolidated soil will not allow free vertical drainage upward because of the low permeability.

Expected Loading

The design must also consider the variation of load expected. The magnitude and shape of expected loading should be reasonably well obtained from the design of the facility being constructed. However, the type of material used for embankment construction has a variability from approximately 100 lb per cubic foot (for certain rock) to 150 lb per cubic foot (for extremely densely compacted long graded soils). Many embankment materials will be placed during a relatively dry period of the year. When heavy rains occur, there is a dramatic increase in loading as a result of the weight of water taken into the soil pores. The loadings predicted for a bridge or other structure are usually not accurately identified for the geotechnical engineer. He is usually supplied with the maximum loading, which does not occur during the construction period; he is rarely, if ever, given the loadings to be expected during construction, when pore pressures are critical.

Pile driving creates a relatively large temporary pore pressure. This pore pressure may exceed 20 psi while a group of piles is being driven. Pore pressures from pile driving have been measured 100 ft or more from the pile-driving area. Their temporary pore pressure dissipates laterally quite rapidly. An approximation to estimate lateral pore pressure dissipation from pile driving follows:

1. Assume a value of 10 psi at a distance of 20 ft from the center of the pile group and
2. Assume total dissipation at a distance of 200 ft.

The contractor's method of operation will influence pore pressure measurements. Although construction procedures cannot be predetermined, some conditions should be considered when designing an instrumentation system. Examples are temporary detours, haul roads, and structure construction. Some temporary loadings can be anticipated on the basis of good knowledge of construction practices and can be designed into the system or controlled during construction by notes in the contract. Most can only be identified during construction, however, and the designer must be prepared to reevaluate the pore pressure measuring system and his interpretation of its reliability on the basis of actual construction procedures.

Groundwater

Normal fluctuations of the groundwater system can give erroneous indications of pore pressure changes.

Most sites adjacent to water crossings have a sand cover over the compressible soil systems. This sand cover allows a relatively rapid change in the groundwater as a result of rainfall or changes in an adjacent stream, lake, or ocean. Contractors' operations--such as local dewatering for sewers, construction of temporary drainage ditches, and construction of temporary retention ponds--influence the local groundwater regimen and cause erratic readings on pore pressure measuring devices. Most of these variations can be identified and their effects eliminated by including, as part of the design of the pore pressure measuring system, a series of surface observation wells to specifically measure local variation in the groundwater table.

The five factors discussed in this section must be addressed during design or corrected for in construction to obtain high-quality pore pressure data.

INSTRUMENT DESIGN

Different instrument designs are discussed thoroughly in the NCHRP synthesis on geotechnical instrumentation (4) and, therefore, will not be discussed here except as they affect the reliability of the instrumentation system.

Many instruments presently used have characteristics that may influence their ability to give correct responses for a specific design. It is generally better to use existing instruments with known different capabilities to accommodate unusual circumstances than to design a special instrument.

Open-well-type piezometers have a good long-term record of performance, but they usually provide too slow response for low permeability soils. Pneumatic cells have demonstrated good reliability and rapid response; however, they do not have the ability to measure dynamic pore pressures. For continuous records of dynamic response an electronic pore pressure cell or a closed-system hydraulic cell may be better.

Each piezometer has its own characteristics and must be matched to the needs of the site being designed.

INSTALLATION

The effects of installation practices on the reliability of pore pressure instrumentation systems are discussed in the NCHRP synthesis on geotechnical instrumentation (4) and in AASHTO specifications (5). Some installation practices that have a direct relationship to the reliability of instrument systems will be discussed here.

One of the easiest items to check during installation is the responsiveness and accuracy of the cell as it is being installed. Pneumatic and electronic cells can be measured in the laboratory before installation and can also be checked when lowered into the installation hole by measuring the height of water above the cell and recording instrument response at different levels.

For closed-system hydraulic piezometers, it is best practice to completely fill all tubing with deaired water before installing the system in the ground. If all connections are then made quickly underwater, the system will usually respond for many years without problems.

Initial readings should be taken immediately after installation and periodically for approximately 1 week or until the pore pressure recorded reflects the groundwater system variations.

The method of installing a cell in the ground often affects the reliability of the cell during its useful life. Installing the cell beyond the tip of a steel casing and leaving the casing in the ground have resulted in many failures because when the protective casing settles it cuts the protruding measuring tubes (conversations with Vermont DOT). This can be avoided by installing the cell in the end of the casing.

Piezometer cells installed in cement grout have had similar types of problems with crimping or pinching of the tubing as a result of the movements of the soft compressible soils around the relatively rigid column of grout. Although a bentonite and sand mixture is difficult to install, it has worked for many years, even in areas of extremely large foundation settlements. On one project, however, it took nearly 2 weeks after installation before the bentonite expanded sufficiently to obtain a good seal.

Installation of leads to the readout location has resulted in numerous system failures. If the trench is too wide or not deep enough, construction traffic may damage or destroy the lines. Lines that cross each other in the trench have been crimped, making them inoperable. Leaving the lines exposed in the trench without backfilling after completion of the connections can result in damage; deterioration of tubing from ultraviolet exposure and large volume change and creation of air pockets in fluid-filled lines are examples. Immediate covering with a 6-in. bedding layer of sand is good practice.

It is essential that the installation inspector be thoroughly familiar with the type of operation he is carrying out. If there is any doubt, hire special trained help.

OPERATOR KNOWLEDGE AND ABILITY

It is New York State experience that the person responsible for reading the geotechnical instrumentation is usually the lowest paid, least experienced inspector on the project. Certain types of instruments are less susceptible to operator error and damage during the life of the project and their use should, therefore, be considered when the knowledge of operators is poor.

It is best practice to educate the operator about the purposes and characteristics of the pore pressure measuring system. It is essential that the operator know how the system operates and what to look for so that he can give early warning of potential problems. Instruction on how to check for the charge in the batteries on electronic systems; how to recharge gas systems; and how to properly store and handle equipment in dusty, hot, or freezing conditions is needed. The operator must also be aware of what to do about changing temperatures and other changes in the vicinity of the readout equipment.

Part of the education of the operator includes setting up a good line of communications between the operator and the engineer responsible for interpreting the data. This can be handled by visits, telephone or written communications, or other similar procedures. One effective way to ensure adequate communication is to periodically visit the inspector to discuss progress and agree on what to do at important times in the construction.

ENGINEERING INTERPRETATION METHODOLOGY

The method used to interpret the pore pressure and to estimate the changes within the soil system influences the interpretation of reliability. To determine what is happening within the soil and determine whether the instruments are recording properly, the following steps are helpful:

1. Obtain complete and accurate information about the construction site including elevation of

fill, adjacent loadings, change in water surface, change in river levels, and other appropriate information.

2. Obtain data immediately after readings are taken and compare the data with changes in fill height and groundwater and expected dissipation rate.

3. Investigate in detail any readings that do not respond in the direction and approximate magnitude estimated by the prediction model chosen in design. If the data obtained do not conform accurately with the prediction model, the reading is wrong or the model is wrong. Check the instrument first and then investigate alternative prediction models.

4. Plan specific check points during the project life to reassess the design model; including stages of construction with waiting periods helps. Look for activities that will cause pore pressure changes (such as structure excavation) and check responses carefully.

5. Normal accuracy of field data needs special consideration at this stage. Field survey of plus or minus two hundredths of a foot (and on ground or fill, plus or minus 1 ft) is normal. Variations of up to 30 percent in the weight of the fill can occur, but, if the weight is different at one fill location, it should be the same at all locations of similar fill.

6. Always check the final zero reading. Unfortunately, the excess pore pressure seldom returns to the "before construction" reading. This is a result of the changes that have taken place during construction. The pore pressure measuring point may have settled to a level further below the groundwater table than it was before the construction started causing a higher reading. The groundwater table may have changed as a result of the construction. When these changes are accounted for, the pore pressure reading should return to zero within the predicted time if the design and instruments are correct.

7. Check the prediction model. The geotechnical model chosen may not be the correct one. If there is not close agreement, construct a revised model using new pore pressure data as a basis for constructing the new model. If the new model is correct, it will show consistent responses through all construction activities. Changes in boundary drainage conditions, such as one-way to two-way drainage, sometimes occur in construction. If there is insufficient instrumentation to verify the change in the model, additional devices must be installed to verify the model and make correct engineering decisions.

CONCLUSIONS

The primary reason for determining the reliability of pore pressure measurement systems is to tell the engineer if the information is sufficient to make correct construction decisions. If the pore pressures are too high, a major failure may occur, destroying the structure being built. If the pore pressures are dissipating too rapidly, the engineer is wasting money on foundation treatment that is not needed. The engineer must be prepared to make decisions during construction in order to economically build the facility and to reduce the risk of a major, disastrous failure.

As can be seen from this discussion, any number of small details of design, installation, or interpretation can adversely affect the reliability of the instrumentation package. Therefore, the engineer must design checks and "memory joggers" into the process so that problems can be corrected immediately. The key to a reliable pore pressure measuring system is a qualified engineer who is responsible for all phases.

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Measurement of Earth Pressure

JOSEPH B. HANNON and KENNETH A. JACKURA

ABSTRACT

Factors that affect the measurement of earth pressure are discussed. Several variables, including cell design, installation procedures, and environmental effects, are cited for their influence on pressure cell behavior. A sampling of pressure cell installation and monitoring experience by various agencies and organizations is presented along with comments on pressure cell failures, potential causes of failure, and suggestions for improved performance on future installations. An example of an alternative procedure for determination of horizontal earth pressure is presented. It is concluded that accurate measurements of earth pressure are difficult to achieve unless pressure-sensing instruments are properly selected, calibrated, and installed by experienced personnel.

Accurate measurement of earth pressure is helpful in evaluating the performance of a structure. Earth pressure measurements may be desirable at any point within a soil mass or at the interface between the soil mass and a structural element.

The most desirable way to detect total or changing stress conditions requires installation of some type of earth pressure sensing cell during construction. Cells for this purpose are diaphragm devices that sense pressure changes. These devices are sold commercially as earth pressure cells or soil stress meters. They are also called earth or soil stress cells. These pressure-sensing devices are pneumatic, hydraulic, vibrating wire strain gauge, or bonded or unbonded resistance strain gauge units. A description of each of these units follows.

Fluid-Filled Cell (oil or mercury)

Pneumatic Readout System

An air- or gas-charged pneumatic readout system to balance out the soil stress imposed on the sensing diaphragm of the cell.

Hydraulic Readout System

Identical to the pneumatic cell except that oil rather than air or gas is used in balancing out the soil stress.

Electrical Readout System

- Vibrating wire strain gauge cell (transducer externally mounted)
- Bonded resistance strain gauge cell (externally or internally mounted transducer)
- Unbonded resistance strain gauge cell (externally mounted transducer)
- Piezoelectric cell (sensor mounted either externally or internally)

Deflecting Diaphragm Sensing Cell (sealed air- or gas-filled cavity)

This cell has an electrical readout system (bonded strain gauge or vibrating wire strain gauge).

Not all earth pressure sensing systems are applicable to specific situations. The performance of a given pressure cell can be affected by its own design and by the environment in which it is installed.

The various factors that influence the accurate measurement of earth pressure will be discussed as will performance information on the reliability of different devices for earth pressure measurements for short- and long-term applications based on a sampling of experience. Suggestions on the use of alternative measurement systems to indirectly estimate earth pressure will also be presented.

DISCUSSION

General

Many agencies and organizations have experienced difficulties on one or more instrumentation projects that have failed to provide reliable data for project needs. Technical difficulties have sometimes been encountered with field data because of poor instrumentation devices and lack of experience with proper installation and data-collection procedures.

It is important to acquire instrumentation devices that provide accurate and reliable field data. To ensure success, instruments should be properly installed and calibrated.

In general, the California Department of Transportation (Caltrans) has experienced adequate success with most instrumentation except soil pressure cells. During the 1960s the data readout from pressure cells was often erratic, with time-dependent drifting of indicated pressures. Also, uncertainty existed as to data accuracy because of the abnormal stress distribution generated by the installation of the pressure cells within the soil.

These problems led Caltrans to research various cell types for accuracy, long-term stability, and best method of calibration (1,2). The effect of various types of bedding soils on pressure cell response was also evaluated. Factors affecting the accuracy of pressure cell (stress cell) measurements, as described by Weiler and Kulhawy (3), are given in Table 1.

Environmental Effects

The main objective in the design of a pressure-sensing device is to minimize the effects of the

TABLE 1 Factors Affecting Accuracy of Pressure Cell Measurements [Weiler and Kulhawy (3)]

Factor	Description of Error	Correction Method
Cell thickness-to-diameter ratio	Cell thickness alters the stress field around the cell	Use relatively thin cells
Soil-to-cell stiffness ratio	Changing soil stiffness may cause a nonlinear calibration	Design cell for high stiffness and use correction factors
Diaphragm deflection (arching)	Excessive deflection changes stress distribution over cell	Design cell for low deflection
Stress concentrations at cell corners	Cause cell to overregister by increasing stress over active cell face	Use inactive outer rims to reduce sensitive area
Eccentric, nonuniform, and point loads	Soil grain sizes too large for cell size used	Increase stress cell active diameter
Lateral stress rotation	Presence of cell in soil causes lateral stresses to act normal to cell	Use correction factors
Stress-strain behavior of soil	Cell measurements influenced by confining conditions	Calibrate cell under near-usage conditions
Placement effect	Physical placing of the cell causes disturbance of the soil	Random error; use duplicate measurements
Proximity of structures and other stress cells	Interaction of stress fields of cell and structure causes errors	Use adequate spacing
Dynamic stress measurements	Response time, natural frequency, and inertia of cell cause errors	Use dynamic calibration
Corrosion and moisture	May cause cell "failure" by attacking the cell material	Use extra waterproofing precautions
Placement stresses	Overstressing during soil compaction may permanently damage cell	Check cell design for yield strength

device on the environment in which it will be placed, so that the actual state of stress can be measured. Numerous investigations have been conducted to determine the effects of pressure cell-soil interaction under loading. If the pressure cell is stiffer or less stiff than the soil around it, stress concentrations can develop and result in overregistration or underregistration, respectively.

Taylor (4) in 1947 found that thin, stiff cells will produce a maximum and determinable overregistration. Compressible cells will underregister but, due to variable soil-cell modular ratio (E_s/E_c) under loading, will not have a constant underregistration value.

This phenomenon was graphically illustrated by Tory and Sparrow in 1967 (5) and is shown in Figure 1. Cell error is shown to be primarily a function of

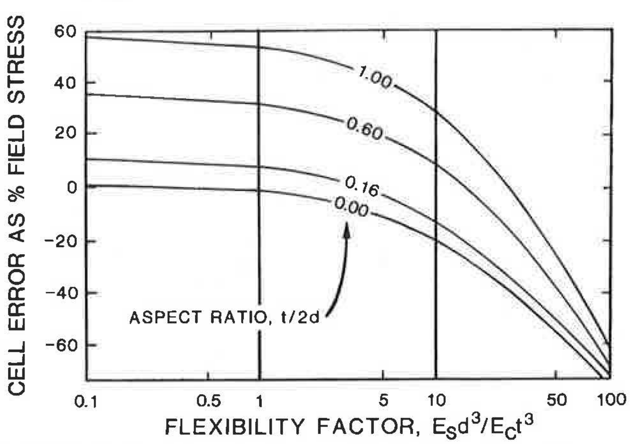

FIGURE 1 Variation of cell error with flexibility factor (5).

the soil-cell modular ratio (E_s/E_c) and the aspect ratio (t/2d), where t and d are cell thickness and diameter, respectively. The ratio E_sd^3/E_ct^3 is called the flexibility factor. Figure 1 shows that for small flexibility factors (stiff cells), cell error remains reasonably constant, but as the flexibility factor increases, cell registration error becomes quite dependent on soil modulus.

Because many soils have a stress-dependent modulus, Tory and Sparrow's information underscores the importance of using stiff cells in order to keep the registration error near the horizontal portion of the curve. The figure also shows that, for a given diameter, thin cells are necessary for greater accuracy in cell readout. Cell overregistration can also be controlled by limiting the cell's sensitive region to the central portion of the cell. Peattie and Sparrow (6) obtained greatly reduced registration error when they limited the sensitive portion to approximately 50 percent of the cell diameter.

Cross Sensitivity

Brown and Pell (7) in 1967 uncovered a problem associated with strain-gauge bonded diaphragm cells in regard to their sensitivity to point loading of in-plane or radial forces. Because the purpose of a pressure cell is to be responsive only to pressures normal to its face, sensitivity to radial point loads, primarily due to Poisson's effect, is an undesirable feature. Insulating the inner sensitive diaphragm from the effects of in-plane loading was described by Weiler and Kulhawy (3) in 1978.

Geometric Shape and Design

The work conducted by Monfore (8) and Peattie and Sparrow (6) and others to minimize registration error by reducing the sensitive region of the cell is one of the innovative advances in soil pressure cell design in recent years. However, Caltrans reinvestigated cell design theory and undertook a finite element study using a program developed by Herrman (9) to evaluate alternative geometric cell shapes with full face sensitivity to reduce registration error. If full face sensitivity could be successfully used with minimum registration error, larger soil masses could be measured (in contrast with cells of the same diameter with only 50 percent face sensitivity) and thus aid field stress measurement reliability. Furthermore, the Caltrans' finite element studies reported by Forsyth and Jackura in 1974 (10) indicated dramatic registration errors for cylindrical cell models having partial face sensitivity to in-plane orientation of the major principal stress (σ_1). This phenomenon was not studied extensively by other investigators. With these factors in mind and assuming that fluid-filled cells would be more advantageous for pressure recordings because they are less sensitive to point loading or soil pressure eccentricities than are diaphragm deflecting cells, the Caltrans finite element study team reached the following conclusions:

1. A cell with cell-soil modular ratio (E_c/E_s) near 10 is desirable to ensure that cell registration error is reasonably independent of variations in soil modulus (E_s). A cell modulus (E_c) of at least 3 x 10^5 psi is recommended.

2. Pressure cell geometry is an important physical characteristic associated with minimizing registration error. For soil principal stress ratios between 2 and 2.5 where the theoretical test study was conducted, cell geometry can be effectively used to

minimize registration error regardless of cell orientation to the major principal stress.

3. The geometric cell shape most conducive to reducing cell registration error is a circular, long edge tapering configuration. If cells are going to be variably oriented to the major principal stresses, uniform thickness cells with either too soft or too rigid annuli are not considered viable pressure cell designs because of their sensitivity to orientation to the major principal stress.

4. The overall dimensions of the pressure cell should be related to soil particle size. The bedding soil contact with the pressure cell diaphragm should not be allowed to vary greatly from its surroundings or redistribution of stress may occur. Kallstenius and Bergau (11) suggest that, for flush-mounted piston-type cells in rigid walls, the cell diameter should be at least 50 times the largest soil particle size. For all other installations or cell types, cell diameter to particle size can be on the order of 5 to 10 (3).

5. Cells should be waterproof and cable entries should be strong enough to resist stresses and deformations during installation.

Calibration

Most manufacturers of earth pressure cells provide a calibration chart established by loadings in either air or water. However, an additional verification of a cell's calibration is generally necessary before use because commercially supplied calibration curves are sometimes insufficient.

The proper method of calibration is still somewhat debatable within the geotechnical community. Dunnicliff (12) and others suggest that pressure cells should be calibrated in a large chamber using the same soil in which the cell is to be embedded and the same installation procedures, unless the installation is made in soft clay. Suitable calibration chambers are described by Hadala (13), Hvorslev (14), Selig (15), and Smith (2).

Jackura (16) reports that calibration can be adequately performed by hydraulic or pneumatic means provided the pressure cell design, construction, and geometry meet established criteria for acceptable performance (9,17,18). He also suggests that, when needed, cell action factors (registration errors) can be estimated by theory.

Laboratory calibration in soil can also become a matter of practicality because large cells would require enormous testing vessels to develop full pressure bulb regions and minimize sidewall frictional effects (16). Taylor (4) suggests minimum test vessel size-to-cell diameter and test vessel depth-to-cell diameter ratios of 8:1 and 4:1, respectively. As an example, for a cylindrical test mold arrangement, assuming the calibration of a 10-in.-diameter soil stress meter, this means soil volume of several cubic yards. Scaled-down sizes can possibly be made effective if side friction is reduced by a greased liner (13). However, even then, transmission pressure must be measured to determine residual sidewall friction. Walter et al. (19) present data on the use of a greased liner, which substantially reduced but did not eliminate sidewall friction.

Unless a user is familiar with the behavior of a particular pressure cell and the proper procedure for its calibration, installation, and bedding within an earth mass, pressure cells may be inappropriate and uneconomical. This is reinforced by the following statement from Weiler and Kulhawy (3, pp. 2-13 through 2-15).

> The present need to calibrate the cells in the soil in which they will be used, as well as the significant amount of time needed to acquire a familiarity with stress cell behavior, makes the use of stress cells uneconomical for most projects. When the cells are "economically" used (meaning no in-soil calibration and no time spent investigating how stress cells behave in soil), the results are nearly always unusable if not incredible. Accurate stress cell measurements are still almost exclusively limited to well-conducted laboratory model investigations.

It therefore appears that although accurate measurement of soil stress is desirable, it may be uneconomical for most installations. Users may sometimes need to settle for reasonable instead of accurate results. Users may be required to rely on alternate measurement systems to provide an estimate of field pressures.

Installation Procedure

A common method of cell installation for point of stress monitoring is to first construct the fill by normal compaction equipment to an elevation approximately 1 ft or more above the instrument level, then to excavate a trench for instrumentation placement. The trenches are then backfilled with several inches of either embankment material screened through a No. 4 or finer sieve or a concrete sand and are hand compacted to at least the same compaction as the surrounding fill. The cells are then fitted into their compacted layer and a 2- to 6-in. layer of the same hand-compacted backfill is placed over the cells to provide sufficient protection for the cells and uniform load transmission. Additional compactive effort is sometimes supplied by hand-guided compaction equipment. Dunnicliff (12) believes that there is no better alternative to procedures of this type. He maintains that considerable underregistration can occur if the backfill is not brought to the same density and compressibility as the surrounding embankment soil.

Trapezoidal or V-bottom trenches with flat side slopes may be more desirable and provide better cell response with less soil bridging than do vertical trenches.

Adequate cell installations, required for measurement of soil stress at a soil-structure interface, are generally more difficult to achieve than that described previously for point stress monitoring. For these installations, it is extremely important to install and grout cells in preformed cavities within the structure's outer face at the soil contact. Control of backfill adjacent to each cell is also important. Representative compactive effort is generally difficult to achieve. Both overregistra- tion and underregistration errors are common with interface stress cell locations because of overcom- pactive effort or insufficient cell-soil contact, respectively.

It is believed that a common cause of underregistration is backfilling around an installed pressure cell that has had insufficient time to adjust to the embankment temperature. Hydraulic cells are most susceptible to volume changes resulting from changes in temperature gradient under load and cannot be calibrated for this phenomenon. Thermal expansion coefficients of 2×10^{-5} in. per degree Fahrenheit for oil-filled cells and one-fifth of that for mercury-filled cells are quite likely. Cell contraction due to cooling will result in measured pressure decreases. Pressure decreases will be more pronounced in dense soils (especially granular types) due to significant soil arching over the contracting cell. Where cells are placed flush mounted in concrete structures, cell cooling is expected to cause even greater pressure decreases, as a result of soil arching over the cell at the cell-structure interface, than are registered by cells placed wholly within soil. A temperature adjustment time of at least 6 hr at 55° ±5° F (average earth temperature) before cell placement will be sufficient to preclude most registration error.

PERFORMANCE AND RELIABILITY

Table 2 gives a sampling of experience by various agencies and organizations with various types of pressure cells and stress meters. The following section relates to Table 2 and provides comments on cell failures, potential causes of failure, and suggestions for improved performance of future installations.

California Department of Water Resources

The California Department of Water Resources (DWR) attributes the malfunction of stress meters to one or more of the following causes:

- Moisture,
- Improper installation,
- Improper backfill,
- Bad connections at junction boxes,
- Strong direct current, and
- Corrosion.

Moisture in contact with the instrument leads at several installations caused false readings and instrument failures when instruments were located below the groundwater surface.

DWR believes that proper installation is the key to good performance, which is dependent on perfect contact between the meter and the adjacent structure-soil interface. DWR engineers believe that improper meter placement, poor compaction, and so forth contributed significantly to the high failure rate of concrete stress meters on three projects. Instead of being encased in a surrounding layer of concrete, the meters were installed directly against the foundation material. These meters were not designed for an interface situation. Sand backfill placed around meters at one pumping plant location was actually transported away from the meter contact faces by rainwater. Meters were left with point contact in rock.

DWR procedures for splicing of lead cables at junction boxes were not standardized for all locations. Improperly sealed wires frequently shorted out. A method that proved successful was to solder the lead wires, cover them with a wire nut, and encapsulate the whole in an epoxy molding compound. Failures could also have been prevented by providing sufficient cable lengths to eliminate the need to splice.

Experience also indicates that any direct current near an electrical resistance stress meter may pass through the meter to the ground causing false or erroneous readings. Recorded stress measurements were generally good at four pumping plant locations before plant start-up. When plants went into operation, the readings gradually decreased toward tension. The same trend was experienced at all four pumping plants. The cathodic protection system at the four plants was also a possible source of the stray direct current. Corrosion was also cited as a possible cause of stress meter failure at one location.

Colorado DOT (Sinco & Gloetzl pneumatic pressure cells)

Typical pressure cell installation procedures consisted of constructing grade to approximate cell elevation, bedding cells, and backfilling 0.5 to 1 ft over cells with fine grained material. Reliance was placed on manufacturer's calibration curves and no independent cell calibration was made.

Georgia DOT (Irad vibrating wire pressure cells)

Experience in Georgia indicates that

1. Cell accuracy was questionable due to probable arching around vertical cells and settlement and installation method on horizontal cells,
2. Cells are electrical but simple to use,
3. Readout equipment is easily calibrated,
4. Protecting lead wires in embankment is somewhat difficult,
5. Overall reliability is considered fair,
6. Cells were installed and monitored by competent personnel,
7. It was assumed that cells were properly installed and their installation did not affect cell behavior,
8. Not much maintenance and care are required, and
9. Readings appeared reasonable but there was no way to double check except by comparing with theoretical readings.

University of Nottingham (Nottingham electrical resistance pressure cells)

Experience at the University of Nottingham indicated that

1. The accuracy of the pressure cells was considered to be within ±15 percent of the true stress in a clay.
2. The Nottingham pressure cell is perhaps the simplest arrangement possible; it uses a full bridge of strain gauges.
3. It is possible to switch in a resistance across one arm of the bridge to simulate a known stress for self-verification during bench testing.
4. Dead weight calibration is used for bench testing, and calibration was done with triaxial cell specimens of the subject soil to determine cell registration.
5. Cells have a titanium body that provides good durability. Failures occurred because of poor adhesion of strain gauges or moisture ingress at cable entry.
6. Cells provided good reliability in fine grained soils and fair reliability in coarser materials when protected with fine grained backfill.
7. Installations should be made by experienced laboratory staff. Minimum disturbance of existing material during installation is considered extremely important.
8. Good, clear, logical records should be kept.
9. Duplicate readings from different instruments are desirable, but measurement of the same parameter with at least three instruments is preferred.
10. It is desirable to gain the confidence of the resident engineer when installing instrumentation.

TABLE 2 Sampling of Pressure Cell Experience

Agency or Organization and Type of Facility	Instrument Type	Instrument Manufacturer	No. Installed	No. Survived	No. Functioning	Remarks
Caltrans						
Reinforced Earth with steel facing (Route 39)	Hydraulic	Gloetzl	50	50	50	Good response for vertical, horizontal, and inclined plane installations
Reinforced Earth with steel facing (Route 39)	Electrical resistance	Gentran	10	10	10	Good response for vertical, horizontal, and inclined plane installations
Mechanically stabilized embankment with concrete facing (Baxter, Calif.)	Electrical resistance (soil stress meter)	Carlson	6	6	6	2 overregister, 2 underregister, and 2 provide reasonable data (horizontal pressure cells incorporated in concrete facing)
Mechanically stabilized embankment with wood facing (Delhi, Calif.)	Pneumatic with valve manifold and nitrogen source	Gloetzl	10	10	10	Horizontal pressure cells with valve manifold and nitrogen source; about half of cells have underregistered
Jail Gulch embankment	Electrical resistance	Gentran	53	34	34	Pressure cell groups installed in trapezoidal trenches (3 ft deep with 1 ft bottom and sides on 6:1 slope); 19 failed initially and 13 more failed within 6 mo
DB & Cross Canyon (large culverts)	Electrical resistance	Cambridge Meters	96	96	90	6 meters failed electrically after 6 mo
	Electrical resistance (transducer)	Kyowa	160	159	159	Electronically most of the cells performed satisfactorily for 3 yr of data collection
	Electrical resistance concrete interface	Carlson	182	180	180	Data from some cells were erratic indicating abnormal pressure readings; no explanation given for this behavior
	Electrical resistance (transducer)	Ormond	170	168	168	It is believed that pressure cells reflected actual conditions
Mechanically stabilized embankments and Reinforced Earth (concrete facing)	Soil stress meters	Carlson	12	12	12	All cells provided reasonable data
California Department of Water Resources						
Several pumping plants	Electrical resistance soil stress meters	Carlson	82	81	45	19 are still functioning after 9 to 13 yr; other meters developed shorts or electrical damage
	Electrical resistance concrete stress meters	Carlson	54	41	27	None functioning after 9 to 13 yr; overregistration on several meters; 26 had erratic readings; 20 shorted out or had electrical damage
Castaic Dam	18-in. dynamic stress meter with Maihak vibrating wire transducers and Carlson-type strain gauge transducers	Aerojet General Corp.	15	15	15	Maihak transducers have performed well for 11 yr
Oroville Dam	Soil stress meters	Carlson	29	-	20	Erratic reading and electrical shorts
	18-in. dynamic stress meter with Maihak vibrating wire transducers and Carlson-type strain gauge transducers	Aerojet General Corp.	27	-	20	Erratic reading and electrical shorts
	30-in. dynamic stress meter with Maihak vibrating wire transducers (dynamic) and CEC resistance type sensors (static)	Aerojet General Corp.	15	15	15	About half function at present time
	Electrical resistance concrete stress meter	Carlson	9	9	9	6 installed in grout gallery and 3 installed on core block foundation; all meters still function
Colorado DOT						
Retaining walls in Glenwood Canyon	Pneumatic	Sinco	25	24	24	A total of 16 additional cells was lost during the first 2-year period for both cell types; remaining cells show a trend but underregister in most cases
	Pneumatic	Gloetzl	19	17	17	
Precast segmented IECO retaining wall	Pneumatic	Gloetzl	45	43	43	For horizontal and vertical pressure measurement; 4 cells cast in foundation; 1 cell provided no reading and 1 cell overregistered; too early to comment on performance of other cells
Georgia DOT (stabilized embankment)	9-in. vibrating wire pressure cell	Irad	10	9	8	5 placed for vertical pressure and 5 placed for horizontal pressure; medium silt-clay backfill around cells; breakage of wire was possible cause of initial failure
Harvard Square Station, Boston, Mass.	Vibrating wire pressure cell	Irad	6	6	0	Cast in slurry wall concrete to measure pressure at back of wall; could not seat at concrete-soil interface; judged unsuccessful
University of Nottingham						
Field studies of pavements and subgrades	Electrical resistance	University of Nottingham	17	17	14	Cells installed to compare vertical subgrade stresses and pavement performance
Laboratory study of pavement subgrades and bases	Electrical resistance	University of Nottingham	12	12	11	Cells installed to measure transient vertical and horizontal stresses in clay subgrades, granular bases, and asphalt mixes

ALTERNATIVE PROCEDURE FOR EARTH PRESSURE DETERMINATION

The following example is furnished to illustrate an alternative procedure for deriving lateral (horizontal) soil pressure when data from installed pressure cells appear unreliable.

This example involves a fully instrumented mechanically stabilized embankment (MSE) (reinforced soil retaining wall) with wood facing and steel bar-mat soil reinforcement (20). The facing for this temporary wall consisted of 1-1/8-in. plywood panels with 4 in. x 6 in. Douglas fir posts on 2-ft centers (Figure 2). The facing was supported by W5 wire (0.252 in. in diameter) bar-mats with 6 in. x 12 in. grid spacings. Bar-mats were secured to 4 in. x 6 in. posts on 2-ft vertical spacings and were embedded in a silty sand backfill material (Figure 3). The maximum wall height was 24 ft. Instrumentation consisted of Gloetzl cells with pneumatic readout installed at five different levels to measure horizontal pressure (Table 2). Strain gauges were placed in both the horizontal and the vertical wall face direction on the plywood facing elements. Strain gauges were also installed on the steel bar-mats behind the wall face.

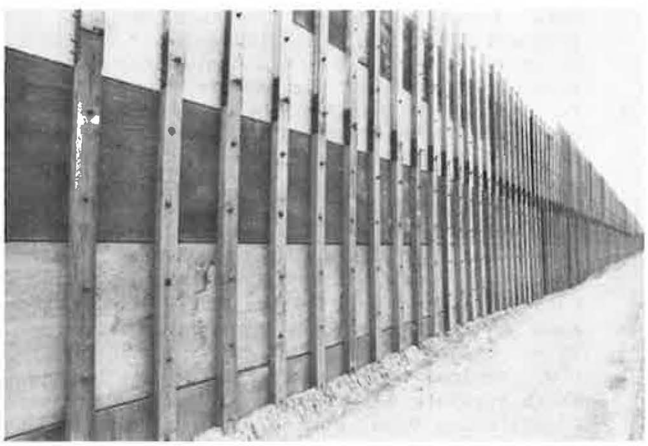

FIGURE 2 Front view of wood faced MSE.

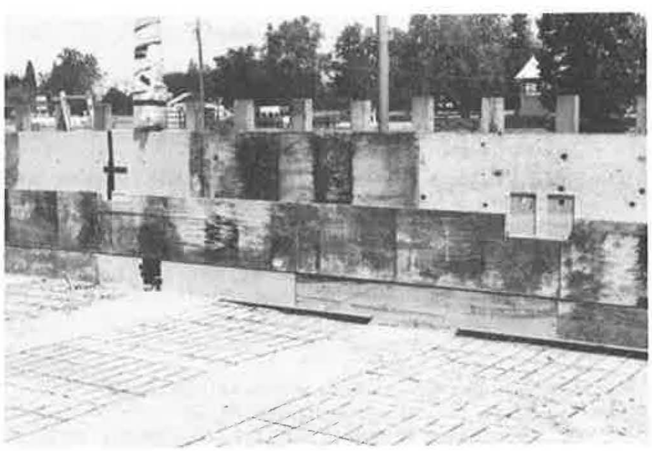

FIGURE 3 Steel bar-mats behind wood faced MSE.

Pressure cell readings during wall construction indicated both under- and overregistration of lateral pressure on the wall face with almost complete pressure relaxation within a month of cell placement. The pressure cells were installed in precut recesses in the wood facing as shown in Figure 4.

FIGURE 4 Field installation of soil pressure cells behind plywood face.

It is assumed that the cell temperature under direct sunlight, before backfilling, rose to above 100°F, thereby creating a temperature differential of about 50°F between cell and soil at the time of soil placement. As suggested in an earlier portion of this paper, the pressure relaxation could be attributed to the volume change of the cells in their soil environment.

Figure 5 shows the results of pressure cell measurements in terms of lateral soil pressure plotted against overburden height in feet. Also shown are calculated lateral soil pressures from bar-mat stresses and strain gauge readings on the wall face and from laboratory vacuum testing of strain gauged plywood facing panels that were tested to model the field condition.

Although the data from pressure cells were quite scattered, data from the other three alternative systems of measurement provided realistic values for lateral pressure.

Figure 6 shows the results of a successful pressure cell installation and monitoring program by Caltrans for lateral (horizontal) wall pressures (21). Pressure measurements for this concrete-faced Reinforced Earth (RE) wall were found to correlate well with Rankine theory. Strain gauges installed on the steel reinforcing strips also provided an alternative means of verifying lateral pressure.

Lateral soil pressure behind the wall face was determined by Carlson soil stress meters carefully installed and flush mounted at the concrete-soil interface (see Caltrans, Soil Stress Meters, Carlson, in Table 2).

SUMMARY

Accurate or reasonable measurements of earth pressure are difficult to achieve unless pressure-

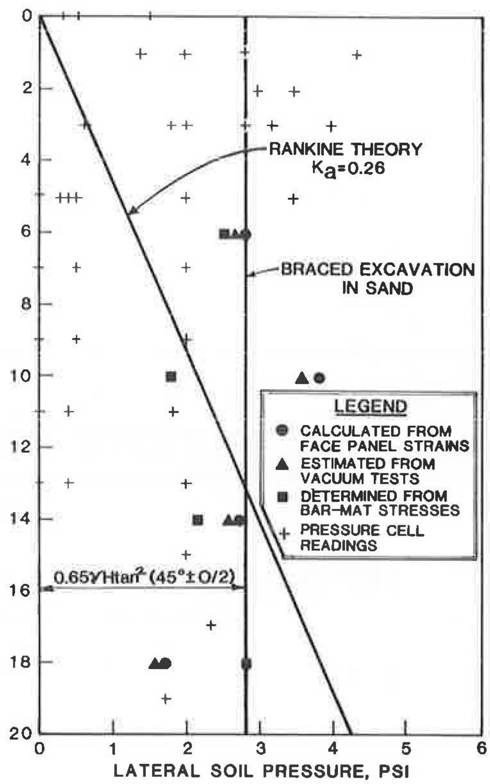

FIGURE 5 Lateral soil pressure test results versus theory (*20*).

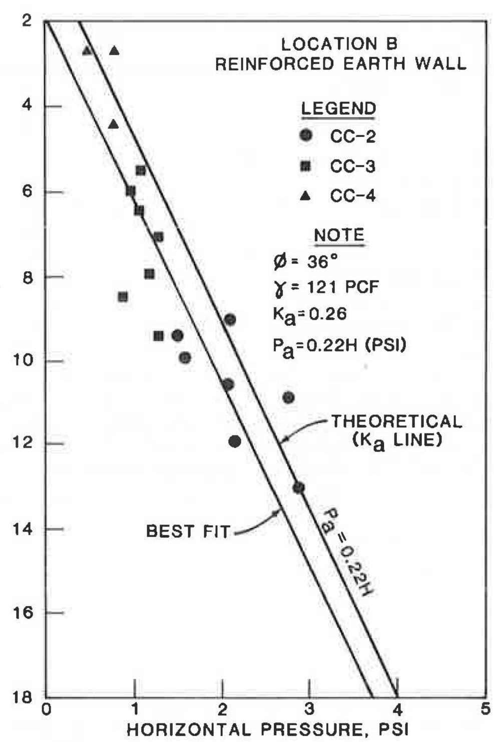

FIGURE 6 Soil pressure on wall face during construction (*21*).

sensing instruments are properly selected and special attention is given to proper installation techniques and calibration.

Instrumentation personnel should be experienced and knowledgeable about potential problems associated with the placement and monitoring of the particular instruments selected for use. All instruments should also be subject to bench or calibration testing, or both to ensure performance and compliance with specifications.

When feasible, alternative procedures should be used for backup to estimate soil stress conditions.

ACKNOWLEDGMENTS

The authors would like to express their thanks to the individuals and organizations who provided information on experience with pressure cell installations. Special thanks are extended to John Campbell and Gus Johnson of the California Department of Water Resources and to John Gilmore and Marrion Wells of Colorado DOT who were most helpful.

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Reliability of Strain Gauges and Load Cells for Geotechnical Engineering Applications

LEE W. ABRAMSON and GORDON E. GREEN

ABSTRACT

Strain gauges and load cells are often used to measure strain and load in concrete or steel components of geotechnical structures. Reliability problems are frequently cited for these instruments. The purposes of this paper are (a) to discuss the factors that affect the reliability of strain gauges and load cells used for geotechnical engineering applications, (b) to suggest the instrument types that have historically performed most reliably and are therefore preferred by some engineers, (c) to indicate which instruments should prove to be the best choice for future projects, and (d) to establish realistic survivability rates to be used in planning instrumentation programs. These objectives were accomplished by searching published literature and by surveying the opinions of more than 40 knowledgeable geotechnical engineers. The results of the survey as well as published information have been compiled and are included in the paper. The primary considerations for instrument reliability are instrument characteristics and selection. Other considerations include proper planning of the instrumentation program, ability of the instrument to perform the intended function, and field installation and handling techniques. Vibrating wire strain gauges are generally preferred for reliable strain measurement. Electrical resistance strain gauged load cells are generally preferred for reliable load measurement. Planning of an instrumentation program should anticipate the probability that one-quarter to one-half of the instruments may not survive installation or the period of monitoring.

Strain gauges and load cells are used to measure the strain and load in concrete or steel components where there are complex soil-structure interaction problems. Strains and total stresses in soils that are measured with extensometers and earth pressure cells are excluded from this paper. Geotechnical instrumentation may be used during construction of or in-service life of a structure to ensure safety, cost economy, and design and construction method adequacy and to monitor long-term performance. A wide variety of strain gauges and load cells is available to the geotechnical engineer for these purposes.

Geotechnical instrumentation programs commonly are plagued with problems relating to instrument-type selection, real performance characteristics, and installation procedures. The purposes of this paper are (a) to discuss the factors that affect the reliability of strain gauges and load cells used for geotechnical engineering applications, (b) to suggest the instrument types that have historically performed most reliably and are therefore preferred by some engineers, (c) to indicate which instruments should prove to be the best choice for future projects, and (d) to establish realistic survivability rates to be used in planning instrumentation programs.

An interpretation of the opinions of more than 40 knowledgeable geotechnical engineers is presented. These engineers participated in a survey conducted by the authors. The purpose of this survey was to determine what the geotechnical practitioner knows and believes about load and strain measurement in structural members. Given the wide variety of instrument types and manufacturers available, it is rarely possible for one or two individuals to maintain hands-on experience with all currently available instruments. Elaborate instrumentation schemes have failed due to lack of attention to critical details by the user, manufacturer, or designer. The distilled experience of many engineers who have suffered the consequences of instrument or program failures should help reduce the incidence of future failures.

TYPES OF STRAIN GAUGES AND LOAD CELLS AVAILABLE

Several types of strain gauges and load cells exist and are available from numerous manufacturers. Cording et al. (1), Dunnicliff and Sellers (2), and Dunnicliff (3) list and describe in great detail many available instruments as well as their use. Tables 1 and 2 give lists of strain gauges and load cells commonly used in geotechnical engineering and example sources. No judgment of the adequacy of these sources is intended in any way. Addresses of common instrument suppliers are given in Dunnicliff and Sellers (2) and a recently published buyers' guide (4).

Strain Gauges

Mechanical strain gauges are used to measure small changes of length between two reference points attached to a structural member typically 2 to 18 in. apart. The gauge consists of a rigid metal bar with a dial gauge and mechanical linkage. During reading, two posts on the gauge are held in temporary contact with the reference points.

Electrical resistance strain gauges are either of the unbonded or the bonded type. In the unbonded resistance wire gauge, the wire is looped around posts fixed to either end of the gauge. The most common, the Carlson gauge, incorporates two wires, which change in length in opposite senses when the gauge is strained and so permit temperature compensation as an added feature. In the more common bonded resistance strain gauge, a wire or foil is bonded to a plastic film that is attached by the user to the structural member being monitored. Great

TABLE 1 Types and Sources of Strain Gauges[a]

Category	Type of Instrument	Example Sources
Surface Mounted Strain Gages	Mechanical	Huggenberger Soiltest Prewitt
	Bonded Electrical Resistance	Micromeasurements Bean
	Weldable Electrical Resistance	Ailtech Hitech Micromeasurements
	Vibrating Wire	Irad Gage Slope Indicator Geokon
Embedment Strain Gages	Bonded Electrical Resistance	BLH Brewer Micromeasurements
	Unbonded Electrical Resistance	Carlson Huggenberger Ailtech
	Vibrating Wire	Irad Gage Geokon Telemac

[a]Modified from Dunnicliff and Sellers *(2)*.

TABLE 2 Types and Sources of Load Cells[a]

Type of Instrument	Example Sources
Telltale	Geokon Local machine shop
Mechanical	Interfels Proceq Roctest
Hydraulic	Gloetzl Soil Instruments Petur
Vibrating Wire	Gage Technique Irad Gage Telemac
Electrical Resistance Strain Gage	Brewer Slope Indicator Irad Gage

[a]Modified from Dunnicliff and Sellers *(2)*.

skill and experience are needed for field installation of bonded gauges. Success in using these instruments depends on many painstaking steps including surface preparation, bonding, waterproofing, and physical protection, which is usually difficult to attain under field conditions. If designed, installed, and used correctly, these gauging systems can be extremely stable and reliable. A third, and less commonly used, resistance strain gauge, the Ailtech gauge, incorporates a friction-bonded wire resistance element inside a small steel tube welded to steel shim stock and is relatively easy to install. Installation problems are also alleviated in the weldable resistance bonded strain gauge in which an electrical resistance strain gauge is bonded to steel shim stock in the factory and integral electrical leads are attached and sealed. The user has only to grind the surface of the metal structural member and weld the gauge in place with a portable battery-powered capacitive discharge spot welder. This is a relatively simple, easily learned technique.

A vibrating wire strain gauge consists typically of a 2- to 6-in. length of tensioned steel wire free to vibrate at its natural frequency when plucked. For surface mounting, the ends of the wire are anchored to posts clamped or welded to the steel structural member. Changes in frequency and hence in wire tension occur when the gauge is strained. The wire is plucked by an electromagnet either intermittently or continuously. The vibrating wire then induces an AC voltage of the same frequency in the plucking coil; this voltage is remotely recorded. Frequency change is related to strain. Potential problems include thermal mismatch between the gauge and the structure, wire creep, slippage at the wire clamps, and wire corrosion. It appears possible to avoid these problems by proper design and material selection. The potential for zero drift remains, however, and prudent users should install dummy gauges from the same batch mounted on free-standing structural elements that experience no stress but are subjected to the same environment, for the same periods of time, as the active gauges.

Strain gauges may be embedded in concrete or shotcrete directly instead of being mounted on a steel member. In this case it is important to recognize that having measured strain it may be desirable to convert it to stress for more meaningful interpretation of forces in structural members. This is easy and reliable for steel because the modulus of steel is constant and creep effects are negligible. For concrete or shotcrete, however, creep and other extraneous strains may be extremely large and under these conditions interpretation of data even from a 100 percent reliable strain gauge can be exceedingly difficult. If these problems are recognized, strain gauges may be used as follows: Resistance wire or foil gauges may be bonded to a reinforcing bar or a short section of a reinforcing bar. Unbonded resistance strain gauges such as the Carlson gauge may be embedded directly in concrete. An Ailtech gauge or a vibrating wire strain gauge mounted between two end flanges may be similarly embedded. In all cases a dummy gauge should be embedded in the same shotcrete or concrete not subject to stress but kept in the same environment as the active gauges.

Load Cells

Load cells measure force, or load, in a structural member. Telltale load cells consist of an unstressed sleeved steel rod usually installed alongside a tieback tendon or rock bolt. The lower end of the rod is attached to the tendon and movement is measured between the upper end and the bearing plate at the anchorage head. Load is determined from in situ calibration during stressing or is based on the tendon dimensions and properties. Direct access is usually needed for readings and telltales can be difficult to install alongside tendons.

Mechanical load cells are infrequently used and few are available in the United States. They may incorporate elastic spring washers or a torsion lever system. Reading is by a dial gauge.

Hydraulic load cells consist of two thin circular steel plates welded together around the edge to form a fluid-filled chamber. The fluid pressure is measured directly by a bourdon gauge or remotely by a pneumatic, hydraulic, or electrical transducer. The hydraulic load cell must be installed between two rigid steel bearing plates and can be provided with a center hole for tieback applications. Hydraulic load cells have successfully withstood driving when mounted on concrete piles and are continuing to function after 2 years according to Green et al. (5).

Both electrical resistance and vibrating wire strain gauge load cells are essentially based on the same concept of operation. A steel or aluminum alloy cylinder is loaded in compression on the ends of the cylinder. Bonded resistance strain gauges are mounted in various bridge configurations typically on the outside of the cylinder at midheight. Alternatively, vibrating wire strain gauges can be similarly mounted or mounted in longitudinal drill holes in the cylinder wall. Solid center load cells for compressive load measurement usually incorporate a spherical seating to avoid edge loading effects. Hollow center load cells, commonly used for tiebacks, are sensitive to eccentric loads and also should be mounted between spherical seat washers or other devices to minimize end effects. Arguments sometimes arise about the true load on a tieback as a result of overlooking eccentricity or inadequate mounting provisions.

COMMON USES

Strain gauges and load cells are commonly used for instrumenting the following types of structures:

- Excavation bracing--struts, soldier piles, and rakers;
- Tiebacks--bar, strand, and wire;
- Retaining walls--cantilevered concrete, steel sheet piles, and Reinforced Earth embankments;
- Tunnels and shafts--steel liner plate, steel sets, cast-in-place concrete, segmented precast concrete, and shotcrete;
- Dams--concrete arch and concrete gravity;
- Locks--concrete;
- Cofferdams--steel sheet piles;
- Pavements--concrete and asphalt;
- Shallow foundations--spread footings and rafts;
- Deep foundations--concrete, steel, or wood piles and caissons;
- Pipelines--water, gas, oil, and sewer;
- Offshore structures--towers and drill rigs; and
- Nuclear waste isolation--in situ tests.

Instrumentation serves a variety of functions depending on the needs of individual projects. Instrumentation can be used during research and development programs or to provide input to the design or remedial treatment of a structure. Construction safety, costs, procedures, and schedules can be controlled with instrumentation as the structure is built. After the structure is built, instrumentation can be used to monitor long-term performance.

FACTORS THAT AFFECT INSTRUMENT RELIABILITY

An attempt was made to identify the most important of the factors that affect the reliability of strain gauges and load cells used for geotechnical applications. Eight factors were listed in the survey distributed to the participating geotechnical engineers. The results of this survey are given in Table 3.

"Instrument characteristics and selection" was chosen as the most important of the factors that affect reliability. Wilder et al. (6) identified several pertinent controlling factors in instrument design, selection, manufacture, and installation. A more complete list to aid the user in selecting the most suitable instrument for a specific application follows:

- Instrument principle;
- Accuracy;
- Sensitivity;
- Measurement range;
- Reliability;
- Environmental factors--temperature limits, humidity, and corrosive agents;
- Operating life;
- Quality control;
- Manufacturer's reputation; and
- Cost.

All of the necessary characteristics of an instrument application should be assessed and then used in selecting an instrument that will perform to those specifications. If no such instrument exists, the specifications must be relaxed or the application modified to reflect the available instruments. In some cases an instrument can be custom designed for a particular job.

Survey respondents considered matching of available instruments to program needs the next most important of the factors that affect reliability. "Proper planning of the instrumentation program," "the ability of the instrument to perform for the intended use and environment," and "field installation and handling" tied for second place.

The other factors listed on the survey had less importance than did the ones just named. Nevertheless they affect instrument or data reliability and include the following (ranked according to survey results):

- Instrument mounting,
- Monitoring procedures and personnel,
- Calibration requirements, and
- Data interpretation.

Some respondents ranked all of the factors as having equal importance. Perhaps some factors are redundant and others too simplistic. This may have led to a problem in ranking. A more extensive survey could have been used to evaluate strain gauges and load cells separately.

Data interpretation is, appropriately, the tail end of the process as the survey results indicate. Interpretation is an engineering or scientific function. Different approaches may yield different interpretations of the same reliable data. But no one, except by accident, will be able to interpret truly unreliable data, except to ignore it.

Other factors, which affect reliability more than do the ones listed, were offered by respondents. These include

- Expertise and motivation of the person or persons doing the work,
- Manufacturer's instrument quality,
- Contract provisions for protection of the instruments, and
- Understanding of the thermomechanical characteristics and limitations of the instrumented structure.

COMMONLY USED TYPES OF STRAIN GAUGES

Although there are many varieties of strain gauges, they fall into three general categories: mechanical, electrical resistance, and vibrating wire as discussed earlier. The second two types can be surface mounted or embedded and read remotely. The decision to use one type of gauge instead of another should be considered on a case-by-case basis. No one type is best for every application and instrumentation team. Manufacturers are often extremely helpful in determining which gauge to use for a particular application. However, it is advisable to consult more than one manufacturer, to remove any bias that may occur, as well as colleagues for up-to-date user in-

TABLE 3 Strain Gauge and Load Cell Reliability Survey[a]

I.	The most important factors which affect the reliability of strain gages and load cells used for geotechnical engineering applications are: (Please rank numerically with 1 being the most important)	**Rank**
	A. Proper planning of instrumentation program	2 (tie)
	B. Instrument characteristics and selection	1
	C. Instrument mounting	3
	D. Field installation and handling	2 (tie)
	E. Calibration requirements	5
	F. Monitoring procedures and personnel	4
	G. Data interpretation	6
	H. Ability to perform for intended use and environment	2 (tie)
	The following factor affects instrument reliability more than any listed above:	(See text)
II.	Respondent has used strain gages and/or load cells for the following types of structures (Check all applicable)	**Instrumentation used by respondents**
	A. Retaining Walls	73 %
	B. Tunnels	73 %
	C. Pavements	33 %
	D. Shallow Foundations	37 %
	E. Deep Foundations	59 %
	F. Dams	53 %
	G. Pipelines	25 %
	H. Excavation Bracing	49 %
	I. Tiebacks	54 %
	J. Other	33 %
III.	Based on actual experience, the following types of instruments are the most reliable: (Indicate which are most reliable and state for what application)	**Preferred by respondents**
	A. Surface Mounted Strain Gages	
	Mechanical	11 %
	Bonded Electrical Resistance	21 %
	Weldable Electrical Resistance	17 %
	Vibrating Wire	51 %
	B. Embedment Strain Gages	
	Bonded Electrical Resistance	33 %
	Unbonded Electrical Resistance	9 %
	Vibrating Wire	58 %
	C. Load Cells	
	Telltale (e.g. for tiebacks or piles)	4 %
	Mechanical (e.g. a proving ring)	4 %
	Hydraulic	20 %
	Vibrating Wire Strain Gage	29 %
	Electrical Resistance Strain Gage (Most common type)	43 %
IV.	The following survivability rates should be used in the planning of instrumentation programs: (Circle one for each category)	**Average for all respondents**
	A. Strain Gages.....25%.....50%.....75%.....100%	62 %
	B. Load Cells......25%.....50%.....75%.....100%	74 %

[a]Total number of respondents = 40 (60 percent of mailing).

formation. In the survey, respondents had the opportunity to indicate the type of gauge they thought was most reliable on the basis of actual experience. A general preference for one type of gauge does not mean that other types should not be used for certain applications.

Mechanical surface strain gauges were least preferred by survey respondents as shown in Section III of Table 3. Mechanical gauges are inexpensive, reusable, rugged, and reliable but offer limited resolution. This type of gauge requires direct physical access to place the gauge on the reference points. Remote reading is not possible. Gauge length can be relatively large, which may be a distinct advantage on concrete, and this type of gauge should not be overlooked where access is available.

Approximately one-third (38 to 42 percent) of the respondents preferred electrical resistance strain gauges for strain monitoring installations. Resistance gauges possess many advantages and provide a higher degree of resolution than do mechanical gauges. Long-term reliability is somewhat doubtful due to the tendency for the gauge zero to drift, the frequent intrusion of moisture, and uncertain temperature effects. Because these gauges function on resistance changes, extremely long lead wires cannot be used without special signal-enhancement electronics. Factors that are of importance when using bonded electrical resistance strain gauges include

- Gauge location and mechanical protection;
- Thermomechanical properties of structure to be gauged and relative stiffness;
- Adhesive--materials used, surface preparation (roughness, cleanliness), clamping pressure, curing temperature and humidity, time for mechanical-thermal equilibrium, and calibration techniques;
- Waterproofing method; and
- Lead wire characteristics and mechanical protection--bridge circuitry, grounding, electrical shielding, connection to gauge, and physical properties (resistance to moisture ingress).

Considerable skill is required to successfully install bonded resistance strain gauges in the field and this job is best left to experts. Many of the problems can be avoided by using weldable resistance strain gauges with integral leads. These gauges can be successfully installed by either skilled engineers or technicians with limited training and practice.

Vibrating wire strain gauges were preferred by more than half (51 to 58 percent) of the survey respondents for most applications. Vibrating wire gauges provide a high level of resolution without being readily affected by moisture or lead wire length. They are sometimes reusable (at least in part) when surface mounted. As mentioned earlier, wire fatigue can occur over extremely long periods of time. Manufacturers have taken steps to correct this by heat treating the wire and limiting the wire tension. Zero drift has also been observed with some gauges but can to some extent be compensated for with dummy gauges. Recently new, low-profile, low-inertia, weldable vibrating wire gauges have become available. These gauges can be easily installed with a portable battery-powered capacitive discharge spot welder (7). These gauges are smaller, easier to protect, and will more readily survive driving when mounted, for example, on driven steel piles. Vibrating wire strain gauges have been used successfully in Europe, including the U.S.S.R., for the past 30 or more years. Only in the last 10 years have they been manufactured in the United States. Interestingly, the European gauges tend to be significantly more expensive and more heavily engineered than their U.S. counterparts and are still preferred by some government agencies that require long-term reliability and are able to justify the extra cost of procurement. The following factors, similar to those that relate to electrical resistance gauges, are important when using vibrating wire strain gauges:

- Gauge location and mechanical protection,
- Thermomechanical properties of the gauged structure relative to the gauge,
- Mounting method, and
- Cable location and protection.

COMMONLY USED TYPES OF LOAD CELLS

Telltale and mechanical load cells were not preferred by the survey respondents for reliable load measurement possibly because of their recent introduction (telltales) or limited availability (mechanical load cells). Telltale load cells are not very precise but can be valuable for measurement of load distribution in tieback tendons or piles. Mechanical load cells are commonly used for soils laboratory testing but few field versions are available.

Twenty percent of the respondents preferred hydraulic load cells for reliable measurement of load in structures. Hydraulic load cells are less readily available in the United States but can be reliable and have the advantage of simplicity and low profile. Important factors to be considered when using all load cells are

- Load cell location and protection;
- Accessibility--need for remote monitoring;
- Eccentricities;
- Insufficient reaction, bending in support plates;
- Calibration requirements;
- End effects;
- Weatherproofing;
- Temperature effects on the load cell and its mountings; and
- Lead protection, where used.

Electrical resistance and vibrating wire strain gauged load cells were preferred by 43 and 29 percent of the respondents, respectively. Factors that affect the use of these load cells are similar to those that affect the use of electrical resistance and vibrating wire strain gauges discussed previously. The major difference between the use of strain gauges and load cells is that load cells come from the manufacturer as one complete unit. The intricacies of mounting the strain gauges in the field are therefore avoided with load cells.

SURVIVABILITY RATES

The respondents to the survey were asked to recommend instrument survivability rates for planning instrumentation programs. The results were averaged and survivability rates for strain gauges and load cells were recommended to be 62 and 74 percent, respectively. Survivability rates recommended by respondents ranged between 25 and 100 percent. The specific numerical results are not as important as the need for owners, designers, manufacturers, and field personnel to face the probability that a significant number (one-quarter to one-half) of the instruments will not survive. Proper planning should be done to compensate for these instrument losses and to ensure that the required number survives to provide sufficient reliable data. Short-term survivability of instruments can be assumed to be better than long term. Both depend on the personnel doing the work, the duration of the instrumentation program, and the environment in which the instrument will be placed.

Many owners effectively encourage low survivability rates by using low-bid procurement procedures. Such procedures can result in the cheapest priced and poorest quality instruments being used. Reputable manufacturers can be forced to cut corners in design and manufacture to underbid their competition. Lengthy specifications aimed at circumventing this will often be unsuccessful. In contrast, adequately funded thorough work by competent organizations with appropriate experience can achieve a high success rate under extremely difficult and challenging conditions (8).

Despite these successes there are many outstanding problems yet to be solved; the nuclear waste disposal industry recently recognized this (9). The particular application of instrumentation to nuclear waste isolation makes extraordinary demands for reliability under extreme conditions not typically encountered in civil works.

CONCLUSIONS

"Instrument characteristics and selection" is the primary consideration for reliable use of strain gauges and load cells in geotechnical engineering. Other important considerations are

- Proper planning of the instrumentation program,
- Ability of the instrument to perform its intended function in the field environment, and
- Field installation and handling.

Vibrating wire strain gauges were preferred for strain measurements by the geotechnical engineers who responded to the survey. Electrical resistance strain gauged load cells were preferred to other types for measuring load in a structure.

Bonded electrical resistance strain gauges should only be installed in the field by experts. Field weldable gauges are available that can be more easily installed by geotechnical engineers or technicians.

Recently available, low-profile, weldable vibrating wire strain gauges possess a number of advantages over traditional gauges.

Conversion of strain to stress or load in concrete can be unreliable even when the best available techniques (i.e., controlled laboratory tests and dummy no-load gauges) are used. Direct measurement of load is preferable where possible (e.g., install a steel load cell across the full diameter of a concrete pile).

Good-quality load cells will give unreliable results if improperly installed between inadequate bearing plates.

Better quality, typically higher priced, instruments are often a better choice because they tend to provide more reliable data. In many cases gauges once installed can never be accessed again and the entire program may end in disaster if the gauges fail.

Planning of instrumentation programs using strain gauges or load cells should assume that one-quarter to one-half of the instruments will not survive through the entire program.

Low-bid procurement procedures encourage low-quality instruments, perhaps designed down to a price. Reliable data are less likely to be obtained from these instruments.

ACKNOWLEDGMENTS

The authors are grateful to those persons within Parsons Brinckerhoff Quade & Douglas, Inc., and Shannon & Wilson, Inc., and outside who assisted in the data gathering and preparation of this paper. The engineers who responded to the survey provided guidance and inspiration, and their contribution is acknowledged. In addition, the authors acknowledge John Dunnicliff and Bill Hansmire both of whom have helped the profession gain a better understanding of planning, design, installation, and monitoring of geotechnical instrumentation programs. Thanks also go to Regina Isidori for her patience and help in preparing the manuscript.

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Some Facts About Long-Term Reliability of Vibrating Wire Instruments

J. L. BORDES and P. J. DEBREUILLE

ABSTRACT

In this review of a manufacturer's experience during the last 50 years, the durability and stability of vibrating wire (acoustic) monitoring instruments are discussed and statistics on survival rates are presented. The causes of instrument failure are analyzed; the factors that affect and the mechanisms of zero drift are discussed. The relationship between instrument failure and cable laying, manufacturing process, and cable connections is discussed. Analysis shows differences in zero drift between plain strain gauges and complex sensors. Mention is made of the influence of wire behavior and sensor structure. There is discussion of factors that affect wire and sensor structure aging. Special tests on aging are described. Various tests on long-term observation of a wire or a sensor are summarized. Examples are given of instruments still faithfully operating several dozen years after installation. Those examples involve dams in operation. Although reliability factors considered are limited to survival and time stability, some consideration is given to quality of sensor response and to other aspects, such as resonance, that affect reliability. On the basis of an analysis of data from two types of sensors, it is concluded that vibrating wire instruments offer an extremely high degree of stability, accuracy, and durability.

The views of a manufacturer, whose only merit has been that of being a pioneer in the application of the vibrating wire instrument to civil engineering projects, are presented. The time span covered is thought to justify this review, which presents facts that may help the reader make up his own mind about the long-term reliability of vibrating wire instruments.

This is not an exhaustive review, but simply a set of facts and figures that cover a long period of time in this relatively new industry. The review begins with a short historical summary of the manufacturer's practical background including a definition of what is meant by long-term reliability, which serves as a basis for comparing the examples cited.

It was between 1928 and 1931 that André Coyne in France, independent of other attempts in Germany, conceived a method of measuring the strains in solids by measuring the change in the frequency of a vibrating piano wire stretched on a frame attached to, and deforming with, the monitored structure. This arrangement, among other advantages, lent itself to remote reading of the output signal. By 1932 the first 36 vibrating wire strain gauges had been installed in Bromme Dam in France; this dam was retired from service about 1950. Then Marèges Dam, the world's highest and first double curvature arch dam, was equipped in 1934 with 78 vibrating wire strain gauges, from which readings were taken until 1953 (1).

Between 1934 and 1938 instruments were built into a bridge, the hoop reinforcement around penstocks, and ground anchors for a wharf wall; the same vibrating wire principle was used to manufacture pressure cells for direct burial in soils and for making dynamic recordings of water hammer (1). Despite the Second World War, many other structures were instrumented from 1938 to 1947: tunnels, underground chambers, gates, and valves as well as dams.

In 1947 A. Coyne and J. Bellier, who together had been responsible for this development, founded a company, Telemac, to promote vibrating wire instruments for civil engineering applications. The next major step forward, in 1954, was a twin-coil instrument in which one coil imparted just enough energy to the vibrating wire to prevent decay while the other could be used for continuously recording the output signal.

Telemac defines long-term reliability as comprising (a) the physical durability of the instrument, which enables it to perform reliably for dozens of years after installation and (b) time stability (i.e., there should be no zero drift and any reading at any time can be referred to the same datum). The second point also includes the concept of minimum change in sensitivity or "dead band" with time (2).

A few examples will be given to illustrate how vibrating wire instruments are intrinsically capable of offering the durability and reliability necessary for civil engineering uses. When reading the statistics, it should be borne in mind that an instrument may in practice become unserviceable, from the operator's point of view, because of minor extraneous faults that can be quickly put right so that some wastage (nonfunctioning instrument) is recoverable.

DURABILITY

The number of instruments manufactured by Telemac since it was founded in 1947 will serve as a measure of the experience acquired by the company. To date, Telemac has built 45,000 vibrating wire strain gauges plus approximately 8,000 vibrating wire pore pressure and total pressure cells, aside from other types of instruments such as inclinometers, the performance analysis of which is extremely complex and the number of which is limited. The overall statistical picture will be examined before a few specific cases are described.

General Statistics

The most accurate records have been kept by Electricité de France (3), covering 3,300 strain gauges, 1,400 piezometer cells in dams, and 2,000 strain gauges in nuclear power stations. These instruments represent 13 percent of total Telemac production.

The oldest instruments (dam strain gauges) had been in continuous use for 30 years in 1975 when the records were compiled and have now been in service for 40 years. Of a total of 3,346 strain gauges, 5 percent became unserviceable in the first year, but only 18 percent were unserviceable after 30 years, an annual rate of loss of 4.1×10^{-3} not counting the first-year failures. Statistics on 3,300 strain gauges during the 8-year period 1974-1982 give an annual loss of 0.7×10^{-3}, making a 50-year total of less than 4 percent. Figure 1(a) shows number of instruments plotted against time installed. Figure 1(b) shows annual losses plotted against time.

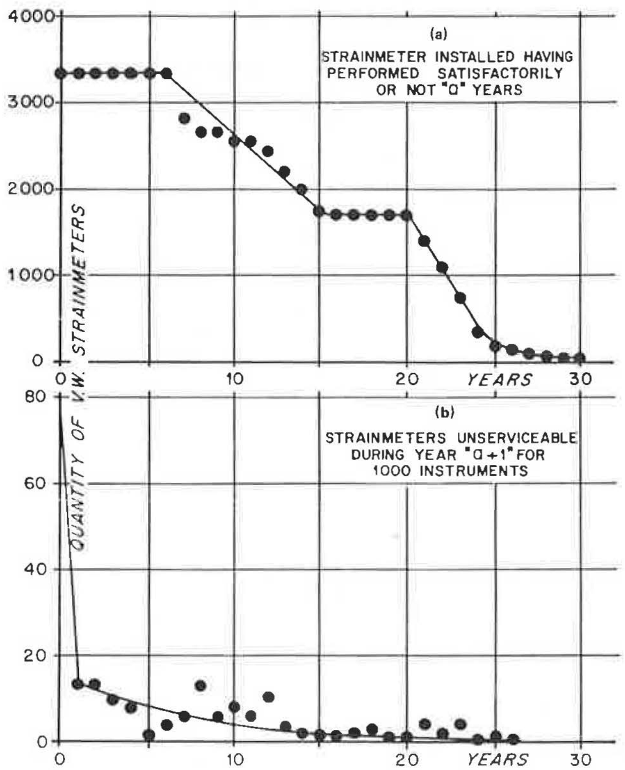

FIGURE 1 Vibrating wire strain gauges in service (from documents of Division Technique Générale d'Electricité de France).

Specific Statistics

Table 1 gives details about 10 of approximately 250 dams fitted with Telemac instruments, ranging from the old Marèges Dam to Grand'Maison, now under construction in France. Other examples are drawn from the American continents, Asia, and Africa. Examples are confined to dams because they represent the oldest single consistent type of engineering structure; nuclear reactor buildings are a relatively recent addition and underground chambers have only been instrumented infrequently in the past. Examining both old and new dams reveals certain definite trends, particularly in connection with the installation of instruments. The dams are chosen from all parts of the world where skills and material resources are different, and these are two factors that may have a significant impact on the success or failure of the instrumentation system. The actual choice of dams is quite arbitrary; others could have served just as well.

An attempt has been made to break down the numbers in the same way Electricité de France did, to distinguish between instrument failures in the first year of installation and those that occur after the dam has been commissioned. Statistics for recent years reveal that only a small percentage (1 to 2 percent) of the total number of instruments may be lost, but this involves questions of workmanship and supervision on site, which are beyond the scope of this paper.

The gross wastage is reasonable (not more than 25 percent not counting the 18 instruments lost at Kariba from extraneous causes). It is important that the unserviceable instruments not be concentrated in critical zones of the structure to such an extent that no information is available, and the numbers must also be corrected to account for the utility of the remainder (e.g., the loss of one strain gauge in a three-axis rosette renders the other two useless). Such considerations are important in instrumentation system design, where a certain degree of redundancy must be built in, and this too is beyond the scope of the present paper.

Nevertheless, the numbers given in Table 1 show that vibrating wire instruments offer excellent durability, which can be further improved by regular maintenance. Maintenance work at dams like Kariba in Zimbabwe has shown that instruments considered unserviceable can be recovered, so the statistics are somewhat overconservative for some of the dams listed. It can be reasonably estimated that, with modern methods and reasonably good site conditions, 80 percent of the instruments can be expected to be operating satisfactorily after 50 years if the notion of utility, mentioned previously, is disregarded. A loss of 20 percent of instruments may be critical in some circumstances.

It is important to note as well that vibrating wire instruments have been used satisfactorily in dynamically loaded structures such as driven piles, railway bridges and tunnel linings, suspension bridges, and offshore platforms. In the last case, continuous measurements were made over periods of several months.

Wastage Factors

By far the most important factor in instrument failure is damage to the cable during installation. Rough handling or difficulties caused by site congestion may cut or puncture the outer casing, enabling water to seep in. Water will impair operation even if it does not reach as far as the instrument itself. Sensors may suffer damage several years after installation in some situations. Such damage to buried cable is irreparable.

Another cause of failure is careless workmanship in manufacturing; failure may occur if metal particles from the machining processes are allowed to fall into the gap between the coil and the wire (such particles sometimes dislodge spontaneously). There have been rare cases of the piano wire corroding quite quickly; this is attributable to the instrument not being watertight.

The last major factor in instrument wastage includes all the defects that can occur in the cable and connections from the junction boxes to the read-

TABLE 1 Dams Fitted with Telemac Instruments

Country	Dam	When Built	No. of Vibrating Wire Instruments	Failures Immediately After Installation		Subsequent Failures		Total Failures	
				No.	Percentage	No.	Percentage	No.	Percentage
France	Marèges	1932-1935	76[a,b]						[b]
	Castillon	1946-1949	104[a]					17	16
Tunisia	Oued Mellegue	1952-1954	95[a]					24	25[c]
Zimbabwe	Kariba, main dam abutment	1955-1959	261[a]	13	5	24	9	37	14
			62[a]	4	6	30	49	34	55[d]
Canada	Daniel Johnson (formerly Manicouagan V)	1959-1968	57[e]					0	0
			1,066[a]					53	5
Pakistan	Tarbela	1968-1976	110[e]	2	2	20	18	22	20
Algeria	Sidi Mohamed Ben Aouda	1974-1977	114[e]	18	16	6	5	24	18[f]
Venezuela	Guri	1979-1982	335[e]					7	2
Argentina	Alicura	1981-1984	132[e]	3	2			3	2
France	Grand'Maison	1979-1985	120[e]	4	3			4	3[g]

[a] Strain gauges.
[b] An attempt is to be made to take readings from the junction boxes themselves on instruments read until 1953 (when cable was cut below junction boxes on downstream face to further maintenance work).
[c] Junction boxes are suspected partial cause of misfunctioning of many of the unreadable sensors.
[d] Eighteen unserviceable due to corrosion of terminals.
[e] CL1 pore pressure cells.
[f] Nine unserviceable at installation (8%), 9 out of use due to cable cut in a slide.
[g] Three unserviceable due to cable cut after differential settlement.

out equipment. Such damage may be reparable and the decision to refurbish junction or switching boxes, or both, and change lengths of multicore cable can sometimes enable significant numbers of instruments to be reinstated. But if the terminals between the boxes and the individual cable from the instruments start to leak, there is no way of recovering that instrument. The vulnerable points in the system should therefore be regularly inspected and any maintenance work needed should be undertaken promptly.

Another cause of instrument failure is lightning, which can destroy both coils rendering the instrument totally irrecoverable. Protection is a simple matter of burying the cables more than 1 m deep. If this is impossible, each cable must be run through an earthed metal pipe conduit. All circuits should be short-circuited whenever there is any risk of direct strikes. Special devices like spark gaps and diode bridges can be incorporated in the switching systems, but there is no guarantee of their effectiveness.

The presence of two coils in modern vibrating wire instruments means that, if one is destroyed by lightning or is defective when it leaves the factory, the sound one can still be used to "pluck" the wire and set it vibrating as well as to pick up the wire's frequency, although in this case the vibrations will decay and the reading procedure is slightly different. Statistics show that some percentage of instruments can be recovered in this way.

At one recent job in an area of high lighting frequency, an installation error enabled lightning to damage nine instruments out of a total of 14; eight of these instruments had only one coil damaged and could still be used for meaningful readings.

Exceptional Damage

In some rare cases, there has been an unusually high mortality rate in the population of strain gauges installed in a structure--as high as 25 to 30 percent over a period of 10 years or even as high as 50 percent over an even shorter period. Such failures have always been found to be due to special circumstances.

The worst case was due to poor workmanship and severe damage caused in laying the cable. A more unusual case involved strain gauges encased in brass and built into a nuclear reactor building. Tests have shown recently that radiation affected the seals and the piano wire itself so that the instruments could not function.

A third case still remains obscure, although it might be that some unnoticed defect in the manufacture of a batch of instruments led to the eventual failure of all of them.

This is a good opportunity to stress that manufacturers should not be overly hasty in introducing new manufacturing processes or new materials. Once installed in a large structure, the instruments are inaccessible and repairs impossible, and there is the added problem of determining what actually went wrong. The difficulty of predicting the long-term effects of design or manufacturing modifications means that manufacturers must think hard before trying to improve their techniques, although strict quality control does help in improving the technology.

TIME STABILITY

Unlike durability, which is relatively easy to quantify, time stability or lack of zero drift is difficult to evaluate objectively. Zero drift is not an all-or-nothing effect, so it is necessary to assess the accuracy of the instrument's response and include the time factor in judging instrument performance. This is a complex task that involves the sorting of a large amount of data. Unfortunately data reaches the manufacturer only in a haphazard fashion, and purchasers and users can only refer to their own limited experience, so it is extremely difficult to attempt any synthesis of information from several clients and engineering consultants.

Despite the sparseness of data, it is still possible to cite a few examples to analyze the intrinsic quality of vibrating wire instruments and the factors that affect zero drift.

Zero Drift Factors and Mechanisms

There are two groups of factors that affect time-stability performance and that depend on the crucial distinction between a plain strain gauge cast into the structural concrete and an instrument in which the piano wire actually measures the deformations of a membrane or other deformable part, as does the pore pressure cell. The second category of instrument also includes plain strain gauges attached to the surface of the structure.

In the case of plain strain gauges, the only factors involved are changes in the piano wire itself and in its end attachments, whereas in the second category, the situation is further complicated by the sometimes complex changes in the membrane or in the method of affixing the strain gauge to the structural surface.

The grade and treatment of the piano wire and the low loads to which it is subjected (9 to 13 percent yield strength) make it inconceivable that there can be any irrecoverable time-dependent strain of any significant magnitude (4). However, there is always the possibility that the end fixings act as stress raisers, eventually causing irrecoverable strain in the wire.

When the stout tube protecting the piano wire was filled with atmospheric oxygen, there might have been a possibility of microcorrosion gradually reducing the diameter of the wire, especially if it had been damaged by an excessive clamping force at the end attachment points. Proof of this argument may be that a reduction in wire diameter causes the resonant frequency of the wire to increase. Several recorded cases of zero drift show an increasing frequency.

Zero drift is difficult to detect because it may be only two or three cycles per second per year or 0.5^{-1} percent of the instrument range per year, small enough to be concealed by the deformations of the instrumented structure. Manufacturers have attempted to overcome the problem to some extent by designing attachment points that do not damage the wire and by filling the instrument casing with a neutral gas like nitrogen or evacuating the casing.

Zero drift caused by the slow deformation of the membrane or other deformable part is more difficult to understand. It is affected by shape and material, and assembly and treatment methods.

Whatever the cause of zero drift, experience shows that it is desirable for the instruments to undergo a period of aging after manufacture, to provide time for strain hardening of the components and attachment points and for the relaxation of internal stresses that are inevitably set up in the manufacturing process. The aging process can be accelerated by high temperature or a shaking table, although the two methods are not equally effective. Temperature accelerates the aging process. Figure 2 shows the reactions of various types of instruments.

Figure 2(a) shows curves of the response frequency of two plain strain gauges (Telemac C110) versus log time. One has undergone a thermal cycle, the other has not; in the latter case, the response stabilizes 3 months after manufacture; the other one stabilizes after 3 days with only one thermal cycle.

Variation in response in a range of less than 1 Hz depends on temperature changes (0.2 Hz per degree Celsius) and atmospheric pressure (1 Hz per 100 millibars).

Figure 2(b) shows the response frequency curve for a Telemac CL1 pore pressure cell from 0 to 1.6 bar under accelerated thermal aging. Note that the wire responds to load-induced changes of the body; stability is reached only after a much larger number of cycles than before. This bears out the statement that the mechanical construction of the gauge is important.

These laboratory tests were done at a temperature of 20°C (in a room accurate to within 1 degree) and in climatic vessels with humidity and temperature monitored.

Whatever time is allocated for aging in the factory, instruments usually spend even more time in transport to and storage on the site. But the extra

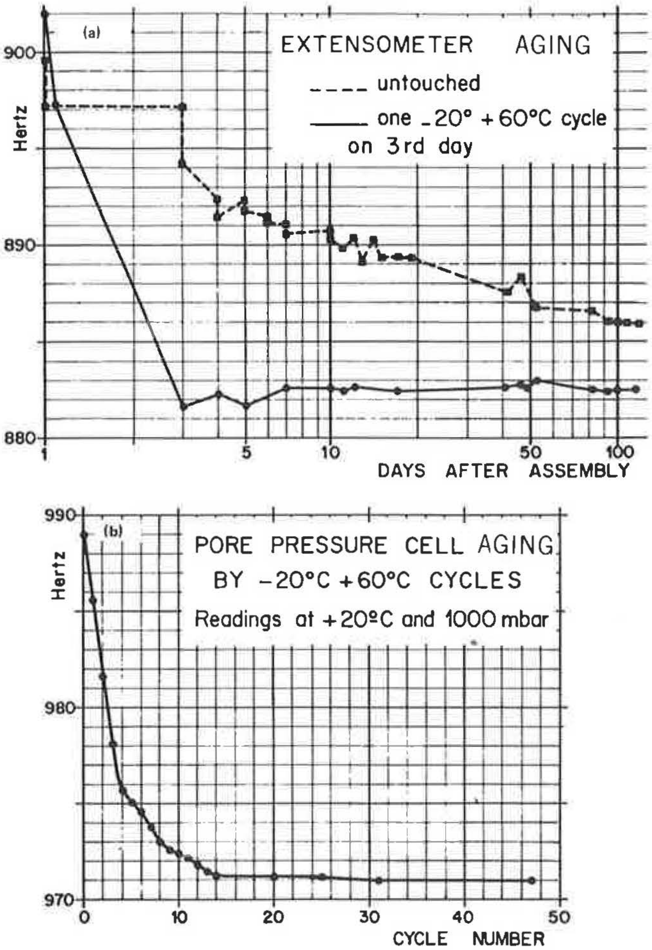

FIGURE 2 Reactions of various types of instruments to aging.

aging is perhaps not totally effective and may adversely affect individual instrument calibration. This will not impair response performance, but the zero shift means that the instrument must be recalibrated before installation. This is the reason for the Telemac procedure for pore pressure cells.

A distinction must be made between two types of zero drift: (a) the short-term process (the length of which depends on the structural design of the instrument) whereby a state of mechanical equilibrium is reached and (b) longer term changes in the wire over as many as several dozen years, which can be controlled by preventing microcorrosion or the other causes of failure already mentioned.

The reliability of readings from vibrating wire instruments is not affected by zero drift alone. There may be irrecoverable strains in the membrane and frame or corrosion of the piano wire leading to a change in sensitivity and in the K factor. Although a minor possibility, this kind of damage has been observed on some pore pressure cells subjected to knocks or uncontrolled forces as they were driven into the ground, causing the membrane to warp. But sensitivity changes represented only a few percent even in the worst cases.

Examples

It must be stressed that one of the most extensive users of vibrating wire instruments has experienced no systematic zero drift over several dozen years. Zero drift is quite exceptional; no statistical figures can be given. In Guri, out of 335 pore pressure cells, two zero drift tendency cases were detected. Such an example is cited to illustrate the general behavior of the sensors. The following examples have no statistical significance; they are merely illustrative. The foregoing section is intended to demonstrate how limited the issue is and how it can be overcome.

That a vibrating wire strain gauge can operate at all after 40 years of service is outstanding, but it is more important that its reliability is coupled with high sensitivity, making it an invaluable and highly effective tool in analyzing structural behavior in civil engineering. Therefore a few case histories of medium- and long-term performance of vibrating wire instruments will be described.

Example 1

Readings were taken over a period of 24 months in 1952 and 1953 from a pore pressure cell on which a constant weight applied a force of approximately 2 kg/cm^2 (5) with corrections for temperature and barometric pressure. The response of the cell remained constant to within $\pm 3 \times 10^{-3}$ of the range.

Example 2

Three pairs of strain gauges, 160 mm, 200 mm, and 260 mm in length, cast in concrete cubes measuring 500 mm on a side were read regularly from 1965 to 1968 (Archives Telemac). The cubes were kept outdoors with no protection from the weather. Concrete surface and internal temperatures were recorded to correct the strain gauge readings. During this 4-year period, there was a shortening of from 120×10^{-6} to 160×10^{-6}, strain shrinkage included, representing 4 to 5 percent of the instrument range. There is a definite scale effect: the recorded shrinkage reading increased with instrument length. It is also true that shrinkage is usually more than 100×10^{-6} strain over 4 years. The shrinkage rate dropped to 20×10^{-6} per year in the fourth year. These simple tests demonstrate the robustness of the instruments and give an approximate idea of their time stability.

Example 3

An unloaded load cell was recorded from 1974 to 1978, so that it responded only to temperature changes (Archives Telemac). The output frequency at 20°C varied by only one cycle per second over 5 years (the actual readings were 824 to 825 Hz), representing a frequency error of 10^{-3} and a strain error of 3×10^{-6} (Figure 3).

Example 4

Extensometers were compared in connection with the construction of the Washington Subway Project (6), including Telemac SC-type surface-mounted strain gauges. Laboratory drift in SC 11 instruments was as much as 40×10^{-6} strain in 50 days with a clear tendency to stabilize after 35 days. No significant zero drift was detected with the SC 8 instrument. Temperature tests between 0°C and 55°C revealed a clear temperature effect: at 55°C, drift in the SC 11 was 80×10^{-6} with no tendency to stabilize after approximately 15 days, but this effect was only approximately 15×10^{-6} for the SC 8, stabilizing after 15 days.

Example 5

Records were kept over a period of 5 years (1975 to 1980) from 20 C110 strain gauges and 30 platinum resistance wire thermometers in the single-span isostatic Condren Bridge in France, as part of a research project of the Laboratoire Central des Ponts et Chaussées (7). Two of the strain gauges, although buried within the concrete, were not subject to structural loads and measured only the concrete shrinkage.

Analysis of the readings, corrected for temperature changes and shrinkage, revealed no detectable zero drift. Strains over the whole period were estimated to within 5×10^{-6} or 3×10^{-3} of the range, similar to the values already mentioned in the first and third examples.

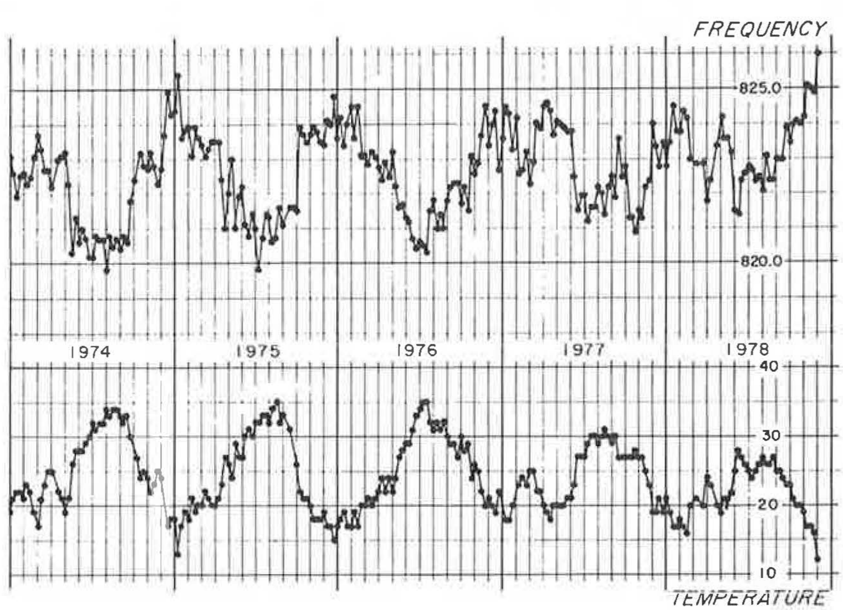

FIGURE 3 Long-term observation of a load cell.

Example 6

During tests conducted in 1981 and 1982 in a pilot underground liquefied natural gas cavity (8), surface-mounted strain gauges were subjected to a temperature of -196°C for several months with no deleterious effects. Earlier tests had shown that their response between 0°C and -196°C was not affected or, at the most, was affected by only 10^{-2} of the range.

Example 7

The examples may be considered rather limited and the reader may wonder why they were not pursued longer. It must be remembered that any laboratory or other setup will be more subject to error, the longer it is maintained. Long-term tests necessarily involve different observers and the results, whether good or bad, are therefore always open to controversy.

It is instructive to discuss here the response curves from four strain gauges cast into the structural concrete of Castillon Dam more than 37 years ago (Figure 4) and from one pore pressure cell placed in the core of Serre Ponçon Dam 25 years ago (Figure 5). The method of analyzing the instrument readings, used by Electricité de France, is described elsewhere (9).

Basically, the readings are corrected to identical average load and temperature conditions. During the last 10 years the responses have been constant and have remained within the same average as during the previous 35 and 25 years. The reduced scatter in the points is due to the introduction of a new type of electronic frequency counter in 1974 (Figure 4), and it must be noted that in 1960-1961 and 1963-1966 the strain gauges were out of order because a cable was out of use (Figure 4).

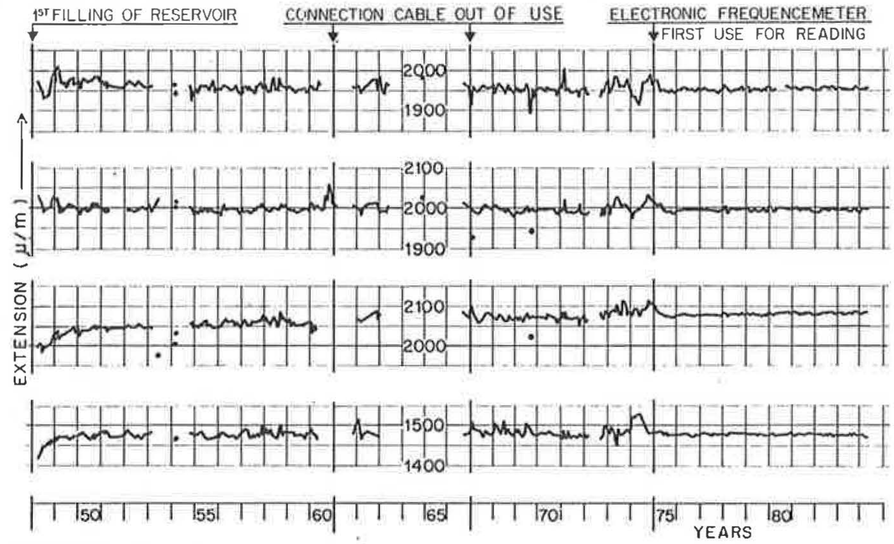

FIGURE 4 Example of elementary deformation measured in Castillon Dam by four vibrating wire extensometers (from documents of Division Technique Générale d'Electricité de France).

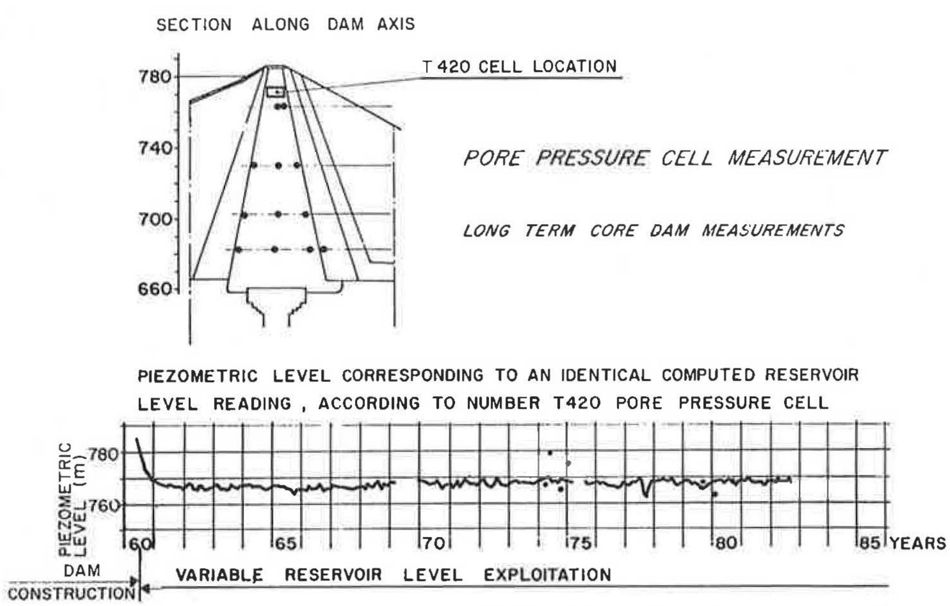

FIGURE 5 Response curve from pore pressure cell in Serre Ponçon Dam (from documents of Division Technique Générale d'Electricité de France).

It is unfortunately impossible to analyze all the other instruments in place throughout the world. But the curves shown in this figure are not exceptional. They are typical of vibrating wire instruments in general.

OTHER ASPECTS OF RELIABILITY

At the beginning of this paper, it was clearly stated that a comprehensive review of all vibrating wire reliability factors was not intended. Reliability is taken in its most restricted sense of durability and zero drift. These are indeed the most important aspects because the instruments are usually inaccessible, once installed. But, even in the gauge itself, there are other factors, and it is important to note that, if reliability is defined as the probability of a measuring system working properly, other important factors are precision, resolution, and reproducibility, which to some degree depend on zero drift and hysteresis.

A detailed discussion would be too long, but it is appropriate to briefly discuss deviations from straight-line responses of the square of the vibrating wire frequency and the porous "pressure inlets" of pore pressure cells.

A study of instrument output signals reveals that deviations from the straight-line response are caused chiefly by resonance of part of the instrument over a usually quite limited frequency band. This is more common when the measured strain is transmitted through the instrument body. Resonance leads to energy loss from the wire and alters the signal, whence the deviation. But this energy loss also weakens the signal so that it may be inaudible at a distance of only 200 or 300 m or even much less, compared with the usual transmission distance of 1000 m. This "blackout" may occur in only an extremely small bandwidth.

The manufacture of reliable instruments requires thorough knowledge of their mechanical properties. Experience shows that resonance can be eliminated by the factory quality control system so that the straight-line error is only 1/1000th of the full range.

As mentioned previously, reliability also requires first-class peripherals, the components of which (except the buried cable) are accessible for repair, but these parts are beyond the scope of this paper.

The pressure inlet tip or porous stone of pore pressure cells is made of a fine porous material to enable the water pressure to gain access to the membrane inside the cell. For fine grained soils, the pores are very fine. The tip is now thoroughly saturated in the factory with deaerated water before delivery. Special tests show that it remains saturated and pressure equilibrium is reached when the instrument is installed, regardless of whether the surrounding soil is saturated. It is important for the porous tip to have proper hydraulic performance if the pore pressure gauge is to be reliable. The problem has now been just about solved and merits a longer discussion.

CONCLUSION

It must be pointed out again that the present analysis is limited to strain gauges and pore pressure cells. It must be remembered that only these two instrument types can be considered statistically meaningful. They have been in use in large numbers for many years, and their response is simple enough for the findings to be universally acceptable. It is remarkable that these instruments have changed little over the years and so the findings are not invalidated by recent changes in construction.

Although all the instruments discussed in this paper come from the same manufacturer, the conclusions drawn have a much wider scope. They apply to all vibrating wire instruments, provided of course that their construction meets a high standard.

It can reasonably be concluded that, after several decades of use, the durability and stability of vibrating wire strain gauges offer outstanding qualities of reliability for the monitoring of civil engineering structures.

Only proper care and skill in installing and maintaining the instruments enables the full benefit to be drawn from them, but simple precautions will ensure faithful service for several dozen years, and 80 percent of an installation can be expected to still be operating after 50 years. Instrument reliability is remarkable; there are few documented cases of significant zero drift, and proper manufacturing, aging, and installation procedures will keep the number of instruments affected to an absolute minimum. Adequate quality control procedures at each stage of design and manufacture are needed to prevent anomalous operation.

ACKNOWLEDGMENTS

The authors would like to thank G. Douillet, Division Technique Générale, Electricité de France; M. Diruy, Laboratoire Central des Ponts et Chaussées, Paris; A. Pujol, Hydronor; L. Pi Botta and C.A. Anderson, Consulting Engineers Group for Alicura Dam, Argentina; Mr. Abid, Tunisian Ministry of Agriculture; J.F. Capelle, Roctest; M. Popiel, Hydro-Québec, Canada; K. de Fries, Consulting Engineer, Caracas; the Water and Power Development Authority, Pakistan; Coyne & Bellier, Consulting Engineers, Paris; and TAMS, New York, for their kind permission to use information in this paper.

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Instrumentation Reliability at Harvard Square Station

ROBERT P. RAWNSLEY, HENRY A. RUSSELL, and WILLIAM H. HANSMIRE

ABSTRACT

As a result of recent emphasis on mass transit, the Massachusetts Bay Transportation Authority has undertaken a large expansion program for its rapid transit facilities. Part of the expansion program was to modify the existing Harvard Square Station in Cambridge, Massachusetts. Slurry walls (concrete cast in place within slurry-filled trenches) were employed for both excavation and final lateral support. In some instances the wall passed within 7 ft of existing buildings of Harvard University. Concern for those buildings resulted in the use of specialized instruments to measure ground, building, and slurry wall movements. The instrumentation program had sufficient redundancy that key parameters were measured by more than one instrument. In addition, detailed field observations were recorded to accurately relate the instrument measurements to construction activity and geologic conditions. In most cases, the instruments performed as well as or better than anticipated. Some instruments were found to be inappropriate for the actual construction conditions. In the cases of poor instrument performance, sufficient redundancy existed that adequate measurements by other instruments were able to serve the monitoring function. The key to the reliability of the instrumentation program was the people involved. Instrumentation installation was done by experienced professionals. Instrument monitoring was performed by trained people who were on the job for extensive periods of time, were interested in the results, and were responsible for interpreting the measurements.

As a result of the recent emphasis on mass transit, the Massachusetts Bay Transportation Authority (MBTA) of Boston, Massachusetts, has undertaken a large expansion program for its rapid transit facilities. A major portion of this expansion program is the extension of the heavy rail Red Line northwest to the Alewife section of Cambridge, Massachusetts. To accomplish the 3-mile extension, the existing Harvard Square Station had to be modified and enlarged to allow realignment of the tracks to enable them to pass through the existing station and make a turn of approximately 90 degrees to the north. The modification of the station required nearly complete demolition of the existing structure and construction of new entrances and passenger platforms.

Slurry walls were employed for both excavation and final support. In some areas the slurry wall passes within 7 ft of existing buildings of Harvard University. Because of concern about ground and building movement, extensive instrumentation was used.

In addition to monitoring movements, strain gauges and pressure cells were installed in the slurry wall at selected locations to better determine the actual slurry wall behavior. That instrumentation was added as an FHWA/UMTA research effort after construction had started. A site plan and typical section of an instrumented panel are shown in Figures 1 and 2, respectively. A summary of the field monitoring is presented by Hansmire et al. (1).

INSTRUMENTATION PROGRAM

The primary purposes of the instrumentation and monitoring were to provide reliable information, to foresee problems, and to allow corrective measures to be taken to prevent damage to adjacent struc-

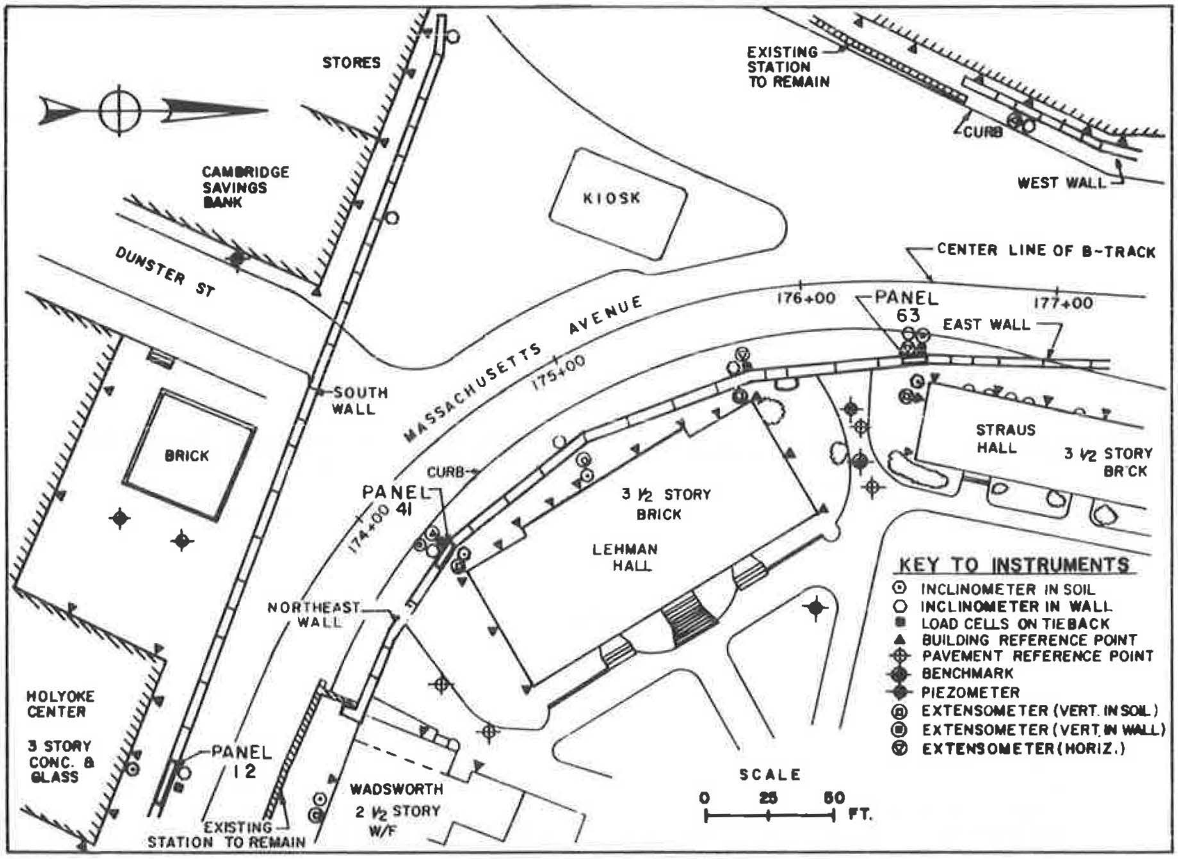

FIGURE 1 Site plan, Harvard Square Station.

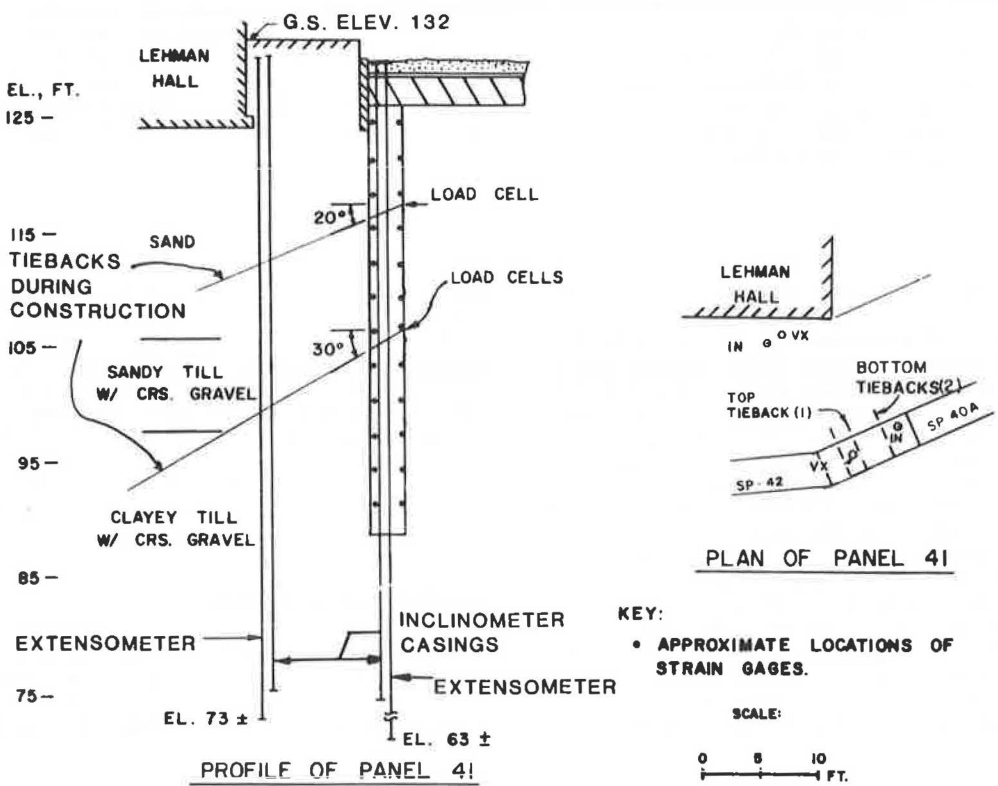

FIGURE 2 Typical instrumented section, plan and profile (Panel 41).

tures. The program also documented the case of only small or no movements and monitored the construction procedures for installation and performance of the tieback system.

Measurements were made of ground and building movements within the zone of influence of the excavation. Redundancy was provided by using more than one instrument to measure key parameters. The monitoring program included detailed field observations to accurately relate the instrument measurements to construction activity and geologic conditions.

A brief summary of the instruments and their specific purposes follows:

- Surface settlement reference points: a reliable method for monitoring settlement of buildings using optical surveying techniques.
- Benchmark: an essential reference for level surveys.
- Inclinometer: determination of lateral movement of slurry wall and soil adjacent to buildings.
- In-place inclinometer: variation of typical inclinometer that provided practically immediate determination of lateral movement at the time of reading.
- Horizontal extensometer: measured lateral movement away from excavation where inclinometers could not be used, such as under buildings; deepest anchor was beyond tiebacks and could verify their adequacy if movement developed.
- Inclinometer with subsurface settlement sensors: inclinometer with additional capability of settlement measurements; used adjacent to buildings.
- Deep settlement sensor: precise determination of settlement of slurry wall or soil at depth between slurry wall and building.
- Tiltmeter: redundant measurement on buildings intended for indirect detection of settlement.
- Load cells: direct measurement of force in tieback to monitor maintenance of required lateral support.
- Piezometers: groundwater level monitoring.
- Strain gauges: direct measurement of strain to compute actual bending moments in slurry wall.
- Earth pressure cells: measurements of soil pressures acting on the slurry wall.

Proper timing of instrument installation and reading was considered important to obtaining the desired results. A summary of the sequence that was followed is given in Table 1.

DESCRIPTION OF INSTRUMENTATION

Surface Settlement

Before any construction was done on the site, a program of preconstruction settlement monitoring was instituted. This program consisted of optical surveys to develop a baseline elevation for future measurements. This program established procedures and provided accurate initial elevations before construction. (See Appendix for requirements for the surveys.)

Six level circuits were performed before construction. A total of 47 reference points on adjacent buildings and 11 pavement markers was surveyed. During construction, additional reference points were added bringing the total number of reference points on buildings to 87. Generally, readings were taken biweekly during construction. Additional surveys were performed as required on selected points in areas of heavy activity or where readings indicated movements had occurred.

TABLE 1 Sequence of Instrument Installation and Construction Schedule

Event No.	Instrument Installation and Initial Measurement	Event No.	Construction Event
1	Preconstruction monitoring program including deep benchmarks, distant survey reference points, and building reference points		
		2	Begin construction contract
3a	Install inclinometer casing in soil near buildings		
3b	Install vertical extensometers in soil next to buildings		
3c	Install tiltmeter reference plates on buildings		
4a	Establish building and surface reference points		
4b	Establish base line for horizontal movements by triangulation and offsets from transit line		
4c	Establish survey reference points on instruments		
5	Install additional piezometers		
6	Complete initial readings on all instruments installed to date		
		7	Begin slurry wall construction
8	Install inclinometer casing in slurry walls		
		9	Install temporary columns and place decking or permanent roof
		10	Begin station excavation and install tiebacks
11	Install tieback load cells at critical sections		
12	Install horizontal extensometers		
13	Continue reading instruments as construction proceeds		

Surface settlement measurements were made on the preconstruction settlement monitoring points and the additional points. These additional points were shallow surface settlement points consisting of cross marks or pins set in pavement, curbs, building steps, and sidewalks. These points were generally located on a 25-ft grid outside the excavation for a distance at least twice the depth of excavation.

Benchmark

Just before construction and as part of the construction contract, three benchmarks were installed to a depth of 6 ft into bedrock. All benchmarks were surveyed, using existing MBTA datum, before construction to establish initial elevations.

Inclinometer

Before all structural excavation, inclinometer casings were installed in the soil between the proposed slurry wall and the existing adjacent buildings. Additional inclinometer casings were installed within the slurry wall of the new station. Inclinometers were used for monitoring lateral deformations of the soil, rock, and slurry walls.

Inclinometer casings placed within the slurry wall were installed through a 6-in.-diameter steel pipe that had been installed before concreting as part of the steel reinforcing cage. The bottom of the steel sleeve was plugged with Styrofoam to prevent the filling of the pipe with concrete. The inclinometer casing within the slurry wall was installed to a minimum depth of 10 ft below the slurry panel to ensure fixity. This was done by drilling through the 6-in.-diameter steel pipe and into the underlying soil or rock by rotary methods.

A 6-in. slip coupling (telescopic) was installed at the base of the slurry wall to prevent buckling of the inclinometer casing, should the concrete panel settle. The annulus between the inclinometer casing and the steel sleeve was filled with low-strength grout.

An inclinometer with high precision was employed to reliably detect movements of less than 0.10 in. The Slope Indicator Co. Digitilt Model 50306 readout and Digitilt Sensor Model 50325 were used. The nominal precision for measurements at the top of the casing was on the order of 1/10,000 of the total length of casing (for a 50-ft casing, this is 0.06 in.). However, experience with the instruments available revealed that a precision on the order of ±0.02 in. at the top of a 50-ft casing could be obtained with good reading techniques.

Field data acquisition was performed manually. To facilitate data processing, a computer was used for calculation. The results of the computer calculations were carefully reviewed to detect and eliminate obvious errors, such as transcription errors. All data were plotted manually.

In-Place Inclinometer

To provide capability for making rapid, timely measurements of horizontal movements adjacent to buildings in an area of heavy construction activity, an in-place inclinometer was used. The in-place uniaxial inclinometer consisted of six separate sensors attached vertically to one another by jointed stainless steel tubing. The sensors were set in the inclinometer casing perpendicular to the direction to be measured. The standard Digitilt Model 50306 readout was used for readings at a junction box located at the top of the casing. Lateral movements were determined at six depths and were easily read and interpreted. The precision of the in-place inclinometer was considered to be significantly greater than that of the regular torpedo device and was ideal for frequent or nearly continuous monitoring. Precision was on the order of ±0.005 in. for a 50-ft casing.

Horizontal Extensometer

The instruments were installed at predetermined locations along the slurry wall as well as through existing busway walls in areas where temporary support was required after removal of the existing roof slab. The instruments were the groutable type (Model E-10G manufactured by Irad Gage, Lebanon, New Hampshire).

The extensometers, which were considered simple and reliable, were located at different elevations to monitor wall and soil movements behind the wall. In particular, horizontal extensometers were used at locations where inclinometers could not be readily installed, such as beneath existing building foundations. The intent was to determine the extent of significant horizontal movements adjacent to the excavation. The extensometers were installed either individually at special sections or in conjunction with other instruments for more comprehensive monitoring of movements.

Eight horizontal extensometers were installed, seven with six anchors and one with four anchors grouted at varying distances from the readout head located on the wall surface. The anchor farthest from the wall was set far enough away from the excavation to be considered to be outside the zone of influence and beyond the longest tieback anchor. This anchor was considered fixed and served as a reference for the determination of absolute horizontal movements. For most installations, an ATLAS-COPCO ODEX system airtrack was used for advancing the borehole; this was the same rig that was used for tieback installation. Extensometers in the slurry wall were presleeved with an 8-in.-diameter steel pipe. In all instances the borehole was drilled at an angle of between 5 and 10 degrees down from horizontal. This slight angle allowed circulation of the drilling fluids for cleanout of the hole and facilitated grouting. When drilled, the borehole was surveyed by a closed hydraulic differential measuring system to locate the elevation of the borehole end.

The extensometer was read with a Mitutoyo micrometer depth gauge with a sensitivity of 0.001 in. The precision of the horizontal extensometer is on the order of ±0.002 in.

Inclinometer with Subsurface Settlement Sensors

Inclinometer casings at selected locations in the soil were equipped with special inductance rings located on a flexible rubber sleeve outside the inclinometer casing to measure settlement. The measurement system consisted of a modified Slope Indicator Co. Sondex Settlement Probe Model 50819. To achieve a higher degree of accuracy the measurement system was modified to use a Mitutoyo Model 192-116 height gauge. All measurements were computed relative to the initial bottom depth reading.

Deep Settlement Sensor

In critical areas, particularly near buildings, settlements were measured at depth with a vertically oriented, multiple-position rod extensometer. The instrument is similar to the horizontal extensometer described previously. All deep settlement sensors were installed by conventional rotary drilling methods.

Each installation contained four or six anchor points at which settlements were determined. The deepest anchor was fixed in rock below the excavation bottom and was considered to be below the zone of possible movements. The accuracy of the surface settlement measured by the deep anchor (which was, in effect, acting as a deep benchmark) could be verified by independent optical settlement surveys on the top of the instrument. This instrument was read with a Mitutoyo depth micrometer to 0.001 in. The precision of the instrument was on the order of ±0.002 in.

Tiltmeter

Ceramic reference plates (tilt plates) were fixed with a polymer epoxy to building walls and columns before construction. Often, however, the surface of the true structural building frame was not, or could not be, exposed. Installation was then often on facades or furred-out walls. The inclination of this plate was determined by a Slope Indicator Co. Model 50344 tiltmeter. The tiltmeter sensor was portable and easy to read. The instrument was sensitive to 10 sec of arc, which corresponds to a tilt of 0.03 in. in 50 ft. Precision of the instrument was on the order of ±20 sec of arc. These instruments were newly available and were believed to have promise as a simple monitoring tool.

Load Cells

Temporary lateral support of the slurry walls was provided by tiebacks. To monitor the tieback loads during and after tensioning, hollow center load cells were used. The cells were generally placed on both tieback levels of specific slurry wall panels at the locations of extensometers and inclinometers. The load cells were manufactured by Soil and Rock Instrumentation, Inc., and were designed for a tieback working load (100 percent design) of 282 kips with an overload factor of 2. The body of the cell was constructed of a high-strength steel, 4.5 in. high and 8 in. in diameter. A 5.5-in. hole in the center of the cell accommodated the tieback strands. Six pairs of temperature-compensating, bonded, resistance gauges and 12 unstressed strain gauges wired in series to form a four-arm Wheatstone bridge were mounted around the steel cell. The readout unit for the load cells was a Vishay Instruments, Inc., Model P-350 portable strain indicator. The sensitivity of the load cell was 0.1 percent of the maximum load or ± 5.6 kips.

Piezometers

In addition to piezometers installed during the design phase exploration program, additional piezometers were installed at selected locations where the soil conditions or adjacent structures required close monitoring of the groundwater. The piezometers were read before construction and routinely during the several years of construction. All piezometers were Casagrande single-tube or double-tube type. Precision of readings was on the order of ± 0.02 ft. The piezometers were read by using an electronic water level device that consisted of a contact probe lowered to the water level in the tube.

Resistance Strain Gauges

Hitec strain gauges were mounted on vertical reinforcing steel of both the front and rear faces of the reinforcing cage for two slurry wall panels. The Hitec HBW-35-125-6 strain gauge is a resistance-type foil gauge preassembled onto a weldable shim for ease of field installation. The shim is approximately 0.25 in. wide by 0.50 in. long. The Hitec gauge is connected to the surface with a four-conductor grounded cable having a polypropolene sheath.

The gauges were mounted on the interior face of the reinforcing steel. Each location was ground flat with a rotary grinder to provide a "slot" for the foil pad and the mounting tabs of the transition zone. When they had been ground smooth, the gauge tabs and ground area of the rebar were cleaned with acetone to remove any oil or other foreign matter.

After cleaning, the gauge was held in position and welded in place using an Ailtic spot-welding machine. The welds were performed following Hitec's recommendations: welding progressed from the center of each side of the tab outward to the end of the tab. This method ensures a good flat bond of the strain gauge shim to the rebar. The transition zone tabs were welded to each side of the rebar to securely fasten the transition zone and cable connection to the steel.

Additional waterproofing of each gauge was accomplished by spray-painting the welded area with a clear lacquer and, when dry, covering the entire area of the gauge with Scotch 2210 elastomeric compound. The cable from the foil to the transition zone was covered with Johns-Manville Duc-Seal to protect the thin wire connecting the foil to the transition zone from any abuse while the steel cage was handled.

Vibrating Wire Strain Gauges

The IRAD embedment gauge, Model EM-5, is a vibrating wire strain gauge approximately 6 in. long with a 1-in. flange at both ends. The flange at each end of the tube contained two 1/16-in.-diameter holes for mounting. The rubber-coated, two-conductor, grounded cable connects at the center of the tube and is sealed into the tube with a plastic potting compound.

The embedment gauges were mounted adjacent to the Hitec gauge locations. The purpose of this mounting was to provide measurements redundant with those made on the reinforcing steel. The gauges were mounted near and parallel to the vertical axis of the steel on the inside face of both sides of the steel cage. The gauges were supported by a 16-gauge wire, and the transition zone was taped to the reinforcing steel.

Earth Pressure Cells

Six earth pressure cells were installed to measure the earth pressure acting directly against the wall. The earth pressure cell chosen was the IRAD vibrating wire earth pressure cell in which the pressure inside an 8-in.-diameter fluid-filled flat jack is measured by a vibrating wire pressure transducer attached to a short tube. The cells were installed on the front and rear of the wall approximately 2 in. outside the reinforcing steel cage. The earth pressure cells were covered with approximately 2 in. of concrete between the cell and the soil. This installation was made despite concern that embedment in concrete would make meaningful measurements impossible. The cells were not jacked into the soil because the glacial till soils contained gravels and cobbles.

INSTRUMENTATION PERFORMANCE

An account of the performance of all the instrumentation used in the Harvard Square Station project is given in this section. Comparison is made between actual performance and predicted performance, and explanations of difficulties encountered and possible solutions to these difficulties are presented.

The overall performance of the instrumentation program at Harvard Square was judged to be good. The intent of the monitoring program was achieved. In most cases, the instruments performed as well as expected, or better. However, some instruments failed to have meaningful use. Detailed accounts of the performance of each instrument type, with suggestions for possible improvements, follow. In a subsequent section reliability is addressed.

Standard Inclinometer

The precision of the standard inclinometer was anticipated to be 1 in 10,000 or 0.06 in. in 50 ft on the basis of manufacturer's information. However, with frequent monitoring and careful reading techniques, it was found that precision of ± 0.02 in. could be achieved. The primary reason such excellent precision was achieved was the frequent monitoring of the instruments for each phase of construction, which yielded a large number of repeated measurements. However, this does not account for the occasional "bad" readings. A high percentage of readings fits the limits of good precision, especially when many readings were taken in a single day.

The only major problem experienced with the inclinometer casings occurred during the winter seasons. Because many of the inclinometers were located within the slurry wall, the groundwater in the casings would freeze as the soil was excavated adjacent to slurry panels. The only feasible solution to the problem was to check the water level in the inclinometer casings regularly and, when needed, bail the casing to a depth equal to or below the top of excavation adjacent to the inclinometer.

In-Place Inclinometer

The precision of the in-place inclinometer was expected to be 1 in 100,000 or 0.006 in. in 50 ft. This precision was attained at Harvard Square.

Problems related to the in-place inclinometer were keeping the connections clean and dry under construction conditions and difficulty in transferring the inclinometer from one location to another. Because of the sensitivity of the in-place inclinometer, great care must be taken to ensure that all connections remain clean and dry. Should the connections become damp, the instrument's precision would be greatly decreased and thus the usefulness of the instrument would be greatly reduced. Further, it is not recommended that the in-place inclinometer be used when frequent relocation of the instrument is anticipated. Because of the nature of the instrument, it is both cumbersome and susceptible to damage when being moved.

Vertical and Horizontal Extensometers

The vertical and horizontal extensometers were expected to perform with a precision of ±0.005 in. It was found that the actual precision of the extensometers was greatly dependent on the care taken in the installation of the instruments. Great difficulty was encountered while installing certain horizontal extensometers because of adverse, tight working conditions. As a result of difficult installation, grout and soil may have entered a disturbed connection in the plastic tubing protecting the stainless steel rods, thus interfering with rod movement and decreasing instrument precision. In the horizontal position, more friction exists between rods and protective casing. In general, it was found that the vertical extensometers had a precision greater than that expected, ±0.002 in., and that the horizontal extensometers varied--some had a precision close to that of the vertical extensometers, others showed a precision worse than that which was expected.

Tiltmeter

Sensitivity of the tiltmeter is approximately one part in 10,000 or 10 sec of arc at 0 degree inclination. The precision of the instrument was expected to be ±20 sec of arc at 0 degree inclination. However, such precision was not achieved in this instance. This failure is attributed in part to temperature influences on the structure and to the fact that most installations were on nonstructural parts of buildings. For the small movements that took place, the tiltmeter was not of value and, therefore, was not a meaningful tool for monitoring on this project. It is recommended that the tiltmeter be used only when general movements or building settlements are expected to exceed 1 in., in which case sufficient tilt would take place to be measurable. Because 1 in. of settlement is usually unacceptable, the tiltmeter as implemented at Harvard Square is not considered suitable for such measurements.

Load Cells

The performance of the load cells was inconsistent. During tensioning of the tiebacks, there was often, but not always, a discrepancy between the jack used for tensioning and the load cell. Occasionally, the jack and the load cell were in agreement to within 5 percent of the applied load. The cause of discrepancy between the jack and load cell is believed to be loading eccentricities. In one instance, the tieback jack and a load cell were loaded together in the laboratory with excellent results. However, when the same load cell and jack were used in the field, the discrepancy was approximately 15 percent. Similar experience is reported by Fellenius for load measurement on piles (2).

When the load cells had been installed, the performance was fair. About one in five of the load cells began to malfunction after a few months of service. The remaining load cells appeared to have measured a change in tieback load with a precision of ±4 kips.

Sondex

It was found that for the small settlements that occurred at Harvard Square, the Sondex system was not sufficiently precise. Precision of measurement was on the order of a = ±0.1 to 0.2 in., which was the order of magnitude of settlement. Reliability was, therefore, not fairly tested.

Settlement Monitoring

The performance of the settlement monitoring instruments was excellent. This is a well-established technique, and other keys to success were the use of good reference points and rigorous procedures. Readings to 0.005 ft, based on a series of routine readings where no construction was taking place, were achieved.

Resistance Strain Gauges

The performance of the resistance strain gauges was extremely poor. Many of the gauges failed shortly after installation, and almost all of the remaining gauges exhibited great variations in readings. It is not recommended that resistance strain gauges be used embedded in concrete.

Vibrating Wire Strain Gauges

The performance of the vibrating wire embedment gauges was excellent. Of the more than 40 gauges installed, only one failed to function. All other gauges recorded consistently for the life of the project. From an engineering standpoint, however, uncertainty about concrete modulus and creep effects made interpretation of stresses quite difficult.

Pressure Cells

The performance of the pressure cells was, not surprisingly, extremely poor. Although the cells recorded consistent data, the magnitudes of earth pressure were inconsistent with what would be expected. It is believed that all cells were mechanically performing well because one cell was eventually removed from the slurry wall and tested in the laboratory with excellent results. The discrepancy in magnitude was most certainly the result of the fatal error of installation in concrete.

CONCLUSIONS

A broad way of answering the question of how reliable was the instrumentation is to ask, "Did the instrumentation do the job intended?" Broadly speaking, the answer was "yes" but in many forms and to different degrees. The instrumentation program was thought to have had thorough planning during design. Despite this, some instruments were found to be unreliable for a variety of reasons ranging from inappropriate application to flawed installation concept (e.g., pressure cells, tilt plates, resistance strain gauges, Sondex for too small movements). Other instruments were extremely reliable.

A discussion of the major conclusions follows.

Settlement monitoring with optical survey leveling was the source of the basic information from which potential for building damage could be evaluated. Reliability was high. Non-instrumentation specialists could understand the monitoring results that were provided with a high degree of precision.

Measurements of lateral movements in inclinometer casings provided confirmation that overall movements were small as indicated by the settlement monitoring. Reliability was high. The inclinometers did an extraordinarily good job of detecting subtle wall movements and consequently had a high degree of reliability to detect larger, possibly damaging wall movements, which, however, did not occur. The in-place inclinometer was difficult to move and should not be considered a readily portable instrument.

Load cells generally indicated that tiebacks carried the design load and their reliability for this function was fair. Where load dropped off, most often there was an inclinometer in the vicinity that showed little or no lateral movement. The reliability of load cells for indirect monitoring for potential building damage was low.

Vibrating wire strain gauges embedded in the slurry walls provided a measure of how much bending was taking place. Reliability was high. The more complex problem of concrete behavior under long-term creep conditions was not evaluated because of limitations in the design of the measuring system, not the instrument per se. On the other hand, the resistance strain gauges embedded in the slurry walls provided no interpretable information. This was believed to be due to instrument malfunction.

Tilt plates failed to give any meaningful information for two reasons. Located mostly on facades of buildings, they were subject to disturbance and non-correctable temperature fluctuations and did not measure true building movement. Further, buildings would have had to move much more than acceptable before movement would have been detected by the tiltmeters. Tiltmeters were unreliable for the intended purpose.

Finally, the most important aspect of instrument reliability, when an instrument worked, was the people involved during implementation. Installation of instruments was done by qualified, experienced professionals who had a reputation to maintain. People reading the instruments were well trained, were on the job for extended periods of time, were interested in the results, and were responsible for interpreting the measurements.

ACKNOWLEDGMENT

The Massachusetts Bay Transportation Authority (MBTA) is the owner of this project. Design of portions of the station and the instrumentation for construction and research was done by Parsons Brinckerhoff Quade & Douglas, Inc. Coparticipant in the research was the Department of Civil Engineering, Massachusetts Institute of Technology. Research was funded by FHWA and UMTA.

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APPENDIX: SPECIFIED REQUIREMENTS FOR SETTLEMENT SURVEYS

Accuracy of Surveys

The accuracy requirements of the preconstruction settlement surveys were considered to be greater than that required for routine elevation surveys. Elevation surveys commonly involve rod readings to 0.01 ft and have a probable error of from ±0.01 ft to ±0.02 ft. For monitoring building settlements, greater accuracy was thought to be required--in the range of ±0.003 ft to ±0.005 ft.

Accuracy involves a number of factors and realistically cannot be obtained without special equipment and surveying procedures. The technical requirements for the preconstruction monitoring are outlined hereafter. They were intended to yield elevations with a probable error of ±0.003 ft (0.04 in.). It was recognized that a true indicator of the real accuracy of the readings would only be known after initial and a series of subsequent readings were made.

Technical Requirements for Instrumentation

Benchmarks

Each benchmark is to be constructed with an outer casing like that shown in Cording et al. (E.J. Cording, A.J. Hendron, Jr., H.H. MacPherson, W.H. Hansmire, R.A. Jones, J.W. Mahar, and T.D. O'Rourke. Methods for Geotechnical Observations and Instrumentation in Tunneling. Report UILU-Eng. 75 2022. University of Illinois, Urbana-Champaign, Dec. 1975.) The exact depth of the benchmark was determined at the time of drilling. In general, benchmarks were seated below the upper layer of highly fractured and weathered bedrock and below structure depth.

Reference Points

Several different types of reference points were required:

• Masonry anchor with machine screw (for use on brick or stone buildings),

• Lag bolt (for use on wood-frame buildings), and

• Masonry "PK" nails (for use on bituminous pavement or joints in concrete sidewalks or streets).

Surveying Equipment

Instrument: Lietz automatic level, Model B1, with a polarizing lens. Rod: Use same rod for all readings. Invar rod should be used and preferably purchased for exclusive use on the project. A Wild Model GPCE 10 was used on this project.

Field Procedures

Field procedures included

• Maximum line-of-sight of 100 ft;

• Balanced backsights and foresights;

• Careful plumbing of rod;

• To the greatest extent possible, use of identical party members for all surveys;

• Identical level circuit to be used for all surveys;

• Use of good quality pavement turning points (i.e., such as PK nails or equivalent); and

• Rod readings to 0.001 ft.

Instrumentation for Load Transfer in Socketed Pier Foundations

R. G. HORVATH

ABSTRACT

An investigation of methods to improve the performance of drilled pier foundations socketed into soft rock was made on full-scale test piers. Instrumentation of the test piers enabled load transfer behavior of the piers to be studied. Flatjack (FREYSSI) load cells were used to measure applied loads and the end-bearing component of pier load support. Vibrating wire concrete embedment strain gauges (Geonor) were used to determine axial and radial stresses at the middepth of the test section. Thus, load distribution along the length of the pier socket could be determined. The description, calibration, and installation of the instruments are briefly summarized. The satisfactory performance and reliability of the instruments are supported by the test data that, in general, were in good agreement with predictions from elastic solutions. Comparison of the results of several piers having different support conditions, and displacement measurements using telltale systems, also support the reliability of these instruments. The versatility of a flatjack load cell to perform three different functions: (a) passive load cell, (b) active (applied) load cell, and (c) void, at the base of one test pier subjected to multiple loading cycles, is also briefly discussed.

Field load testing of six full-scale, instrumented concrete piers socketed into weak rock was carried out to investigate methods of improving the performance of this type of foundation system (1,2). A summary of the load testing program is given in Table 1.

To gain a better understanding of the load transfer mechanisms operating in socketed pier foundations, measurements were made of

• The portions of load supported by shift resistance and end bearing,

• Displacements at the top, middepth, and bottom of the socket,

• Displacements in the rock adjacent to the pier socket, and

• Strains within the concrete pier at middepth.

Presented in the following sections are a brief summary of the test conditions and a description and discussion of each instrument, which includes calibration, installation, performance, and reliability.

TEST CONDITIONS

To minimize construction costs and to simplify inspection, instrumentation installation, and construction, a shale quarry was selected as the test site because the work could be carried out directly

TABLE 1 Summary of Field Load Testing Program

	PIER DESCRIPTION		TEST DESCRIPTION
P1	Smooth Shaft Conventional construction Void at base		Shaft resistance only
P2	Smooth shaft Conventional construction Load Cell at Base		Shaft resistance and End-bearing resistance
P3	Roughened shaft Shaft grooved (AFE) Void at base		Shaft resistance only
P4	Roughened shaft Shaft grooved (AFE) Load cell at base		Shaft resistance and End-bearing resistance
P5	Smooth shaft Conventional construction Preload cell at base		Preload applied to base Shaft resistance and End-bearing resistance
		A	Preload = 0.89 MN
		B	Preload = 1.78 MN
		C	Preload = 4.00 MN
		D	Shaft resistance only
P6	Roughened shaft Shaft grooved (ART) Void at base		Shaft resistance only

Notes: All test piers were auger excavated and had the following dimensions:
Socket diameter, D_s = 710 mm
Socket length, L_s = 1370 mm
Aspect ratio, L_s/D_s = 1.9
All load tests were axial compression tests.

on the exposed rock surface. The site was located in Burlington, Ontario.

Material Properties

The rock exposed at the site consisted of predominantly weak red mudstone (Queenston shale) that became massive with depth. A summary of classification data and engineering properties of the shale is given in Table 2.

The average properties of the concrete were

Uniaxial compressive strength	σ_c = 49 MPa
Elastic modulus	E_s = 35 GPa
Poisson's ratio	ν = 0.27

More detailed information concerning the material properties may be found elsewhere (1,2).

Test Pier Details

The dimensions of the test section of each pier were socket diameter D_s = 710 mm and socket length L_s = 1.37 m (Figure 1). The top of each test section was located approximately 0.6 m below the ground surface. The sockets for each test pier were excavated with a truck-mounted auger (Hughes-Williams LDH 100) that produced shafts with relatively smooth sides. Three test piers were constructed in shafts of this type. Three piers were constructed in shafts with grooves approximately 25 mm deep and 40 mm high cut into the wall to increase the roughness. Three of the test piers were constructed with voids between the bottom of the pier and the bottom of the socket to eliminate end bearing. The remaining three piers were constructed with load cells at their bases.

TABLE 2 Summary of Engineering Properties of Mudstone (Queenston shale)

Test Description			Test Results Range	Ave.
Unit Weight	γ	(kN/m^3)	25.8 to 26.1	25.9
Water Content	w	%	4.1 to 4.8	4.6
Liquid Limit	w_L	%	--	22
Plasticity Index	I_P	%	--	3
Rock Quality Designation	RQD	%	29 to 88	70
Shore Scleroscope Hardness	S_h		14 to 19	16
Point Load Strength	$I_s(50)$	(MPa)	0.56 to 0.91	0.69
Brazilian Tensile Strength	σ_t	(MPa)	0.21 to 1.03	0.64
Uniaxial Compression Test				
Compression strength	σ_c	(MPa)	4.70 to 11.10	6.75
Secant elastic modulus	E_s	(MPa)	400 to 1180	695
Poisson's ratio	ν		0.19 to 0.35	0.30
Triaxial Compression Test (σ_3 = 0.7 to 3.5 MPa)				
Cohesion	c	(MPa)	--	1.2
Friction	ϕ	(deg)	--	43
Secant elastic modulus	E_s	(MPa)	500 to 1600	1000
Poisson's ratio	ν		--	0.22
Direct Shear Test				
Peak: (σ_n = 0.3 to 0.6 MPa)	c	(MPa)	--	0.3
	ϕ	(deg)	--	54
Residual: (σ_n = 0.3 to 2.8 MPa)	c	(MPa)	--	0
	ϕ	(deg)	--	29
Goodman jack				
Elastic Modulus	E_g	(MPa)	740 to 1420	1085

Reactor Frame

The main components of the load-reaction system were the reaction beam, anchor piers, and the anchor connections (Figure 2). The system was designed for a maximum safe test load of 8.7 MN.

Instrumentation

Suitable monitoring equipment was selected to provide the basic data necessary to study the load transfer behavior of socketed piers. Profiles of the

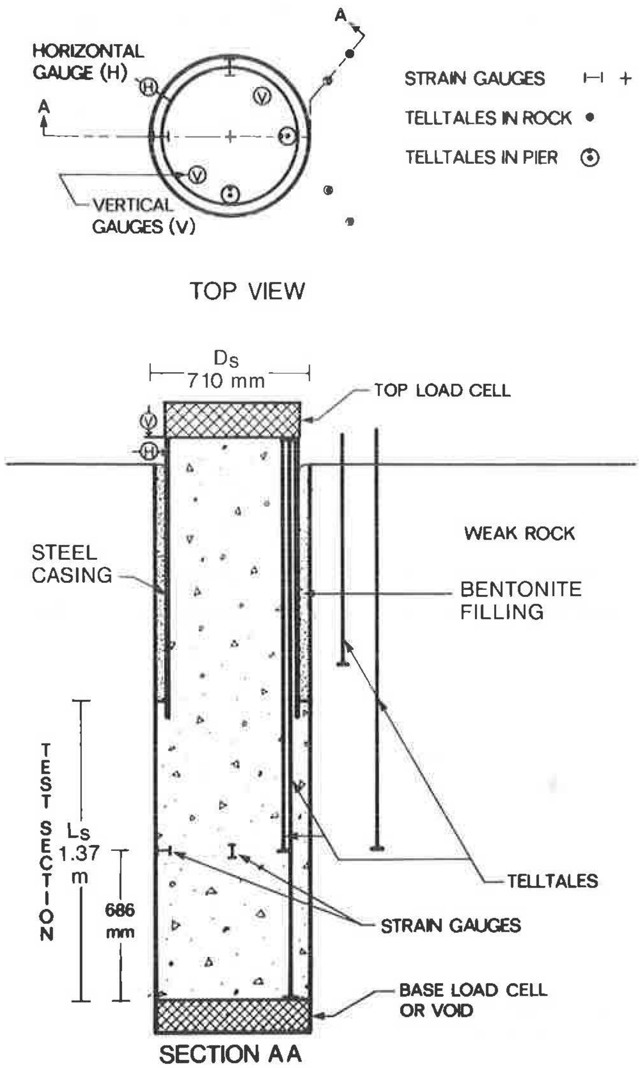

FIGURE 1 Typical test pier and instrumentation.

wall of each socket were made using a simple profilometer. The profilometer was basically a pantograph instrument, consisting of a feeler arm that followed the surface of the rock and a tracing arm that recorded the profile on paper.

Measurements of vertical displacements were made at two locations on top of each pier with dial indicator gauges fastened to wooden reference beams supported outside the influence of the test (approximately 2.5 m from the pier). Within each pier and in the surrounding rock, vertical displacements were measured using telltale rods and dial indicator gauges (Figure 1).

The test loads were applied and measured by means of oil-filled FREYSSI flatjacks. This type of flatjack was also used to measure end-bearing loads. Vibrating wire strain gauges were installed within the test piers for the purpose of estimating axial and radial stresses in the concrete during loading.

Construction and Instrumentation Installation

Each test pier was constructed individually to ensure that the concrete was placed on the same day that the socket shaft was drilled. The test piers were constructed between April 3 and April 18, 1980, and tested between May 17 and June 18, 1980. The sequence of construction and instrument installation was generally the same for each test pier. The steps were

1. The shaft was drilled with the auger to the required depth.
2. The shaft was visually logged, photographed, and tested with a Schmidt Hammer. Four profile traces, at 90-degree spacing, were made with the profilometer at the top and bottom of the test section.
3. Grooving of the socket wall (P3 and P4) was carried out at this point. Step 2 was repeated except for the Schmidt Hammer testing.
4. The bottom of the drilled socket was cleaned as required. Test piers P2, P4, and P5 were thoroughly cleaned by hand to provide a good clean surface for the bottom load cells.
5. A thin layer of grout was placed on the bottom of the socket (P2, P4, and P5 only) to provide a smooth contact surface for the load cells.
6. The load cell (P2, P4, and P5), or the void-forming device (P1, P3, and P6), was placed in position at the base of the socket.
7. Concrete was placed up to the middle of the test section and vibrated.
8. The concrete embedment strain gauges were installed (except P6).
9. The upper casing and telltale assemblies were placed in position.
10. Concrete was placed up to the top of the test section and vibrated.
11. After the concrete in the test section had set (next day) the casing above the test section was filled with concrete and vibrated.

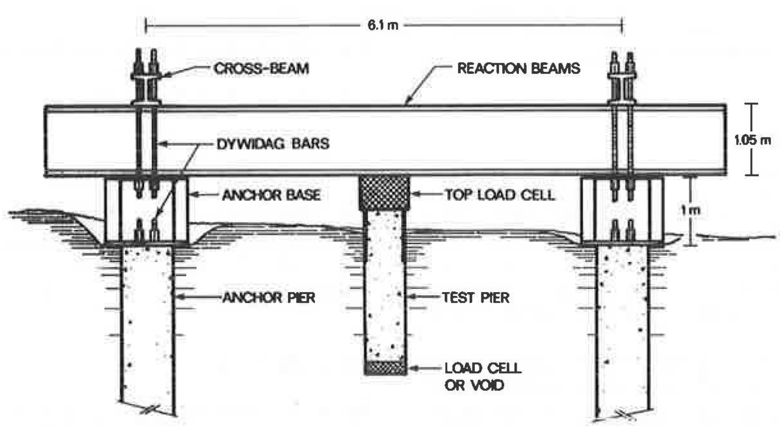

FIGURE 2 Load reaction frame system.

12. The annulus between the casing and the shaft, above the test section, was filled with bentonite to seal off any surface water.

13. Just before testing, the top of the pier was dry packed with grout to provide a smooth, level surface for installing the load transfer plates for the jacking system.

Load Application Procedure

A method of maintaining a constant rate of loading, similar to that used by Bozozuk et al. (3), was used for the tests. Load increments of 22 kN were applied at 15-min intervals. Each load was maintained for 13 min, and 2 min were allowed for adding the next load increment. Vertical displacement gauges at the top of the pier were read at 0-, 3-, 6-, 9-, and 13-min intervals for each load increment. Reading of all instruments, including horizontal displacement, rock displacement, strain gauges, and survey level, were taken at the 13-min mark.

Two test piers, P2 and P4, representing typical pier loading conditions (both shaft resistance and end-bearing resistance) were loaded in increments to 4.45 MN. This load was maintained for approximately 36 hr to observe possible changes of load-transfer behavior with the passage of time. On completion of the maintained-load portion of the test, incremental loading was resumed.

LOAD MEASUREMENTS

The loads applied to the top of the test piers and, in some tests, the loads transferred to the base of the socket in end bearing were both measured with load cells. The basic unit for each load cell was a FREYSSI flatjack pressure capsule that measures load hydraulically. All the load cells were calibrated before they were used in the field. The calibrations were obtained by duplicating in-service conditions as closely as possible.

Bourdon-type pressure gauges were used to measure the hydraulic pressure in the flatjacks. Two sets of gauges (0 to 6.9 MPa and 0 to 34.5 MPa) were used. The low-capacity pressure gauges were used to improve the accuracy at the lower stress levels. The gauges selected were accurate within ±1.5 percent.

Top Load Cell

The top load cell is unusual in that flatjacks were used for both applying and measuring the test loads. This load cell consisted of three to six 920-mm-diameter flatjacks positioned in series (Figure 3). The rated capacity of the flatjacks at a working hydraulic pressure of 14 MPa was 8.68 MN. Each flatjack was capable of expanding to a maximum opening of 25 mm.

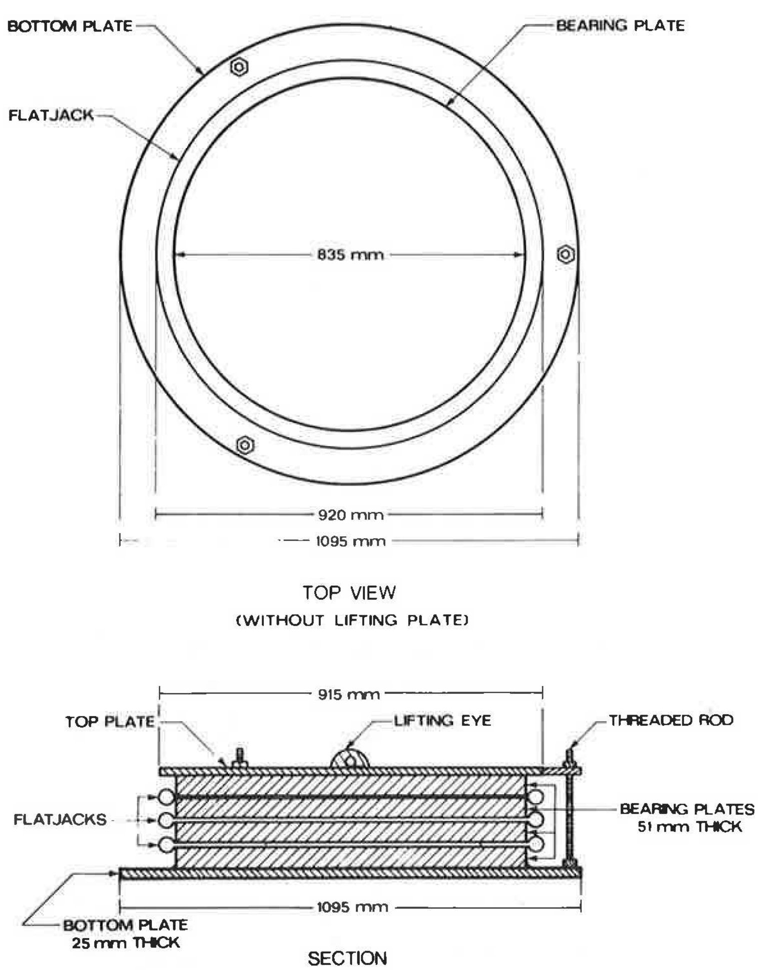

FIGURE 3 Top load cell.

Initially two of the flatjacks (partially expanded) were used as passive load cells. The remaining jacks were used as active jacks; that is, they were expanded by pumping hydraulic fluid into the jack thus applying the desired load to the top of the test pier. However, due to problems related to the lateral stability of the flatjack system, it was necessary to modify the procedure. Subsequently, all jacks in the system were initially deflated. A single flatjack was then inflated to near its maximum aperture (25 mm), closed off using the appropriate valves, and maintained as a passive load cell. If additional vertical movement was necessary, another flatjack was activated and the procedure was repeated.

The advantages of using flatjacks for applying the test load include

- At high loads, there is no problem with ram friction that can occur with piston-type hydraulic jacks;
- As long as the top surface of the pier is perpendicular to the shaft, the applied load will be parallel to the pier axis; and
- It is relatively easy to ensure that the applied load is concentric.

The top load cell performed well throughout the testing program. However, two incidents of equipment failure occurred that must be mentioned because they concern safety. It is important to note that both mishaps could have been avoided.

In the first case, a section of steel pressure tubing connected to one of the flatjacks pulled out of its fitting at a load of about 8 MN. Consequently a high-velocity stream of hydraulic fluid was expelled from the flatjack and covered a car about 10 m away. After this accident, all hydraulic fittings were changed to threaded-type connections. A partial shield (sheetmetal) was also installed around the top load cell to protect personnel.

In the second case, one flatjack ruptured at a load of about 7.5 MN. The high-velocity stream of oil expelled from the flatjack bent the protective shield and traveled about 10 m. The rupture occurred because the test pier had been accidentally constructed approximately 150 mm off the centerline of the load reaction frame. Consequently, the reaction frame and load cell were subjected to severe lateral and twisting distortion during loading.

Base Load Cells

The base load cells were made up of two or three 600-mm-diameter flatjacks positioned in series (Figures 4 and 5). The rated load capacity of these flatjacks was 3.46 MN at a working pressure of 14 MPa. The maximum displacement capacity was 25 mm for each flatjack. Two flatjacks were expanded approxi-

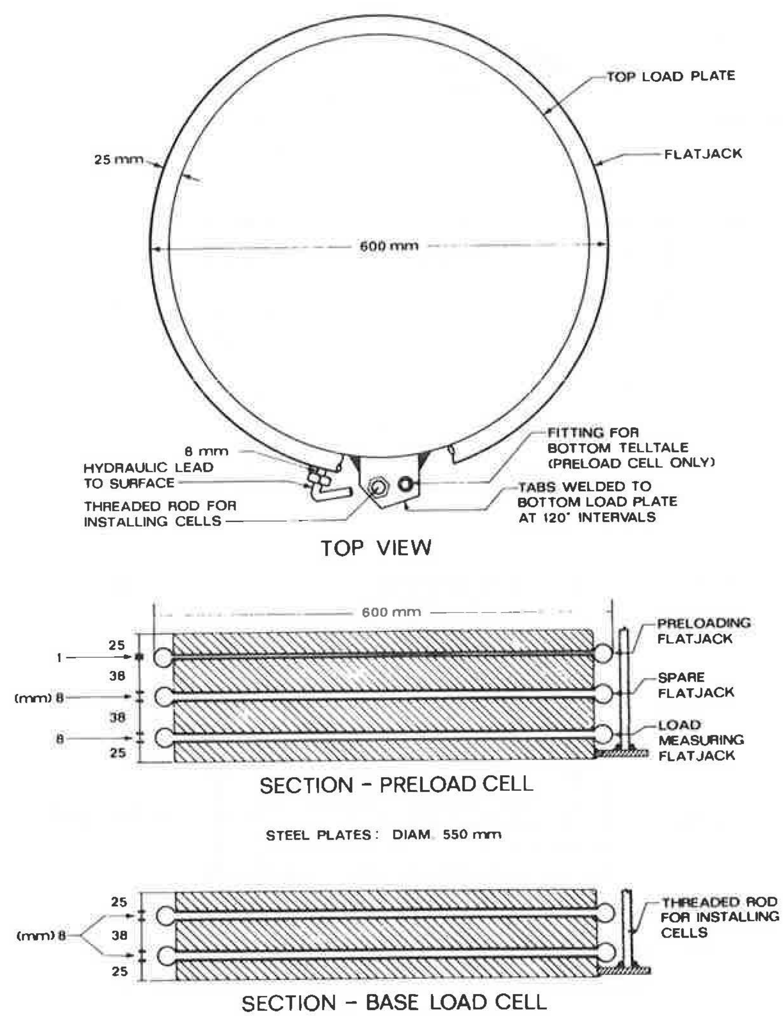

FIGURE 4 Base load cells.

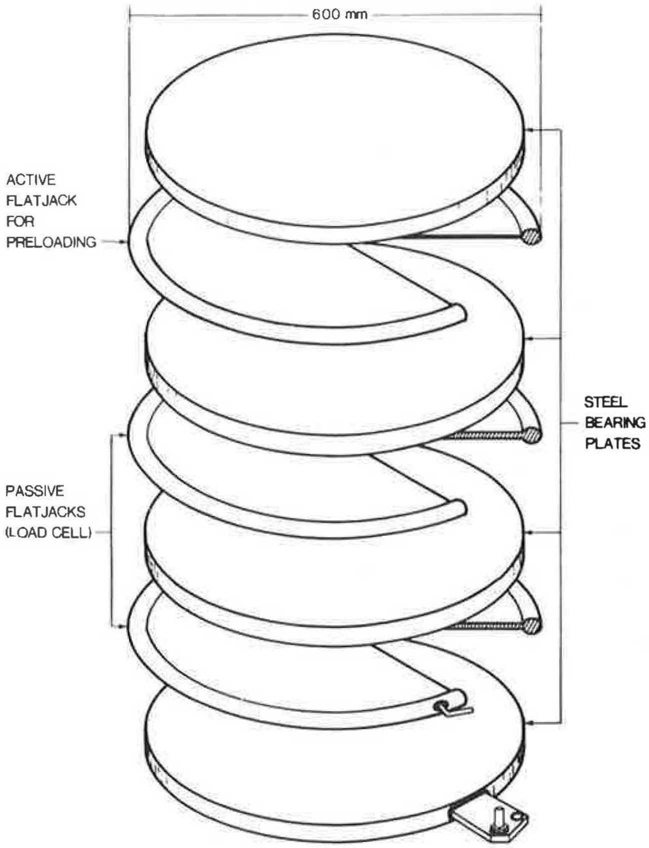

FIGURE 5 Base load cell—exploded view.

mately 6 mm before being placed at the base of the excavation. These cells were connected to pressure gauges in a closed hydraulic system and were used to monitor the load transferred to the bottom of the test piers. Only one flatjack was necessary for this purpose. However, a second one was included as insurance in case there was a malfunction of the first one.

A multiple-loading test procedure was developed to observe the effects of preloading the base of a drill pier in weak rock to improve the load-displacement performance (1). To implement the multiple-loading method, a suitable preload cell was required at the bottom of the test pier. This preload cell was capable of performing three different functions: (a) measuring the load transferred to the end bearing at the pier base (passive load cell), (b) applying a load at the pier base (active load cell), and (c) eliminating end-bearing load (void). The preload cell used for the cycled loading test on this project (1) consisted of three oil-filled FREYSSI flatjacks positioned in series (Figures 4 and 5). All three flatjacks were partly expanded to about 6 mm and calibrated before being placed in the test pier shaft.

A brief explanation of the versatility of the preload cell may be provided by a description of how the cell could function during different loading cycles. During loading cycle A (combined shaft resistance and end-bearing resistance), all three flatjacks could be used as passive load cells. Each flatjack would be connected to a pressure gauge (or transducer) in a closed hydraulic system to monitor the load transferred to end bearing at the bottom of the pier. For loading cycle B (end-bearing resistance only), one flatjack, an active load cell, would be connected to a hydraulic pump. This flatjack could then be expanded by pumping oil into it so that a load would be applied to the pier base. The other two flatjacks would continue to function as passive load cells. Loading cycle C (shaft resistance only) would be carried out with the valves for all three flatjacks opened. Thus, there would be no resistance to compression of the flatjacks back to their original shape. End-bearing resistance would therefore be essentially eliminated. Details of a recommended method for field testing drilled piers using a multiple-loading method are presented elsewhere (4).

Including a spare flatjack in the load cells proved to be a worthwhile precaution because one of the flatjacks in the preload cell began to leak during the initial portion of a test. The faulty flatjack was isolated from the system by opening the valve to the atmosphere, while pumping oil into the other two flatjacks until the faulty flatjack was completely compressed. The load test was then restarted using the two remaining flatjacks in the base load cell.

Calibration

The flatjacks for the top load, base load, and preload cells were calibrated using the 5.3-MN Baldwin Testing Machine at the University of Toronto. The calibration of the top load cell (8.25-MN capacity) was also verified on the 17.8-MN capacity testing machine at the Department of Mines and Resources, Elliot Lake Laboratory for Mining Research, Canadian Centre for Mineral and Energy Technology.

Representative calibration curves for the various load cells are shown in Figures 6-8. The gauge pressure-versus-load calibration curves for flatjack 6 of the top load cell and the top flatjack of the base load cell (cell 2) are shown on Figures 6 and 7, respectively. These curves were determined using linear regression analyses. The curve for flatjack 6 of the top load cell (Figure 6) is based on 190 data points (10 loading and unloading cycles) and had a coefficient of correlation, $r^2 = 0.9997$. The curve for the top flatjack of base load cell 2 (Figure 7) is based on 144 data points (8 loading and unloading cycles) having $r^2 = 1.0000$.

The load-versus-displacement calibration curve for base load cell 2 is shown in Figure 8. This curve was determined using the continuous plotting device on the testing machine. Eight cycles of loading and unloading were used and the curve of best fit (straight line) for each cycle was almost identical. The maximum deviation of any data "point" from the average curve was about 6 percent.

Reliability

The flatjack load cells performed well and, on the basis of the calibration testing and load testing results, the loads measured during the field testing were presumed to be accurate to about ±2 percent.

An indirect evaluation of the reliability of the base load cell measurements may be made by comparing the load-displacement curves for two test piers, P1 and P2, that were both constructed using conventional auger techniques. P1 had a void at its base (shaft resistance only) and P2 had a load cell at its base (combined shaft and end-bearing resistance). A comparison of the shaft resistance-versus-displacement curves for the two piers indicates almost identical behavior in terms of shaft resistance (Figure 9). Shaft resistance for P1 was measured

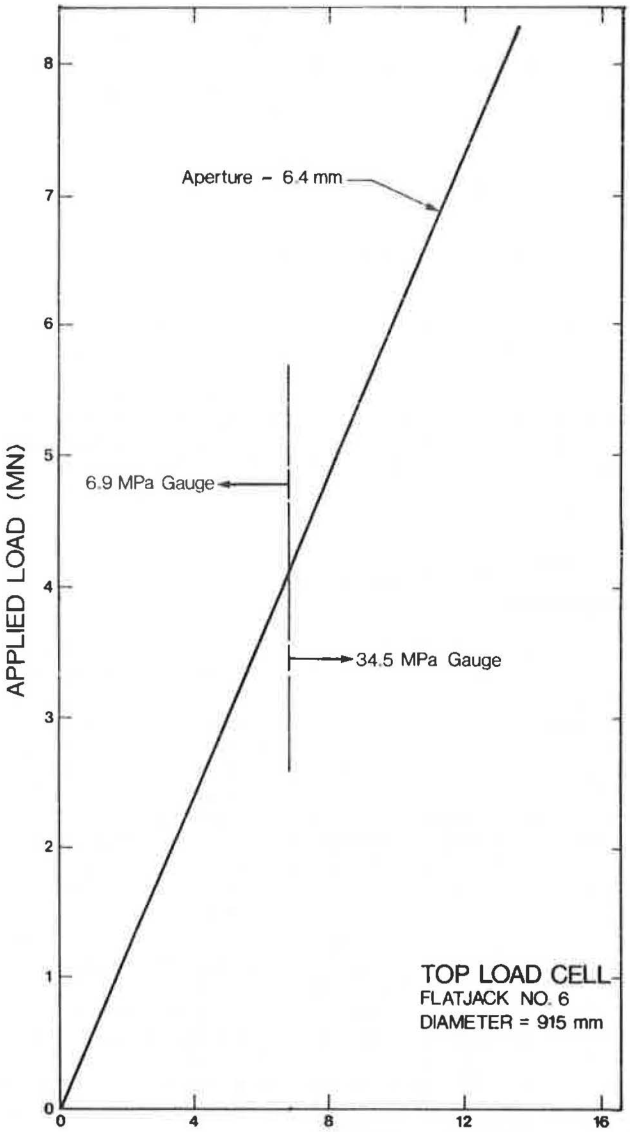

FIGURE 6 Calibration curve, applied load cell, flatjack 6.

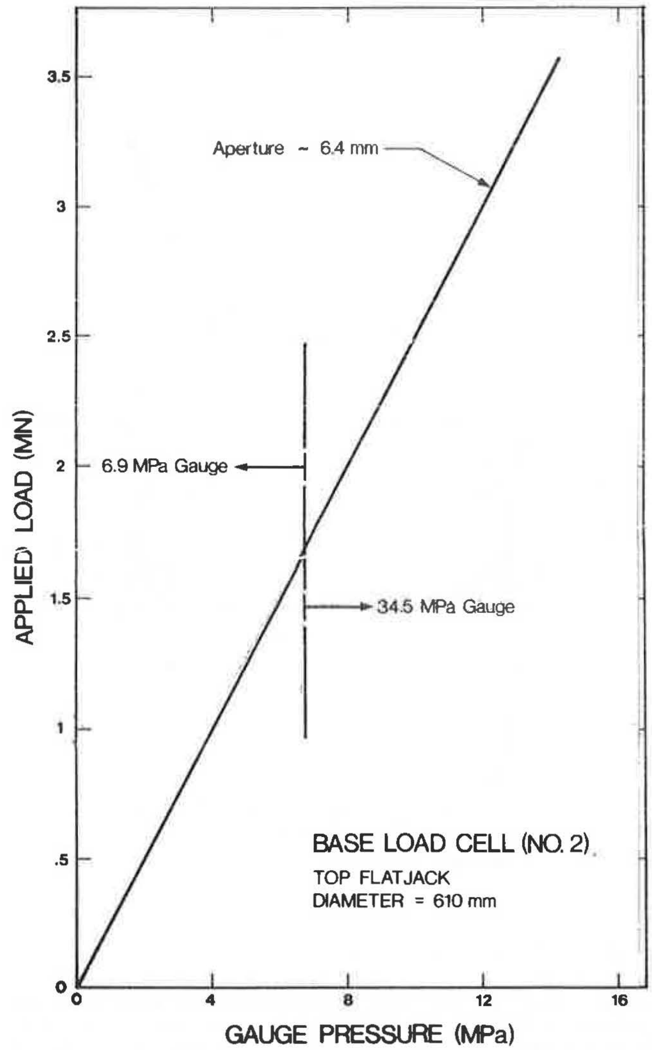

FIGURE 7 Calibration curves, base load cell 2, top flatjack.

directly, whereas shaft resistance for P2 was calculated using

$$Q_s = Q_t - Q_b \tag{1}$$

where

Q_s = shaft resistance load,
Q_t = applied load, and
Q_b = end-bearing load (measured using load cell).

This comparison suggests that the end-bearing loads measured by the base load cell are reliable. The reliability of the base load cells was also supported by a comparison of the results with behavior predicted on the basis of elastic analyses. A summary of a comparison of measured values of base load and predicted values using elastic solutions (5) is given in Table 3. The agreement between measured and predicted values is extremely good.

Only one flatjack, out of the seven used for the base load cells, failed to operate. This failure was presumed to have been caused by a leak in a pressure-tubing connection, which may have been damaged during installation.

DISPLACEMENT MEASUREMENTS

Vertical and horizontal displacement measurements for the piers were made using dial indicator gauges (0.025 mm per division). The measurements were referenced to timber beams supported on steel rods driven into the soft rock. A survey level and steel scales (1-mm divisions) were used to cross check vertical displacement at the top of the test piers, to measure vertical uplift of the anchor piers, and to verify that the reference beam supports did not move.

Pier Displacements

Vertical displacements were measured at the top of the test pier at two locations 180 degrees apart (Figure 1), at middepth, and at the bottom of the test section at two locations each 90 degrees apart,

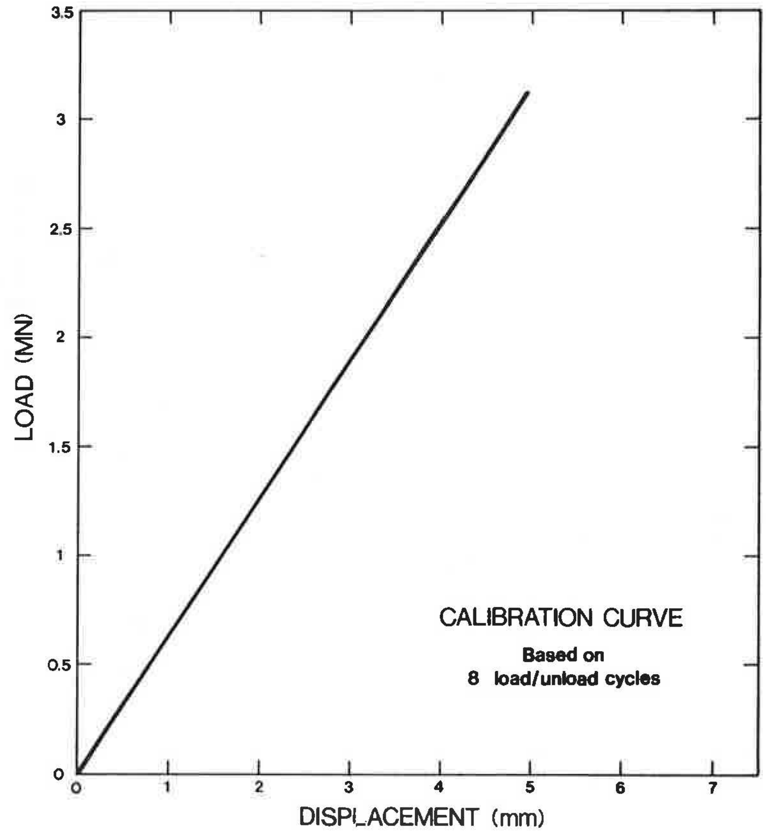

FIGURE 8 Calibration curves, base load cell 2—load versus displacement.

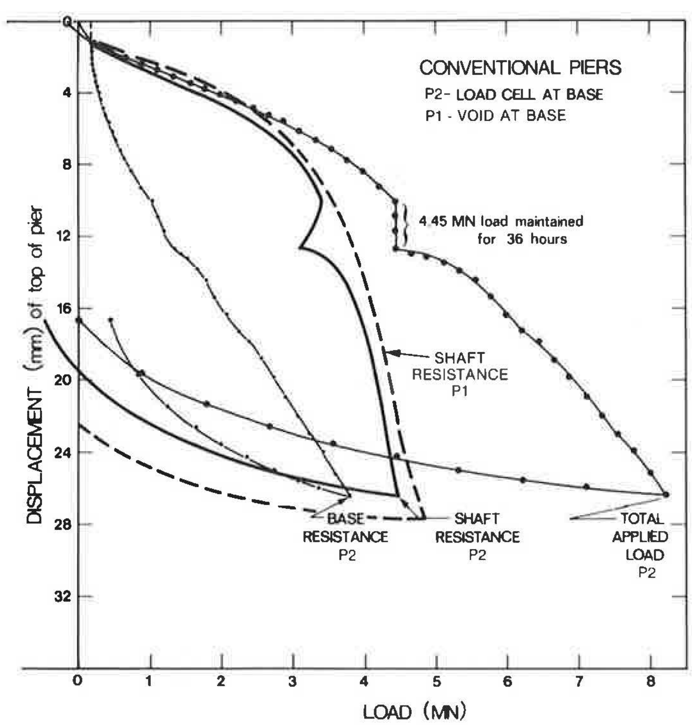

FIGURE 9 Comparison of load-displacement behavior for test piers P1 (shaft resistance only) and P2 (shaft resistance and end bearing).

TABLE 3 Comparison of Measured and Predicted Values of Base Load (Q_b/Q_t)

	TEST PIER		
	P2	P4	P5
Applied Load, Q_E (MN)	2.2	2.4	*2.1
Q_b/Q_t (measured)	.15	.20	*.20
Q_b/Q_t (predicted by elastic analyses)	.20	.20	*.20

* average value based on several test cycles

using a system of telltale rods (Figure 1). The telltale system consisted of a threaded rod inside a copper tube sleeve. A large washer was fastened to the base of the rod for embedment in the concrete. Tape was used as a spacer between the washer and the tubing to allow vertical movement of the rod and to seal the bottom of the tube to prevent concrete from seeping in. Dial gauges referenced to the top of the test pier were used to measure these telltale displacements. The telltale systems were only intended to measure displacements and did not provide the precision or accuracy necessary to determine strains within the pier. Typical load-displacement curves for a test pier are shown in Figure 10.

Horizontal displacement was measured at the top of the test pier at one location.

Rock Displacements

Vertical displacements within the rock mass adjacent to the test piers were measured at approximate depths of 0.9 m and 1.8 m at two locations using telltale rods (Figure 1). The telltale system consisted of threaded rods grouted at the bottom of a 50-mm-diameter percussion-drilled hole. The holes were located at distances of about 300 mm and 600 mm outside the pier-rock interface. Displacements were measured using dial gauges attached to the wooden reference beams. Typical load-displacement curves for the rock adjacent to the test pier are shown in Figure 11.

STRAIN MEASUREMENTS

Description

Geonor P-250 embedment vibrating wire strain gauges were installed in all of the test piers (except P6) at the midpoint of the test section.

Three gauges were used in each pier: a single gauge located on the axis and oriented to measure axial strain and two gauges (90 degrees apart) located near the perimeter and oriented to measure radial strain (Figure 1). The three gauges were fastened to a frame, made from copper tubing, and were placed in position after the concrete had been placed up to the midpoint of the test section.

The strain gauge measurements were used to estimate axial and radial stresses acting at the midpoint of the pier test sections during load testing so that load or stress distribution along the socket length could be determined. The axial and radial stresses may be calculated from strain measurements using the following linear elastic equations for an axisymmetric pier (6):

$$\sigma_z = \{[E_c(1 - \nu_c)]/[(1 - 2\nu_c)(1 + \nu_c)]\}\{\varepsilon_z + [\nu_c/(1 - \nu_c)]\varepsilon_r + [\nu_c/(1 - \nu_c)]\varepsilon_\theta\} \quad (2)$$

$$\sigma_r = \{[E_c(1 - \nu_c)]/[(1 - 2\nu_c)(1 + \nu_c)]\} \times \{[\nu_c/(1 - \nu_c)]\varepsilon_z + [\nu_c/(1 - \nu_c)]\varepsilon_\theta\} \quad (3)$$

where

E_c = Young's modulus of the concrete,
ν_c = Poisson's ratio of the concrete,
ε_z = the axial strain (measured),
ε_r = the radial strain (measured), and
ε_θ = the circumferential strain ($\varepsilon_\theta = \varepsilon_r$ assumed).

The pier is assumed to be isotropic and elastic and to have a uniform distribution of radial and axial strain across the pier section.

It should be noted that the use of strain measurements to estimate stress in concrete is not an

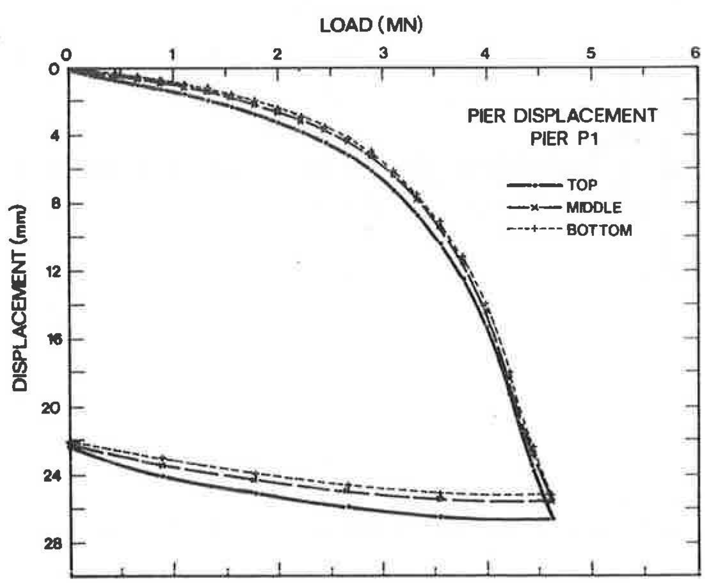

FIGURE 10 Typical load-displacement curves for test pier.

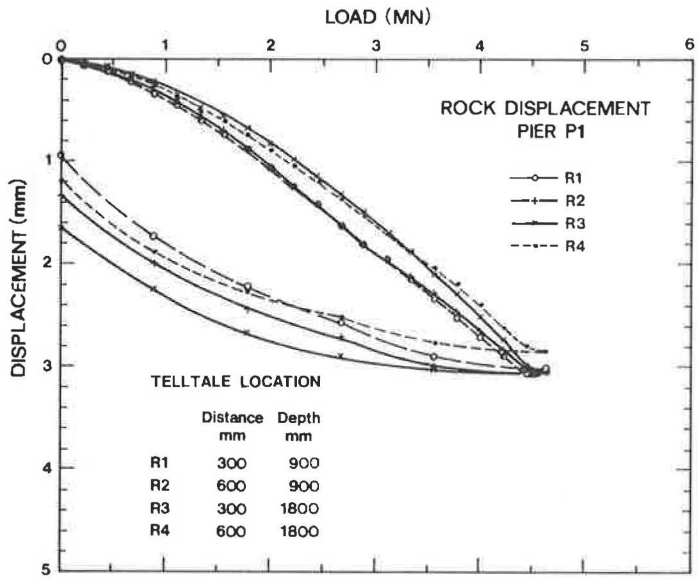

FIGURE 11 Typical load-displacement curves for rock adjacent to test piers.

ideal approach to the determination of load distribution. Knowledge of the elastic modulus of the concrete is necessary, and nonhomogeneity, stresses induced by curing, and creep behavior of the concrete can cause difficulties in the interpretation of the data.

The testing was of short duration; therefore, the influence of concrete creep on strain measurements would be negligible. Strain measurements were also obtained when the concrete was placed and at various intervals before testing (concrete curing period). All of the strain gauges indicated that concrete expansion occurred during curing.

Calibration

The strain-frequency calibration factor supplied by Geonor was verified in the laboratory. The reported accuracy of these gauges is ±2 microstrain over a range of 1250 microstrain.

Reliability

Some difficulties experienced with the electronics during load testing on piers P3 and P4 were traced to inadequate grounding of the readout instrument. After this problem was corrected, the strain measuring system functioned well. The reliability of the strain gauge measurements may be indirectly assessed by comparing the measured strains to expected values of strain for the various load support conditions.

Load tests were performed on conventional piers that had essentially zero base resistance. Thus, the applied load was supported only through shaft resistance. In these tests, the measured radial strains were negative (compression) indicating that the pier was being compressed radially inward during loading (Figure 12a). This behavior indicates the tendency for shear dilation (volume expansion) to occur at the pier-rock interface or within the rock, or both. Also, the strain data were consistent in that both axial and radial strains decreased (compression) with increasing applied load. Load tests were also carried out on piers with combined shaft resistance and base resistance components. In all of these tests, the measured radial strains were positive, indicating that the pier was expanding in a radial direction (Figure 12b). This behavior is comparable to that which occurs during a compression test on a concrete cylinder and indicates Poisson's effect. These strain data were consistent in that axial strain decreased (compression) and radial strain increased (expansion) with increasing applied load. Thus, all of the measured strain data correctly reflected the anticipated strain behavior for the load support conditions tested.

A summary of a comparison of values of axial strain measured using the strain gauges with values measured using the telltale systems and with values estimated using a simplified elastic analysis is given in Table 4.

The reliability of the data obtained from the vibrating wire strain gauges may also be assessed by examining the load distribution in the test piers. This is discussed in the next section. In the simplest case, test pier P1 (shaft resistance only), the determined load distribution was exactly as anticipated. Thus the strain gauge performance in this case can be judged to be extremely good. In the other test piers, the conditions were more complicated: combined end-bearing and shaft resistance, grooved shafts, preloading of base, and cycled loading were involved. Thus the reliability of the strain gauge measurements in these cases could not be evaluated.

LOAD DISTRIBUTION IN TEST PIERS

Load distribution in test piers P1 through P5 was determined from measurements of applied load at the top of each pier, axial and radial strain at the middepth of the test sections, and end-bearing load (or known boundary condition) at the base of each pier. Thus, the load distribution curves determined for all the piers were based on data from the same three locations. The top and bottom loads were measured with load cells except in the case of piers that had voids at their bases for which zero end-bearing load was assumed. The load (or stress) at pier middepth was determined indirectly using vibrating wire strain gauges embedded in the concrete pier.

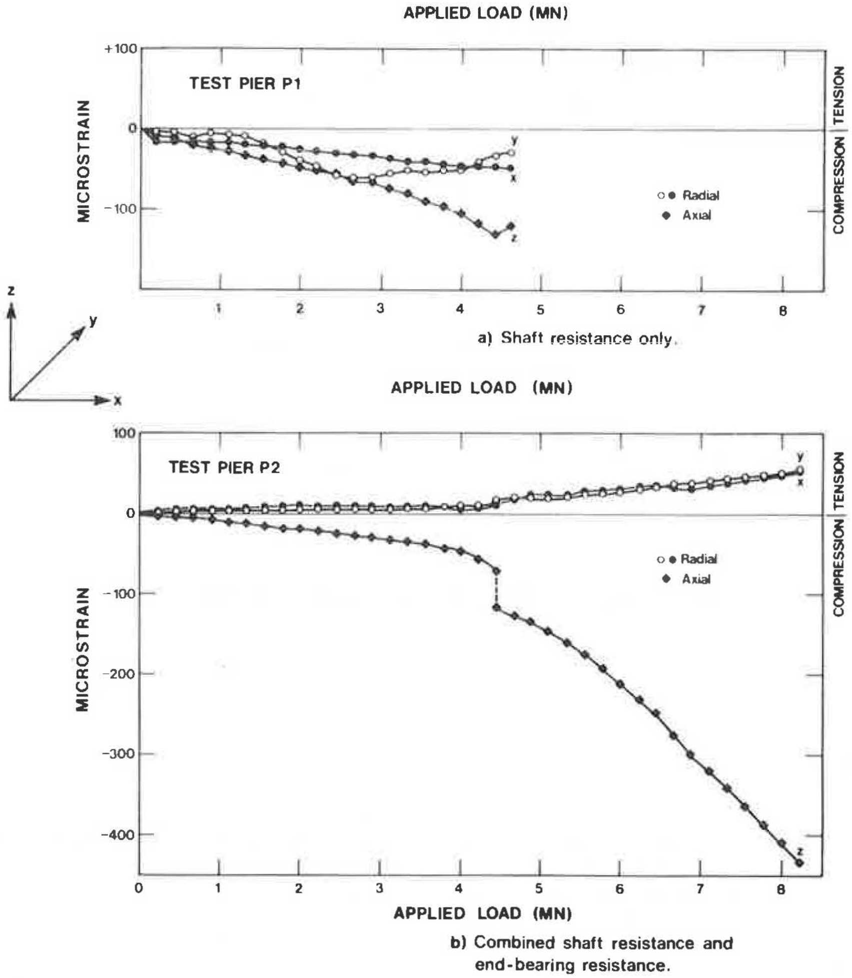

FIGURE 12 Measured values of axial and radial strain.

TABLE 4 Summary of Estimated and Measured Values of Axial Strain

	PIER DESCRIPTION	STRAIN IN PIER @ Q_a = 2 MN (x 10^{-6}) CALCULATED Upper Limit	CALCULATED Best Estimate	MEASURED Strain Gauge	MEASURED Telltales*
P1	Conventional - void at base	41	21	46	110
P2	Conventional - end bearing	30	17	18	0
P3	Grooved - void at base	41	20	89**	37
P4	Grooved - end bearing	31	18	10**	110
P5A	Conventional - end bearing preload	30	23	29	130
P5B	Conventional - end bearing preload	30	28	42	70
P5C	Conventional - end bearing preload	30	30	48	110
P5D	Conventional - void at base	30	15	28	37

* Strain based on telltale measurements were determined using difference between middle and bottom telltale and only provide a crude approximation of strain in the test pier. (Accuracy of dial gauge = .001 in. is equivalent to a strain of 37 x 10^{-6})

** Readings may be erroneous due to difficulties with readout box.

The load distribution curves for conventional socketed piers, P1 and P2, at various magnitudes of applied load are shown in Figure 13. The distribution curves for test pier P1 (void at base) indicate that shaft resistance was distributed uniformly over the length of the socket for all values of applied load (Figure 13a). This distribution behavior is consistent with the load distribution predicted from analytical studies based on elastic theory (7). The distribution curves for pier P2 (load cell at base) are distinctly different from those of P1. In the elastic loading range $Q_T < 2.2$ MN, little shaft resistance was mobilized in the lower half of pier P2 (Figure 13b).

This behavior is not consistent with analytical solutions that predict uniform distribution of the load (constant slope) over the socket length (7). The reasons for this discrepancy are unclear. However, the similarity in the shape of the distribution curves for other similar piers suggests that the initial small preload applied to the base load cells (to ensure seating) may be the cause of this inconsistent behavior.

During the maintained load increment for pier P2 (applied load = 4.45 MN), the slopes of the upper and lower portions of the distribution curves began to equalize and load distribution or shaft resistance along the socket became essentially uniform at an applied load of 6 MN (Figure 13b).

CONCLUSIONS

The instrumentation used in a field testing program to investigate methods of improving the performance of rock socketed piers has been described and discussed. The results obtained from the test program demonstrate the reliability of the instruments used.

The load cells consisting of a series of FREYSSI flatjacks performed well. The load cells were reliable and the results obtained using these cells were estimated to be accurate to within ±2 percent.

In the case of test pier P1, the Geonor P-250 embedment vibrating wire strain gauges were judged to be reliable and satisfactory. All of the strain measurements for the test piers were consistent with anticipated behavior. When questionable data were obtained, the cause was improper grounding of the readout equipment and not a fault of the instruments themselves.

Loads (stresses) calculated using the strain gauge data enabled determination of the load distribution within the pier-socket system.

Displacement at the top, middepth, and bottom of the test piers and in the rock immediately adjacent to the piers was reliably measured using a combination of dial indicator gauges and telltale systems.

The versatility of a flatjack load cell to perform three different functions has been described.

ACKNOWLEDGMENTS

The work presented in this paper was performed by Western Caissons Limited. Financial support for the project was provided by Supply and Services, Canada, and the National Research Council (NRC) of Canada, under contract 1SX79.00053 and by the Natural Sciences and Engineering Research Council. The project was sponsored by the Division of Building Research, NRC. M. Bozozuk acted as scientific authority, made valuable suggestions for this work, and reviewed the

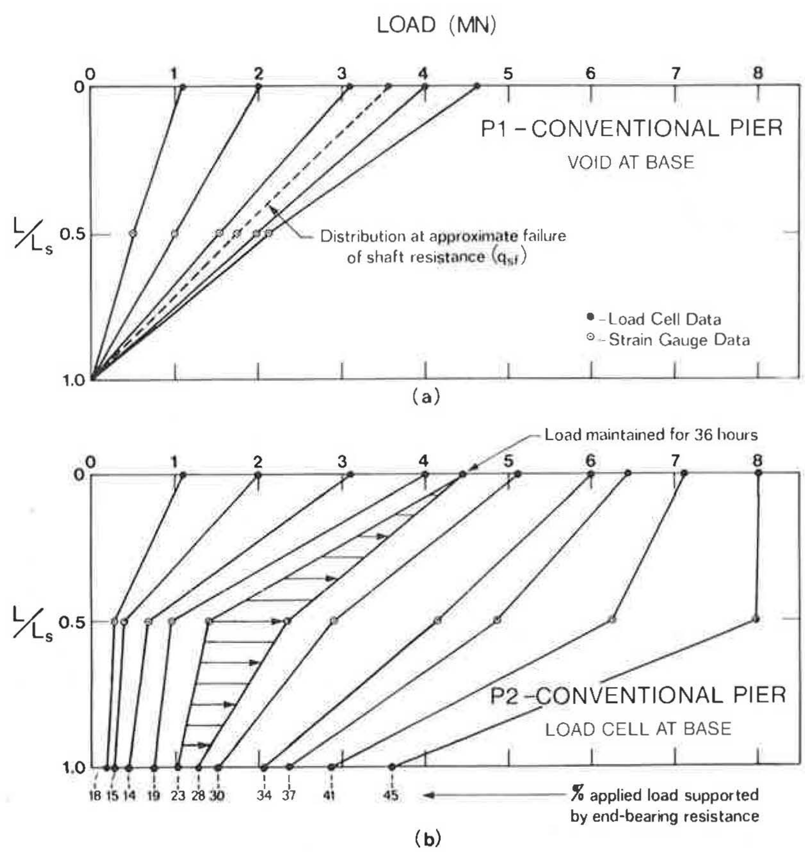

FIGURE 13 Typical load distribution curves for conventional piers.

manuscript. National Sewer Pipe provided use of the test site.

The author also acknowledges T.C. Kenney for his valuable assistance and D. Allan, J. Franklin, B. Fyfe, S. Horvath, P. Kozicki, P. Luk, E. Magni, R. Mills, B. Pluhator, and W. Trow for their contributions.

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Closing Remarks on Reliability of Geotechnical Instrumentation

JOHN DUNNICLIFF

ABSTRACT

In the closing remarks delivered at the Symposium on Reliability of Geotechnical Instrumentation, three subjects are discussed: a "recipe" for reliability, the parameters that can be measured most readily, and a plea to users of instrumentation.

These closing remarks will address three topics: First, a recipe for reliability. Second, which parameters can be measured most reliably? Third, a plea to users of instrumentation.

A RECIPE FOR RELIABILITY

When this symposium was being planned, I wrote a recipe for reliability. Having now read the six papers that have been presented, I have made a few changes and will define what I believe are the major ingredients. There are two types: instrument ingredients (three of these) and people ingredients (five of these).

Instrument Ingredients

Simplicity

Follow the KISS (keep it simple, stupid) principle. For example, mechanical and hydraulic devices are generally more reliable than electrical devices.

Self-Verification

This term means that instrument readings can be verified in place. For example:

- Telltales on a rod extensometer with a method of disconnecting the rod from the anchor, so that a check can be made for free sliding;
- Duplicate transducers (e.g., a vibrating wire and a pneumatic transducer packaged within the same housing to create a piezometer with two independent methods of reading); and
- Checking remote-reading borehole extensometers with a dial gauge at the head.

Durability in the Installed Environment

The transducer must have proven longevity to suit the application. Cables, tubes, or pipes that connect the transducer to its readout must be able to survive imposed pressure changes, deformation, water, sunlight, and chemical effects such as corrosion and electrolytic breakdown.

People Ingredients

Thorough Planning

Details of planning requirements are given by Dunnicliff (1). The ingredients include

- McGuffey's "System Design," (see paper by McGuffey in this Record) including development of the best predictive model.
- Use of the best contract practices. Abramson and Green (see their paper in this Record) say, "Many owners effectively encourage low survivability rates by using low-bid procurement procedures." That is a succinct statement about a very large and serious problem.
- Rawnsley, Russell, and Hansmire (see their paper in this Record) address backup and redundancy, in their discussion of Harvard Square Station: "Redundancy existed, with key parameters being measured by more than one instrument." Hannon and Jackura (see their paper in this Record) say in their summary, "When feasible, alternative procedures should be used for backup to estimate soil stress conditions."
- Comprehensive factory calibration and quality assurance are important. Note that this is a people ingredient not an instrument ingredient. Hannon and Jackura say, "All instruments should also be subject to bench or calibration testing, or both, to ensure performance and specification compliance."

Installation Care

Planning for installation usually includes gaining the cooperation of the construction contractor. Without this, reliability is hard to achieve.

Regular Maintenance and Calibration

The need for regular maintenance and calibration is well demonstrated in the paper in this Record by Bordes and Debreuille. For example, portable readout units should be calibrated frequently.

Care During Data Collection

For example, in the field the person reading an instrument should always study changes with respect to the previous reading. Substantial changes may indicate a reading error or the need for rapid remedial action.

Care During Data Processing and Interpretation

This ingredient includes McGuffey's "Engineering Interpretation Methodology."

Summary

In summary, experience and knowledge are vital to the people ingredients. Rawnsley, Russell, and Hansmire say:

> The key to the reliability of the instrumentation program was the people involved. Instrumentation installation was done by experienced professionals. Instrument monitoring was performed by trained people who were on the job for extensive periods of time, were interested in the results, and were responsible for interpreting the measurements.

In their summary Hannon and Jackura say,

> Instrumentation personnel should be experienced and knowledgeable about potential problems associated with the placement and monitoring of the particular instruments selected for use.

McGuffey had operator knowledge as one of his five major items contributing to reliability. It is therefore agreed that experience and knowledge are vital. But perhaps even overriding these is motivation. Discussing responsibility for instrumentation, Baker (2) said:

> Who has the motivation? Who cares about the data? The person with the greatest vested interest in the data should have direct line responsibility for producing it accurately.

Conclusion to a Recipe for Reliability

In my view, unreliability can more often be attributed to the people ingredients than to the instrument ingredients. The message is clear: We, the users, need to make a strong effort to improve the state of the practice.

WHICH PARAMETERS CAN BE MEASURED MOST RELIABLY?

The four parameters, pore pressure, total stress, deformation, and load and stress in structural elements, need to be rated. They will be rated here in order of increasing reliability.

Total Stress in Soil

The difficulties are well illustrated by Hannon and Jackura. They divide them into two groups, first the ability of the cell to measure the stress around it (cell design) and, second, underregistration because the cell is in a soft cocoon of backfill (cell placement). I believe the larger problem is the second one. S.D. Wilson (personal communication, 1984), on the basis of his extensive experience measuring total stress within embankments dams, states:

> When earth pressure cells are installed in a horizontal plane in compacted fills for embankment dams, the cells typically register only 50 to 70 percent of the added vertical stress as embankment construction continues.

There is a need to develop a method of hand compaction around the cells that prestresses the soil to match the prestress in the remainder of the fill without damaging the cells. This is, of course, extremely difficult to do. The Comision Federal de Electricidad at experimental laboratories in Mexico City has constructed a large laboratory facility to test the response of embedment earth pressure cells to applied loads. It is hoped that improved installation techniques will result from tests now in progress.

Finally, I rate total stress as the least reliable parameter because of one other fundamental factor: Measurements are point measurements in a heterogeneous environment, and therefore a small number of measurements may not be representative of overall conditions.

Pore Pressure

The instruments are satisfactory. Installation problems are difficult, but they can be solved. The main problem, well illustrated by McGuffey, is the same as the one mentioned last for total stress: Measurements are point measurements in a heterogeneous environment. The problem is not as severe with pore pressure measurements, but is still significant.

I want to say a few words about the paper by Bordes and Debreuille, in this Record, because their conclusions apply to pore pressure measurements. A most believable and impressive case for vibrating wire instruments is presented in their paper. I have been looking for such a paper for more than 10 years and welcome this clear and convincing information. However, I am going to disagree with their Parisian graciousness. They say:

> Although the instruments discussed in this paper come from the same manufacturer, the conclusions drawn therefrom have a much wider scope. They apply to all vibrating wire instruments, provided of course that construction is of a high standard.

I will mention briefly two experiences with vibrating wire instruments from another "leading manufacturer," from whom "construction of a high standard" might be expected.

1. During first filling of the reservoir behind an embankment dam, a vibrating wire piezometer indicated a piezometric level that caused concern. Filling was stopped. The piezometer reading continued to rise. When the indicated piezometric level rose above pool level, everybody discounted the measurements and filling continued.
2. Vibrating wire pressure transducers have recently been used to measure oil level in oil tankers. Many have been unreliable, and several hundred have been returned to the manufacturer.

I truly believe that the conclusions drawn by Bordes and Debreuille do not necessarily apply to all vibrating wire instruments. How do users know whether all the details discussed by the writers are handled with similar care by all manufacturers? As one example, is the aging issue raised by Bordes and Debreuille handled adequately by other manufacturers?

Load and Stress in Structural Elements

I rate this parameter more reliable than pore pressure because it is measured on or in a material made and controlled by people. Discussion of this parameter is subdivided into use of three types of instrument: load cells, strain gauges on elastic elements, and strain gauges in or on concrete.

1. Load cells serve extremely well. Abramson and Green indicate the need for good bearing plates and taking care of eccentricity. Rawnsley, Russell, and Hansmire talk about problems with using hydraulic jacks for load measurement and confirm what many others have found:
   - Up to 20 percent overregistration during loading and
   - Up to 5 percent underregistration during unloading.

   The problem is caused by friction between the piston and the cylinder, and hydraulic jacks should not be relied on for load measurement.
2. Strain gauges on elastic elements also serve well. Measurements on structural steel, for which a reliable conversion from strain to stress can be made, have a long and successful history.
3. Strain gauges on or in concrete cause problems in converting strain to stress, and the problem is aggravated if measurements are other than extremely short term. Horvath and Abramson and Green discuss this problem, and Abramson and Green recommend three methods of dealing with it:
   - Controlled laboratory tests;
   - Dummy no-load gauges; and
   - Measuring the load directly, where possible (e.g., across a concrete pile).

   My experience has been that
   - Controlled laboratory tests rarely model field conditions adequately and
   - Dummy no-load gauges are of little use because they do not account for strain caused by creep under load. This leaves three options:
   - Measure load directly, as suggested by Abramson and Green;
   - Where possible, use concrete stress meters instead of strain gauges, taking great care to ensure intimate contact between the instrument and the concrete, either by following the installation methods recommended for the Carlson stress meter or by using a poststressing tube as provided in the Gloetzl stress meter; and
   - Create, as part of the structure in the field, an "unconfined compression test specimen," under known load, and measure strain with strain gauges or multiple telltales in this part of the structure, to determine modulus. This can be done at the top of piles and drilled piers during test loading, sleeving if necessary below the ground surface to create the "specimen."

Deformation

Deformation can be measured with greatest confidence. Instruments can often be simple. A single instrument can provide data for a large and representative zone. If you can answer your geotechnical question with deformation measurements, please do so.

The extensive topic of deformation measurements has not been covered in this symposium, and at first this seems to be a shortcoming. However, I do not think its inclusion would alter my view that the main impediment to reliability is the people ingredient of inadequate experience, knowledge, and motivation.

MY PLEA TO THE USERS OF INSTRUMENTATION

Hansmire says, in his introduction to this Symposium, "The negative experiences, the failures, are not often reported." Abramson and Green say, when talking of the failure of instrumentation schemes: "The distilled experience of many engineers who have suffered the consequences of instrument or program failures should help reduce the incidence of future occurrences."

It is clear that we learn from our mistakes and the mistakes of others. I have described about 20 mistakes, of which I have been guilty or with which I have been associated, in a series of three articles to be published in Geotechnical News (published quarterly by Bitech Publishers, Ltd., 801-1030 West

Georgia Street, Vancouver, British Columbia, Canada, V6E 2Y3, and distributed to registered members of the Canadian Geotechnical Society and the United States National Society of ISSMFE; others may subscribe by contacting the publisher). My purpose in writing these articles, entitled "Lessons Learned from Imperfect Field Monitoring Programs," was to help others to avoid making the same mistakes. In each case the mistake and the lesson learned are stated. This is planned as an ongoing section in Geotechnical News. The ball will soon be rolling; please keep it rolling by contributing lessons learned from your mistakes so that I may avoid adding them to my already long list.

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