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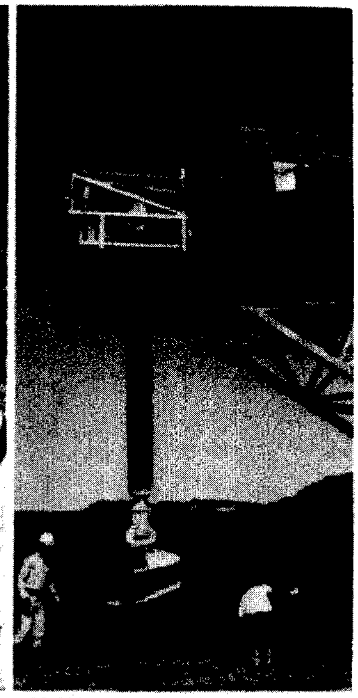
GEOTECHNICAL INSTRUMENTATION

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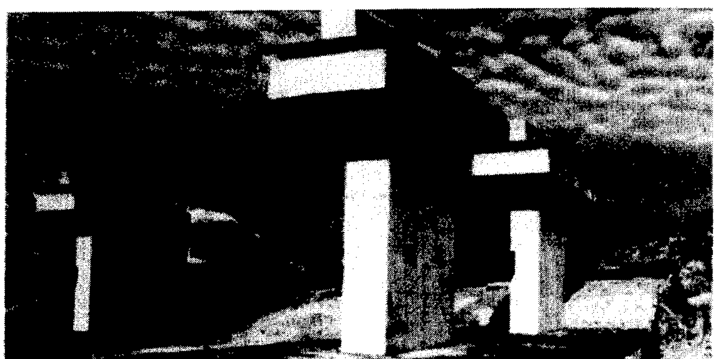
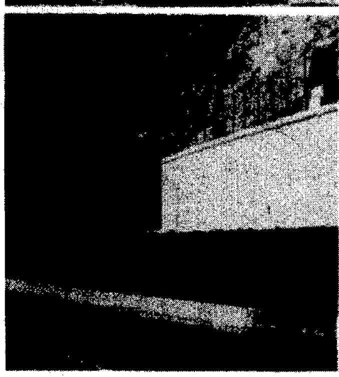



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FHWA Technical Consultant - J.A. DiMaggio, A. Muñoz, P. Osborn
National Highway Institute CO - J. Mowery III; COTR - L. Jones

16. Abstract

This manual provides the practicing geotechnical engineer with a thorough understanding of the use of geotechnical instrumentation in highway construction. The manual presents an overview of measurement methods and tools, and recommends a systematic and complete approach to planning monitoring programs. Issues that lead to the use of instrumentation for various types of construction such as soil and rock slopes, embankments, deep foundations, earth retaining structures, preloading programs, and grouting are identified and appropriate instruments are recommended. Finally, general guidelines are provided for various field tasks such as calibration, maintenance and installation of instruments, collection of instrumentation data, processing and presentation of collected data, interpretation of processed data and reporting of the results. A practical checklist for planning a geotechnical instrumentation program is included. Student exercises for planning monitoring programs and for data evaluation are also included.

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PREFACE

This Module is the eleventh in a series of eleven Modules that constitute a comprehensive training course in geotechnical and foundation engineering. Sponsored by the National Highway Institute (NHI) of the Federal Highway Administration (FHWA), the training course is given at different locations in the U.S. The intended audience is geotechnical specialists involved with the design and construction of transportation facilities.

The reference manual for this Module is intended to be a stand-alone document and is geared towards providing the practicing engineer with a thorough understanding of the use of geotechnical instrumentation for highway applications. Accordingly, the manual presents an overview of measurement methods and recommends a systematic and complete approach to planning monitoring programs, which should always be followed. The manual then proceeds to identify issues that may lead to the use of geotechnical instrumentation during various types of construction, and suggests appropriate instruments. Finally general guidelines are presented for various field tasks. The organization of the manual is presented below.

Chapter 1 identifies the scope of the module, the general role of instrumentation, and the key to successful performance monitoring.

Chapters 2 through 5 provide an overview of geotechnical instrumentation hardware for measurement of groundwater pressure, deformation, total stress in soil, and load and strain in structural members.

Chapter 6 outlines a systematic approach to planning monitoring programs, and is the hub of the Module. The greatest shortcoming in the state-of-the-practice of geotechnical instrumentation is inadequate planning of monitoring programs, and therefore readers should give their first concentrated attention to Chapter 6. Appendix A provides a checklist of the planning steps.

The systematic approach is illustrated with a case history of an embankment on soft ground. Overhead viewgraphs used during the course, and that supplement Chapter 6, are included in Appendix B.

Chapters 7 through 11 are companions to other Modules. They include a discussion of instrumentation for the following types of construction identified in the other Modules:

- Embankments on soft ground
- Cut slopes in soil
- Landslides in soil
- Driven piles
- Drilled shafts
- Internally braced excavations
- Externally braced excavations
- Slurry walls
- Mechanically stabilized earth walls
- Soil nailing walls
- Surcharging
- Vertical drains
- Staged construction
- Grouting
- Cut slopes in rock
- Landslides in rock

For each type of construction, there are two sections in Chapters 7 through 11. The first section indicates the general role of geotechnical instrumentation. The second section suggests the principal geotechnical questions that may arise and indicates the types of instruments that may be used to help provide answers to those questions.

Chapters 7, 8 and 9 include student exercises for planning monitoring programs, and examples of planning results are included in Appendixes C, D and E.

Chapter 12 includes general guidelines on instrument calibration, maintenance and installation, and on collection, processing, presentation, interpretation and reporting of instrumentation data. The chapter also includes a student exercise on evaluation of data, with suggestions for the evaluations included in Appendix F.

Chapter 13 presents appropriate references.

A primary source of text and figures for this Module has been *Dunnicliff, J. (1988, 1993). "Geotechnical Instrumentation for Monitoring Field Performance," John Wiley & Sons, Inc., New York, 577 p.* Reprinting of this material is by permission of John Wiley & Sons, Inc., and is gratefully acknowledged.

Finally, this manual is developed to be used as a living document. After attending the training session, it is intended that the participant will use it as a manual of practice in everyday work. Throughout the manual, attention is given to ensure the compatibility of its content with those of the reference manuals prepared for the other training Modules. Special efforts are made to ensure that the included material is practical in nature and represents the latest developments in the field.

ACKNOWLEDGEMENTS

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NOTICE

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MODULE #11
GEOTECHNICAL INSTRUMENTATION

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CHAPTER 1.0
INTRODUCTION

1.1 SCOPE OF THIS MODULE

There are two general categories of measuring instruments. The first category is used for subsurface investigations to determine soil or rock properties, for example, strength, compressibility, and permeability, normally during the design phase of a project. Examples are given in Module 1 (Subsurface Investigations). The second category is used for monitoring performance, normally during the construction or operation phase of a project, and may involve measurement of ground water pressure, total stress, deformation, load, or strain. This Module is concerned only with the second category.

This Module includes an overview of geotechnical instrumentation hardware for measurement of groundwater pressure, deformation, total stress in soil and load and strain in structural members. It outlines a systematic approach to planning monitoring programs, and includes a discussion of instrumentation for various types of construction identified in other Modules. Finally, general guidelines are provided for calibration, maintenance and installation, and on collection, processing, presentation, interpretation and reporting of instrumentation data.

1.2 GENERAL ROLE OF GEOTECHNICAL INSTRUMENTATION

Peck (1988, 1993) indicates that every geotechnical design is to some extent hypothetical, and every construction job involving earth or rock runs the risk of encountering surprises. These circumstances are the inevitable result of working with materials created by nature, often before the advent of human beings, by processes seldom resulting in uniform conditions. The inability of exploratory procedures to detect in advance all the possibly significant properties and conditions of natural materials requires the designer to make assumptions that may be at variance with reality and the constructor to choose equipment and construction procedures without full knowledge of what might be encountered. Field observations, including quantitative measurements obtained by field instrumentation, provide the means by which the geotechnical engineer, in spite of these inherent limitations, can design a project to be safe and efficient, and the constructor can execute the work with safety and economy. Thus, field instrumentation is vital to the practice of geotechnics, in contrast to the practice of most other branches of engineering in which people have greater control over the materials with which they deal. For this reason geotechnical engineers, unlike their colleagues in other fields, must have more than casual knowledge of instrumentation: to them it is a working tool.

The engineering practice of geotechnical instrumentation involves a marriage between the capabilities of measuring instruments and the capabilities of people.

During the past few decades, manufacturers of geotechnical instrumentation have developed a large assortment of valuable and versatile products for the monitoring of geotechnically related parameters. Those unfamiliar with instrumentation might believe that obtaining needed information entails nothing more than pulling an instrument from a shelf, installing it, and taking readings. Although successful utilization may at first appear simple and straightforward, considerable engineering and planning are required to obtain the desired end results.

The use of geotechnical instrumentation is not merely the selection of instruments but a comprehensive step-by-step engineering process beginning with a definition of the objective and ending with implementation of the data. Each step is critical to the success or failure of the entire program, and the engineering process involves combining the capabilities of instruments and people.

1.3 THE KEY TO SUCCESS

Full benefit can be achieved from geotechnical instrumentation programs only if every step in the planning and execution process is taken with great care. The analogy can be drawn to a chain with many potential weak links: this chain breaks down with greater facility and frequency than in most other geotechnical engineering endeavors. Shortcomings in instrumentation programs can usually be attributed to a weakness in one or more links. The 21 major links in the planning process are listed in Table 1-1 and defined in Chapter 6. The 10 major links in the execution process are discussed in Chapter 12 and listed in Table 1-1. The strength of these 31 links depends both on the capabilities of measuring instruments and the capabilities of people. The success of performance monitoring will be maximized by maximizing the strength of each link.

Peck (1984) states "The legitimate uses of instrumentation are so many, and the questions that instruments and observation can answer so vital, that we should not risk discrediting their value by using them improperly or unnecessarily."

1.4 PRIMARY REFERENCE

A primary source of text and figures for this Module has been Dunnicliff (1988, 1993). More detail on many of the topics discussed in this Module can be found in that reference.

1.5 COURSE OBJECTIVES

Upon completion of this course, participants should be able to:

1. Demonstrate knowledge of available instrumentation through proper selection of monitoring equipment.
2. Understand how to plan instrumentation programs in a systematic way.
3. Understand the general role of instrumentation, the principal geotechnical questions, and the applicable instrumentation for:
   - Soil slopes and embankments
   - Deep foundations
   - Earth retaining structures
   - Ground improvement techniques
   - Rock slopes
4. Understand practical methods for:
   - Calibration and maintenance of instruments
   - Installation of instruments
   - Collection of instrumentation data
   - Processing and presentation of instrumentation data
   - Interpretation of instrumentation data
   - Reporting of conclusions
5. Demonstrate knowledge of the golden rule: **Every instrument on a project should be selected and placed to assist with answering a specific question: if there is no question, there should be no instrumentation.**

TABLE 1-1
STEPS TO FOLLOW IN DEVELOPING A SUCCESSFUL MONITORING PROGRAM USING GEOTECHNICAL INSTRUMENTATION

Phase	Step
Planning	Define the project conditions
	Predict mechanisms that control behavior
	Define the geotechnical questions that need to be answered
	Define the purpose of the instrumentation
	Select the parameters to be monitored
	Predict magnitudes of change
	Devise remedial action
	Assign tasks for design, construction, and operation phases
	Select instruments
	Select instrument locations
	Plan recording of factors that may influence measured data
	Establish procedures for ensuring reading correctness
	List the specific purpose of each instrument
	Prepare budget
	Prepare instrumentation system design report
	Write instrument procurement specifications
	Plan installation
	Plan regular calibration and maintenance
	Plan data collection, processing, presentation, interpretation, reporting, and implementation
	Write specifications for field instrumentation services
	Update budget
Execution	Procure Instruments
	Perform pre-installation acceptance tests
	Install instruments
	Perform post-installation acceptance tests
	Calibrate and maintain instruments
	Collect data
	Process and present data
	Interpret data
	Report conclusions
	Implement

CHAPTER 2.0
OVERVIEW OF HARDWARE FOR MEASUREMENT OF GROUNDWATER PRESSURE

2.1 INTRODUCTION

In this Module the term piezometer is used to indicate a device that is sealed within the ground so that it responds only to groundwater pressure around itself and not to groundwater pressures at other elevations. Piezometers are used to monitor pore water pressure in soil and joint water pressure in rock. The term pore pressure cell is sometimes used as a synonym for piezometer. An observation well is a different type of instrument: it has no subsurface seals, and creates a vertical connection between strata.

Applications for piezometers fall into two general categories. First, for monitoring the pattern of water flow and second, to provide an index of soil or rock mass strength. Examples in the first category include monitoring subsurface water flow during large-scale pumping tests to determine permeability in situ, and monitoring the long-term seepage pattern in slopes. In the second category, monitoring of pore or joint water pressure allows an estimate of effective stress to be made and thus an assessment of strength. Examples include assessing the strength along a potential failure plane behind a cut slope in soil or rock, and monitoring of pore water pressure to control staged construction over soft clay foundations.

In general, piezometers used for measuring pore water pressure in soil are no different from piezometers used for measuring joint water pressure in a rock mass: the difference is in the installation arrangements. Piezometers in soil need to be installed to monitor changing pore water pressure, hence the installation procedure will usually entail placement of a column of sand in the borehole, around the piezometer, typically about 750 mm long. Piezometers in rock need to be installed to monitor changing joint water pressure, hence the column of sand placed during installation will typically be much longer than in soil, to ensure that the zone intersects a discontinuity in the rock mass.

The following sections provide an overview of hardware for measurement of groundwater pressure. Advantages and limitations are summarized in Table 2-1.

Section 2.7 includes recommendations on selection among the various instruments, and Section 2.8 provides guidelines on miscellaneous issues associated with measurement of groundwater pressure.

2.2 OBSERVATION WELLS

As shown in Figure 2-1, an observation well consists of a perforated section of pipe attached to a riser pipe, installed in a sand- or gravel-filled borehole. The surface seal, with cement mortar or other material, is needed to prevent surface runoff from entering the borehole, and a vent is required in the pipe cap so that water is free to flow through the wellpoint. The elevation of the water surface in the observation well is determined by sounding with one of the probes described in Section 2.3 for measurements within open standpipe piezometers.

Appropriate applications for observation wells are very limited. In current practice they are frequently installed in boreholes during the site investigation phase of a project, ostensibly to define initial groundwater pressures and seasonal fluctuations. However, because observation wells create a vertical connection between strata, their **only** application is in continuously permeable ground in which groundwater pressure increases uniformly with depth. This condition can rarely be assumed. The author believes that many practitioners do not appreciate the need to make pore water pressure measurements at

TABLE 2-1
INSTRUMENTS FOR MEASURING GROUNDWATER PRESSURE

Instrument Type	Advantages	Limitations*
Observation well (Figure 2-1)	Can be installed by drillers without participation of geotechnical personnel	Provides undesirable vertical connection between strata and is therefore often misleading; should rarely be used
Open standpipe piezometer (Figure 2-2)	Reliable Long successful performance record Integrity of seal can be checked after installation by performing falling head permeability test Can be converted to remote-reading piezometer by insertion of a pressure transducer	Long time lag Subject to damage by construction equipment and by vertical compression of soil around standpipe Freezing problems if piezometric level rises above frost line Extension of standpipe through embankment fill interrupts construction and causes inferior compaction Porous filter can plug owing to repeated water inflow and outflow
Pneumatic piezometer (Figure 2-6)	Short time lag Calibrated part of system accessible Minimum interference to construction No freezing problems	Readings somewhat operator-dependent Accuracy reduced if reading under a gas flow condition
Vibrating wire piezometer (Figure 2-10)	Easy to read Short time lag Minimum interference to construction Lead wire effects minimal No freezing problems Can readily be connected to a datalogger	Need for lightning protection should be evaluated
Multipoint piezometer (e.g. Figure 2-11)	Provides detailed pressure-depth measurements Unlimited number of measurement points Calibrated part of system accessible Great depth capability	Complex installation procedure Periodic manual readings only

* Diaphragm piezometer readings indicate the head above the piezometer, and the elevation of the piezometer must be measured or estimated if piezometric elevation is required. All diaphragm piezometers, except those provided with a vent to the atmosphere, are sensitive to barometric pressure changes.

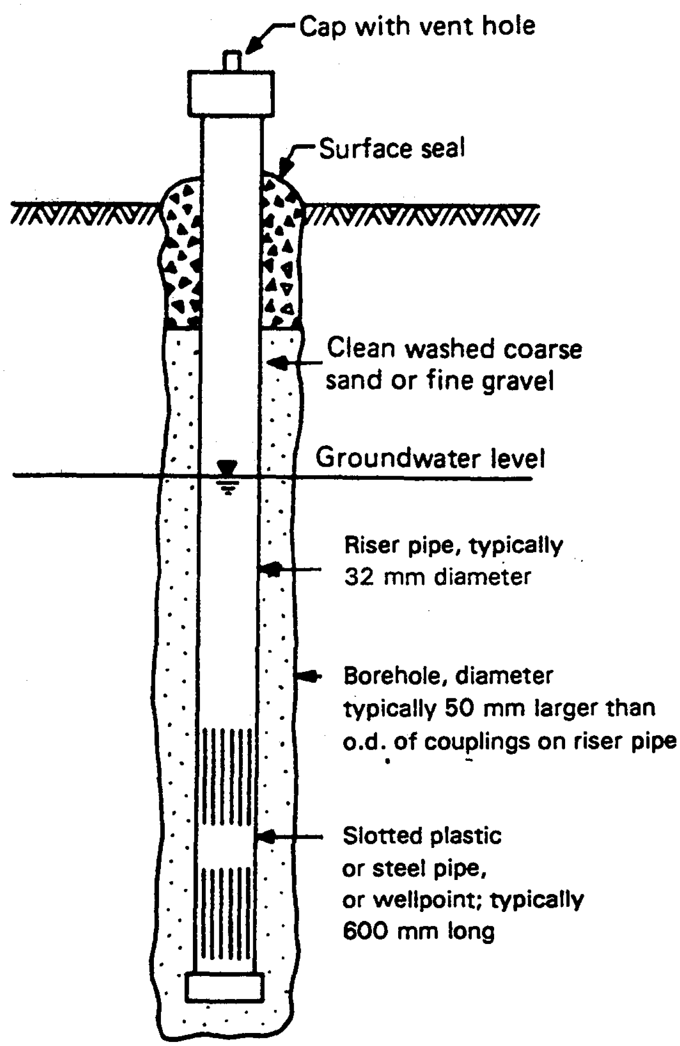


Figure 2-1: Schematic of Observation Well. (After Dunnicliff, 1988, 1993)

many different depths, rather than measuring at one or two points and assuming a straight line pressure–depth relation. Detailed pore water pressure data provided by recently developed multipoint piezometers (Section 2.6) have shown that pressure–depth profiles can be quite irregular and very different from that which one would infer from measurements at one or two points. Perhaps the major reason for the frequent and continued use of observation wells is that they can be installed by drillers without the participation of geotechnical personnel: this is generally not the case for installation of piezometers. Although observation wells are inexpensive, in the view of the author they are often misleading and they should be used only when the groundwater regime is well known. If the groundwater regime is well known, measurements may be unnecessary: therefore, a practitioner must have a strong argument in favor of an observation well before selecting this option.

If an observation well is installed through ground in which groundwater pressure does **not** increase uniformly with depth, the water level within the observation well is likely to correspond to the head in the most permeable zone and will usually be misleading. At sites where a contaminant exists in one aquifer, installation of an observation well also leads to contamination of other aquifers.

The term observation well should not be confused with monitoring well, which is a system for sampling and monitoring water quality in a particular aquifer, requiring an arrangement similar to an open standpipe piezometer.

2.3 OPEN STANDPIPE PIEZOMETERS

An open standpipe piezometer requires sealing off a porous filter element so that the instrument responds only to groundwater pressure around the filter element and not to groundwater pressures at other elevations. Piezometers can be installed in fill, sealed in boreholes (Figure 2-2), or pushed or driven into place.

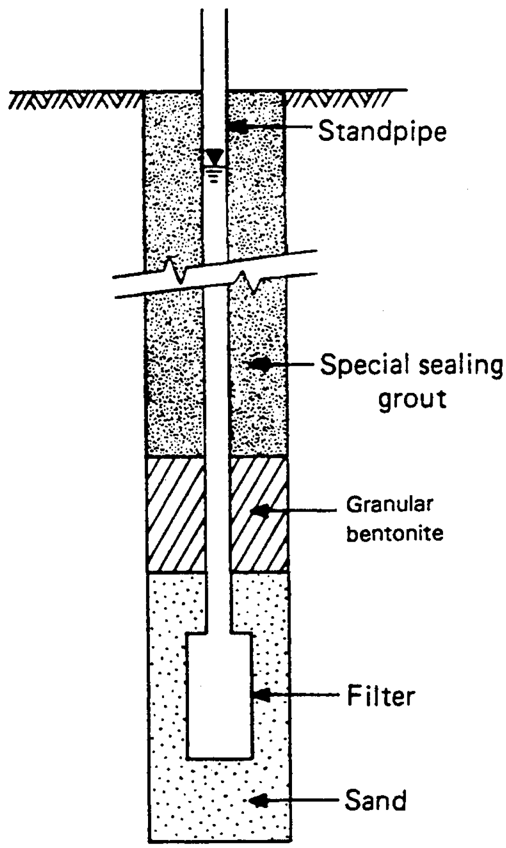


Figure 2-2: Schematic of Open Standpipe Piezometer Installed in a Borehole. (After Dunnicliff, 1988, 1993)

The components are identical in principle to components of an observation well, with the addition of seals. The water surface in the standpipe stabilizes at the piezometric elevation and is determined by sounding with a probe. Care must be taken to prevent rainwater runoff from entering open standpipes, and an appropriate roadway box can be used, ensuring that venting of the standpipe is not obstructed.

The open standpipe piezometer is also referred to as a Casagrande piezometer, after publication of measurement methods for monitoring pore water pressure during construction of Logan Airport in Boston. Preferred materials are a filter of high-density porous hydrophilic polyethylene (Figure 2-3), and a standpipe of **flush-coupled** Schedule 80 PVC or ABS pipe.

When threaded couplings are used, they should be of the self-sealing type shown in Figure 2-3. These special threads are undersized at the female end and create a watertight connection without any sealer, merely by tightening with vice grips or small pipe wrenches. Conventional tapered pipe threads are not suitable for flush couplings, and conventional square threads will not sustain significant internal water pressure unless O-rings are added.

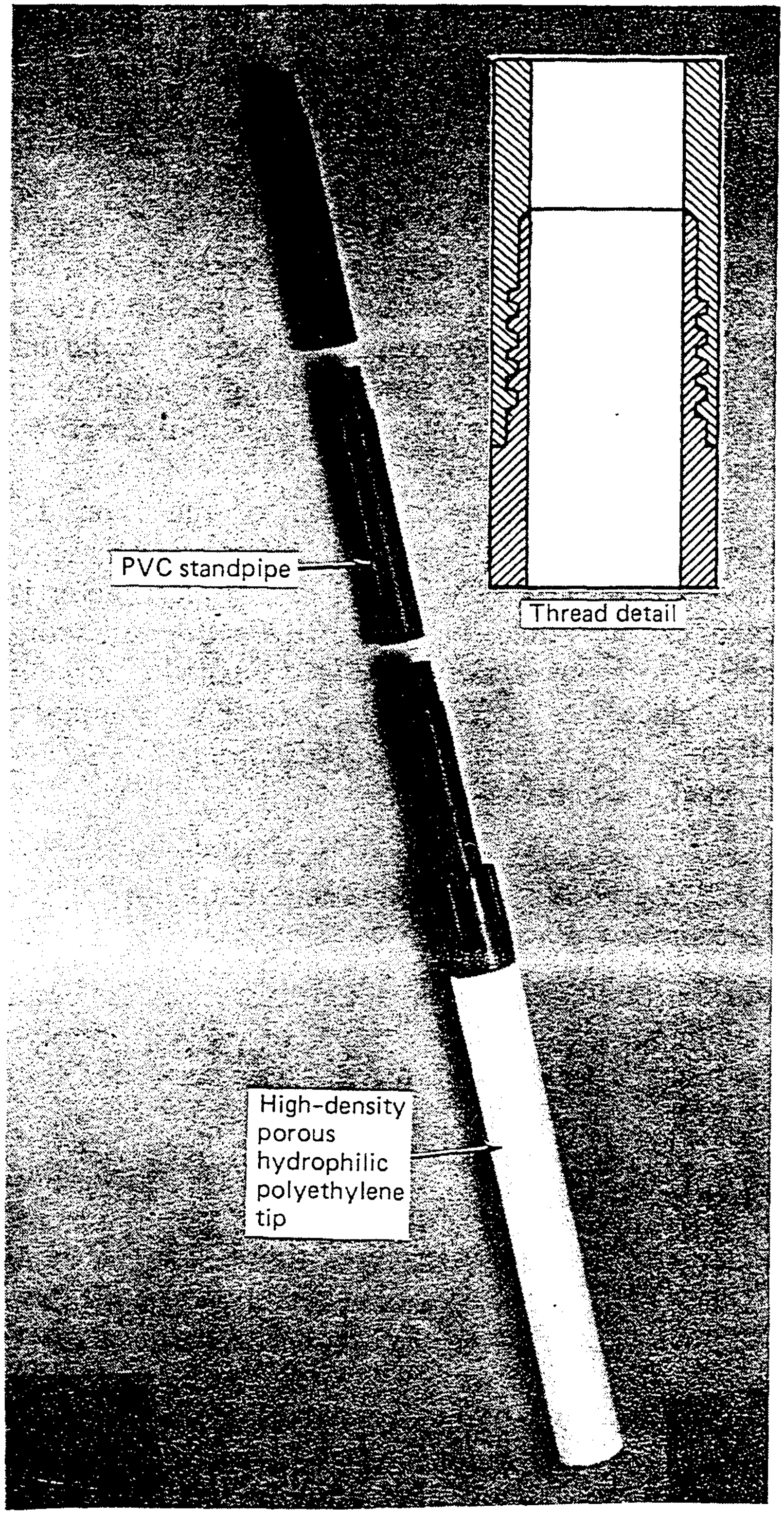


Figure 2-3: Open Standpipe Piezometer with Porous Polyethylene Filter and Self-Sealing Threaded PVC Standpipe. (Courtesy of Piezometer Research & Development, Bridgeport, CT)

Open standpipe piezometers can be packaged as push-in piezometers: the term push-in piezometer is used in this Module for piezometers that are connected to a pipe or drill rod and pushed or driven into place. A frequently used open standpipe push-in piezometer is the Geonor Model M-206 field piezometer (Figure 2-4). It was developed for use in soft sensitive clays in Norway and is pushed into place by attachment to EX-size drill rods.

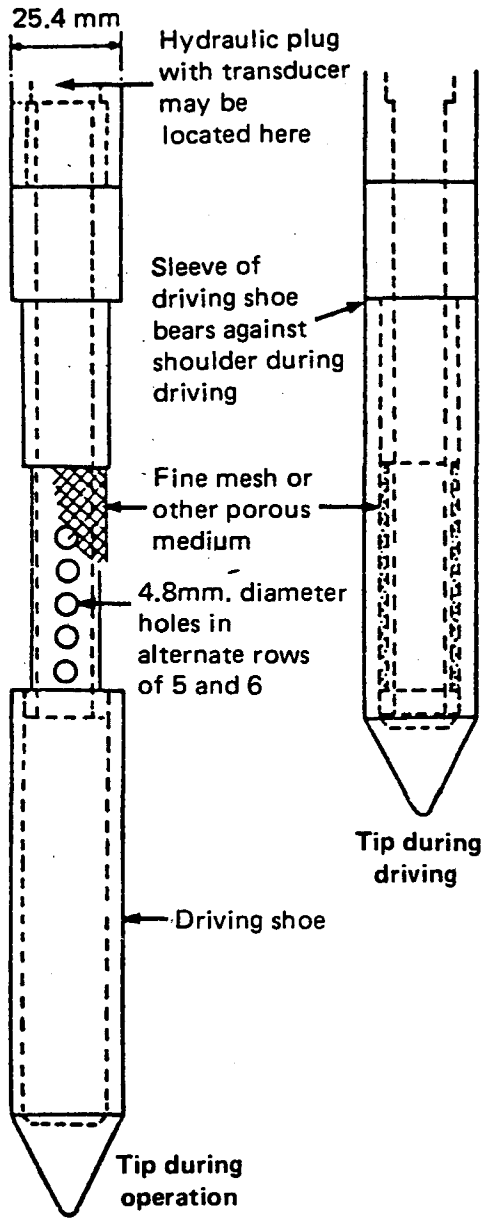


Figure 2-4: Geonor Model M-206 Field Piezometer. (Courtesy of Geonor A/S, Oslo, Norway and Geonor, Inc., Milford, PA)

The most common method for reading open standpipe piezometers is use of an electrical dipmeter (Figure 2-5), consisting of a two-conductor cable with a cylindrical stainless steel weight at its lower end. The weight is divided electrically into two parts, with a plastic bushing between, and one conductor is connected to each part. The upper end of the cable is connected to a battery and either an indicator light, buzzer, or ammeter. When the probe is lowered within the standpipe and encounters the water surface, the electrical circuit is completed through the water and the surface indicator is actuated.

Figure 2-5: Electrical Dipmeter. (Courtesy of R. S. Technical Instruments Ltd., Coquitlam, BC)

2.4 PNEUMATIC PIEZOMETERS

Figure 2-6 shows the basic arrangement of a pneumatic piezometer. An increasing gas pressure is applied to the inlet tube and, while the gas pressure is less than the pore water pressure, it merely builds up in the inlet tube. When the gas pressure exceeds the pore water pressure, the diaphragm deflects, allowing gas to circulate behind the diaphragm into the outlet tube, and flow is recognized either by noting a peak on the pressure gage, or by hearing the gas flow emerging from the outlet tube. The gas supply is then shut off at the inlet valve, and any excess pressure in the tubes bleeds away, such that the diaphragm returns to its original position when the pressure in the inlet tube equals the pore water pressure. This pressure is read on a Bourdon tube or electrical pressure gage.

The above description of operation is for the case when the piezometer is read under a condition of no gas flow. This requires absolutely gas-tight connections in the system, including the quick-connects between the installed tubing and the portable readout unit. Some manufacturers include an arrangement for reading the piezometer under a gas flow condition, by incorporating a gas flow controller and a gas flow meter in the gas supply tube. However, accuracy is reduced because small changes in gas flow create changes in head losses, and hence affect the readings.

As for the open standpipe piezometer, pneumatic piezometers can be packaged as push-in piezometers. A special version of push-in pneumatic piezometer, manufactured by Slope Indicator Company, is shown in Figure 2-7. This version is suitable for use in very soft clay (standard penetration test blowcount of less than about 2 blows per foot).

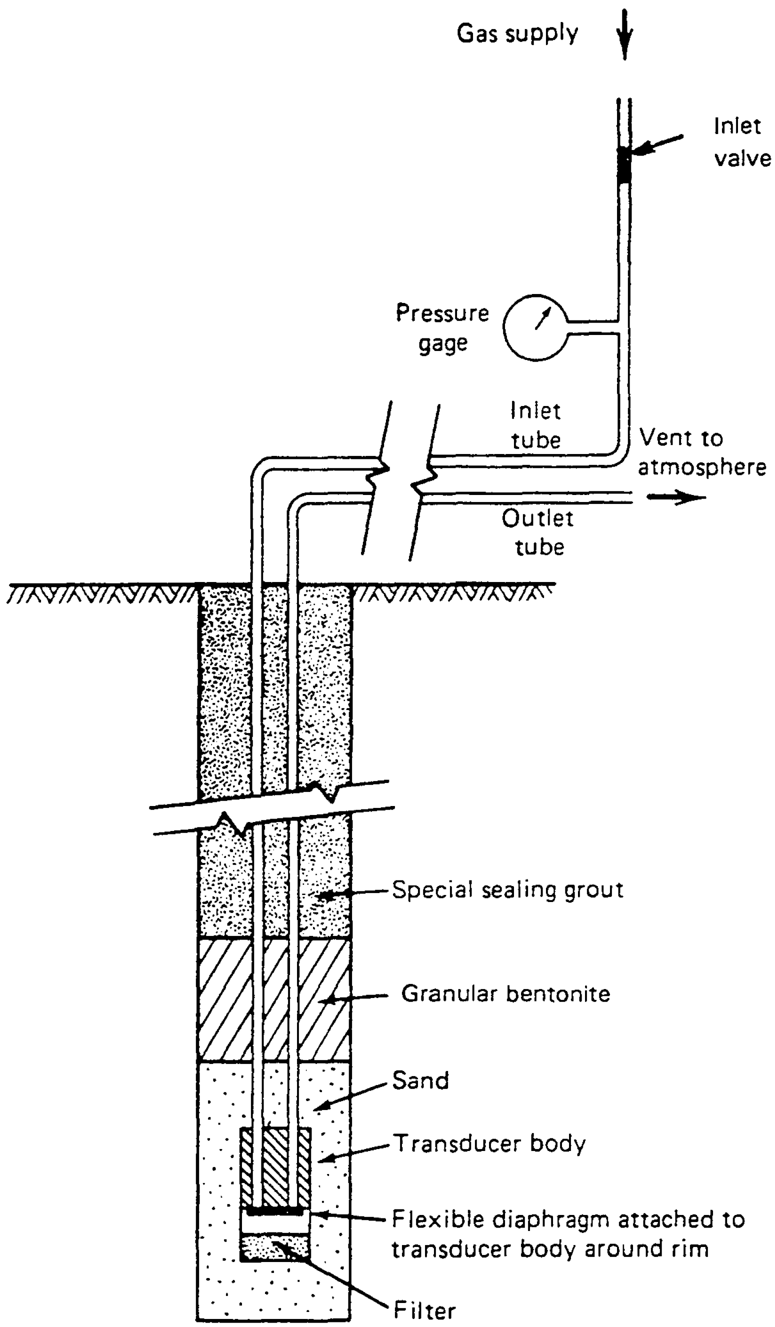


Figure 2-6: Schematic of Pneumatic Piezometer Installed in a Borehole, Read Under a Condition of No Gas Flow. (After Dunnicliff, 1988, 1993)

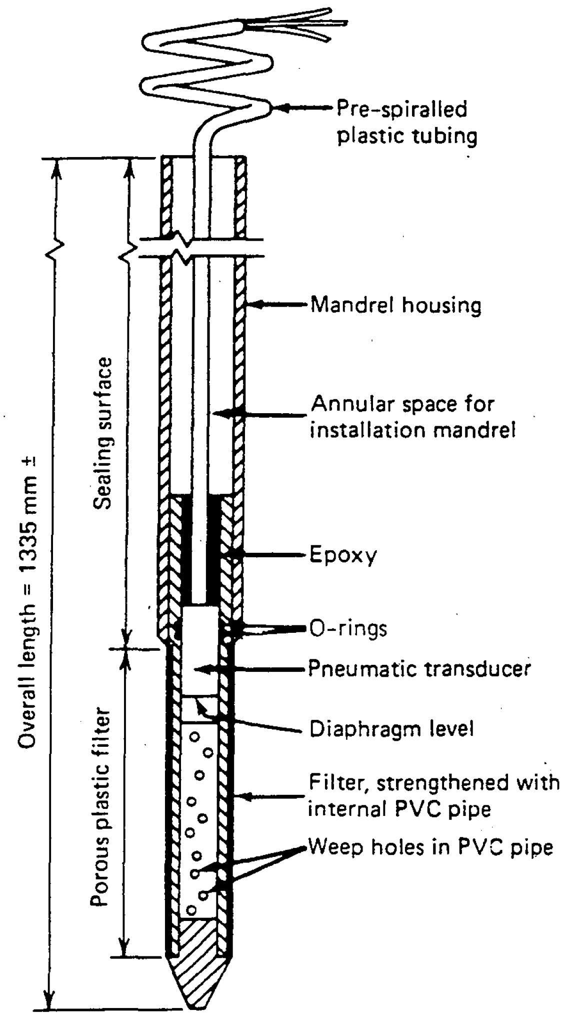


Figure 2-7: Pneumatic Piezometer for Installation by Pushing in Place below Bottom of a Borehole in Soft Clay. (after Dunnicliff, 1988, 1993)

2.5 VIBRATING WIRE PIEZOMETERS

The vibrating wire transducer, illustrated as a surface-mounted strain gage, is shown in Figure 2-8. A length of steel wire is clamped at its ends and tensioned so that it is free to vibrate at its natural frequency. As with a piano string, the frequency of vibration varies with the wire tension, and thus with small relative movements between the two end clamps. The wire can therefore be used as a strain gage by plucking the wire, measuring natural frequency, and relating frequency change to strain. The wire is plucked magnetically by an electrical coil attached near the wire at its midpoint, and either this same coil or a second coil is used to measure the frequency of vibration by using a frequency counter.

When used as a transducer in a vibrating wire piezometer, the arrangement is as shown in Figure 2-9. The piezometer has a metallic diaphragm separating the pore water from the measuring system. The tensioned wire is attached to the midpoint of the diaphragm such that deflection of the diaphragm causes changes in wire tension, and measurements are made as described above.

Vibrating wire piezometers can be packaged as push-in piezometers. A commercial version is shown in Figure 2-10. After pushing, the upper drill rod is disconnected at the left/right adaptor and withdrawn, while injecting soft grout down the upper drill rod. The reaction wings allow the disconnect to be made easily.

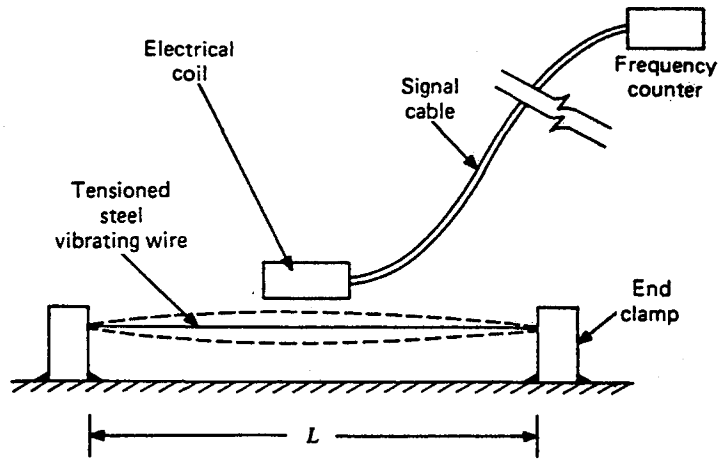


Figure 2-8: Schematic of Surface-Mounted Vibrating Wire Strain Gage. (After Dunnicliff, 1988, 1993)

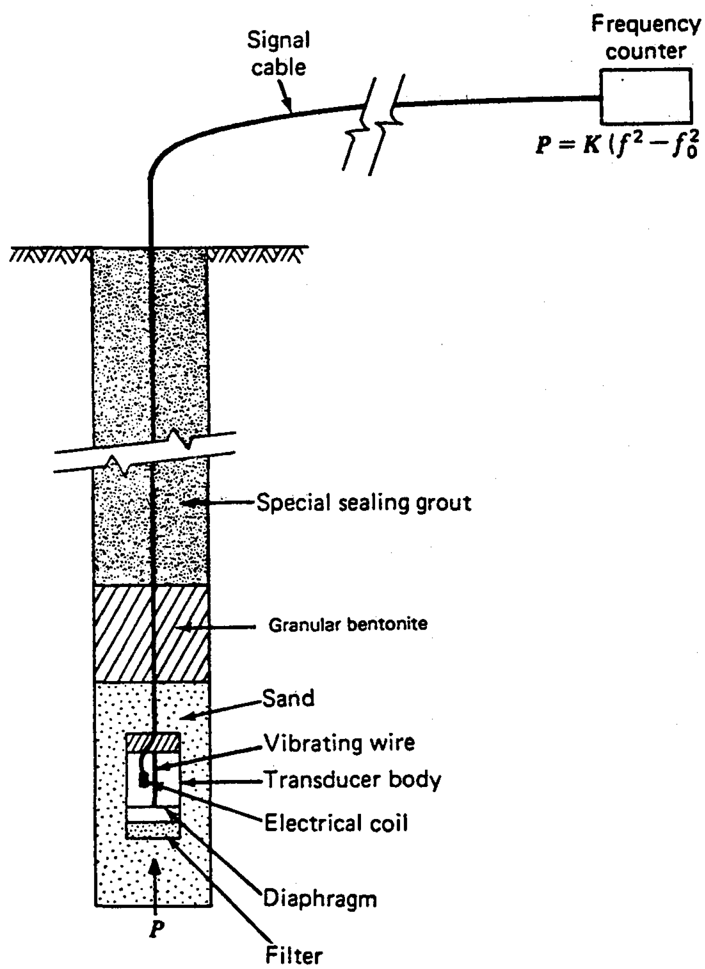


Figure 2-9: Schematic of Vibrating Wire Piezometer Installed in a Borehole. (After Dunnicliff, 1988, 1993)

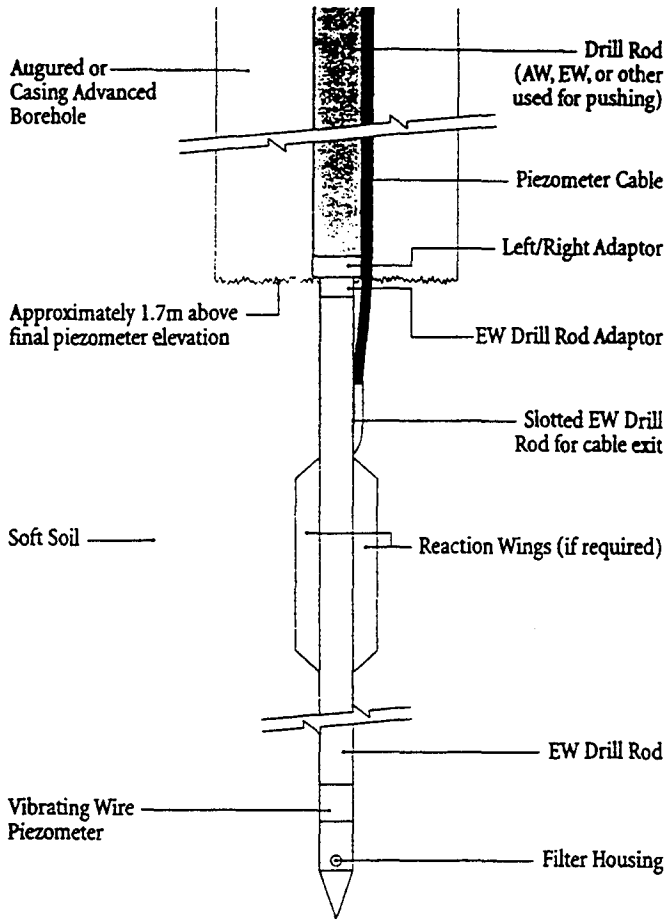


Figure 2-10: Vibrating Wire Piezometer for Installation by Pushing in Place below Bottom of Borehole in Soft Clay. (Courtesy of Geokon, Inc., Lebanon, NH)

A special version of vibrating wire piezometer, referred to as a vibrating strip piezometer, is manufactured by Slope Indicator Company. It differs from the vibrating wire piezometer shown in Figure 2-9 in the following ways:

- The vibrating wire is replaced by a narrow steel strip
- An increase in pore water pressure increases, rather than decreases the wire tension
- The arrangement is based on the principal of a force transducer rather than a strain transducer, thereby reducing any possible errors caused by temperature changes and by zero drift of mechanical components

The vibrating strip piezometer is somewhat more expensive than a vibrating wire piezometer. Also, it is more delicate and must therefore be handled with more care. However, when temperature change is an issue or where highest possible accuracy is required (for example, when monitoring hydraulic gradients to determine in which direction pollutants are moving, or when monitoring water levels in weirs), the vibrating strip piezometer can be considered.

2.6 MULTIPOINT PIEZOMETERS

When there is a need to determine groundwater pressure of several levels within the ground, for example when evaluating the impact of groundwater pressures on landslides, two options are available. First, one or two piezometers can be installed in each of several boreholes (not more than two: see Section 2.8.5), but this entails large drilling costs. Second a multipoint piezometer can be used.

A pipe is installed and sealed within a borehole and a movable probe periodically inserted within the pipe. The development of multipoint piezometers has created a significant breakthrough in the measurement of groundwater pressure. They allow a detailed pressure-depth profile to be defined, with an almost unlimited number of measurement points. Although they have a high initial cost, their use may result in substantial cost savings by ensuring a reliable knowledge of the pressure-depth profile.

The most frequently used multipoint piezometer is the Westbay Instruments Ltd MP (multiple piezometer). Installations can be made in soil and rock. Figure 2-11 shows the general arrangement and the probe. The system consists of pipe, couplings and packers permanently installed in a borehole, a portable pressure measurement probe, and installation tools. Measurement port couplings are installed in the pipe wherever groundwater pressure measurements are desired. Each of these couplings has a hole through its wall, with a filter on the outside and spring-loaded check value on the inside. The remainder of the pipe is connected with sealed couplings, and a packer is installed around the pipe above and below each measurement port coupling. The assembly is lowered into the borehole, and the packers inflated one at a time with water, using a probe temporarily inserted within the pipe. To take readings, a pressure measurement probe is lowered within the pipe to locate a measurement port, jacked against the opposite wall of the pipe to open a check valve, and a measurement made. The procedure is repeated at each measurement port.

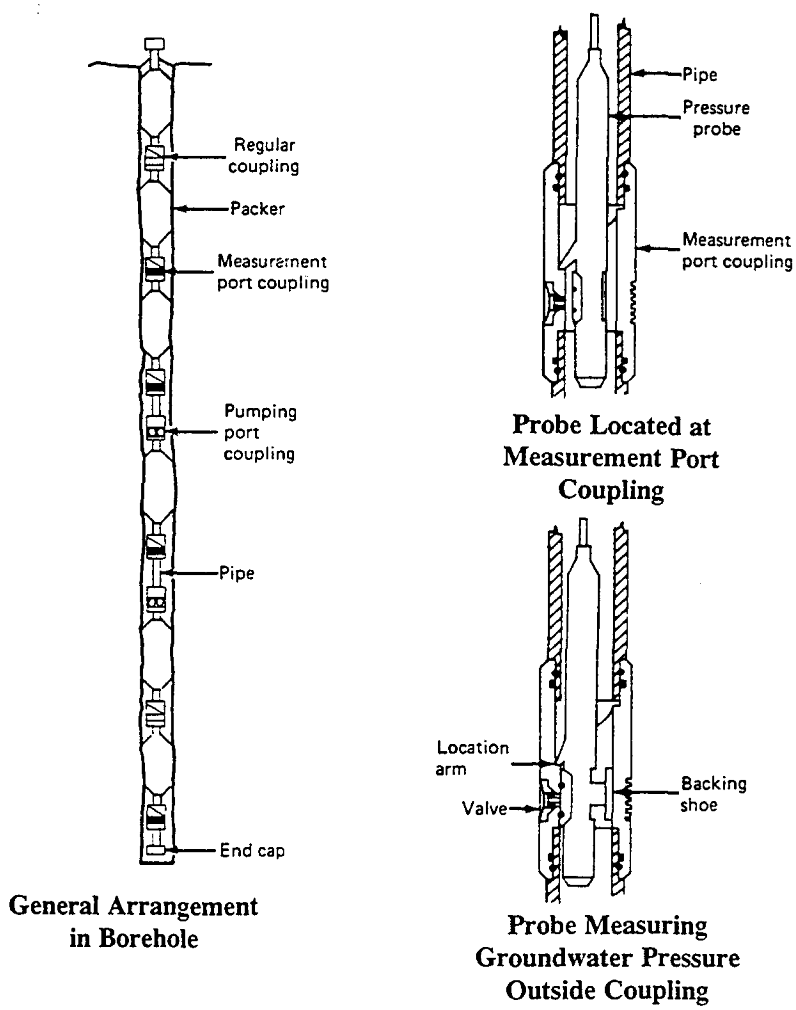


Figure 2-11: Multiple Piezometer (MP) System® (Courtesy of Westbay Instruments, Inc., North Vancouver, BC, Canada)

2.7 RECOMMENDED INSTRUMENTS FOR MEASURING GROUNDWATER PRESSURE

General guidelines on the selection of instruments are given in Chapter 6 and a recommendation is made to maximize reliability by using the simplest instrument that will achieve the purpose.

Reliability and durability are often of greater importance than sensitivity and high accuracy. The fact that the actual head may be in error by 400 mm as a result of time lag may not matter in some cases provided the piezometer is functioning properly. If a malfunction occurs, it is of little importance that the apparent head can be recorded to 10 mm. It is generally believed that if the instrument is installed correctly, that it is functioning, and that no time lag remains, the accuracy of all piezometers can be within 150 mm of water head. High-accuracy requirements of course necessitate selection of high-accuracy components, such as test quality pressure gages in pneumatic readout units.

As indicated in Section 2.2, applications for observation wells are limited to monitoring in continuously permeable ground in which groundwater pressure increases uniformly with depth. This condition can rarely be assumed.

For single-point measurements of pore or joint water pressure, an open standpipe piezometer is the first choice and should be used provided that the time lag and other limitations listed in Table 2-1 are acceptable. When these limitations are unacceptable, a choice must be made among the remaining piezometer types. The choice between pneumatic and vibrating wire piezometers will depend on the factors in Table 2-1, on the user's own confidence in one or the other type, and on a comparison of cost of the total monitoring program.

When detailed monitoring of the pressure–depth profile is required, multipoint piezometers will normally be the instruments of choice.

When the economics of alternative piezometers are being evaluated, the **total** cost should be determined, considering costs of instrument procurement, calibration, installation, maintenance, monitoring, and data processing. The cost of the instrument itself is rarely the controlling factor and should never dominate the choice.

2.8 MISCELLANEOUS ISSUES ASSOCIATED WITH MEASUREMENT OF GROUND-WATER PRESSURE

2.8.1 Hydrodynamic Time Lag

When a piezometer is installed and groundwater pressure changes, the time required for water to flow into or out of the piezometer to effect equalization is called the hydrodynamic time lag. It is dependent primarily on the type and dimensions of the piezometer and the permeability of the ground. Open standpipe piezometers have a much greater hydrodynamic time lag than diaphragm piezometers because a much greater movement of pore or joint water is involved. The term slow response time is used to describe a long hydrodynamic time lag.

The order of magnitude of the time required for 90% response of several types of piezometers installed in homogeneous soils can be obtained from Figure 2-12, in which the Geonor open piezometer is the push-in instrument shown in Figure 2-4. The 90% response is considered adequate for many practical purposes, and of course the time for 100% response is infinite. The response time of open standpipe piezometers can be estimated from equations given by Penman (1960). For example,

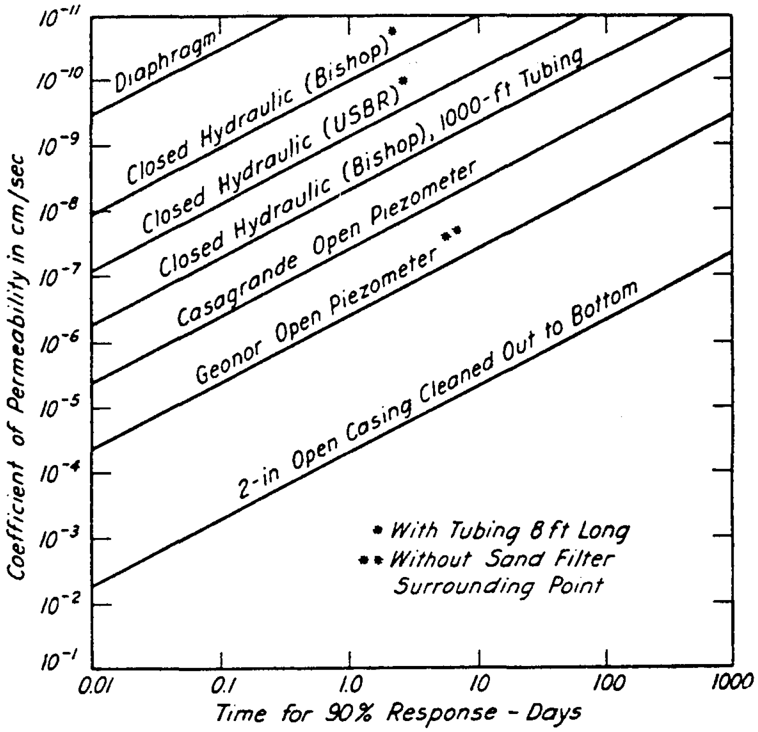


Figure 2-12: Approximate Response Times for Various Types of Piezometer (After Terzaghi and Peck, 1967) Reprinted by Permission of John Wiley & Sons, Inc.

$$t = 3.3 \times 10^{-6} \; \frac{d^2 \ln\left[L/D + \sqrt{1 + (L/D)^2}\right]}{kL}$$

where
- t = time required for 90% response in days,
- d = inside diameter of standpipe in centimeters (cm),
- L = length of intake filter (or sand zone around the filter) in centimeters (cm),
- D = diameter of intake filter (or sand zone) in centimeters (cm),
- k = permeability of soil in centimeters per second (cm/sec).

Figure 2-13 gives an example for an open standpipe piezometer surrounded by a sand zone.

The significance of hydrodynamic time lag depends primarily on the purpose of the measurements and on the anticipated fluctuations of the groundwater pressure. For example, if measurements are made to determine joint water pressure in a rock slope in which pressure fluctuations are not likely to be significant, an open standpipe piezometer may be suitable. If an embankment is being constructed on soft ground, and piezometers are used to monitor gain in strength, the rate of embankment placement and anticipated rates of pore water pressure dissipation will enter into judgments concerning time lag. If pore water pressure measurements are made with a push-in piezometer by leaving it in place for a short time and then pushing it deeper for additional measurements, a long time lag will not be acceptable. If the groundwater pressure is subject to daily fluctuations, for example near the ocean, a time lag of more than a few hours would obscure real variations in pressure and the measurements might have no value. Time lag criteria should be evaluated on a case-by-case basis.

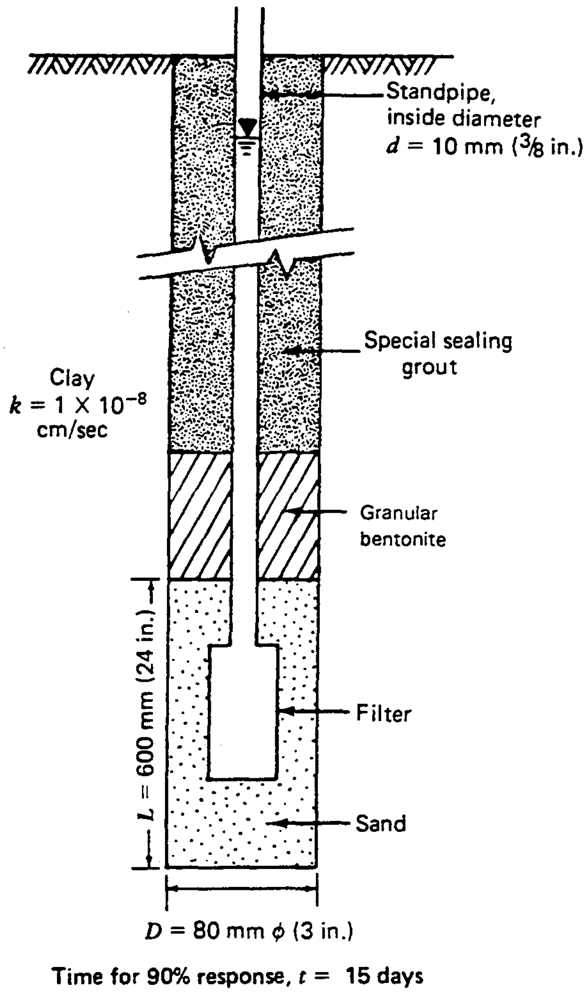


Figure 2-13: Example of Time Lag for Open Standpipe Piezometer. (After Dunnicliff, 1988, 1993)

2.8.2 Types of Filter

All piezometers include an intake filter. The filter separates the pore fluid from the structure of the soil in which the piezometer is installed and must be strong enough to avoid damage during installation and to resist the total stresses without undue deformation. Filters can be classified in two general categories: high air entry and low air entry.

High air entry filters have low permeability (hence the term tends to be confusing), and are applicable only for ensuring that pore water pressure rather than pore gas pressure is measured in unsaturated fills, such as the compacted clay cores of embankment dams. They have no application for monitoring pore water pressure in saturated soils, hence should not be used for applications discussed in this Module. Low air entry (high permeability) filters should be used, typically with a pore diameter of 0.02–0.08 mm (20–80 microns).

Filters, and the cavities between filters and diaphragms, should always be saturated with water prior to installation. If this is not done, response time will be increased. Filters can be saturated by forcing water through the pores. A filter on an open standpipe piezometer can be saturated by immersing the filter in water and applying a suction to the standpipe, for example, with a bilge pump: when a piezometer is installed in a borehole, this can usually be done within the borehole. If a filter on a diaphragm piezometer can be removed from the piezometer, it can be saturated by disconnecting it from the piezometer, forcing water through the pores, filling the cavity with water and reconnecting the piezometer. If the filter cannot be removed, a vacuum should be applied to the dry filter (having determined from the manufacturer that the instrument will not be damaged by the vacuum) and the filter then allowed to flood with water as the vacuum is released. The cycle should be repeated two or three times until no gas bubbles appear from the filter as the vacuum is applied.

Filters should preferably be saturated at the site rather than before shipment. When filters are saturated before shipment and shipped in a sealed saturated container, there is a risk that they may be damaged if freezing occurs in transit. After saturation, filters should be stored under water. When transferring a saturated filter from one container to another or from a container to a water-filled borehole, a water-filled plastic bag or rubber sheath can be placed around the filter and later removed by tearing.

2.8.3 Granular Bentonite Seal Above Piezometers

Compressed dry bentonite pellets (formed by compressing powdered bentonite into small spherical balls) have been shown to form an adequate seal provided that they can be inserted in the borehole at the required location. However, most people who have installed piezometers in boreholes with bentonite pellets have experienced the problem of pellets becoming stuck part way down the borehole, and the problem is aggravated if pellets are poured too fast. There is no reason to experience this problem, because preferred materials are available from various suppliers of bentonitic grouts. Pit-run granular bentonite (material direct from the bentonite pit, rather than pellets that have been prepared from powder) does not become sticky when immersed in water, as quickly as pellets, and falls down water-filled boreholes much more readily. 10 mm size is appropriate, and typically a 1.2 m height of granular bentonite seal is used. Two commercial sources (Dunnicliff, 1994) are:

- Enviroplug Medium - Wyo-Ben, Inc., P.O. Box 1979, Billings, MT 59103. Tel. (800) 548-7055.
- Holeplug, 3/8 in. Size - Baroid Drilling Fluids, Inc., P.O. Box 1675, Houston, TX 77251. Tel. (713) 987-6057.

2.8.4 Grout Above Granular Bentonite Seal

The "special sealing grout" shown on Figures 2-2, 2-6 and 2-10 needs to have low permeability, and high compressibility. It is possible to create such grouts by mixing powdered bentonite, cement and water, but it is difficult to mix these ingredients into an appropriate creamy consistency, and the product is highly dependent on the sequence of adding the ingredients and also on the chemistry of the water. A preferred option is to use a single-component proprietary bentonitic grout that set up to the consistency of soft clay. One commercial source (Dunnicliff, 1994) is:

- Benseal/EZ-Mud Slurry, from Baroid, as above. Use 135 lbs of EZ-Mud per 100 gallons of water, not 150 lbs as in the Baroid product information.

2.8.5 Sounding Hammer

A tamping hammer is often used to tamp the bentonite seal and to center the piezometer tubes, wires or standpipe. In fact the tamping hammer serves three more purposes: to measure depths, to center the piezometer in the borehole, and to assist in dislodging any bridges of backfill material that may form in the borehole. However, granular bentonite should **not** be tamped, because a very compact zone of bentonite can cause either excessive pore water pressures as a result of swelling pressures, or reduced pore water pressures as hydration of the bentonite progresses, either process causing pore water pressure measurements to be incorrect for a significant time, perhaps several days or weeks. The rules are: *Install piezometers as early as possible, well before data are needed. Make sure that the pellets are where they should be, and leave them alone. Do not tamp them.*

Despite the above rules, the hammer is required for the other three purposes. By persisting with the term tamping hammer, users are encouraged to use the device for tamping, and the author proposes the term "sounding hammer."

Chapter 12 describes installation procedures for piezometers in boreholes, and also for use of a sounding hammer. This section includes guidelines on materials for the sounding hammer.

When used around a single piezometer lead or a multiple-lead with a single jacket, the sounding hammer should be a steel cylinder with outside diameter 8–12 mm less than the inside diameter of the casing or augers, inside diameter approximately 5 mm larger than the outside diameter of the piezometer lead, a length of not less than 600 mm, and a weight of approximately 7 kg. The diameter limits are necessary so that sand and granular bentonite are forced to the bottom of the borehole, the length limit so that the hammer does not become angled and stuck in the borehole, and the weight limit for handling purposes. For small-diameter boreholes, the hammer can be made from solid steel, but usually it is necessary to weld one pipe inside another, with donut-shaped top and bottom plates, to keep the weight adequately low. If the required outside diameter cannot be achieved by using standard steel pipe, the next smaller pipe size can be used and the top and bottom plates made to protrude beyond the outside diameter by the required amount. All corners on the hammer should be thoroughly rounded. A bridle should be attached to the top of the hammer and a cable attached to the bridle with a secure smooth connection that will not damage the piezometer lead. A 3 mm stainless steel airplane cable, covered with a plastic sheath, is ideal for the bridle and cable. The cable should be graduated, using a hot stamping machine, with zero at the bottom of the hammer. Figure 2-14 shows a typical sounding hammer.

The arrangement shown in Figure 2-14 requires that the sounding hammer is inserted over the entire length of piezometer lead: a laborious task if leads are to be carried for a significant distance to a remote terminal. This can be overcome by milling a slot along one side of the cylinder so that the lead can be inserted sideways. Removable top and bottom plates, each with a slot, are attached to the cylinder with their slots rotated 180 degrees with respect to the slot in the cylinder, so that they can also be inserted sideways but can retain the lead within the cylinder.

When a piezometer has two separate leads, for example, a pneumatic piezometer for which a common tubing jacket has not been used, the tubes must be separated by the sounding hammer. If the arrangement for a single lead is used, the two leads will be forced into close contact and a seal may not form along the contact. Separation is achieved by use of a bottom plate with two holes or by welding two separate small-diameter pipes within a larger pipe. A clear distance of at least 20 mm should be created between the two leads. The same need arises, above the upper piezometer, when two piezometers are installed in the same borehole.

Recognizing the problem of sealing between two leads, the maximum number of piezometers in a single borehole should be two. There should be at least 6 m of granular bentonite seal between the two piezometers.

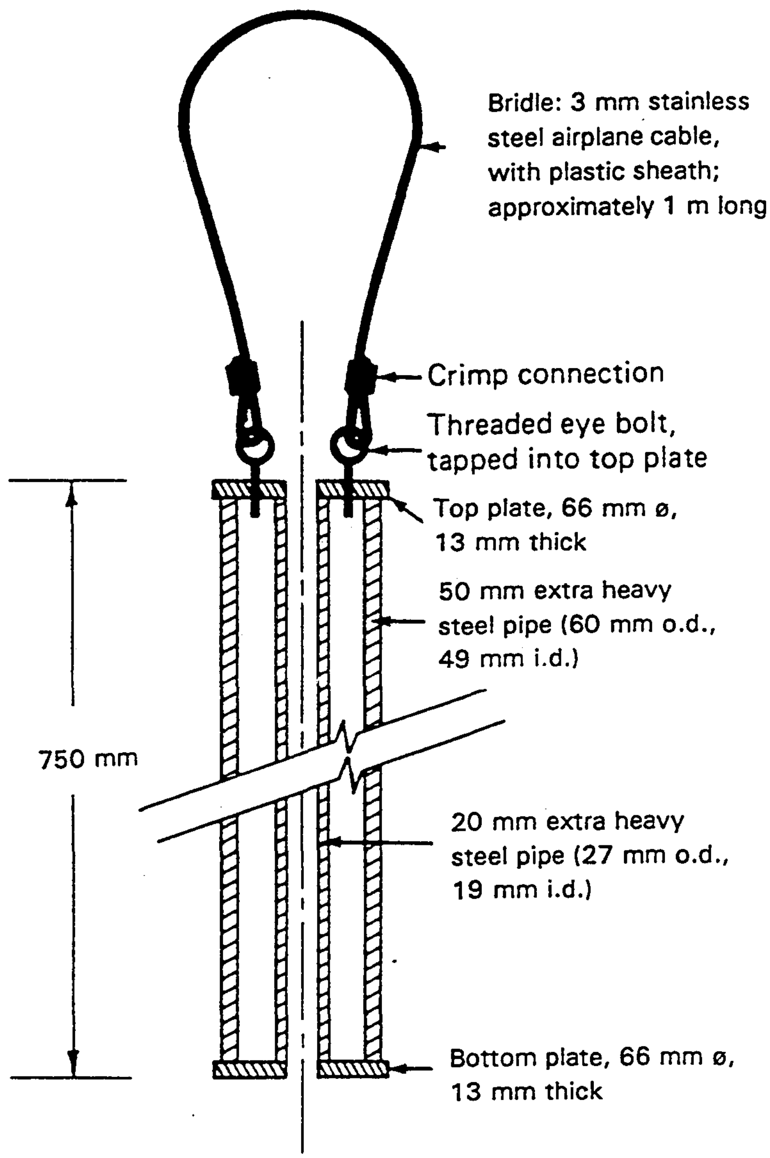


Figure 2-14: Typical Sounding Hammer for Use Inside NW Flush-Joint Casing (Casing i.d. 76 mm). Note: (1) Plates and Pipe Welded Together; (2) All Corners Well Rounded; (3) Weight Approximately 7 kg; (4) Graduated Cable Attached to Bridle with Loop and Crimp Connection. (After Dunnicliff, 1988, 1993)

CHAPTER 3.0
OVERVIEW OF HARDWARE FOR MEASUREMENT OF DEFORMATION

3.1 INTRODUCTION

Instruments for measuring deformation can be grouped in the categories listed in Table 3-1. Definitions of each category, together with an indication of typical applications, are given in later sections of this chapter.

The following sections provide an overview of hardware for measurement of deformation.

TABLE 3-1
CATEGORIES OF INSTRUMENTS FOR MEASURING DEFORMATION

Category	Type of Measured Deformation						Section
	Horizontal	Vertical	Axial	Rotational	Surface	Subsurface	
Surveying Methods	✓	✓	✓		✓		3.2
Crack Gages	✓	✓	✓		✓		3.3
Convergence Gages	✓	✓	✓		✓		3.4
Tiltmeters				✓	✓	✓	3.5
Probe Extensometers	✓	✓	✓			✓	3.6
Settlement Platforms		✓				✓	3.7
Subsurface Settlement Points		✓				✓	3.8
Fixed Borehole Extensometers	✓	✓	✓			✓	3.9
Series Extensometers	✓	✓	✓			✓	3.10
Inclinometers	✓					✓	3.11
Horizontal Inclinometers		✓				✓	3.12
In-Place Inclinometers	✓	✓			✓	✓	3.13
Shear Plane Indicators	✓					✓	3.14
Liquid Level Gages		✓				✓	3.15
Telltales	✓	✓	✓		✓	✓	3.16
Convergence Gages for Slurry Trenches	✓					✓	3.17
Metallic Time Domain Reflectometry	✓	✓	✓		✓	✓	3.18
Acoustic Emission Monitoring	✓	✓	✓			✓	3.19

Table 3-2 indicates major advantages and limitations, and approximate accuracies.

TABLE 3-2
INSTRUMENTS FOR MEASURING DEFORMATION

Instrument Type	Advantages	Limitations	Approximate Accuracy
Mechanical crack gages	Inexpensive	Span often limited Manual reading	±0.005 to 5 mm, depending on type
Electrical crack gages	Can be read remotely and automatically	Range and span often limited	±0.005 to 0.15 mm
Convergence gages		Manual reading	±0.15 mm in 10 m span, decreasing with increasing span
Tiltmeters	Can be read remotely and automatically	Temperature compensation and correction required	±3 to 50 arc-seconds
Probe extensometers	Large range Settlement/heave measurements at many depths along one installation	Manual reading	±3 to 5 mm
Settlement platforms	Inexpensive	Requires survey crew for reading Pipe must be carried up through fill	±3 to 25 mm
Subsurface settlement points		Requires survey crew for reading Only single measurement of heave/settlement	±3 to 25 mm
Fixed borehole extensometers	Deformation measurements at many depths along one installation Can be read remotely and automatically	Limited range	±0.05 to 0.15 mm
Series extensometers	Can be recovered for re-use Can be read remotely and automatically	Limited range	±0.05 to 0.15 mm
Inclinometers and horizontal inclinometers	Full profile of deformation measurements along casing	Manual reading	±1 to 15 mm in 30 m
In-place inclinometers	Can be read remotely and automatically	Check-sum data not available	±0.5 to 1.0 mm in 3 m
Shear plane indicators	Inexpensive	Very crude	Depends on hardware details
Liquid level gages	No interference to construction	Error prone, due to temperature changes and discontinuity in liquid	±0.1 to 40 mm

3.2 SURVEYING METHODS

Surveying methods are used to monitor the magnitude and rate of horizontal and vertical deformations of structures, the ground surface, and accessible parts of subsurface instruments in a wide variety of construction situations. Frequently, these methods are entirely adequate for performance monitoring, and geotechnical instruments are required only if greater accuracy is required or if measuring points (points on which surveying observations are made) are inaccessible to surveying methods, as in the case for subsurface measurements. In general, whenever geotechnical instruments are used to monitor deformation, surveying methods are also used to relate measurements to a reference datum: a benchmark for vertical deformation monitoring or a horizontal control station for horizontal deformation monitoring.

Surveyors who work on construction sites often have little experience with the accuracies required for deformation monitoring, and a well-trained survey crew is essential when maximum accuracy is required. Measurement accuracy is controlled by the choice and quality of surveying technique and by characteristics of reference datums and measuring points. Survey instrument technology is well established, and most reputable manufacturers include a statement of accuracy in their instrument specifications, which can be relied on if the instrument is calibrated and operated in accordance with instructions.

A detailed discussion of surveying methods is beyond the scope of this chapter, and readers should make use of information provided by manufacturers of surveying equipment. Chrzanowski (1993) provides descriptions of modern surveying techniques. Greening plans to write an article describing current techniques, which is expected to be published in Geotechnical News during 1997 and 1998. Dunnicliff (1996a) describes precise measurements with a digital level and a bar coded staff.

3.3 CRACK GAGES

Crack gages (sometimes called jointmeters) are typically used for monitoring tension cracks behind slopes and for monitoring cracks in concrete or other structures, or joints or faults in rock.

3.3.1 Mechanical Crack Gages

There are numerous mechanical devices available for monitoring the width of cracks. The most common methods are described in the following subsections. All methods require access to the gage location for monitoring.

Pins and Tape

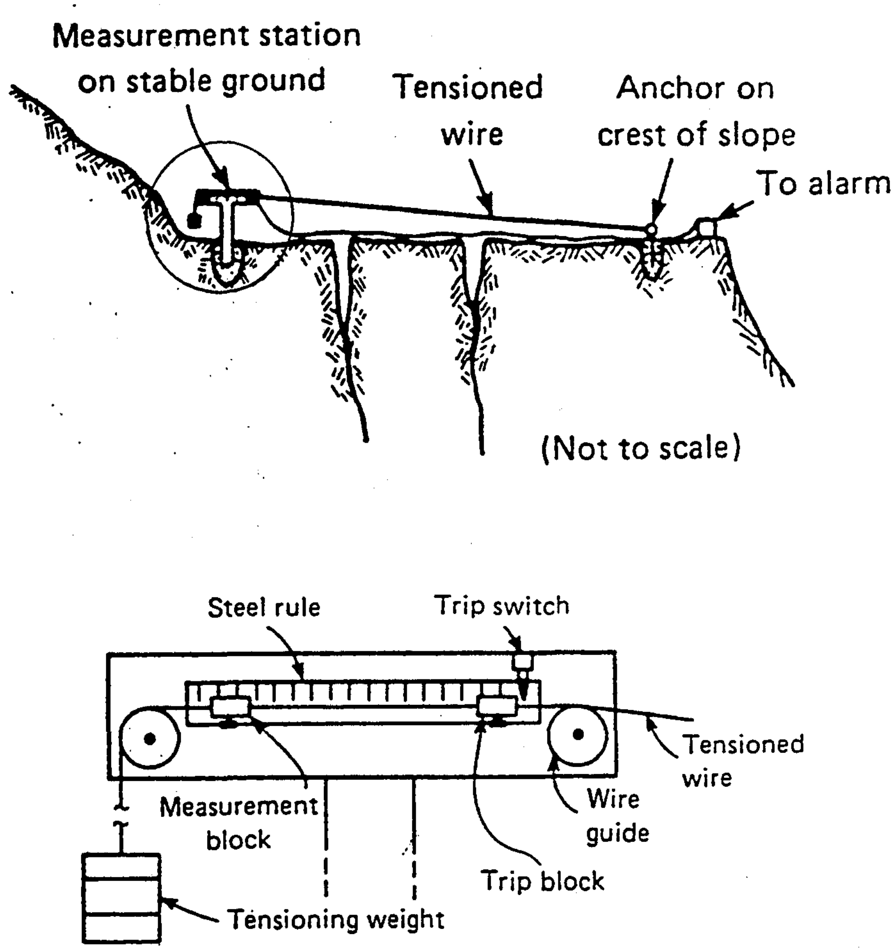
A pin is set on each side of a discontinuity and separation monitored using a steel tape. The type and dimensions of the pins and the fixing system to be used should be appropriate to the condition of the ground or structure to be monitored, to ensure that the pins are rigidly attached to the surface and will remain attached throughout the monitoring program.

Pins and Steel Rule or Calipers

Pins are set as described previously and separation monitored using a steel rule or calipers. Measurements are more precise than when using a tape, but the span is limited.

Pins and Tensioned Wire, Using Weight

Figure 3-1 illustrates the use of a pin and tensioned wire gage for monitoring tension cracks at the top of



Measurement Station

Figure 3-1: Mechanical Crack Gage, Using Pins and Tensioned Wire. (After Hoek and Bray, 1981)

a rock slope. A wire is stretched across the discontinuities between an anchor on one side and a pulley mounted on a measurement station on the other side. A weight on the wire below the pulley maintains tension. A scale is attached to the measurement station behind the wire, and a measurement block is fixed to the wire. Observation of the position of the measurement block with respect to the scale provides movement data. A trip block can be added to the wire, arranged to contact a trip switch on the scale when a predetermined movement occurs. This can be connected to an alarm if required.

Grid Crack Monitor

The grid crack monitor, also called a calibrated crack monitor and calibrated telltale, consists of two overlapping transparent plastic plates, one mounted on each side of the discontinuity. As shown in Figure 3-2, crossed cursor lines on the upper plate overlay a graduated grid on the lower plate. Movement is determined by observing the position of the cross on the upper plate with respect to the grid.

Mechanical Strain Gage

Surface-mounted mechanical strain gages (Chapter 5) can be used for measurements across discontinuities.

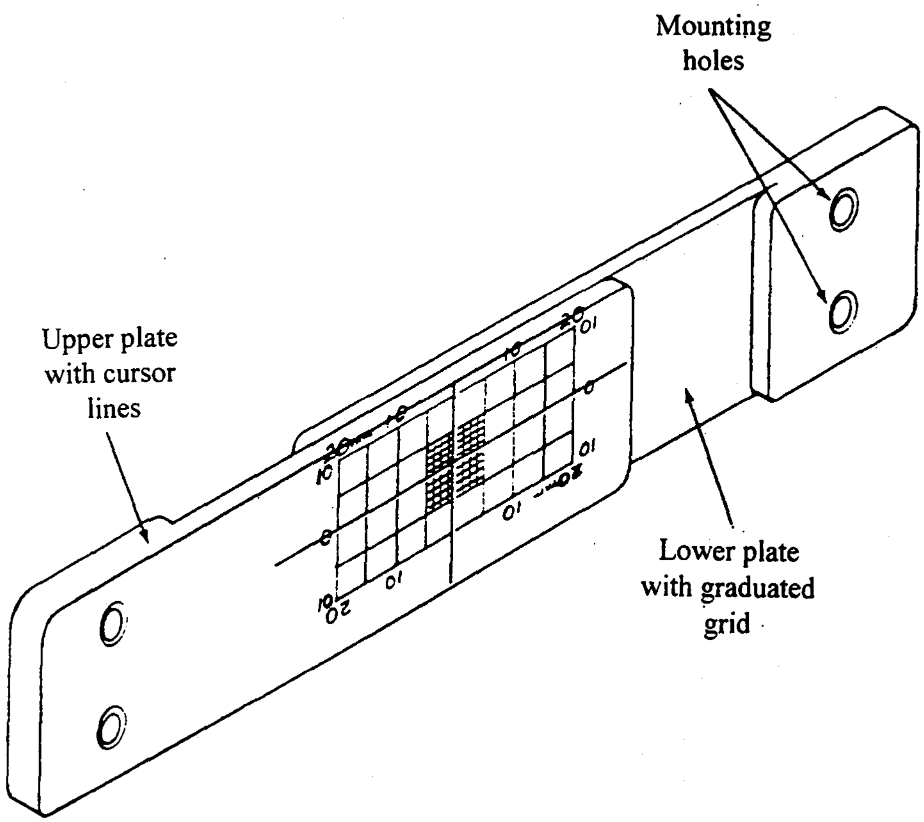


Figure 3-2: Grid Crack Monitor. (Courtesy of Avongard Products, U.S.A. Ltd., Irvine, CA)

Dial Indicator

A dial indicator can be attached temporarily or permanently to a bracket on one side of the discontinuity and arranged to bear against a machined reference surface on the other side. Three-axis versions are also available for measurements in orthogonal directions. Portable micrometers can be used instead of dial indicators.

3.3.2 Electrical Crack Gages

When access to the gage location is not available for monitoring, or when continuous monitoring is needed, a remote reading electrical gage is required.

Three general arrangements are possible. First, an electrical linear displacement transducer can be attached to a bracket on one side of the discontinuity and arranged to bear against a machined reference surface on the other side. Second, anchor points can be located on either side of a discontinuity and the transducer attached to the anchor points via ball joints, as shown in Figure 3-3. Third, an electrical linear displacement transducer can be incorporated in the mechanical system shown in Figure 3-1.

Electrical crack gages are more expensive than mechanical gages, and their range is limited. However, range can usually be extended by resetting. Precision is between ± 0.003 and 0.15 mm, depending on the transducer.

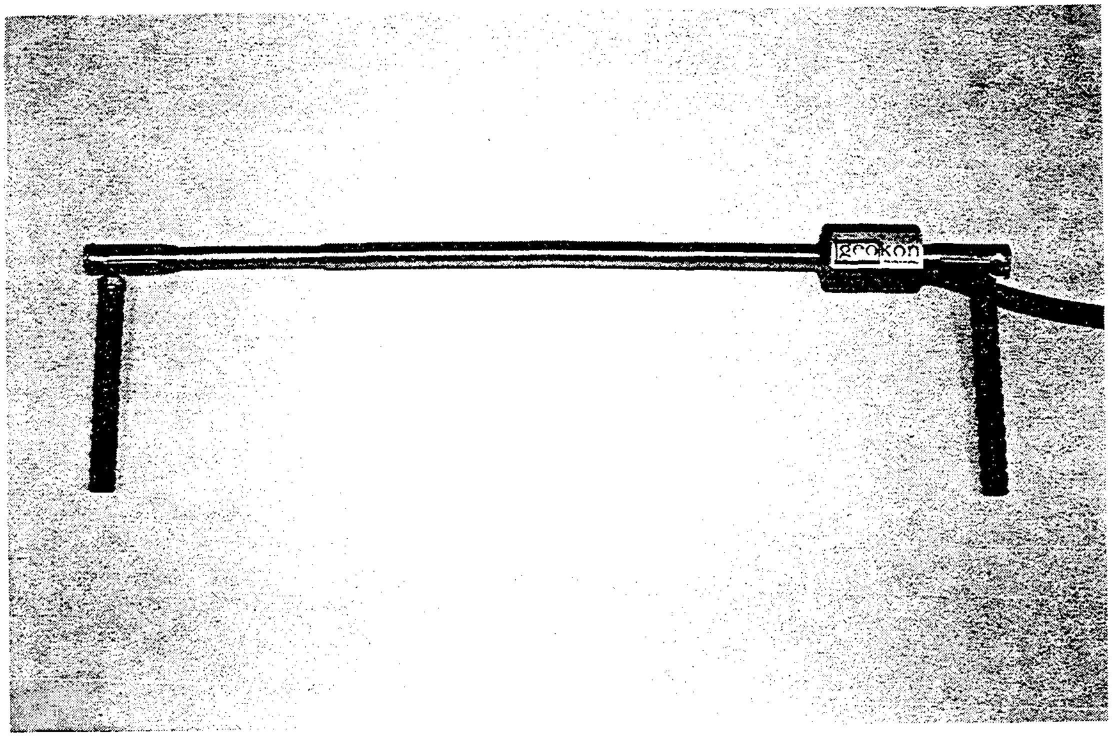


Figure 3-3: Electrical Crack Gage. (Courtesy of Geokon, Inc., Lebanon, NH)

3.4 CONVERGENCE GAGES

Convergence gages are typically used for monitoring convergence across braced excavations.

A typical convergence gage, often called a tape extensometer, is shown in Figure 3-4. The tape has punched holes at 50 mm intervals. Grouted rebar and expansion shell anchors are shown, but anchors can be welded or bolted. The tension in the tape is controlled by a compression spring, and to standardize tension the collar is rotated until the scribed lines are in alignment. After attachment of the extensometer to the anchors and standardizing the tension, readings of distance are made by adding the dial indicator reading to the tape reading. Precision is typically ± 0.15 mm in a 10 m span, decreasing with increasing span. Maximum span is approximately 60 m.

3.5 TILTMETERS

Tiltmeters are used to monitor the change in inclination (rotation) of points on or in the ground or a structure.

A tiltmeter consists of a gravity-sensing transducer within an appropriate housing, and housings are available for installation either on or below the surface of the ground or structure. Surface versions my be either fixed-in-place or arranged as portable devices by mating with reference points permanently attached to the surface. Subsurface versions are usually fixed-in-place within boreholes.

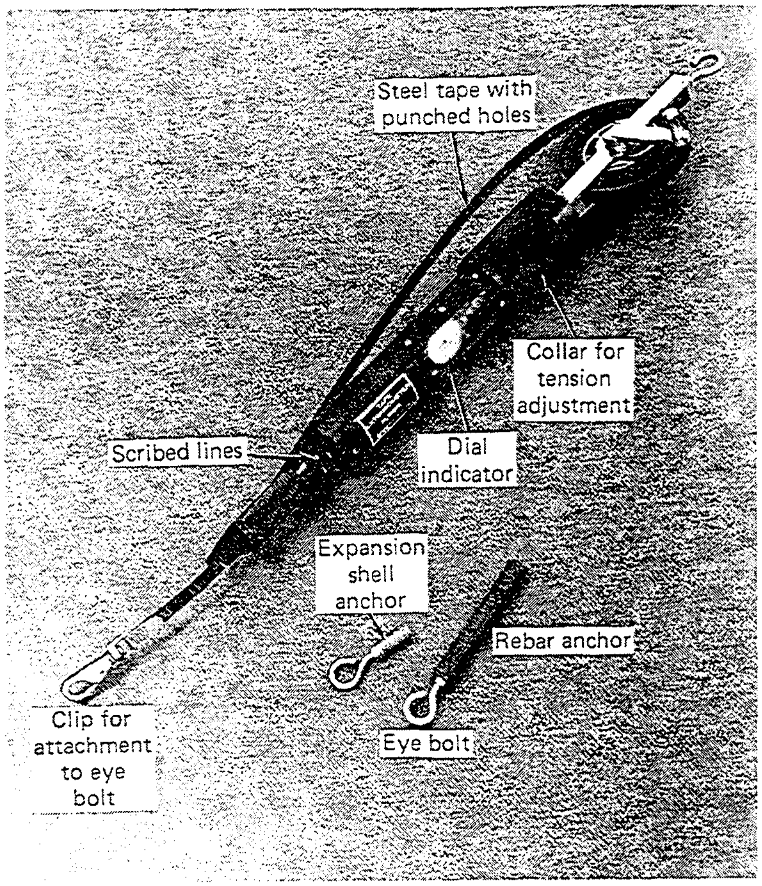


Figure 3-4: Tape Extensometer. (Courtesy of Slope Indicator Company, Bothell, WA)

Applications of tiltmeters include monitoring tilt of retaining walls and monitoring landslides in which the failure mode can be expected to contain a rotational component. Very precise tiltmeters can sometimes be used during a short time period to provide a rapid indication of deformation trends. Tiltmeters have been used for monitoring safety of buildings alongside excavations in an attempt to provide forewarning of distress, but unless a rotational component of deformation is expected, settlement measurements are likely to be more meaningful.

3.5.1 Tiltmeters With Accelerometer Transducers

A force balance accelerometer transducer consists of a mass suspended in the magnetic field of a position detector (Figure 3-5). When the mass is subjected to gravity force along its sensitive axis, it tries to move, and the motion induces a current change in the position detector. This current change is fed back through a servo-amplifier to a restoring coil, which imparts an electromagnetic force to the mass that is equal and opposite to the initiating gravity force. The mass is thus held in balance and does not move. The current through the restoring coil is measured by the voltage across a precision resistor. This voltage is directly proportional to the input force.

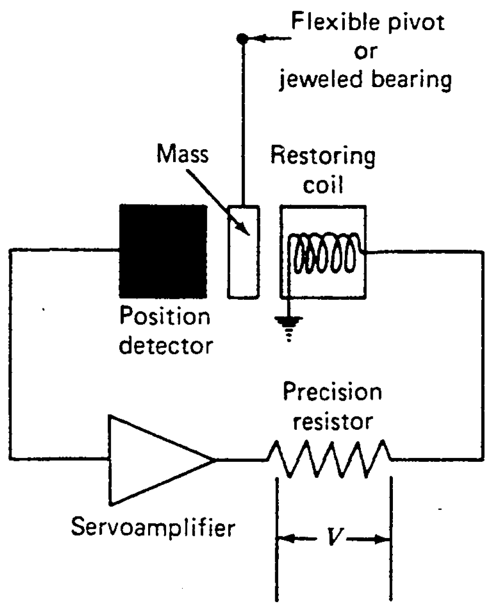


Figure 3-5: Schematic of Force Balance Accelerometer. Voltage V is proportional to the Force Required to Hold the Mass in the Null Position. (After Dunnicliff, 1988, 1993)

The transducer is packaged as a tiltmeter either as a portable or a fixed-in-place device. A portable tiltmeter for measuring tilt in both a horizontal and vertical plane is shown in Figure 3-6. A measurement is made by placing the tiltmeter in an exactly reproducible position on a reference plate, taking a reading, turning the tiltmeter 180 degrees, and again taking a reading. This method allows use of the check-sum procedure described for inclinometers in Section 3.11. The reference plate is either metallic or ceramic and must be securely bonded or bolted to the monitored surface.

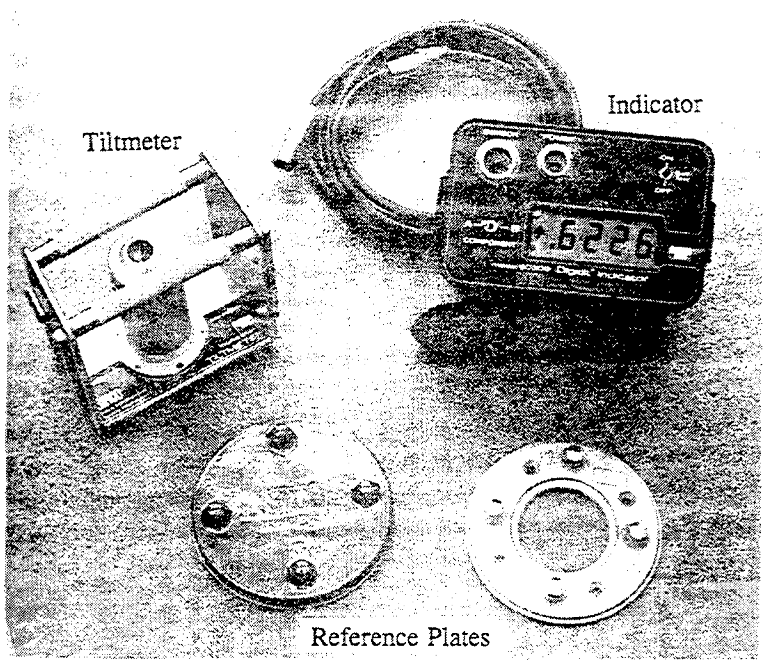


Figure 3-6: Tiltmeter with Accelerometer Transducer. (Courtesy of Slope Indicator Company, Bothell, WA)

The fixed-in-place version is attached to the monitored surface with appropriate anchors. The output voltage is read with a portable readout unit or datalogger. Because there is no inaccuracy caused by the mechanical mating with a reference plate, the fixed-in-place version is more accurate than the portable version.

3.5.2 Tiltmeters With Electrolytic Level Transducers

An electrolytic level (Figure 3-7) consists of a sealed glass vial similar to the vial on a conventional builder's level, partly filled with a conductive liquid. Output resistance is read with a portable readout unit or datalogger.

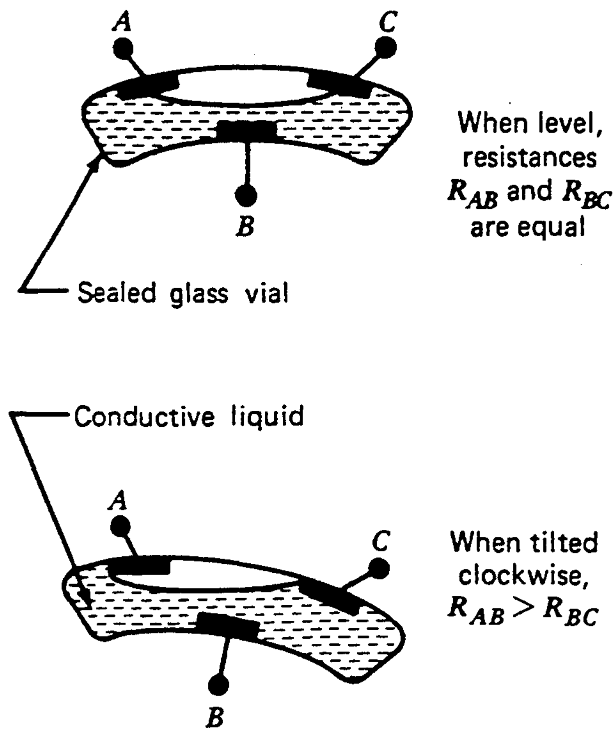


Figure 3-7: Schematic of Electrolytic Level. (After Dunnicliff, 1988, 1993)

The electrolytic level is packaged as a tiltmeter (Figure 3-8) by incorporating it in an appropriate housing. Versions are available for mounting on vertical surfaces, horizontal surfaces, and within boreholes, with either uniaxial or biaxial transducers.

3.6 PROBE EXTENSOMETERS

Probe extensometers are defined in this Module as devices for monitoring the changing distance between two or more points along a common axis, by passing a probe through a pipe. Measuring points along the pipe are identified by the probe, and the distance between points is determined by measurements of probe position.

Typical applications of probe extensometers are monitoring vertical compression within embankments or embankment foundations, settlement alongside excavations, and heave at the base of open cut excavations.

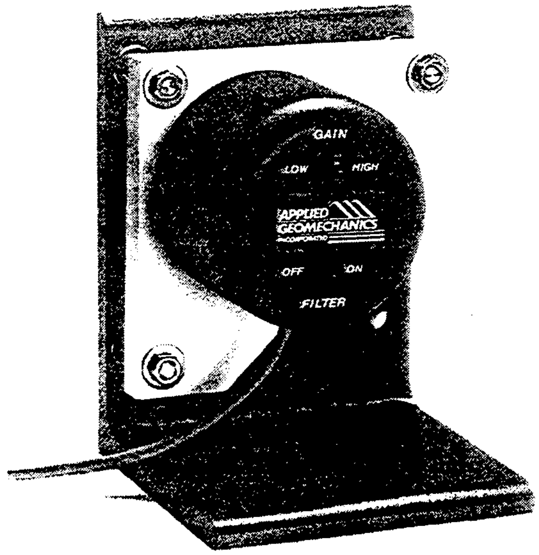


Figure 3-8: Biaxial Wall-Mount Tiltmeter with Electrolytic Level Transducer on Universal Mounting Bracket. (©1997, Courtesy of Applied Geomechanics, Inc., Santa Cruz, CA)

3.6.1 Induction Coil Gages

An induction coil transducer consists of a steel wire ring and an electrical coil connected to a power source. When the coil is placed inside the ring, a voltage is induced in the ring, which in turn alters the current in the coil because its inductance changes. The current in the coil is a maximum when it is centered inside the ring; thus, by measuring current in the coil the transducer can be used as a proximity sensor. The readout unit contains an ammeter for current indication.

When packaged as a probe extensometer, the embedded part of the instrument consists of a telescoping pipe surrounded by steel rings at the required measuring points. The reading device consists of a coil housed within a probe and an attached signal cable connected to a current indicator. Figure 3-9 shows a schematic of one commercial version.

Depth measurements are made either with a survey tape attached to the probe or by using graduations on the signal cable: the former is very preferable because the cable can experience significant change in length during its life. Use of a composite survey tape and signal cable is preferable (similar to the composite tape and cable shown in Figure 2-5 for use with an electrical dipmeter when reading open standpipe piezometers), allowing accurate measurements to be made without the need for dual lines running along the pipe. Readings are made by traversing the probe along the pipe and noting the tape graduation when output current is a maximum

For determination of absolute vertical deformation data, one steel ring must be at a location not subject to settlement. The lowest ring should therefore be placed at sufficient depth to serve as a reference datum, and precision can be maximized by installing two such measuring points and averaging readings.

The device is electrically very sensitive and can locate a steel ring to within ± 0.5 mm. When noting the tape reading at which output current is a maximum, precision with respect to the deep measuring point will generally be ± 3-5 mm.

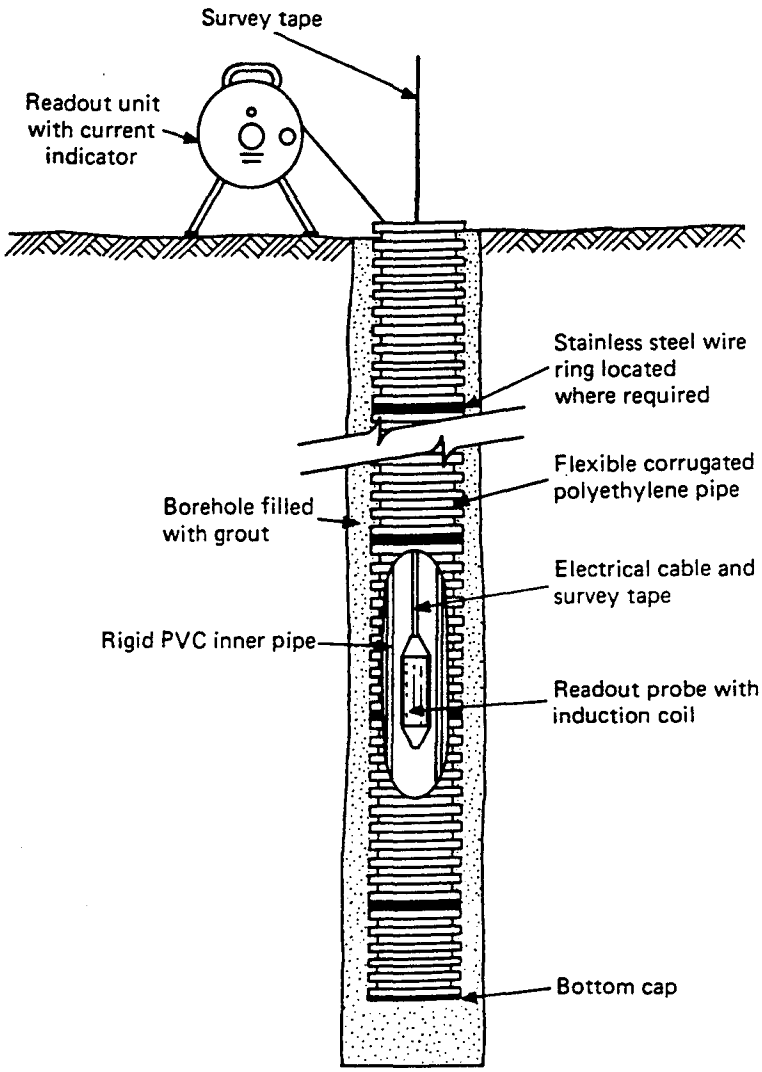


Figure 3-9: Schematic of Slope Indicator Company Sondex Probe Extensometer, Installed in a Borehole. (After Dunnicliff, 1988, 1993)

3.6.2 Magnet/Reed Switch Gages

A magnet/reed switch transducer is an on/off position detector, arranged to indicate when the reed switch is in a certain position with respect to a magnet. Figure 3-10 shows an early version of the transducer, which used a ring magnet. Current versions use a ring of bar magnets, located around the reed switch. The switch contacts are normally open and one of the reeds must be magnetically susceptible. When the switch enters a sufficiently strong magnetic field, the reed contacts snap closed and remain closed as long as they stay in the magnetic field. The closed contacts actuate a buzzer or indicator light in a portable readout unit.

When packaged as a probe extensometer the gage is referred to as a magnetic probe extensometer or magnetic extensometer. A schematic of a borehole installation is shown in Figure 3-11. Precision of reed switch closure is between ± 0.03 and 0.3 mm depending on guidance arrangements of the probe. System precision is similar to that of the induction coil gage: ± 3-5 mm.

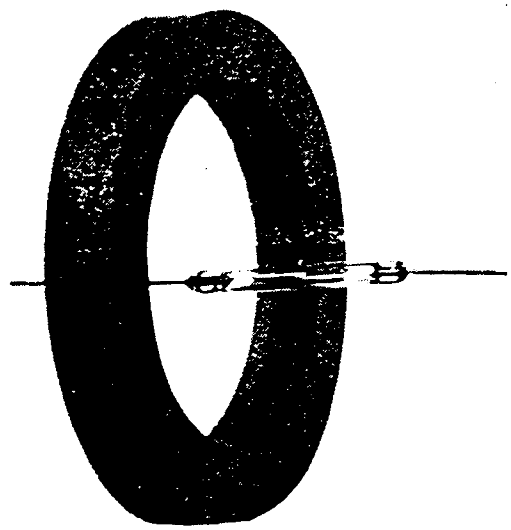


Figure 3-10: Magnet/Reed Switch. (After Burland et al., 1972). Reprinted by Permission of Institution of Civil Engineers, London. (After Dunnicliff, 1988, 1993)

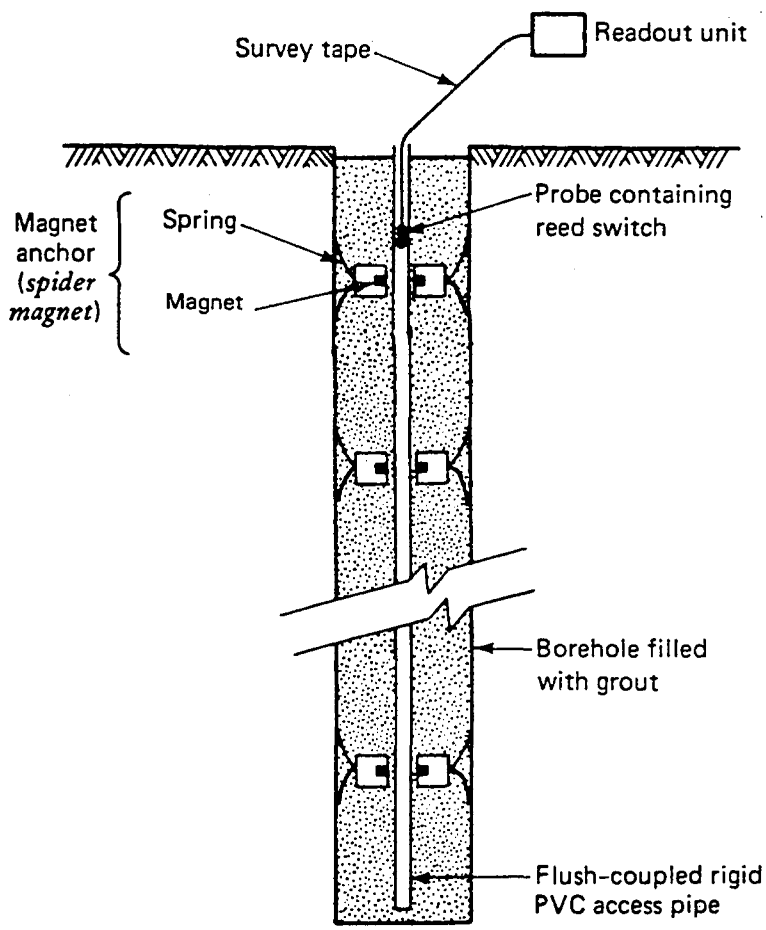


Figure 3-11: Schematic of Probe Extensometer with Magnet/Reed Switch Transducer, Installed in a Borehole. (After Dunnicliff, 1988, 1993)

The magnet/reed switch gage can be arranged to provide measurements of heave at the bottom of braced or other open cut excavations, in a way that does not interfere with excavation work. As shown in Figure 3-12, the installation is made prior to the start of excavation. After taking initial readings, the access pipe is sealed 2-3 m below the ground surface, using an expanding plug set with an insertion tool, and the pipe is cut with an internal cutter just above the plug. A good survey fix is made on the plan position and, just before general excavation reaches the top of the pipe, the pipe is carefully located, a reading made, and the pipe again sealed and cut. The procedure is repeated until excavation is complete. Clearly, vigilance on the part of supervisory personnel is required, but the method is far less prone to damage and malfunction than heave gages which extend above the excavated surface.

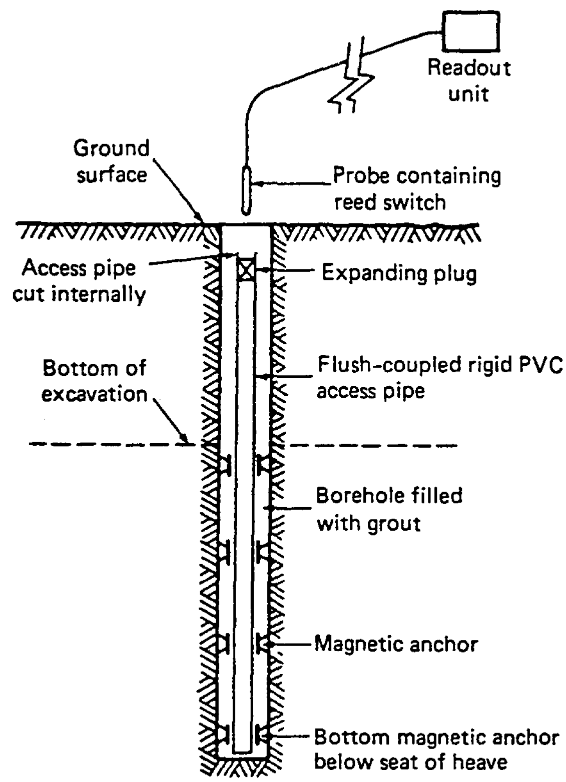


Figure 3-12: Schematic of Probe Extensometer with Magnet/Reed Switch Transducer, Arranged for Monitoring Heave at the Bottom of Open Cut Excavations. (After Dunnicliff, 1988, 1993)

3.7 SETTLEMENT PLATFORMS

Settlement platforms are typically used for monitoring settlement below embankments on soft ground.

A settlement platform consists of a square plate of steel, wood, or concrete placed on the original ground surface, to which a riser pipe is attached (Figure 3-13). Optical leveling measurements to the top of the riser pipe provide a record of plate elevations. The plate is typically 1 or 1.2 m square, and the riser pipe is typically 50 mm standard black iron pipe with threaded couplings. Either the pipe and plate are welded together or a floor flange is bolted to a wooden plate or embedded in a concrete plate. A sleeve pipe is sometimes placed around the riser pipe, with a gap between the bottom of the sleeve pipe and the plate to prevent downdrag forces on the riser pipe from being transmitted to the plate. Reasonable practice appears to require a sleeve pipe only if the embankment is over about 8 m high or if the plate is seated on highly compressible material such that downdrag forces might punch the plate below the original ground surface.

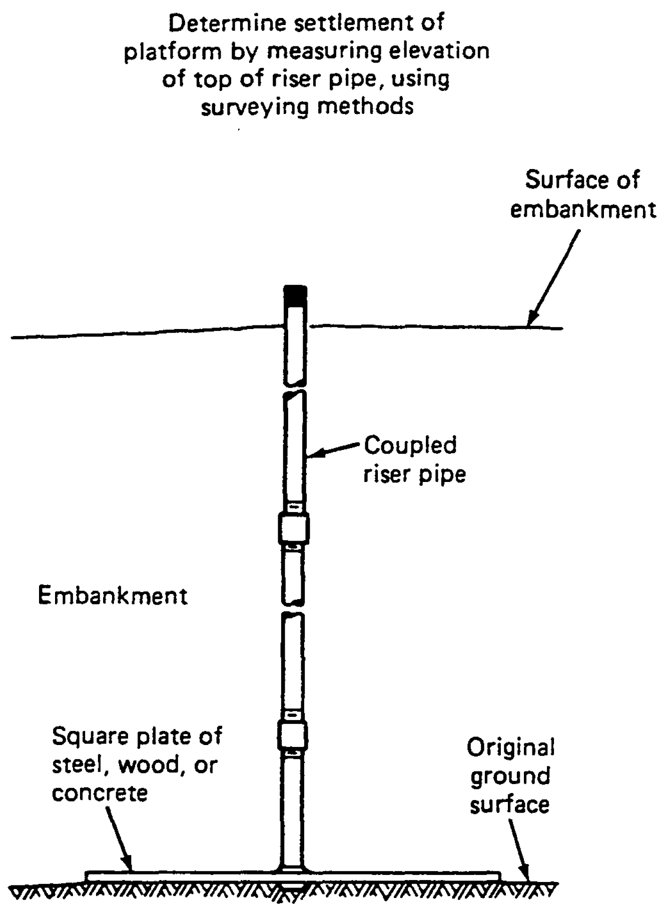


Figure 3-13: Typical Settlement Platform. (After Dunnicliff, 1988, 1993)

Care must be taken to maintain pipe verticality, and an accurate record of added pipe length must be made as fill is placed.

3.8 SUBSURFACE SETTLEMENT POINTS

Typical applications of subsurface settlement points are for monitoring settlement below embankments and preloads, or adjacent to braced excavations, and for monitoring uplift during grouting operations.

The device consists essentially of a riser pipe anchored at the bottom of a vertical borehole and an outer casing to isolate the riser pipe from downdrag forces caused by settlement of soil above the anchor. Settlement of the anchor is determined by measuring the elevation of the top of the riser pipe, using surveying methods. Two arrangements are in common use and are described in the following subsections.

3.8.1 Driven or Grouted Anchor

A typical arrangement, sometimes referred to as a deep settlement point, is shown in Figure 3-14. Outer pipe casing is driven to the required depth and cleaned out. If the casing is set in a predrilled hole, the annular space between the casing and borehole should be backfilled with sand, pea gravel, or grout, and any grout should be allowed to set before the riser pipe is installed. The riser pipe is then inserted and driven 300 mm to 1 m below the bottom of the casing. A rounded reference surface is often attached to the top of the riser pipe and the arrangement protected by a surface cover.

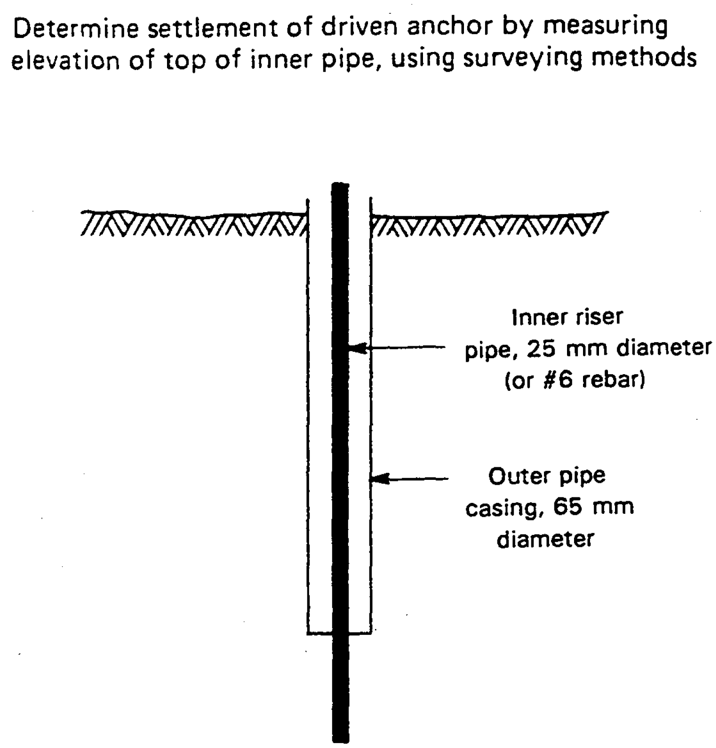


Figure 3-14: Schematic of Subsurface Settlement Point with Driven Anchor. (After Dunnicliff, 1988, 1993)

When a more secure anchor is required, a measured quantity of grout can be tremied to the bottom of the borehole before inserting the riser pipe, the riser pipe driven through the grout, and the outer casing bumped back so that its bottom remains about 300 mm above the top of the grout.

3.8.2 Borros Anchor

The Borros anchor, or Geonor settlement probe, is shown in Figure 3-15. The anchor consists of three steel prongs housed within a short length of 25 mm steel pipe, with points emerging from slots in a conical drive point. The upper end of the 25 mm pipe has a left-hand thread, and 6 mm steel pipe is welded to the tops of the prongs.

A borehole is advanced to a few feet above the planned anchor depth and the anchor inserted by attaching extension lengths of riser pipe and outer pipe. All threads are wrench-tight, except the left-hand thread, which is greased and hand-tight. When the point reaches the bottom of the borehole, it is driven 300 mm to 1 m by driving on the top of the outer pipe. The prongs are then ejected by driving on the riser pipe, the left-hand thread opened by turning the outer pipe clockwise, and the outer pipe bumped back a distance larger than the anticipated vertical compression of the soil above the anchor. Any drill casing used to advance the borehole is then withdrawn.

The Borros anchor provides a more positive anchorage than the driven anchor. However, although a frequently used and simple device, a problem can arise owing to binding of the riser pipe where it exists from the bottom of the outer pipe, such that downdrag forces cause settlement of the prongs. The problem can be minimized by installing an O-ring bushing or a length of greased garden hose in the annular space at the bottom of the pipes.

Determine Settlement of Prongs by Measureing
Elevation of Top of Inner Pipe, Using Surveying Methods

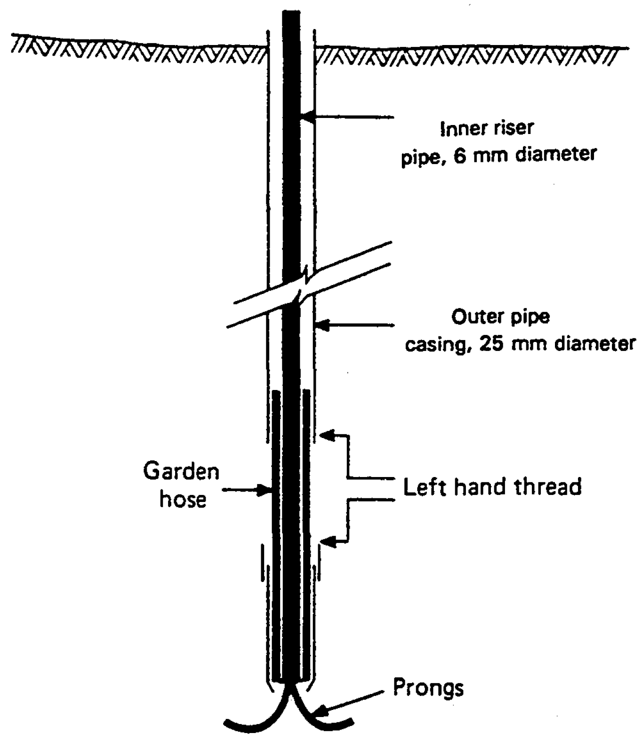


Figure 3-15: Schematic of Borros Anchor. (After Dunnicliff, 1988, 1993)

3.9 FIXED BOREHOLE EXTENSOMETERS

Fixed borehole extensometers are defined in this Module as devices installed in boreholes in soil or rock for monitoring the changing distance between two or more points along the axis of a borehole, without use of a movable probe. When the location of one measurement point is determined with respect to a fixed reference datum, the devices also provide absolute deformation data.

Typical applications are monitoring deformations around underground excavation in rock and behind the faces of excavated slopes.

The operating principle is shown in Figure 3-16.

The distance from the face of the collar anchor to the end of the rod is measured using either a mechanical or an electrical transducer. The device shown is a single-point borehole extensometer (SPBX), but several downhole anchors can be located in a single borehole, each with an attached rod from the downhole anchor to the collar anchor, to create a multipoint borehole extensometer (MPBX). MPBXs are used to monitor the deformation or strain pattern along the axis of an appropriately oriented borehole, for example so that potential failure zones can be located and dangerous deep seated movements separated from surface spalling.

Many types of fixed borehole extensometers are available, the primary variables being choices of anchor type, SPBX or MPBX, transducer type, and extensometer head. Available downhole anchor types including expanding wedge, spring-loaded, groutable and hydraulic anchors, the choice depending primarily on soil or rock type, borehole inclination and drilling method. Mechanical transducer types include dial indicators, depth micrometers and suspended weights. Electrical transducers can be installed in the heads to allow remote monitoring, with a datalogger if required, choosing among LVDTs (linear variable differential transformers), DCDTs (direct current differential transformers), linear potentiometers, and vibrating wire transducers connected in series with coil springs to create adequate range.

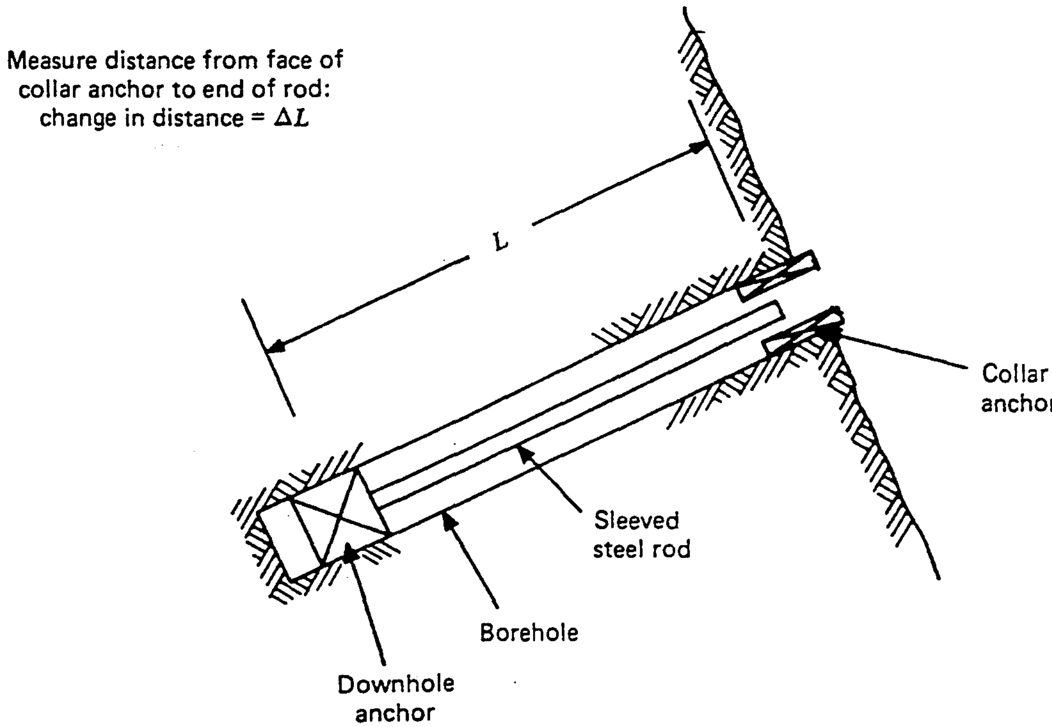


Figure 3-16: Operating Principle of Fixed Borehole Extensometer. (After Dunnicliff, 1988, 1993)

3.10 SERIES EXTENSOMETERS

Series extensometers are designed to measure deformations in boreholes in rock, as alternatives to fixed borehole extensometers, and to monitor strains during load testing of deep foundations. A schematic of the system is shown in Figure 3-17 and a commercial version in Figure 3-18. The system is designed so that lengths of extension tubing can be selected to suit the application, and for complete recovery after use. The measurement module contains one of the electrical transducer types described in Section 3.9 for fixed borehole extensometers. Anchors are either mechanical or hydraulic type.

3.11 INCLINOMETERS

Inclinometers are devices for monitoring horizontal deformation below the ground surface. Typical applications include determining the zone of landslide movement, monitoring the extent and rate of horizontal movement of embankments on soft ground and alongside open cut excavations, and monitoring the deflection of bulkheads, piles, or retaining walls.

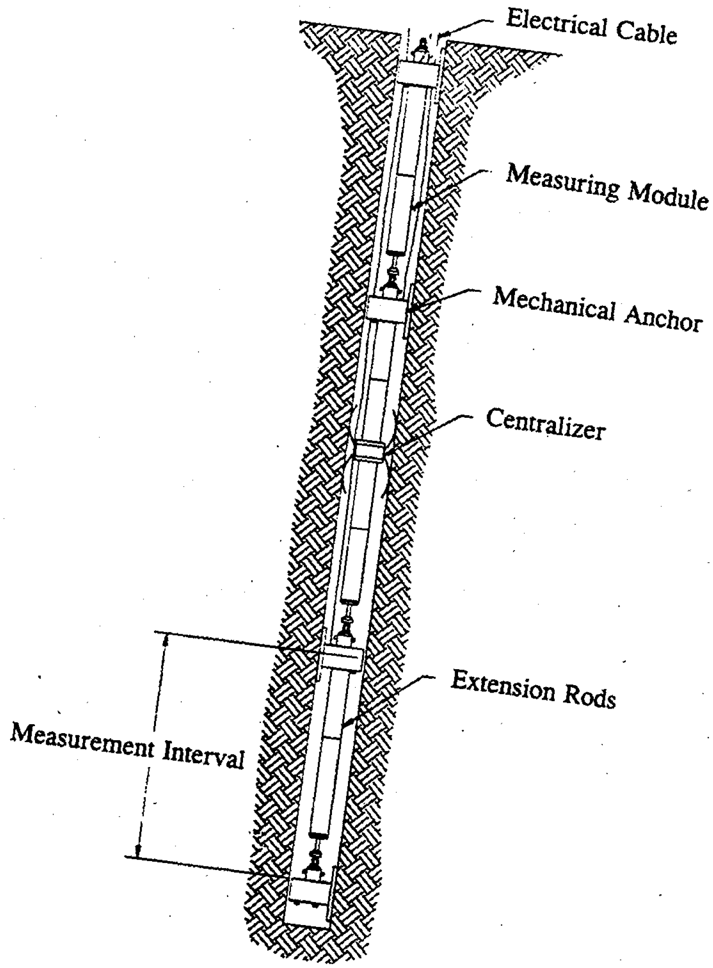


Figure 3-17: Schematic of Series Extensometer. (Courtesy of Roctest, Inc., Plattsburgh, NY)

Figure 3-18: Model BOF-EX Series Extensometer. (Courtesy of Roctest, Inc., Plattsburgh, NY)

Inclinometers have four major components. First, a permanently installed guide casing, usually made of plastic or fiberglass, installed in a near-vertical alignment. The guide casing has tracking grooves for controlling orientation of the probe. Second, a portable probe containing a tilt sensor. Third, a portable readout unit for power supply and indication of probe inclination. Fourth, a graduated electrical cable linking the probe to the readout unit. Examples of these components are shown in Figure 3-19.

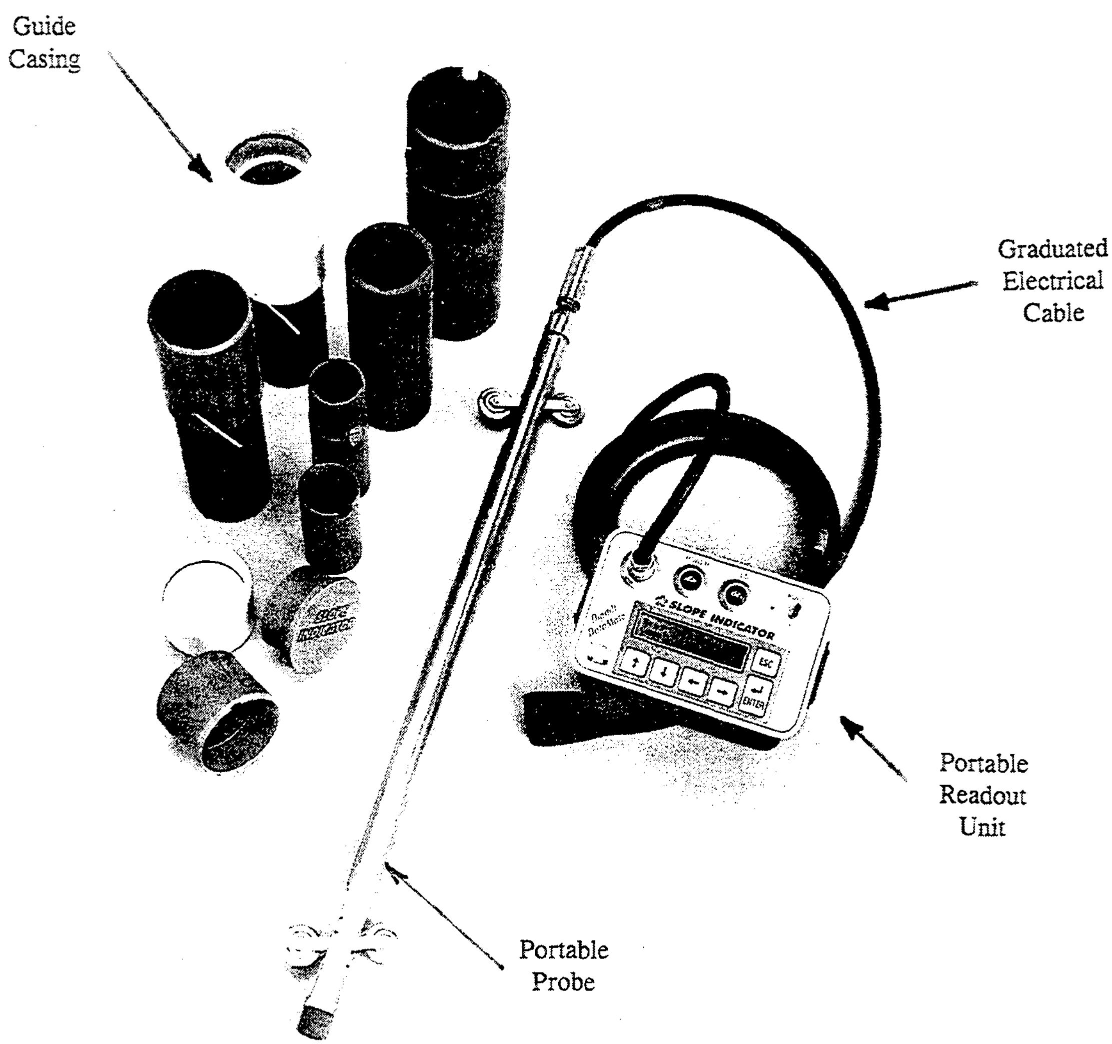


Figure 3-19: Inclinometer System: Slope Indicator Company Digitilt® System. (Courtesy of Slope Indicator Company, Bothell, WA.)

Figure 3-20 shows the normal principle of inclinometer operation. After installation of the casing, the probe is lowered to the bottom and an inclination reading is made. Additional readings are made as the probe is raised incrementally to the top of the casing, providing data for determination of initial casing alignment. The differences between these initial readings and a subsequent set define any change in alignment. Provided that the lower end of the casing is fixed from translation by installing it in material such as bedrock, that will not move laterally, these differences allow calculation of absolute horizontal deformation at any point along the casing.

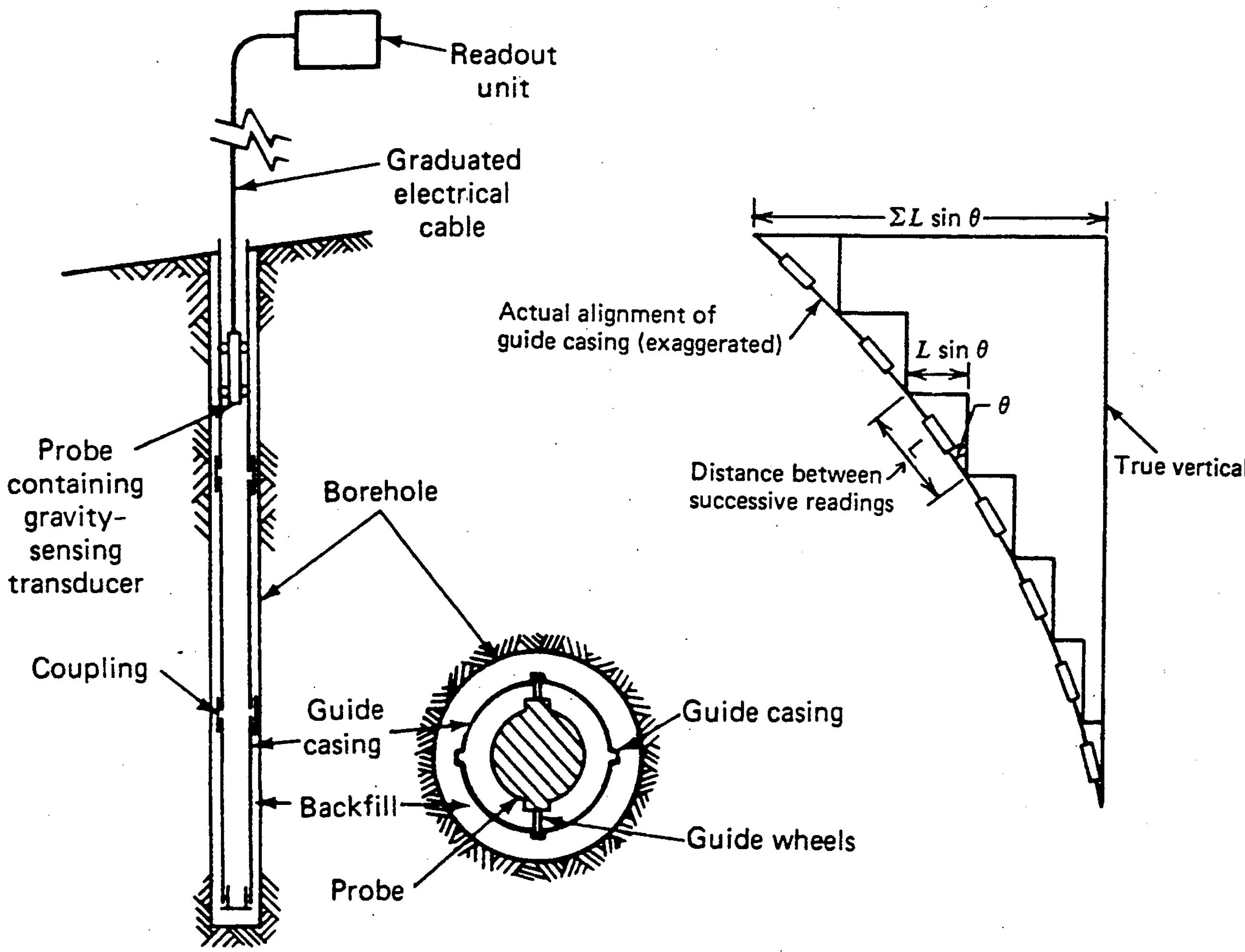


Figure 3-20: Principle of Inclinometer Operation. (After Dunnicliff, 1988, 1993)

Most commercially available inclinometers made use of a force balance accelerometer transducer (Section 3.5.1), and have portable readout units that store data in memory. If accurate data are to be obtained, care must be given to various aspects of the installation and reading procedure. When vertical compression of more than one percent is predicted, telescoping couplings should be used in the casing. Casing should be installed as near to vertical as possible. As indicated above, the bottom of the casing must be installed deep enough to ensure base fixity, preferably over at least the bottom 5 m of casing. The annulus between casing and ground must be filled completely, preferably with grout. Buoyancy forces during grouting must be overcome by applying weight at the bottom of the casing, not the top, to avoid snaking of the casing. The casing must not be twisted during installation, to avoid spiraling of the grooves, and misleading data. When talking readings, great care must be taken to ensure repeatability of probe depth, to use a reading depth interval equal to the wheelbase of the probe, and to allow enough time for temperature stabilization. Immediately after taking readings within an installed casing, the "check sums" (sum of two readings at the same depth, 180 degrees apart) should be examined for data errors. The manufacturer's maintenance procedures should be followed carefully, and regular checks on probe functioning should be made by checking reading repeatability with time within one of the base fixity zones. Manufacturers provide guidance on types of plots, and some provide computer programs that allow for various plot formats. A particularly useful plot is a plot of shear deformation across a small depth interval, versus time.

A remote reading inclinometer, together with methods for maximizing reading accuracy across thin shear zones, is described by McKenna et al. (1994) and McKenna (1995).

3.12 HORIZONTAL INCLINOMETERS

An inclinometer can be used to measure vertical deformation, by installing the casing horizontally. The primary application is for measuring settlement below embankments on soft ground. Use of horizontal inclinometers in this application, in contrast to use of settlement platforms, overcomes the need to extend a pipe upward as filling proceeds, and also overcomes the need to protect the pipe from construction activities. However, a special inclinometer probe is required, different from the probe used for inclinometer measurements in near-vertical casings, because the transducer must be mounted such that its sensitive axis is perpendicular to the length of the probe. A method of traversing the probe along the inclinometer casing must be devised. The simplest method, used where access is available to both ends of the pipe, involves leaving a traction wire within the pipe, clipping the probe to one end of the wire when readings are required, and pulling on the other end. When only one end of the pipe is accessible, an endless traction wire can be installed. The wire can pass around a pulley at the inaccessible end of the pipe and return along the same pipe, but this arrangement runs a severe risk of the probe becoming entangled with the return wire, such that it cannot be moved, and is not recommended. A better arrangement is to install a second pipe, parallel to and of smaller diameter than the main pipe, connecting the two pipes at the inaccessible end with two large-radius 90 degree steel elbows, such that the wire passes along both pipes. Plastic elbows are not suitable, because repeated passage of the traction wire may wear a slot in the inside wall, allowing soil particles to enter and block the pipe or causing the wire to become jammed.

Typical accuracy of settlement measurements is ± 1 to 15 mm per 30 m of casing, and of course it is necessary to determine the elevation of one end of the casing using survey methods, to convert instrument data to absolute settlement data.

3.13 IN-PLACE INCLINOMETERS

In-place inclinometers are typically used for monitoring subsurface deformations around excavations or within slopes, when rapid or automatic monitoring is required.

An in-place inclinometer is generally designed to operate in a near-vertical borehole and provides essentially the same data as a conventional inclinometer (Section 3.11). The device, shown schematically in Figure 3-21, consists of a series of gravity-sensing transducers joined by articulated rods. Systems are available with electrolytic level, accelerometer and vibrating wire gravity-sensing transducers. Uniaxial or biaxial transducers can be used. The transducers are positioned at intervals along the borehole axis and can be concentrated in zones of expected movement. Movement data are calculated using the same methods as for conventional inclinometers.

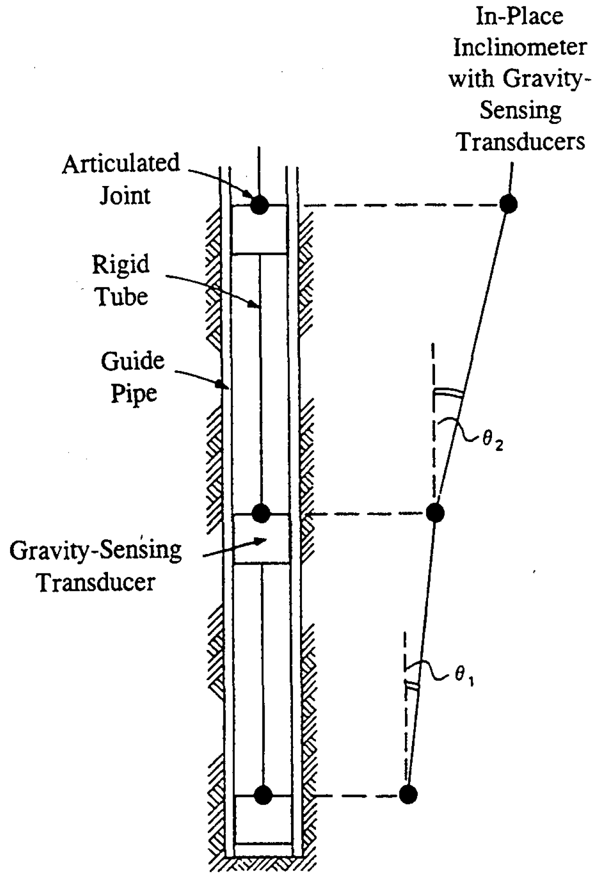


Figure 3-21: Schematic of In-Place Inclinometer. (After Dunnicliff, 1988, 1993)

The device generally uses standard inclinometer casing as guide pipe and can be removed for repairs. However, data continuity will be interrupted when the device is removed and replaced. When compared with conventional inclinometers, advantages include more rapid reading, an option for continuous automatic reading by using a datalogger, and an option for connection to a real-time automatic data acquisition system for transmission of data to remote locations or for triggering an alarm if deformation exceeds a predetermined amount. Disadvantages include greater complexity and expense of the hardware. When conventional inclinometers are read, any long-term drift of the gravity-sensing transducers is removed from calculations by taking a second set of readings with the inclinometer rotated 180 degrees (the check-sum procedure), but this is not possible with the in-place version. Although the transducers are generally stable, there is always the possibility of a "rogue transducer," and this possibility should be recognized if one is planning to use an in-place inclinometer for long-term applications where high precision is required.

In-place inclinometers can be used effectively in combination with a conventional inclinometer. An in-place version can first be installed to define the location of any horizontal deformation, with minimal labor costs for reading. If deformation occurs, the in-place system can be removed and the moving zone monitored with a conventional inclinometer. Alternatively, a conventional inclinometer can be used first to indicate any deformation and an in-place version later installed across a critical deforming zone to minimize subsequent effort and perhaps to provide an alarm trigger.

3.14 SHEAR PLANE INDICATORS

Shear plane indicators are occasionally used as indicators of subsurface horizontal deformation when landslides occur in soil. They are inexpensive and crude alternatives to inclinometers. The most frequently used shear plane indicator is the shear probe (also referred to as a poor man's inclinometer, slip indicator, and poor boy). It consists of plastic tubing or thin-walled polyvinylchloride (PVC) pipe, installed in a nominally vertical borehole. The depth to the top of the shear zone is determined by lowering a rigid rod within the tubing or pipe and measuring the depth at which the rod stops at a bend. The depth to the bottom of the shear zone can be measured by leaving a rod with an attached graduated nylon line at the bottom of the tubing or pipe and pulling on the line until the rod stops. Approximate curvature can be determined by inserting a series of rigid rods of different lengths and noting the depth at which each rod will not pass further down the tubing or pipe.

3.15 LIQUID LEVEL GAGES

Liquid level gages are defined in this Module as instruments that incorporate a liquid-filled tube or pipe for determination of relative vertical deformation. Relative elevation is determined either from the equivalence of liquid level in a manometer or from the pressure transmitted by the liquid.

The primary application for liquid level gages is monitoring settlements within embankments or embankment foundations. In general, they are alternatives to probe extensometers, settlement platforms, and subsurface settlement points, allowing installation to be made without frequent interruption to normal fill placement and compaction, and minimizing the potential for instrument damage. Most liquid level gages also allow measurements to be taken at a central reading location.

The gages only provide a means of measuring **relative** elevations between two or more points. If **absolute** settlement or heave is required, as is usually the case, data must be referenced to a benchmark. If one end of the gage cannot be mounted directly on a benchmark, a surveying method will normally be used, and accuracy may be dependent on accuracy of the surveying method.

In general, liquid level gages are sensitive to liquid density changes caused by temperature variation, to surface tension effects, and to any discontinuity of liquid in the liquid-filled tube. The greatest potential source of error is discontinuity of liquid caused by the presence of gas, and great care must always be taken to ensure absence of gas.

3.15.1 Single-Point Gages

Single-point gages can have both ends at the same elevation, the readout unit can be higher than the cell, or the cell and liquid-filled tube can be installed in a borehole. The three versions are discussed in turn below.

Gages With Both Ends at the Same Elevation

The most frequently used instruments with both ends at the same elevation are called overflow gages, or alternatively hydraulic leveling devices, water overflow pots, or overflow weirs. They are commonly used as alternatives to settlement platforms during construction of embankments on soft ground, overcoming the need to extend a riser pipe through the embankment.

The instrument is shown schematically in Figure 3-22. The gage is normally read by adding liquid to the liquid-filled tube at the readout station, causing overflow in the cell such that the visible level at the readout station stabilizes at the same elevation as the overflow point. The vent tube is essential to maintain equal pressure on both surfaces of liquid, and the drain tube is needed to allow overflowed liquid to drain out of the cell. A four-tube version, with duplicate liquid-filled tubes, provides a verification of reading correctness and is the preferred instrument.

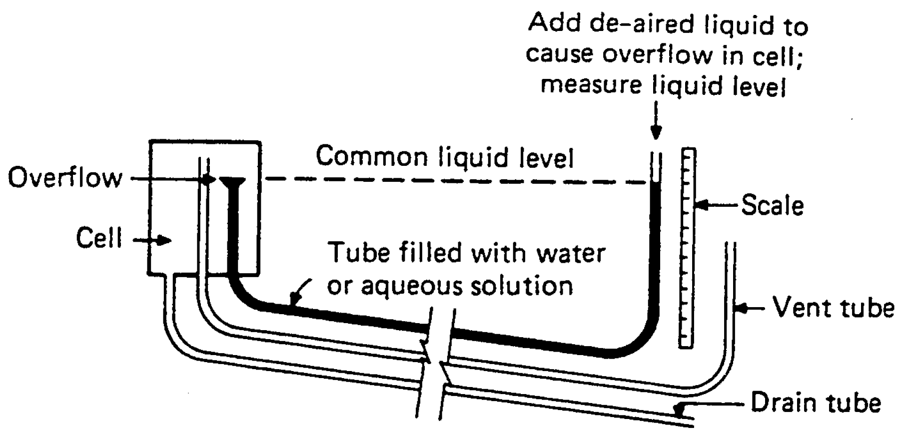


Figure 3-22: Schematic of Overflow Gage with Both Ends at Same Elevation. (After Dunnicliff, 1988, 1993)

Gages With Readout Unit Higher than Cell

Gages that allow the readout unit to be higher than the cell include a pressure transducer at the bottom of a column of liquid.

The arrangement is shown schematically in Figure 3-23. The pressure transducer can be either a pneumatic or vibrating wire type. The upper surface of the liquid column is at a known elevation at the readout location; therefore, relative elevation of the transducer and reservoir can be determined from the pressure measurement and liquid density. Figure 3-23 shows only one liquid-filled tube, but in practice it is preferable to have two tubes so that flushing is possible and independent measurements can be made on each tube as a check.

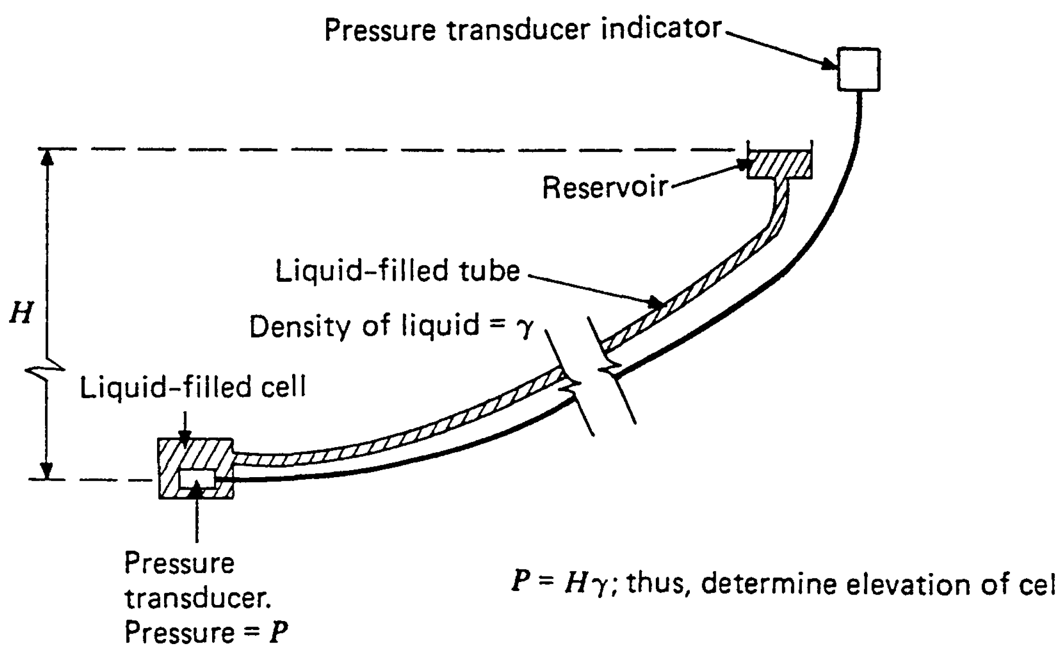


Figure 3-23: Schematic of Liquid Level Gage with Readout Unit Higher than Cell. (After Dunnicliff, 1988, 1993)

Precision of these gages is dependent primarily on whether or not there are any discontinuities in the liquid. High quality de-aired liquid should be used, preferably prepared in the Nold DeAerator™. The apparatus, shown in Figure 3-24, consists of a sealed tank, electric motor, impeller, and electric vacuum pump or water-powered aspirator for obtaining the necessary vacuum, which is applied to the space above the liquid. The phenomena of cavitation and nucleation generate an ultrahigh vacuum that vaporizes dissolved gases and volatile liquids. Centrifugal force directs the released vapors outward, where they bubble up to the partially evacuated space above the liquid surface. From there the gases are withdrawn through the aspirator or vacuum pump to the atmosphere.

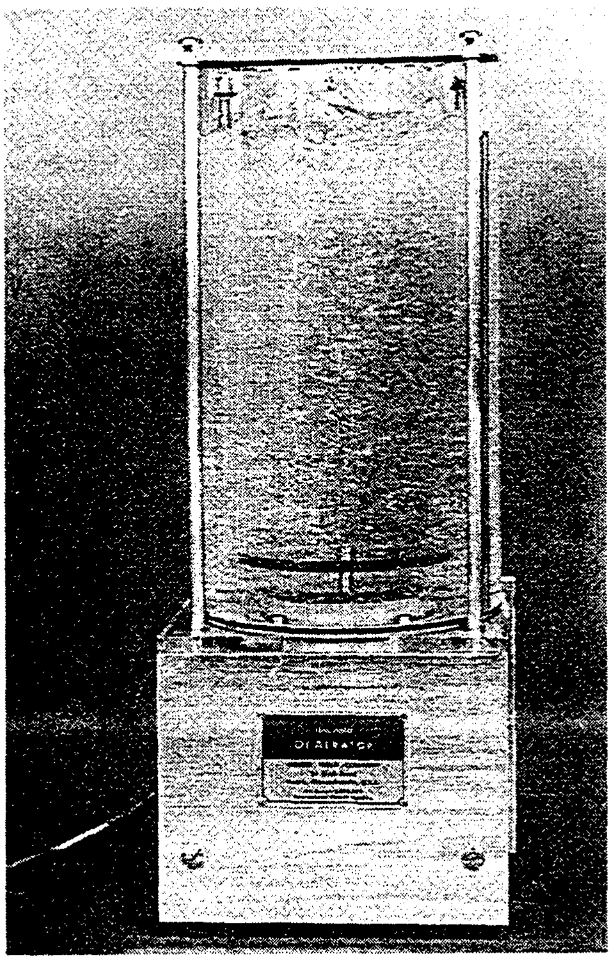


Figure 3-24: Nold DeAerator™. (Courtesy of Walter Nold Company, Inc., Natick, MA)

Preferred versions of these gages are available with a method of applying a measured backpressure of air to the surface of the reservoir. By increasing the backpressure, any gas pockets in the liquid will be driven into solution. By noting the relationship between each increment of backpressure and the response of the pressure transducer, confidence in the absence of discontinuities occurs when the relationship is one-to-one. Moreover, several subsequent readings can be made under increasing backpressure, allowing a best one-to-one straight line to be drawn, thereby increasing accuracy.

Gages With Cell and Liquid-Filled Tube Installed Within a Borehole

This type of gage also includes a pressure transducer at the bottom of a column of liquid. However, this version allows all components except the electrical cable and readout unit to be installed within a borehole, thereby avoiding the need for liquid-filled tubes to be extended to the readout unit, and also avoiding sensitivity to liquid density changes caused by temperature variation.

A schematic of the gage is shown in Figure 3-25. The sensor pressure transducer is grouted at the bottom of a borehole, at a depth below possible vertical deformation, so that the transducer does not move. The reservoir is attached to a plate at the borehole collar, so that the change in liquid head equals vertical deformation. The change in head is monitored by the vibrating wire transducer. The vent tube ensures that the measured head is not affected by changes in air pressure within the reservoir. The system is therefore a self-compensating closed system.

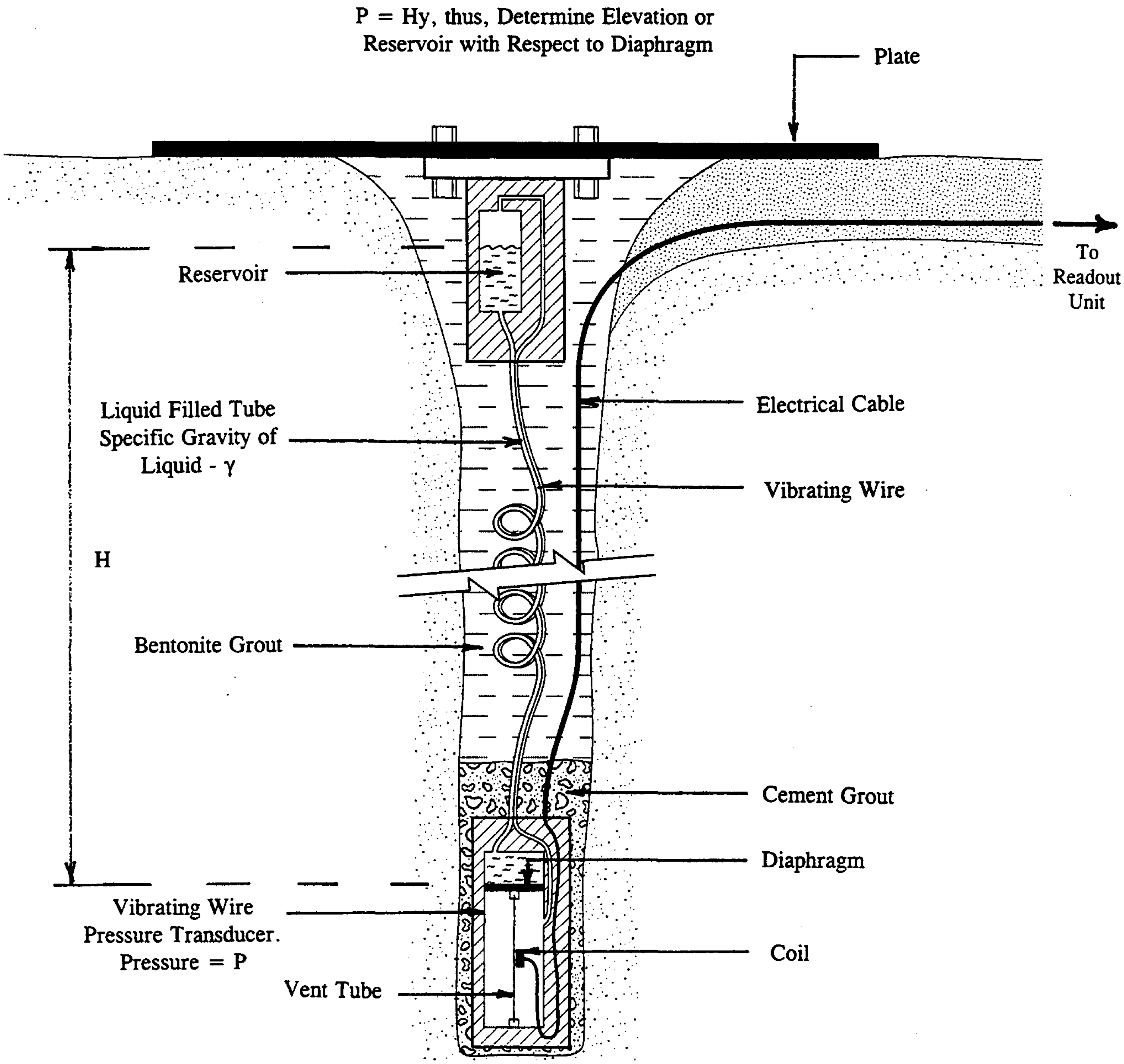


Figure 3-25: Schematic of Liquid Level Gage with Cell and Liquid-Filled Tube Installed within a Borehole. (Courtesy of Geokon, Inc., Lebanon, NH)

3.15.2 Full-Profile Gages

Full-profile gages consist of a near-horizontal plastic pipe and in instrument that can be pulled along the pipe. Readings are made at points within the pipe, and the entire vertical profile can be determined. Differences in vertical profile with time provide data for determination of vertical deformation.

These gages are particularly appropriate where vertical deformation is likely to be nonuniform, such that many single-point gages would otherwise be required. They provide the same data as a horizontal inclinometer. Survival records are generally excellent, since no delicate parts are buried, and the instruments can be checked on a day-to-day basis and any malfunctions corrected in the laboratory. The expensive and calibrated part of the system is portable and can be used at several locations on one project or on several projects.

The most common type of full-profile gage is identical to the single-point version shown in Figure 3-23, with the cell packaged as a movable probe. The backpressuring feature described in Section 3.5.1 is essential.

3.16 TELLTALES

When a sleeved rod or wire is attached to an inaccessible point, routed to an accessible point, and used with a transducer for monitoring the changing distance between the two points, the device is often referred to as a telltale. Telltales can be installed in or on a driven pile or drilled shaft for determining toe settlement during a load test.

A typical telltale for installation in or on a driven pile or drilled shaft consists of the arrangement shown in Figure 3-26. Two options are available for the top arrangements, as shown in ASTM (1981). First, elbows can be used to bring the top of the telltale rod to an accessible point alongside the reference beam. Second, a load transfer assembly can be used to avoid the reduced accuracy that inevitably occurs if elbows are used.

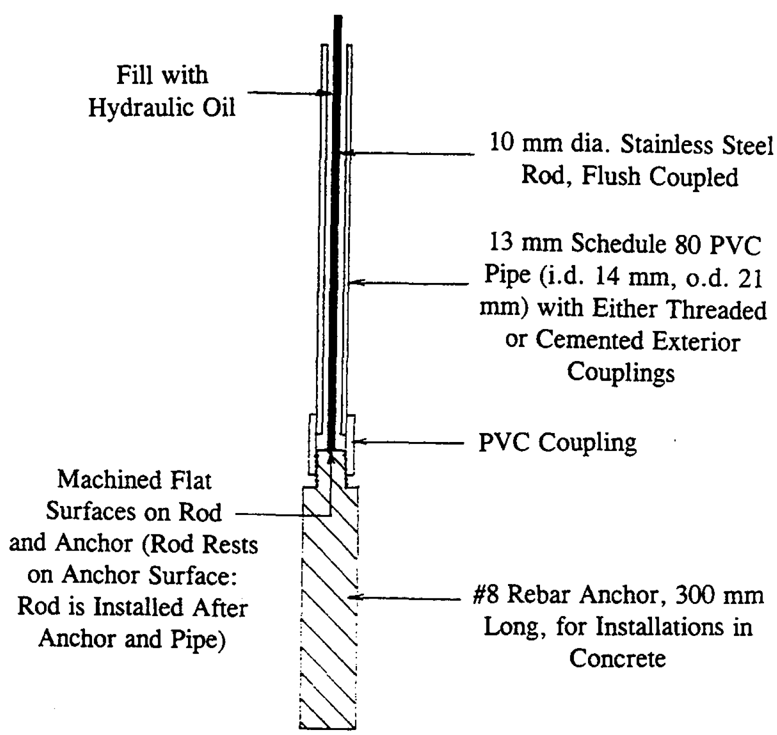


Figure 3-26: Schematic of Telltale.

As described in Chapter 5, multiple telltales can be used for determination of strain and load in structural members. Figure 3-27 shows one of the telltales used for this purpose during a drilled shaft load test with which the author was involved. The top of the top anchor was installed about 300 mm below the butt of the shaft.

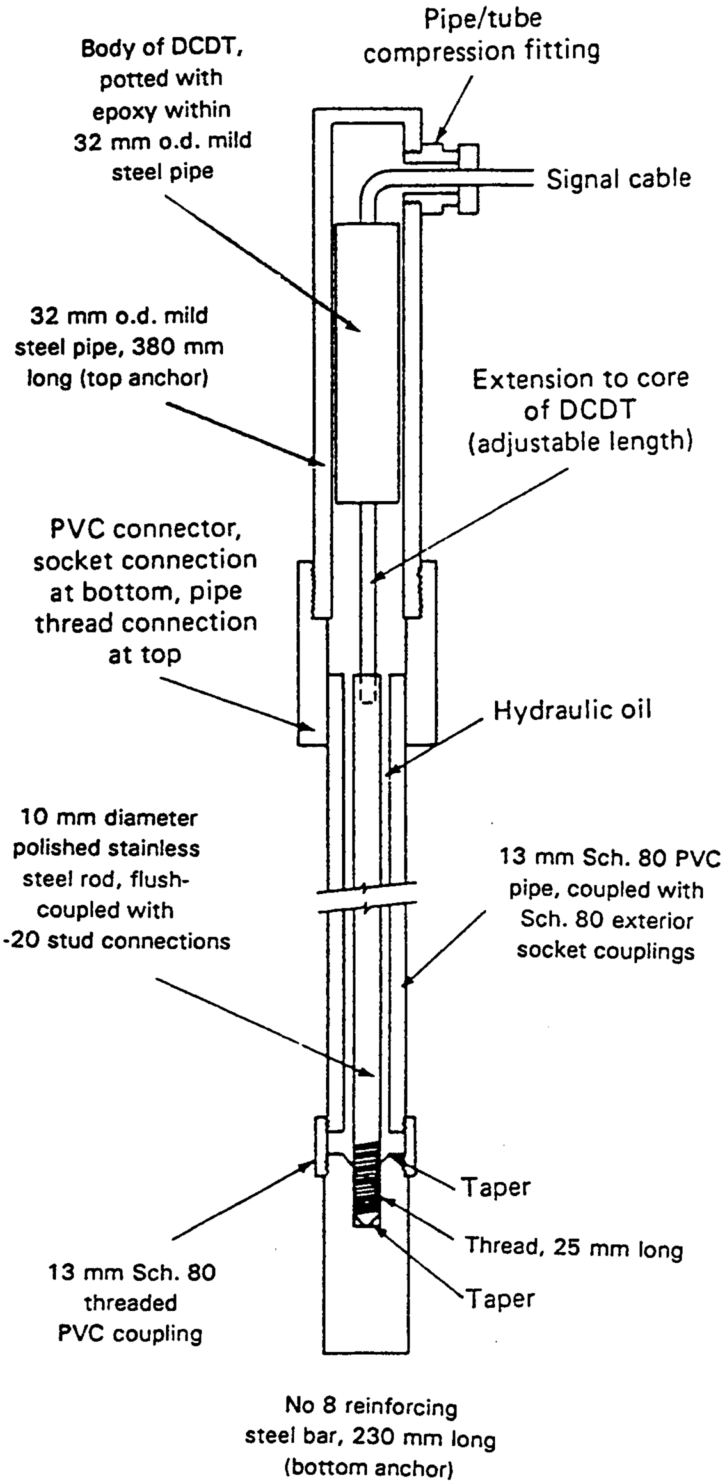


Figure 3-27: Schematic of One Telltale that Forms Part of a Remotely-Read Multiple Telltale System for a Drilled Shaft Load Test. (After Dunnicliff, 1988, 1993)

3.17 CONVERGENCE GAGES FOR SLURRY TRENCHES

Section 9.4.2 refers to an application for a special gage to measure convergence across slurry trenches.

A hydraulic gage is described by DiBiagio and Myrvoll (1972) and shown in Figure 3-28. It consists of a piston within an oil-filled piston chamber, set horizontally across the trench. End bearing plates contact opposite walls of the trench, one attached to the end of the piston rod, the other to the opposite end of the assembly. A standpipe rises vertically from the piston chamber. A reduction in the width of the trench causes movement of the piston and an upward flow of oil into the standpipe; thus, the level of oil in the standpipe can be related to the width of the trench. The gage is attached to the reinforcing cage prior to installation in the trench.

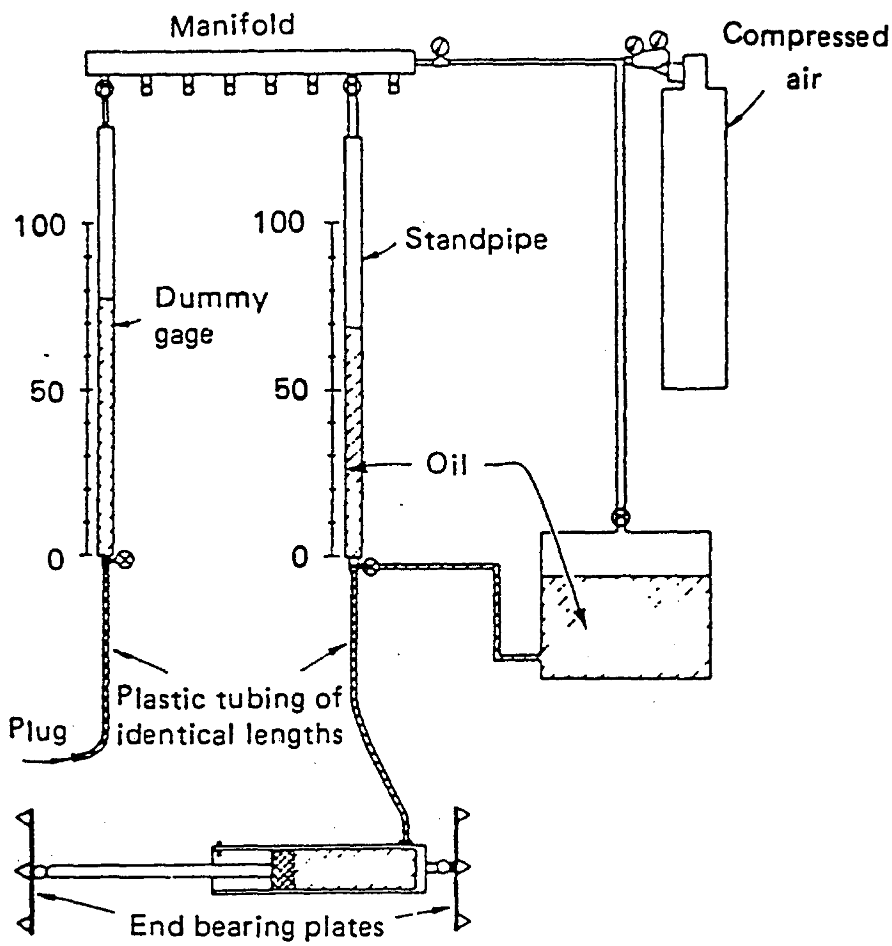


Figure 3-28: Schematic of Hydraulic Gage Used to Measure Convergence of Slurry Trench. (After DiBiagio and Myrvoll, 1972)

3.18 METALLIC TIME DOMAIN REFLECTOMETRY

Metallic time domain reflectometry (MTDR, O'Connor, 1996, Figure 3-29), has been used for monitoring unstable slopes, and also for detecting roadway subsidence caused by collapse of mine voids below roadways. The term MTDR is used to distinguish the technology from optical time domain reflectometry (OTDR), which uses fiberoptic cables instead of coaxial electrical cables. The primary application of

MTDR has been to supplement inclinometer data by installing MTDR cables alongside inclinometer casings, but the technology holds promise as a time-saving and economic replacement for inclinometers in appropriate situations, and for monitoring the onset of scarps during investigation and monitoring of landslides.

The California Department of Transportation has successfully used MTDR cables installed in landslides to determine slide depths (Kane and Beck, 1996). MTDR was originally developed to locate breaks in power line cables. The equipment consists of a coaxial electrical cable grouted in a vertical borehole and a standard MTDR cable tester. The cable tester is used to transmit an electrical pulse along the cable and to monitor the return signal, and faults in the cable such as crimps, short circuits, or breaks are indicated as characteristic return pulses. The return pulse is compared with the transmitted pulse, and a determination made of the distance to any deformation in the cable. By addition of a datalogger, cellular telephone and modem, the technology allows real-time slope monitoring from any location.

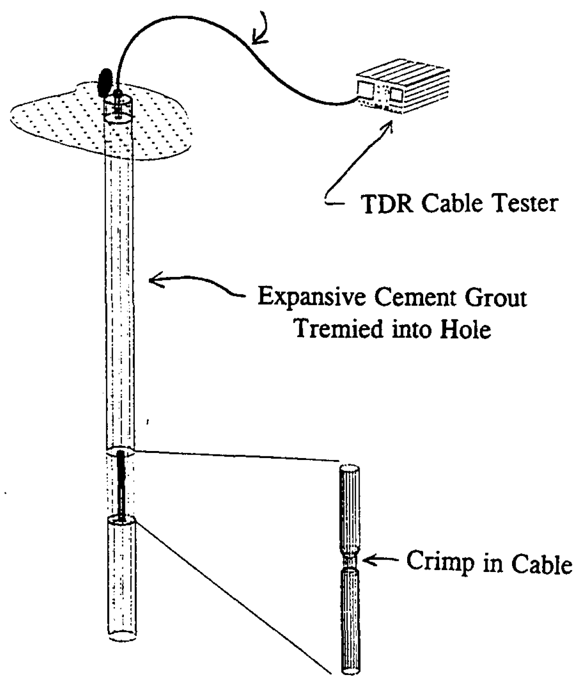


Figure 3-29: The Basic TDR System for Monitoring Deformation in Rock and Soil. (After O'Connor, 1996)

3.19 ACOUSTIC EMISSION MONITORING

As indicated in Chapter 11, acoustic emission monitoring can be used to provide an indication of subsurface deformation in rock slopes.

Acoustic emissions are sounds generated within a material that has been stressed and subsequently deforms. Sometimes these sounds are audible, for example, wood cracking, ice expanding, or soil and rock particles abrading against one another, but more often they are not, owing to their low amplitude or high frequency or both.

A piezoelectric transducer is generally used as a "pickup" to detect the acoustic emissions and produces an electrical signal proportional to the amplitude of sound or vibration being detected. The signal is then amplified, filtered, and counted or recorded in some quantifiable manner. Unwanted machine and environmental noise are electronically filtered from the signal or separately quantified and subtracted from the measurements. The counts or recordings of the emissions are then correlated with the basic material behavior to determine empirically the relative stability of the given material. Usually, if no acoustic emissions are present, the material is in equilibrium and therefore stable. If emissions are observed, the material is not in equilibrium and may be in a condition that eventually leads to failure.

The method is sometimes referred to as microseismic detection and subaudible rock noise monitoring, but the term acoustic emission, or simply AE, is becoming the accepted term.

As shown in Figure 3-30 the components of an acoustic emission monitoring system consist of a waveguide to bring the signals from within the ground to a convenient monitoring point, a transducer (geophone, accelerometer, or hydrophone) to convert the mechanical wave to an electrical signal, a preamplifier to amplify the signal if long cable is being used, filters to eliminate undesirable portions of the signal, an amplifier to amplify the signal further, and a quantification system. The dashed lines in Figure 3-30 indicate components that are grouped in self-contained boxes.

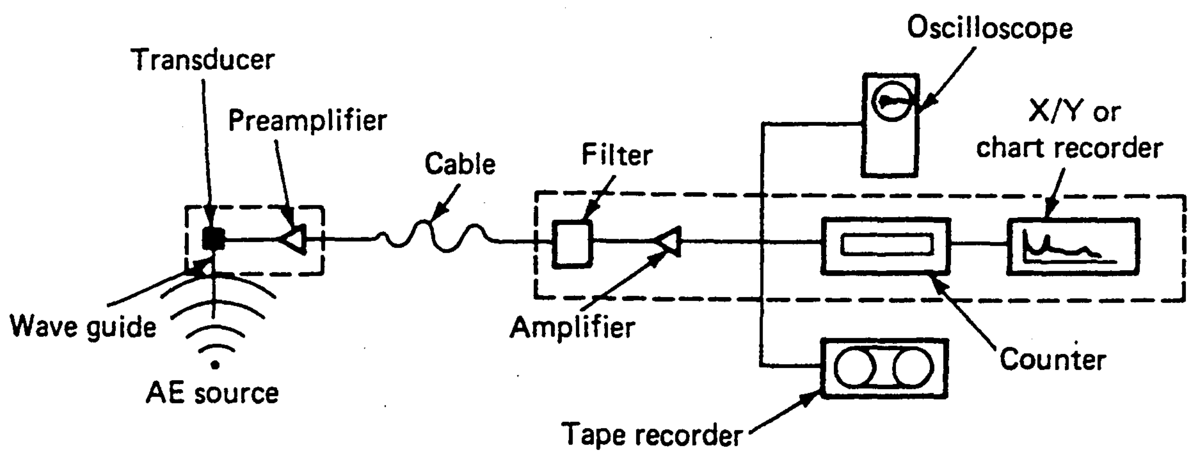


Figure 3-30: Schematic of Basic Single-Channel Acoustic Emission Monitoring System for Recording Total Counts or Count Rate. (After Koerner et al., 1981). Reprinted with Permission from ASTM STP 750. Copyright ASTM

CHAPTER 4.0
OVERVIEW OF HARDWARE FOR MEASUREMENT OF TOTAL STRESS IN SOIL

4.1 INTRODUCTION

Total stress measurements in soil fall into two basic categories: measurements within a soil mass and measurements at the face of a structural element. Instruments are referred to as earth pressure cells, soil stress cells, and soil pressure cells, and in this chapter the terms embedment earth pressure cells and contact earth pressure cells will be used for the two basic categories.

Embedment earth pressure cells are installed within fill, for example, to determine the distribution, magnitude, and direction of total stress within an embankment. Applications for contact earth pressure cells include measurement of total stress against retaining walls, culverts, piles and slurry walls.

The primary reasons for use of earth pressure cells are to confirm design assumptions and to provide information for the improvement of future designs; they are less commonly used for construction control or other reasons. When concerned with stresses acting on a structure during construction or after construction is complete, it is usually preferable to isolate a portion of the structure and to determine stresses by use of load cells and strain gages within the structure. For example, this approach has been used successfully in the determination of earth pressures in braced excavation from measurements of support loads.

The following sections provide an overview of hardware for measurement of total stress in soil.

4.2 EMBEDMENT EARTH PRESSURE CELLS

Attempts to measure total stress within a soil mass are plagued by errors resulting from poor conformance (See Section 4.2.2) because both the presence of the cell and the installation method generally create significant changes in the free-field stress. It is difficult and expensive to match the elastic modulus of the earth pressure cell to that of an individual soil. It is also very hard to place the cell under field conditions so that the material around the cell has the same modulus and density as the surrounding soil and with both faces of the cell in intimate contact with the material. It is also very difficult and costly to perform a truly representative calibration in the laboratory to determine the cell response or calibration factor. Therefore, it is usually impossible to measure total stress with great accuracy.

4.2.1 Typical Embedment Earth Pressure Cells

Most earth pressure cells available in the United States are hydraulic cells. A cell consists of two circular or rectangular steel plates, welded together around their periphery, with liquid filling the intervening cavity and a length of high-pressure steel tubing connecting the cavity to a nearby pressure transducer (Figures 4-1 and 4-2). Total stress acting on the outside of the cell is balanced by an equal pressure induced in the internal liquid. It is essential that the cell is filled with de-aired liquid and that no gas bubbles are trapped within the cavity during filling.

Accuracy is increased by machining grooves around the edge to increase edge flexibility, so that the active face tends to work as a piston (Figure 4-1), hence the cell is less sensitive to eccentric, non-uniform and point loads. In addition, the layer of liquid is thin (0.5 - 2 mm) so that the stiffness of the cell is high and more closely matches that of the surrounding soil, and the installed cell experiences minimal effects caused by thermal expansion and contraction of the liquid.

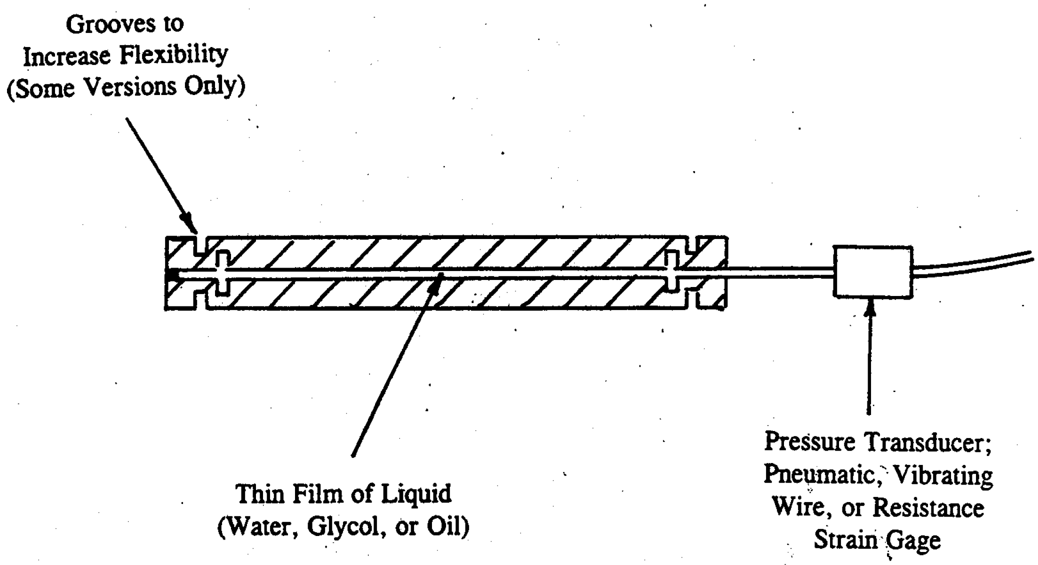


Figure 4-1: Schematic of Embedment Cell Earth Pressure Cell.

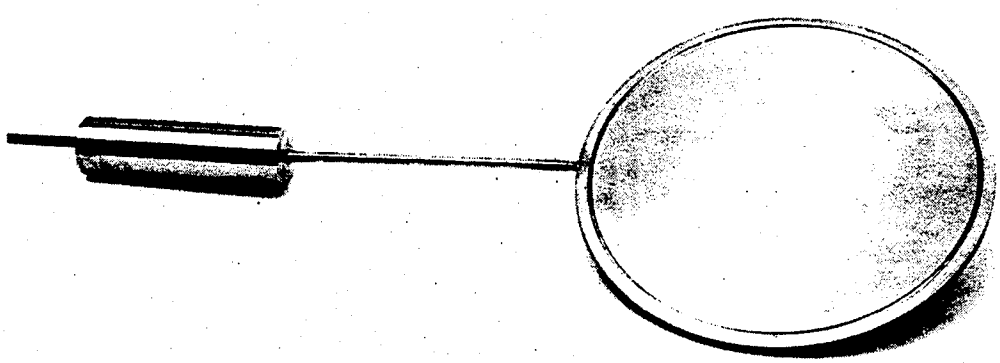


Figure 4-2: Earth Pressure Cell with Vibrating Wire Transducer. (Courtesy of Roctest, Inc., Plattsburg, NY)

4.2.2 Limitations Imposed by Soil Environment

Measurement of total stress at a point within a soil mass requires the following:

1. An earth pressure cell that will not appreciably alter the state of stress within the soil mass because of its presence (conformance).

2. A large enough sensing area to average out local nonuniformities.

3. Minimum cell sensitivity to nonuniform bedding.

4. A method of installation that will not seriously change the state of stress.

The last requirement generally limits these measurements to fills and other artificial soil conditions. However, some success has been achieved in measuring horizontal stress in soft soils by pushing specially designed earth pressure cells downward into natural ground (e.g. Massarsch, 1975). Other instruments used for this purpose include dilatometers and pressuremeters (See Module 1 - Subsurface Investigations), Chapter 5).

Attempts to measure stress in soil by advancing a large-diameter borehole, inserting earth pressure cells, and backfilling around the cells are generally subject to gross conformance errors.

4.2.3 Factors Affecting Measurements

Table 4-1, based on a table by Weiler and Kulhawy (1982), with substantial revisions, summarizes the major factors that affect measurements, and indicates correction methods. The many factors that affect measurements can result in substantial errors, so that measurements with embedment earth pressure cells can rarely be made with high accuracy. Figure 4-3 shows the effects of aspect ratio (ratio of cell thickness to diameter) and soil/cell stiffness ratio (ratio of soil stiffness to cell stiffness).

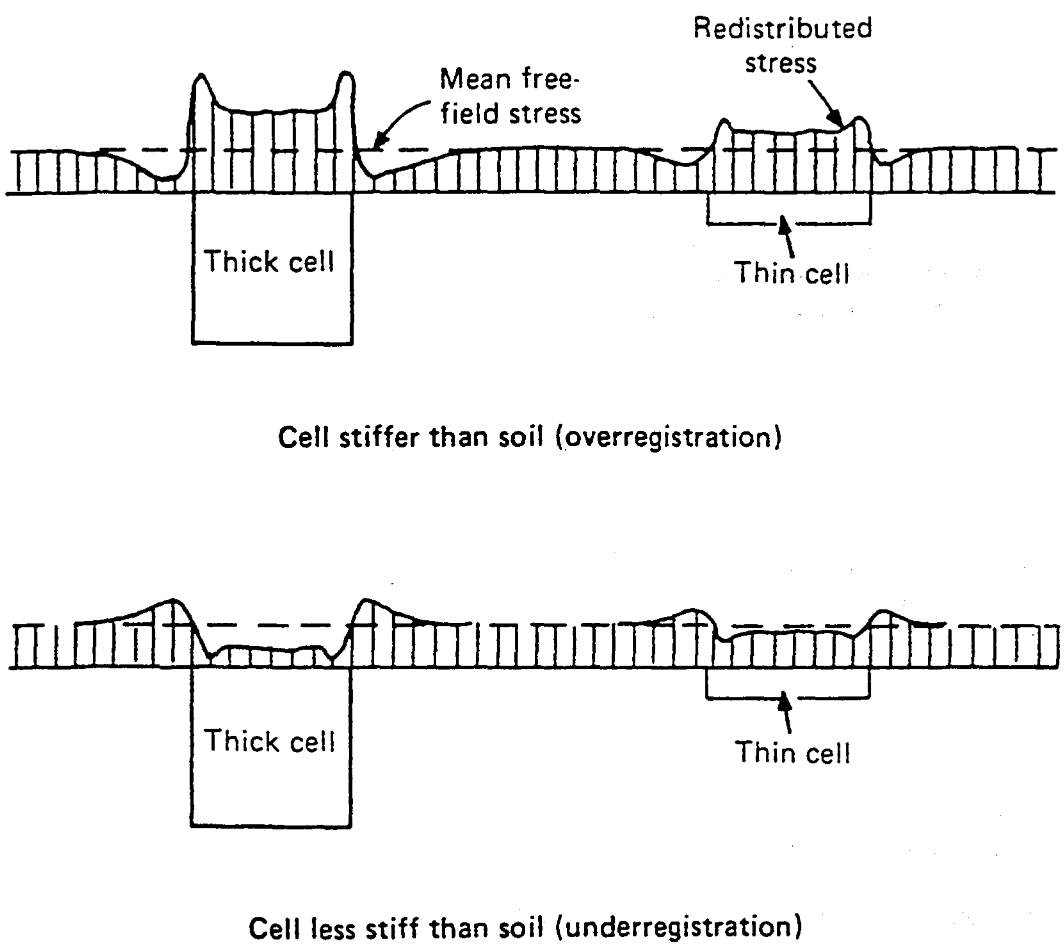


Figure 4-3: Effect of Embedment Earth Pressure Cell Aspect Ratio and Soil/Cell Stiffness Ratio. (After Selig, 1964)

Wilson (1984) comments that when earth pressure cells are installed in a horizontal plane in compacted fills for embankment dams, the cells typically register only 50-70% of the calculated added vertical stress as embankment construction continues. Wilson also comments that, because it is often difficult to shape the bottom of the excavation to the exact shape of the cell, bedding may be uneven and the application of vertical stress may deform the cell and cause additional error. The best choice of cell appears to be the type shown in Figure 4-1, with grooves to increase edge flexibility. Exact figures for accuracy cannot be given, but in the view of the author it would be unusual to achieve an accuracy better than ±20% of the true free-field stress.

4.3 CONTACT EARTH PRESSURE CELLS

Measurements of total stress against a structure are not plagued by so many of the errors associated with measurements within a soil mass, and it is possible to measure total stress at the face of a structural element with greater accuracy than within a soil mass. However, cell stiffness and the influence of temperature are often critical.

4.3.1 Standard Type of Contact Earth Pressure Cell

When installing an earth pressure cell to measure contact stress between soil and concrete, there is a possibility of uncoupling between the cell and concrete when temperature rises and the liquid in the cell expands during concrete cure. It is therefore preferable to construct the cell with one steel plate approximately 12 mm thick and to place that plate against the concrete, thereby ensuring that any expansion occurs outward. The layer of liquid should be as thin as possible. This arrangement is shown in Figure 4-4. This is similar to the embedment cell shown in Figure 4-1, with one of the active faces replaced by a thick inactive face.

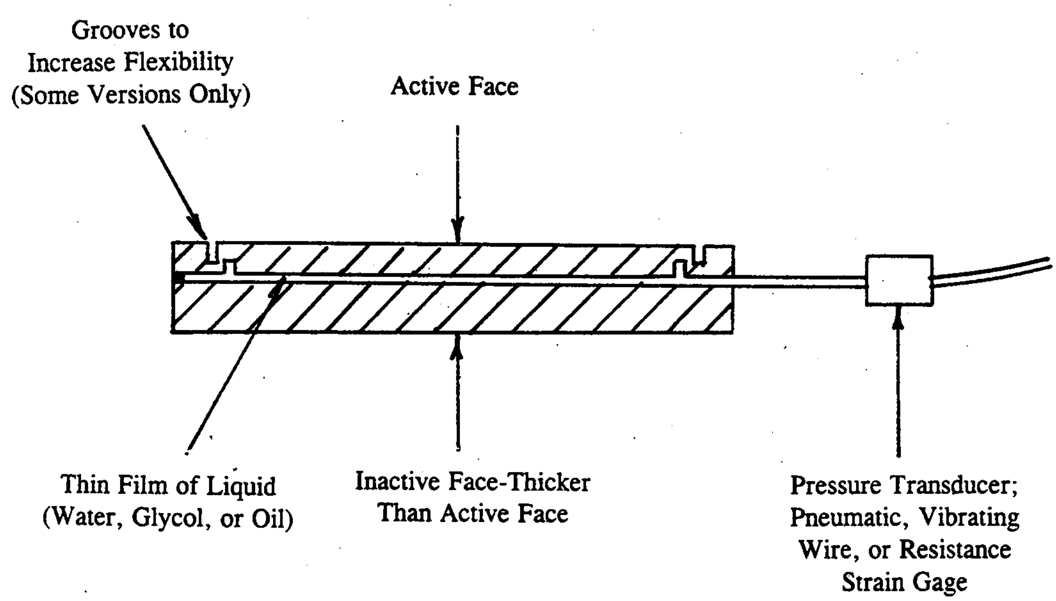


Figure 4-4: Schematic of Contact Earth Pressure Cell.

TABLE 4-1
MAJOR FACTORS AFFECTING MEASUREMENTS WITH EMBEDMENT EARTH PRESSURE CELLS

Factor	Description of Error	Correction Method[a]
Aspect ratio (ratio of cell thickness to diameter)	Cell thickness alters stress field around cell	Use relatively thin cells ($T/D < 1/10$)
Soil/cell stiffness ratio (ratio of soil stiffness to cell stiffness)	May cause cell to under- or over-register Error will change if soil stiffness changes	Design cell for high stiffness and use correction factor
Size of cell	Very small cells subject to scale effects and placement errors Very large cells difficult to install and subject to nonuniformbedding	Use intermediate size of cell: typically 230–300 mm diameter
Stress-strain behavior of soil	Measurements influenced by confining conditions	Calibrate cell under near-usage conditions[b]
Placement effects	Physical placement and backfilling causes alteration of material properties and stress field around cell	Use placement technique that causes minimum alteration of material properties and stress field[b]
Eccentric, nonuniform, and point loads	Soil grain size too large for cell size used Nonuniform bedding causes nonuniform loading	Increase active diameter of cell[b] Use hydraulic cells with grooved active faces in preference to other types[b] Take great care to maximize uniformity of bedding[b]
Proximity of structures and other embedded instruments	Interaction of stress fields near instruments and structure causes errors	Use adequate spacing
Concentrations of normal stress at edges of cell	Causes cell to under- or over-register, depending on stiffness of cell relative to soil	Use grooved active face and thin layer of liquid
Deflection of active face	Excessive deflection of active face changes stress distribution around cell by arching	Design cell for low deflection: use thin layer of liquid[b]
Placement stresses	Overstressing during soil compaction may permanently damage cell	Check cell and transducer design for yield strength (cells with pneumatic transducers have high overload capacity)[b]
Temperature	Temperature change causes change of cell reading	Design cell for minimum sensitivity to temperature; if significant temperature change is likely, measure temperature and apply correction factor determined during calibration[b]

[a] D = cell diameter; T = cell thickness; d = diaphragm diameter.
[b] Applies also to contact earth pressure cells. See Section 4.3.5

4.3.2 Cells for Measurement of Load at Toes of Driven Piles

Various types of cells have been developed for measurements of load at the toes of driven piles. The cells can be considered as load cells but are classified in this Module as contact earth pressure cells.

A cell with pneumatic transducers is described by Green et al. (1983) for measurement of toe load on driven prestressed concrete piles. The cell is divided into four independent quadrants as shown in Figure 4-5, each with a pneumatic pressure transducer, and covers almost the entire area of the pile toe. Design criteria includes adequate robustness to withstand driving forces and a thin enough active face to transmit toe pressure to the liquid in the cell.

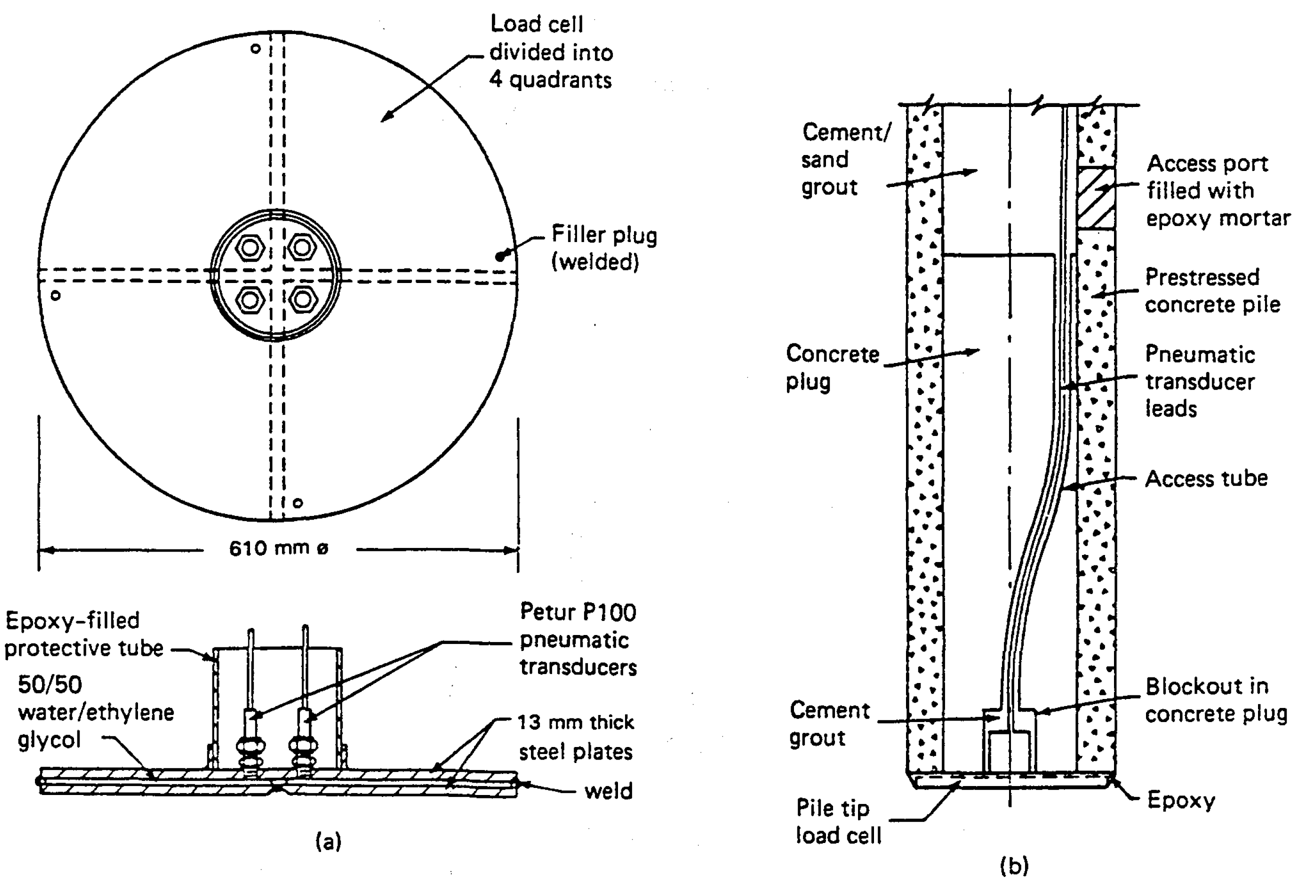


Figure 4-5: Contact Earth Pressure Cell for Measurement of Pile Toe Load: (a) Construction Details of Cell and (b) Cell Mounted on Pile. (After Green et al., 1983)

4.3.3 Cells for Measurement of Load at Toes of Drilled Shafts

Figure 4-6 shows a cell for measurement of load at the toe of a drilled shaft. The circular steel plates are held together by three steel bolts, one through the hollow-core of each load cell. The skirt and inflatable tube ensure that no load can bridge the cell during the load test.

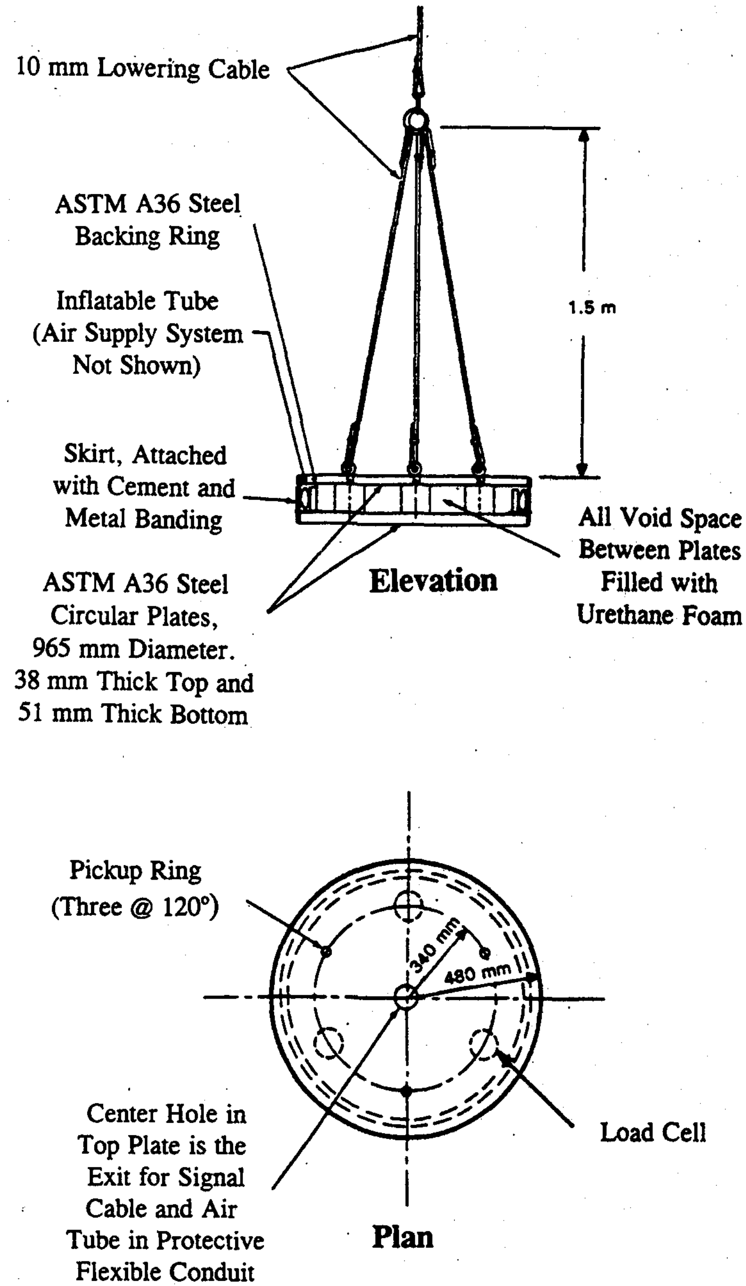


Figure 4-6: Schematic of Contact Earth Pressure Cell for Measurement of Load at the Base of a Drilled Shaft: Vibrating Wire Load Cells Mounted Between Two Thick Circular Plates. (After Dunnicliff, 1988, 1993)

4.3.4 Cells for Measurement of Stress on Sides of Driven Piles

Various types of cells are commercially available for attachment to sheet and H-piles, but measurement difficulties are great in both cases. If a cell is attached on the face of a pile, the stress field is likely to be disturbed significantly, with the possible exception of piles driven through soft clay. If the cell is attached within a cutout in an attempt to place the sensitive face in the same plane as the face of the pile, the cutout will usually weaken the section excessively. Even if an accurate measurement could be made, stresses on the surfaces of the sheet and H-piles are likely to be irregular, and a few point measurements are therefore of limited value.

Measurements on the sides of pipe piles are also difficult, because of the need to mount the sensitive face flush with the outside of the pile. Possibly the best approach is to manufacture special hydraulic cells with curvature to match the outside of the pipe and to use a thick grooved outer face. If the grooves are omitted, the curved face is likely to prevent flexing. Alternatively, rectangular cells could be used, with the long side parallel to the axis of the pipe, but the face would not be flush with the outside of the pile. For either option, the cells should be installed in cutouts, and measurement accuracy is likely to increase with increasing pile diameter.

Measurements on the sides of driven concrete piles can be made satisfactorily (e.g. Clemente, 1979), provided that a flat face (piles square or octagonal) is available.

Cells with pneumatic transducers have been shown to survive pile driving and are generally preferred.

4.3.5 Cells for Measurement of Stress on Sides of Slurry Walls

Contact earth pressure cells can be packaged for measurement of total stress on the sides of slurry walls. The cells are sometimes referred to as jack-out cells. The cell, together with a rigid steel support plate, a reaction plate of similar dimensions, and a hydraulic jack spanning between the support and reaction plates, is installed in the reinforcing cage. When the cage is in position in the slurry trench, the jack is activated and locked, forcing the cell into contact with the soil. The trench is then concreted.

4.3.6 Factors Affecting Measurements

Overall requirements for contact earth pressure cells are: good conformance, adequate sensing area, minimum sensitivity to nonuniform bedding, and a method of installation that will not seriously change the state of stress. These requirements can usually be accomplished by installing a flat-faced cell of adequate size with its sensitive face absolutely flush with the surface of the structure and by attention to correction methods for minimizing various sources of error. These methods are identified by a superscript *b* in Table 4-1. Various additional factors that affect measurements are discussed in the following two paragraphs.

Considering all these factors, the best possible accuracy at an individual cell is likely to be no better than $\pm 10\%$ of the time contact stress. If there is significant temperature variation, accuracy is likely to be much worse than $\pm 10\%$.

Although contact stresses may be reasonably uniform for the structure as a whole, stresses measured over areas the size of most contact earth pressure cells may be very irregular, owing to local variations in soil conditions. Measurements with contact earth pressure cells therefore often show considerable scatter. The more cells the better, but it is usually difficult to determine whether scatter results from real variations in stress or from measurement errors.

Embedment earth pressure cells are not usually subject to temperature changes, but contact cells are often near the atmosphere or near concrete pours that create changes in temperature, and temperature effects can cause significant measurement errors. The following discussion will elaborate. If a somewhat rigid container is filled with liquid, then the container is warmed, the liquid will expand and generally its pressure will increase. A hydraulic earth pressure cell will behave in the same way, hence will be sensitive to temperature changes. Most manufacturers provide a temperature calibration for the transducer, and a temperature sensor within the transducer. But it is not possible for manufacturers to provide a temperature calibration for the cell itself. When temperature changes at an installed earth pressure cell, the "correction" depends on the extent of restraint given to the cell by its surroundings. It would be possible to develop a cell calibration by immersing the cell in water at various temperatures, but this does not model field conditions correctly, because during calibration there is no restraint. In a full-embedment installation, where cells are buried within fills, this issue is rarely important, because the cells are usually below the zone of temperature change. However, if contact earth pressure cells are exposed to changing ambient temperature, such as at the faces of mechanically stabilized earth walls, soil-nailed walls and other types of retaining walls, data accuracy can be severely downgraded, and there does not appear to be a viable method for temperature correction.

CHAPTER 5.0
OVERVIEW OF HARDWARE FOR MEASUREMENT OF LOAD AND STRAIN IN STRUCTURAL MEMBERS

5.1 INTRODUCTION

Instruments for measuring load and strain in structures fall into two groups: load cells and strain gages. In each case, the transducers are used to measure small extensions and compressions. Load cells are interposed in the structure in such a way that structural forces pass through the cells, and strain gages are attached directly to the surface of the structure or are embedded within the structure to sense the extensions and compressions in the structure itself.

Typical load cell applications include load testing of piles, drilled shafts, tiebacks and rockbolts, and long-term performance monitoring of tiebacks and end-anchored rockbolts.

Strain gages are used where load cells cannot be interposed in the structure for reasons of geometry, capacity, or economy and where load and stress can be calculated with adequate accuracy from knowledge of the relationship between strain and stress. Strain gages are used, for example, to determine temporary or permanent stresses or loads in struts across braced excavations, in rockbolts, retaining walls, slurry walls and deep foundations.

5.2 LOAD CELLS

The types of load cell in general use are described below, and comparative information is given in Table 5-1.

Cells for measurement of load at the toes of driven piles and drilled shafts are classified in this Module as contact earth pressure cells, and are described in Chapter 4.

5.2.1 Hydraulic Load Cells

As shown in Figure 5-1, a hydraulic load cell consists of a flat liquid-filled chamber connected to a pressure transducer. The edges of the cell have either a bonded rubber/metal seal or a welded joint. A central hole can permit use with tiebacks or rockbolts. The pressure transducer can be a Bourdon tube pressure gage, requiring visual access to the cell for reading, or an electrical or pneumatic pressure transducer (Chapter 2) can be used to allow remote reading.

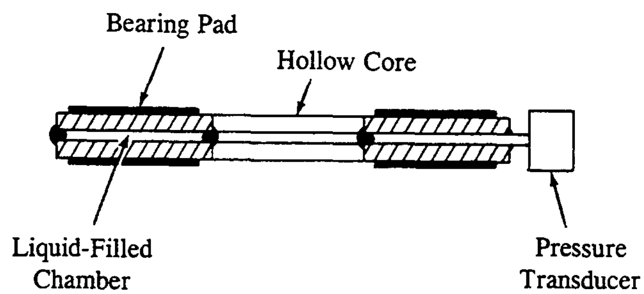


Figure 5-1: Schematic of Hydraulic Load Cell. Note: The Cell is Circular in Plan. (After Dunnicliff, 1988, 1993)

TABLE 5-1
LOAD CELLS

Type of Load Cell	Advantages	Limitations	Approximate Accuracy
Hydraulic (Figure 5-1)	Low profile Remote readout is possible	Requires large-area rigid bearing plates Potential for temperature sensitivity	±2–10%*
Electrical resistance (Figure 5-2)	Remote readout Readout can be automated	Low electrical output Lead wire effects Errors owing to moisture and electrical connections are possible Need for lightning protection should be evaluated Errors arise due to size mismatch between cell and adjacent components	±2–5%*
Vibrating wire	Remote readout Lead wire effects minimal Readout can be automated	Need for lightning protection should be evaluated	±2–5%*
Calibrated hydraulic jack (e.g. Figure 5-3)	Readily available	Low accuracy Error usually on unsafe side for load testing Should not be used alone for load measurement	±10–25%

* These are accuracies of the load cells when used with adequately designed and installed mounting arrangements. However, because of misalignment of load, off-center loading, and end effects, system accuracy is often no better than ±5–10%, and may be significantly worse if mounting arrangements are inadequate. Guidelines on mounting arrangements are given in Section 5.2.5.

5.2.2 Electrical Resistance Load Cells

Most electrical resistance load cells used in geotechnical applications consist of a cylinder of steel or aluminum alloy, with electrical resistance strain gages bonded to the outer periphery of the cylinder at its midsection as shown in Figure 5-2.

An electrical resistance strain gage is a conductor with the basic property that resistance changes in direct proportion to change in length. Several strain gages are used at regular intervals around the periphery. Half the gages are oriented to measure tangential strains and half to measure axial strains. They are connected to form a single full Wheatstone bridge network, thereby integrating individual strain gage outputs and reducing errors that result from load misalignment and off-center loading. In critical situations, the gages may be duplicated to provide two full bridge networks, one serving as a backup.

Sellers (1994) reports on tests made to examine the error caused by a size mismatch between electrical resistance load cells and adjacent hydraulic jacks. If the hydraulic jack is larger than the load cell there

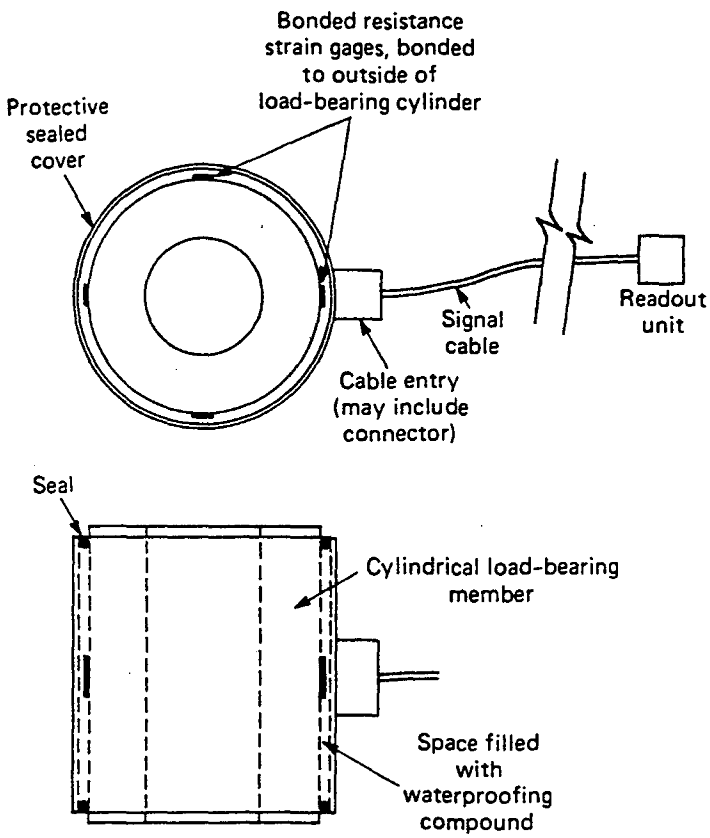


Figure 5-2: Schematic of Electrical Resistance Load Cell. (After Dunnicliff, 1988, 1993)

is a tendency for it to try to wrap the intervening bearing plate around the load cell and to bend the cylinder walls inward. If the hydraulic jack is smaller than the load cell it will try to push the intervening bearing plate through the hole in the load cell and to bend the cylinder walls outward. Sellers reports on errors of up to eight percent.

5.2.3 Vibrating Wire Load Cells

In most vibrating wire load cells, deformation of the load-bearing member is measured by using three or more vibrating wire transducers, and outputs from each transducer must be measured separately and averaged. The arrangement is similar to the electrical resistance load cell shown in Figure 5-2, but with vibrating wire transducers instead of bonded resistance strain gages. However the vibrating wire strain gages are installed at a diameter midway between the inside and outside diameters of the cylindrical load-bearing member, thereby almost eliminating errors caused by size mismatch between the cell and adjacent components (Section 5.2.2). Dunnicliff (1995a) reports on tests made by Sellers, subsequent to the tests reported above, to examine the errors caused by a size mismatch between vibrating wire load cells and adjacent hydraulic jacks. Maximum errors were only one percent.

5.2.4 Calibrated Hydraulic Jacks

Calibrated hydraulic jacks (Figure 5-3) are used for the application of load to rockbolts, tieback anchors, piles, drilled shafts, cross-lot struts, and other structural elements. They are also used for later determination of load in cross-lot struts and tieback anchors by lift-off testing, by determining the load required to free a loaded strut or anchor from its bearing surface.

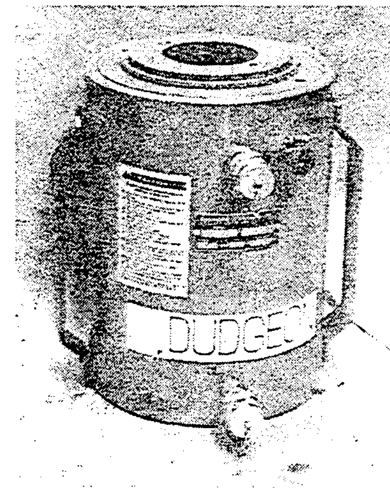


Figure 5-3: Calibrated Hydraulic Jack with Center Hole. (Courtesy of Richard Dudgeon, Inc., Bridgeport, CT)

Reliance on the measured pressure in the jack fluid as the sole method for load determination can often lead to significant inaccuracies. Misalignment of load, off-center loading, nonparallel bearing plates, and transverse relative movement of bearing plates all cause friction between the piston and cylinder, so that hydraulic pressure does not give a good indication of load. Temperature changes and pressure gage inaccuracy may cause additional errors. During laboratory calibration, the jack will usually be inserted in a high-quality testing machine, fitted with a spherical seating in the loading head, and rigid bearing plates. The jack will be placed with its axis in line with the loading axis. In the field, however, the jack bearing plates may flex and will rarely be parallel, loading will usually be misaligned and off-center, and consequently friction between the piston and cylinder will be significant. When used as the active member while load testing piles or tiebacks, errors will be on the unsafe side during loading, because the actual load will be less than the load implied from the laboratory calibration. When unloading is accomplished by releasing jack fluid pressure, the actual load will be higher than the load implied from the laboratory calibration.

Davidson (1966) and Fellenius (1984) report on significant errors when relying on calibrated hydraulic jacks for load measurement. Figure 5-4 shows data from pile load testing and indicates that during loading the jack typically overregisters by 10–25%, and during unloading it typically underregisters by 5%.

A further source of error in the use of calibrated hydraulic jacks results from incorrect calibration. If the fluid pressure in the jack must be used for load determination, the calibration procedure should model the method of field loading, in which the jack is usually in the active mode as the load is both increasing and decreasing. For example, when calibrating a jack for tieback proof testing, the distance between the loading plates of the testing machine should initially be set equal to the expected jack height at alignment load, the jack activated to alignment load, and the testing machine read as the passive member. The distance between loading plates should then be increased to model the expected piston travel to the next load increment, the jack activated to that load, and a calibration reading taken. The procedure should be repeated in increments to the maximum load. Calibration during load decrease should be made in a similar way, with the jack in the active mode.

In summary, load measurements based on jack fluid pressure may be significantly in error, may be on the unsafe side, and are usually unacceptable. Good practice when load testing piles, drilled shafts, and tiebacks includes use of a load cell in series with the hydraulic jack. When piles and drilled shafts are being load tested, a swivel head should also be used. When lift-off tests to determine load in a structural element are being performed, a load cell should be used in series with the jack.

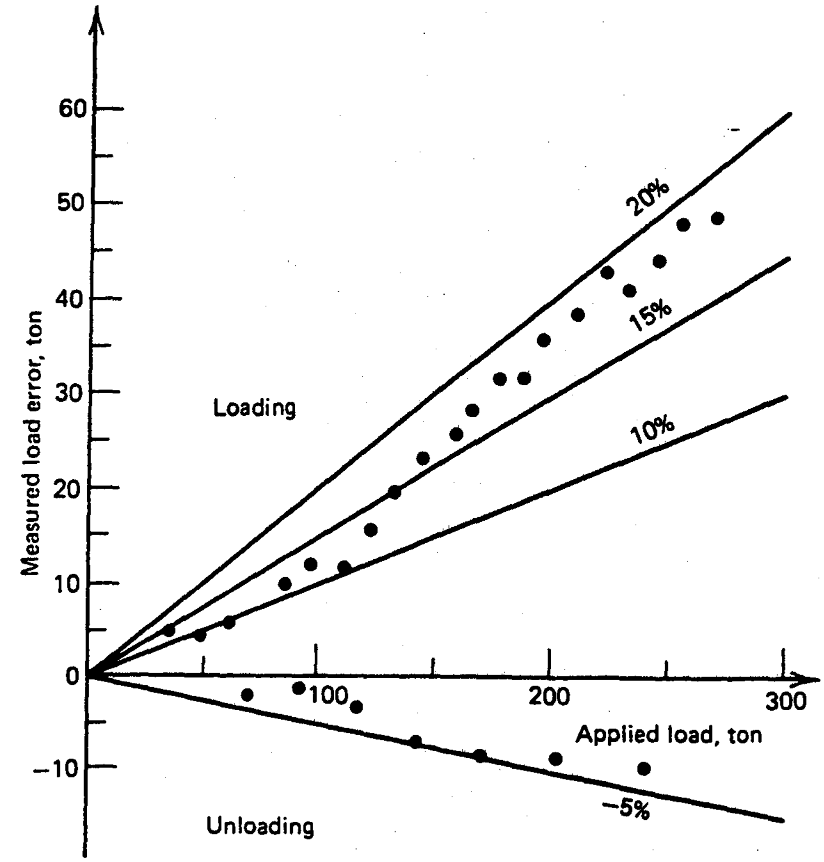


Figure 5-4: Typical Error in Load Determined from Fluid Pressure in Hydraulic Jack. (After Fellenius, 1984). Reprinted from Geotechnical News, Vol. 2, No. 4, December 1984

5.2.5 Guidelines on Use of Load Cells

General guidelines on instrumentation use are given in Chapter 12. The following additional guidelines apply specifically to load cell use. Table 5-1 provides guidelines for selecting among the different types of load cell.

The major sources of inaccuracy in hollow-core load cells are off-center loading, misalignment of load, and end effects. Errors can be minimized by:

- Use of swivel bearing washers and/or soft washes.
- Selecting a load cell with a height at least four times the cylinder wall thickness.
- Selecting a cell which is gaged for eccentric loading, with a minimum of four gaged points at 90°.
- Avoiding any significant size mismatch between the cell and adjacent components (Sections 5.2.2 and 5.2.3).
- Selecting a cell which is gaged at a diameter midway between the inside and outside diameters of the cylindrical load-bearing member (i.e. favoring a vibrating wire load cell).
- Use of thick bearing plates, ground flat, smooth and parallel. Typical thicknesses are 38, 63 and 76 mm for cell capacities of 0.7, 1.8 and 2.2 MN respectively.
- Installing the cell, bearing plates and hydraulic jack in alignment within 3 mm, using centralizer bushings as necessary.

5.3 SURFACE-MOUNTED STRAIN GAGES

Strain gages in general use for measurements on structural surfaces are described in the following sections and compared in Table 5-2.

5.3.1 Surface-Mounted Mechanical Strain Gages

Portable gages with dial indicators are the most common mechanical gages for measurement of surface strain in geotechnical applications.

A typical gage is shown in Figure 5-5. The instrument consists of an invar bar with a conical locating point at each end, one fixed to the bar and the other pivoting on a knife edge. The pivoting movement is transmitted to a dial indicator via a lever arm. A separate invar bar is supplied with the gage for use as a reference standard. Small stainless steel disks with central indentations are either cemented directly to the structure or mounted on anchor pins set in drilled holes. A setting-out bar is used to ensure correct initial disk spacing.

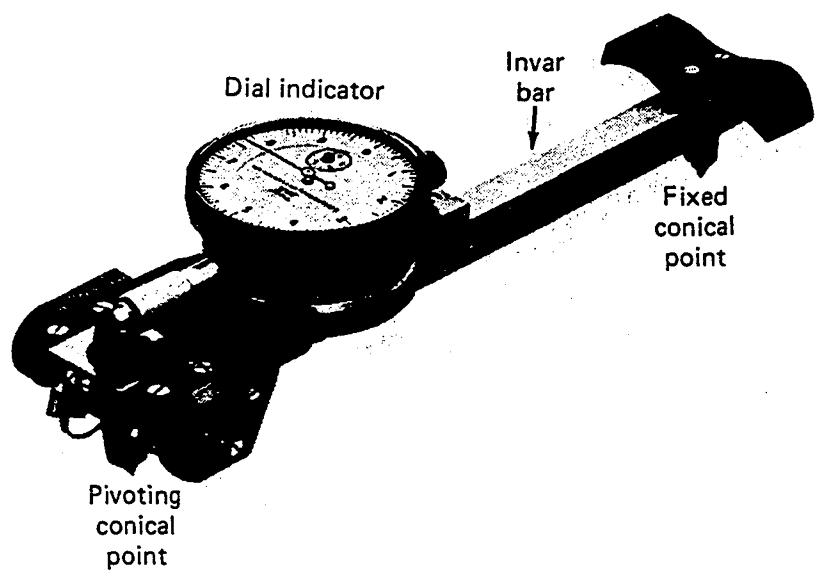


Figure 5-5: Demec Strain Gage. (Courtesy of Mayes Instruments, Ltd., Windsor, England)

5.3.2 Surface-Mounted Vibrating Wire Strain Gages

The vibrating wire transducer is described in Chapter 2, and a schematic of a surface-mounted gage is shown in Figure 2-9. The surface-mounted gage is available in the two basic configurations illustrated in Figure 5-6.

The configuration designed for installation by arc welding (Figure 5-6a) or bolting is generally between 100–200 mm long. The end blocks are normally installed by arc welding to the structure, using a solid spacer bar to hold the blocks at their correct spacing. Alternatively, the end blocks can be bolted to the structure, requiring appropriate threaded studs or holes in the blocks. The remainder of the gage is then fixed into the end blocks and bolted in place.

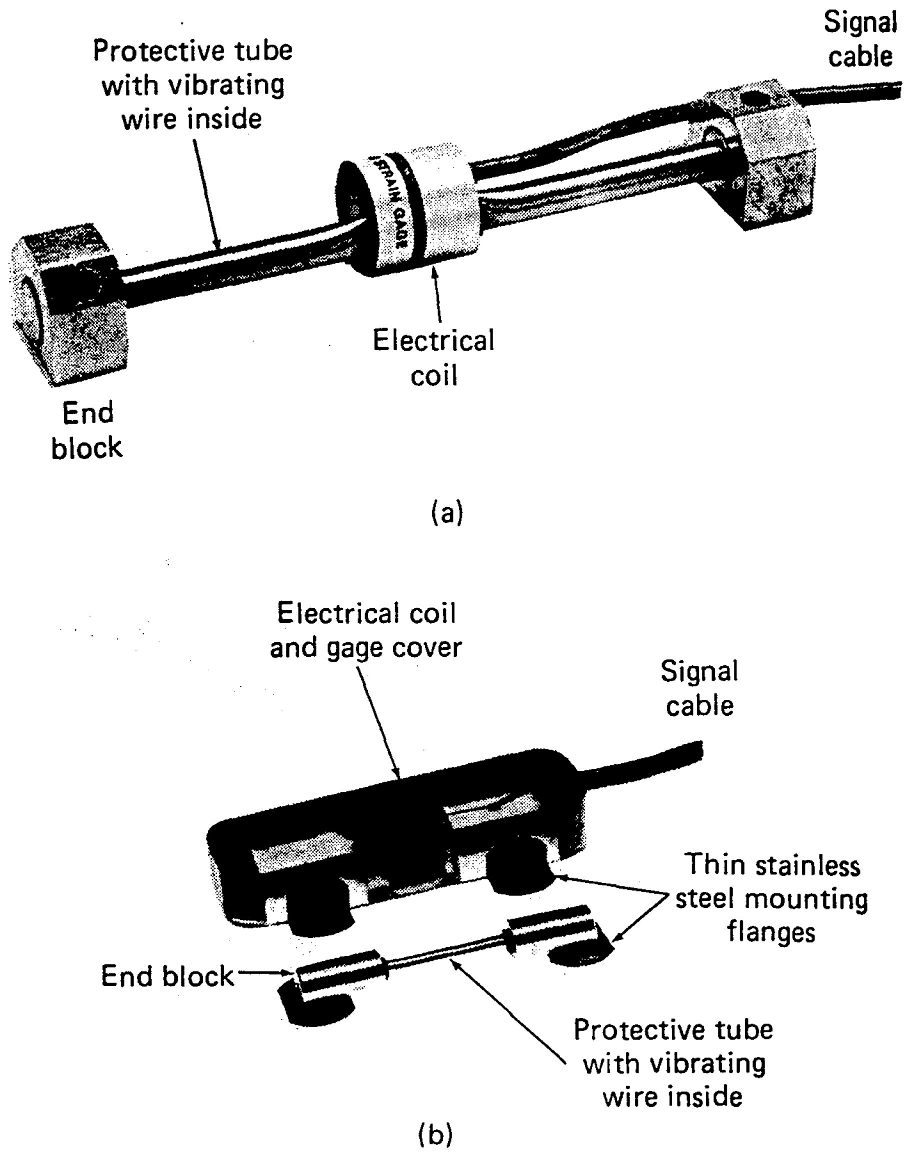


Figure 5-6: Surface-Mounted Vibrating Wire Strain Gages: (a) Gage Installed by Arc Welding (Gage Can Also Be Fitted with Different End Blocks for Installation by Bolting) and (b) Gage Installed by Spot Welding. (Courtesy of Geokon, Inc., Lebanon, NH)

The spot welded gage is typically 50 mm long. The version shown in Figure 5-6b has O-ring seals between the protective tube and end blocks, so that the tube remains unstressed. An internal spring holds the wire at an initial tension, normally preset at midrange, but the initial tension can be set during installation, allowing for maximum range in compression or tension as required. Another version has the protective tube welded to a thin stainless steel mounting flange along the entire length of the gage, such that the tube is stressed as strain occurs, and in this version no field adjustment of wire tension is made. All versions are installed on steel by using a small hand-held capacitive discharge spot welder to weld the mounting flange to the structure, with weld points at approximately 1.5 mm spacing.

TABLE 5-2
SURFACE-MOUNTED STRAIN GAGES

Gage Type	Advantages	Limitations	Typical Gage Length	Typical Range (microstrain)	Sensitivity (microstrain)	Approximate Accuracy (microstrain)
Mechanical gage (e.g. Figure 5-5)	Simple Inexpensive Waterproofing not required Calibration can be checked at any time No delicate parts attached to structure	Requires access to structure Requires extreme care to read	50–2000 mm	Up to 50,000	3–50	±5–200
Vibrating wire gage (e.g. Figure 5-6)	Remote readout Lead wire effects minimal Readout can be automated Factory water-proofing Arc-welded or bolted version is reusable	Cannot be used to measure high-frequency dynamic strains Need for lightning protection should be evaluated	100–200 mm	3000	0.2–2	±5–50
Electrical resistance gage (e.g. Figures 5-7, 5-8)	Remote readout Readout can be automated Suitable for monitoring dynamic strains	Low electrical output Lead wire effects Errors owing to moisture, temperature, and electrical connections are possible Installation of bonded gages requires great skill and experience Need for lightning protection should be evaluated	0.25–150 mm	20,000	1–4	±1–100

When compared with the arc-welded gage, advantages of the spot welded gage are small size, an installation procedure not requiring arc welding, and minimum errors that result from bending of the structure because of the close proximity between the vibrating wire and the structure surface. However, if adequate space is available, the larger arc-welded version is generally preferred.

A special vibrating wire gage, referred to as a "strand gage" is available for monitoring strains in stranded tieback tendons (Dunnicliff, 1994).

5.3.3 Surface-Mounted Electrical Resistance Strain Gages

An electrical resistance strain gage is a conductor with the basic property that resistance changes in direct proportion to change in length. Two types are used for surface strain measurements in geotechnical applications; bonded foil (e.g. Figure 5-7) and weldable gages (e.g. Figure 5-8). When selecting between bonded foil and weldable strain gages, the primary issues are environmental conditions, accuracy, cost, and size. Environmental conditions often override all other considerations, leading to the selection of weldable gages.

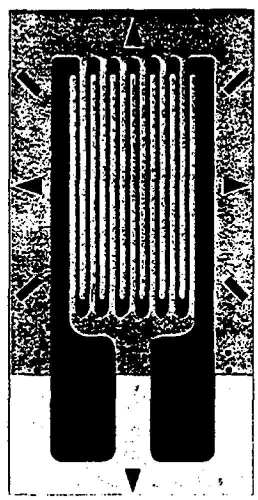


Figure 5-7: Bonded Foil Electrical Resistance Strain Gage. (After Dunnicliff, 1988, 1993)

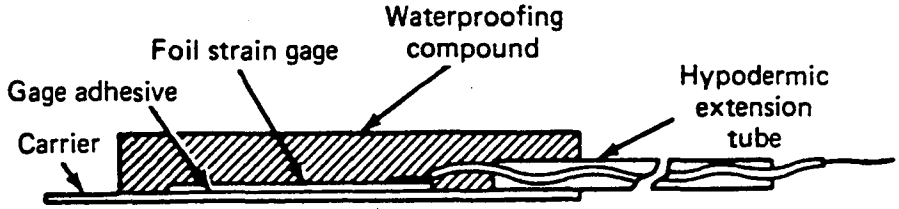


Figure 5-8: Weldable Electrical Resistance Strain Gage with Bonded Foil Transducer. (After Dunnicliff, 1988, 1993)

5.3.4 Relationship Between Strain and Stress

Strain data are rarely of interest; the data are merely a step in the determination of stress. When making measurements on steel, provided that modulus is known and temperature is measured, conversion from strain to stress is straightforward. However, stress determination in concrete is by no means straightforward, and accurate results should not be expected.

Strain in concrete can be caused by several factors other than stress change, and a strain gage responds to all causes of strain. Strain other than that caused by stress may be due to creep (strain under constant stress), shrinkage and swelling (moisture content change), temperature change, and the progress of autogenous volume change (dimensional change that is self-generated and not due to temperature, moisture change, or stress); other strain may also occur as concrete cures. Strain due to creep is generally more extensive in highly stressed prestressed concrete than in conventional reinforced concrete; therefore, for structural members such as pre-stressed concrete piles, creep strains are particularly significant.

A special effort must therefore be made to determine the relationship between strain and stress. Conversion methods are summarized below.

All Measurements

- Determine whether gages are thermally matched to structure by using laboratory tests. Develop any necessary temperature correction factors, either in the laboratory or by using the dummy gage procedure. If temperature correction factors are required, measure temperature at each gage location.
- Wherever possible, take readings when temperature is uniform throughout the member.

Measurements on Steel

- Use modulus of steel and any necessary temperature correction factors.

Measurements in or on Concrete

Use one of the following five options.

- Subject a specimen of the same concrete to the same influences as the prototype, in a known stress field, and measure resulting strain. This is sometimes attempted in the laboratory, but it is usually not possible to match laboratory and prototype conditions. A better approach is to install strain gages on a part of the prototype that is subjected to a known stress, thereby ensuring that the specimen behaves in a similar way to the prototype. For example, while load testing a concrete pile or drilled shaft, an unconfined compression test specimen can be created near the pile butt or shaft top, either above the ground surface or within a sleeved section just below the ground surface. It is important to model moisture conditions correctly, because strains caused by moisture content change can be a significant part of total strain. The specimen at a pile butt or shaft top should therefore be kept wet if the subsurface gages are below the water table.
- Interpose a load-measuring device across at least one entire cross section and relate the known load to strain measured at nearby strain gage locations. The same relationship is applied to strains measured elsewhere. For example, a contact earth pressure cell can be installed at the toe of a pile or drilled shaft, or a load cell can sometimes be installed at a splice in a driven pile.
- Use creep curves developed for each particular project.
- Use averaged empirical data.
- Use combinations of the above options.

5.4 EMBEDMENT STRAIN GAGES

The primary application for embedment strain gages is measurement of strain in concrete. Embedment strain gages in general use are described in the following sections and compared in Table 5-3.

When using embedment strain gages in concrete, most geotechnical applications involve simple cross sections subjected to compression and some bending, for example, driven piles and drilled shafts. We are not well able to monitor strains in complex cross sections involving rapid strain gradients, large tensile stresses, and/or cracked tensile sections.

5.4.1 Embedment Strain Gages: Vibrating Wire Types

There are two types of embedment strain gage with a vibrating wire transducer. The first type is similar to the arc-welded surface-mounted gage described in Section 5.3.2, except that large end flanges (Figure 5-9) replace the end blocks. Most gages are supplied with a preset tension, which can be specified by the user.

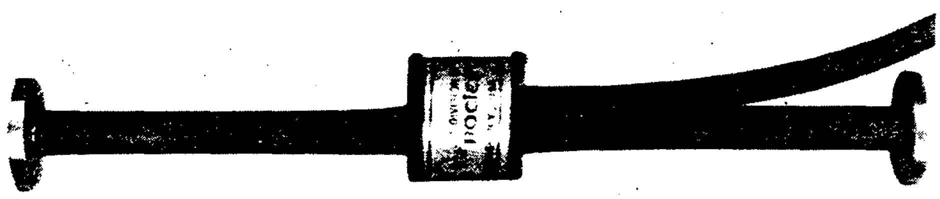


Figure 5-9: Vibrating Wire Strain Gage for Embedment in Concrete. (Courtesy of Roctest, Inc., Plattsburgh, NY)

The second type of gage consists of a vibrating wire transducer mounted in the central portion of a length of steel bar, such as reinforcing steel. The use of a length of steel bar, embedded in concrete to measure strain, is sometimes referred to as a sister bar, or a rebar strain meter. Figure 5-10 shows a sister bar suitable for installation in a drilled shaft, to provide strain data during a load test.

5.4.2 Embedment Strain Gages: Electrical Resistance Types

Embedment strain gages with electrical resistance transducers have largely been superseded by the vibrating wire gages described above. Descriptions and evaluations of four electrical resistance types are given by Dunnicliff (1988, 1993): Carlson elastic wire strain meters, bonded foil or weldable resistance sister bars, Mustran cells and plastic encased gages.

5.4.3 Relationship Between Strain and Stress

Relationships between strain and stress for surface measurements are discussed in Section 5.3.4 and apply also to measurements made **within** concrete.

An additional problem for embedment strain gages is the inclusion effect, whereby the presence of the gage may distort the strain field so that the measured strain is significantly different from the strain that would occur if the gage were not present. Guidelines for minimizing the inclusion effect are given by Dunnicliff (1988, 1993).

TABLE 5-3
EMBEDMENT STRAIN GAGES

Gage Type	Advantages	Limitations	Typical Gage Length	Typical Range (microstrain)	Sensitivity (microstrain)	Approximate Accuracy (microstrain)
Vibrating wire; type similar to arc-welded surface-mounted gage (e.g. Figure 5-9)	Lead wire effects minimal Remote readout Readout can be automated Factory waterproofing	Cannot be used to measure high-frequency dynamic strains Need for lightning protection should be evaluated	100–200 mm	3000	0.2–2	±5–50
Vibrating wire; sister bar type (Figure 5-10)	Robust Easy to install Lead wire effects minimal Remote readout Readout can be automated Factory waterproofing	Cannot be used to measure high-frequency dynamic strains Special design features required to minimize inclusion effects Sister bar must be small relative to size of structural member Need for lightning protection should be evaluated	De-bonded length, plus 100 x bar diameter	3000	0.2–2	±5–50

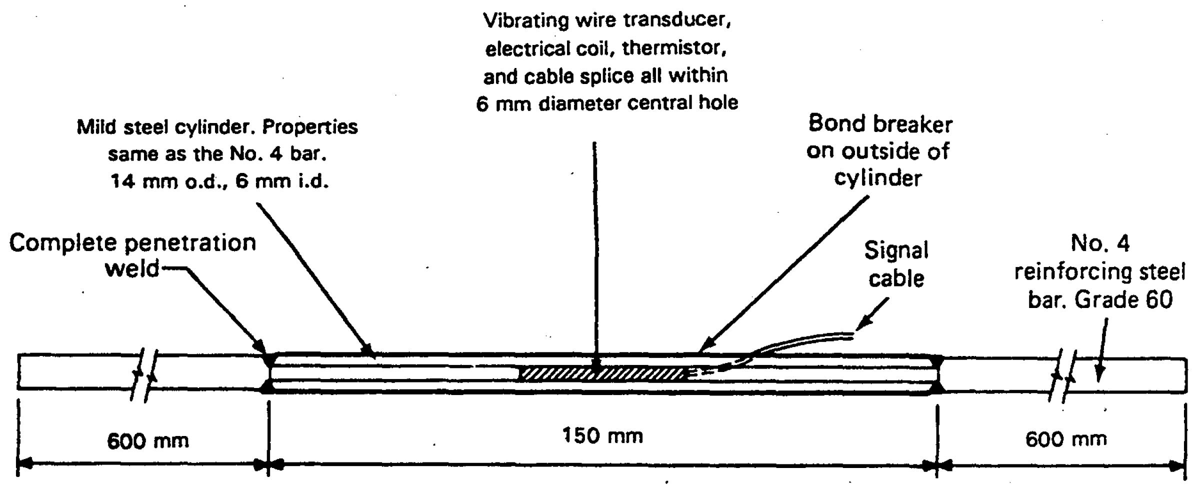
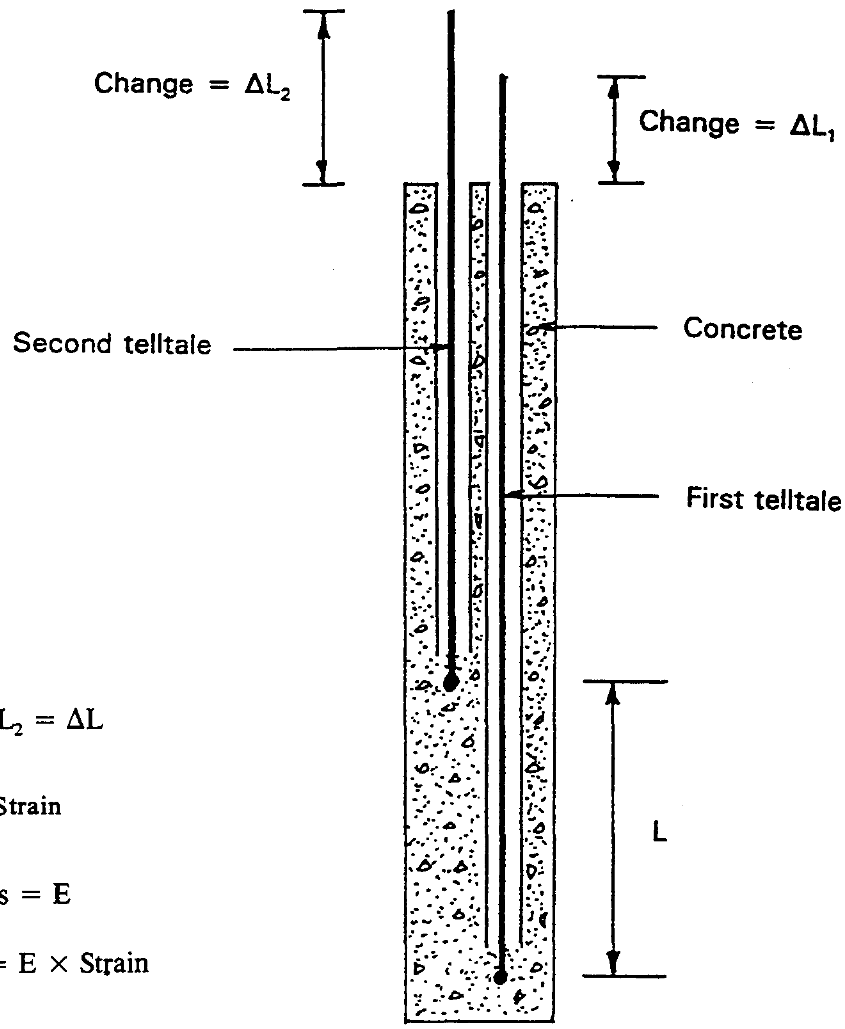


Figure 5-10: Schematic of Sister Bar with Vibrating Wire Transducer. (After Dunnicliff, 1988, 1993)

5.5 MULTIPLE TELLTALES

A pair of telltales can be used to monitor strain. The arrangement is shown in Figure 5-11, and is essentially the same as a multipoint fixed borehole extensometer (Section 3.9). The average strain between the two attachment points is calculated as shown on the figure. It is necessary, however, to assume the shape of the load transfer plot between the telltale attachment points: if a straight line is assumed, the calculated stress will be attributed to the location midway between the telltale attachment points. If the load at one attachment point is known, the load at the other can be calculated if the same straight line assumption is made, but if the line is not straight, there may be a large error in the calculated load. Fellenius (1980, 1987) provides guidelines on the interpretation of telltale data. When planning multiple telltale measurements in driven piles and drilled shafts, one should recognize that any telltale measurement may be in error by at least ± 0.5 mm, and this can create a significant error in the calculated stress. Errors in load transfer data can be minimized by installing telltales at more than two attachment points, so that data from each pair can be evaluated in light of data from adjacent pairs. Because load transfer calculations are extremely sensitive to any error in telltale data, it is advisable to install duplicate telltales at each attachment point. If they agree, they provide confidence in the data, and if they disagree the adjacent telltales are used to evaluate which is likely to be more correct.

Electrical linear displacement transducers can be mounted at the accessible ends of the telltales to permit remote monitoring. An example is shown in Figure 3-27.



$\Delta L_1 - \Delta L_2 = \Delta L$

$\frac{\Delta L}{L}$ = Strain

Modulus = E

Stress = E × Strain

Figure 5-11: Multiple Telltales for Stress Determination.

CHAPTER 6.0
SYSTEMATIC APPROACH TO PLANNING MONITORING PROGRAMS

6.1 INTRODUCTION

Planning a monitoring program using geotechnical instrumentation is similar to other engineering design efforts. A typical engineering design effort begins with a definition of an objective and proceeds through a series of logical steps to preparation of plans and specifications. Similarly, the task of planning a monitoring program should be a logical and comprehensive engineering process that begins with defining the objective and ends with planning how the measurement data will be implemented.

Unfortunately, there is a tendency among some engineers and geologists to proceed in an illogical manner, often first selecting an instrument, making measurements, and then wondering what to do with the measurement data. Franklin (1977) indicates that a monitoring program is a chain with many potential weak links, and breaks down with greater facility and frequency than most other tasks in geotechnical engineering.

Systematic planning requires special effort and dedication on the part of responsible personnel. The planning effort should be undertaken by personnel with specialist expertise in applications of geotechnical instrumentation. Recognizing that instrumentation is merely a tool, rather than an end in itself, these personnel should be capable of working in a *team-player* capacity with the project design team.

Planning should proceed through the steps listed below. The steps are summarized in checklist form in Appendix A. All steps should, if possible, be completed before instrumentation work commences in the field.

During the course the systematic approach to planning will be illustrated with an example of an embankment on soft ground. Overhead viewgraphs used during the course are included in Appendix B.

6.2 DEFINE THE PROJECT CONDITIONS

If the engineer or geologist responsible for planning a monitoring program is familiar with the project, this step will usually be unnecessary. However, if the monitoring program is planned by others, a special effort must be made to become familiar with project conditions. These include project type and layout, subsurface stratigraphy and engineering properties of subsurface materials, groundwater conditions, status of nearby structures or other facilities, environmental conditions, and planned construction method. If the monitoring program has been instigated to assist in finding facts during a crisis situation, such as a landslide, all available knowledge of the situation should also be assimilated.

6.3 PREDICT MECHANISMS THAT CONTROL BEHAVIOR

Prior to developing a program of instrumentation, one or more working hypotheses must be developed for mechanisms that are likely to control behavior. The hypotheses must be based on a comprehensive knowledge of project conditions, as described above.

6.4 DEFINE THE GEOTECHNICAL QUESTIONS THAT NEED TO BE ANSWERED

Every instrument on a project should be selected and placed to assist in answering a specific question: if there is no question, there should be no instrumentation. Before addressing measurement methods themselves, a listing should be made of geotechnical questions that are likely to arise during the design, construction, or operation phases. Various potential geotechnical questions are posed in Chapters 7 through 11 in this Module.

6.5 DEFINE THE PURPOSE OF THE INSTRUMENTATION

Instrumentation should not be used unless there is a valid reason that can be defended. When using this chapter to assist with planning a monitoring program, if engineers or geologists are unable to define a clear purpose for the program, they should cancel the program and proceed no further through this chapter. Peck (1984) states, "The legitimate uses of instrumentation are so many, and the questions that instruments and observation can answer so vital, that we should not risk discrediting their value by using them improperly or unnecessarily".

6.6 SELECT THE PARAMETERS TO BE MONITORED

Parameters include groundwater pressure, joint water pressure, total stress, deformation, load and strain in structural members, and temperature. The question *which parameters are most significant?* should be answered.

Variations in parameters can result both from *causes* and *effects*. For example, the primary parameter of interest in a slope stability problem is usually deformation, which can be considered as the *effect* of the problem, but the *cause* is frequently groundwater conditions. By monitoring both cause and effect, a relationship between the two can often be developed, and action can be taken to remedy any undesirable effect by removing the cause.

Most measurements of pressure, stress, load, strain, and temperature are influenced by conditions within a very small zone and are therefore dependent on local characteristics of that zone. They are often essentially *point* measurements, subject to any variability in geologic or other characteristics, and may therefore not represent conditions on a larger scale. When this is the case, a large number of measurement points may be required before confidence can be placed in the data. On the other hand, many deformation measuring devices respond to movements within a large and representative zone. Data provided by a single instrument can therefore be meaningful, and deformation measurements are generally the most reliable and least ambiguous.

6.7 PREDICT MAGNITUDES OF CHANGE

Predictions are necessary so that required instrument ranges and required instrument sensitivities or accuracies can be selected.

An estimate of the maximum possible value, or the maximum value of interest, leads to a selection of instrument range. This estimate often requires substantial engineering judgment, but on occasion it can be made with a straightforward calculation, as is the case with maximum pore water pressure in a clay foundation beneath the centerline of an embankment.

An estimate of the minimum value of interest leads to a selection of instrument sensitivity or accuracy. There is a tendency to seek unnecessarily high accuracy, when in fact high accuracy should often be sacrificed for high reliability if the two are in conflict. High accuracy often goes hand in hand with delicacy and fragility. In some instances, high accuracy may be necessary where small changes in the measured variable have significant meaning, or where only a short time is available for defining trends, for example, when establishing the rate of slide movement from inclinometer data. Parametric studies can often be carried out to assist in establishing range, accuracy and sensitivity.

If measurements are for construction control or safety purposes, a predetermination should be made of numerical values that indicate the need for remedial action. These values are referred to as hazard warning levels, response values, or alert levels. They will often be in terms of rate of measured change, rather than absolute magnitude. Hazard warning levels may be based on clearly defined performance criteria - for example, where an acceptable differential settlement has been established for a structural foundation - or may be based on substantial engineering judgment, requiring a general assessment of ground behavior modes and mechanisms of potential problems or failures. When in doubt, several hazard warning levels should be established. The concept of green, yellow, and red hazard warning levels is also useful. Green indicates that all is well, yellow indicates the need for cautionary measures including an increase in monitoring frequency, and red indicates the need for timely remedial action.

6.8 DEVISE REMEDIAL ACTION

Inherent in the use of instrumentation for construction purposes is the absolute necessity for deciding, in advance, a positive means for solving any problem that may be disclosed by the results of the observations (Peck, 1973). If the observations should demonstrate that remedial action is needed, that action must be based on appropriate, previously anticipated plans.

As described above, several hazard warning levels may be identified, each requiring a different plan. Planning should ensure that required labor and materials will be available so that remedial action can proceed with minimum and acceptable delay and so that personnel responsible for interpretation of instrumentation data will have contractual authority to initiate remedial action. An open communication channel should be maintained between design and construction personnel, so that remedial action can be discussed at any time. A special effort will often be required to keep this channel open, both because the two groups sometimes tend to avoid communication and because the contract for design personnel may have been terminated. Arrangements should be made to determine how all parties will be forewarned of the planned remedial actions.

6.9 ASSIGN TASKS FOR DESIGN, CONSTRUCTION, AND OPERATION PHASES

When assigning tasks for monitoring, the party with the greatest vested interest in the data should be given direct line responsibility for producing it accurately. The various tasks involved in accomplishing a monitoring program, together with alternative choices of the parties available for performing them, are listed in Table 6-1. It is useful to complete this chart during the planning stage by indicating the responsible party of each task.

Several of the tasks involve the participation of more than one party. In cases where the state is also the designer, there will be no design consultant. Instrumentation specialists may be employees of the state or the design consultant, or may be consultants with special expertise in geotechnical instrumentation. All tasks assigned to instrumentation specialists should be under the supervision of one individual.

If construction contractors have economic or professional incentive to contribute toward good data, they should be assigned major responsibilities. If the instrumentation program has been instigated by the construction contractor, clearly the contractor will have responsibility for all tasks. However, if the instrumentation program has been instigated by the state, as is usually the case, the construction contractor will often regard it as an interference with normal construction work and the contractor's participation should be minimized. The contractor will usually be responsible for providing support services during installation, and access during the data collection phase. Instrument selection and procurement, factory calibration, installation, regular calibration and maintenance, and data collection, processing, and presentation should preferably be under the direct control of the state or instrumentation specialist selected by the state. When any of these tasks are performed by the construction contractor, data quality is often in doubt. Data interpretation and reporting should be the direct responsibility of the state, the design consultant, or instrumentation specialist selected by the state.

While completing Table 6-1 it may become evident that personnel are not available for all tasks, leading either to assignment of additional personnel or to a change in direction of the monitoring program. For example , if personnel available for data collection are insufficient, it may be appropriate to turn towards use of automatic data acquisition systems; this decision will affect instrument selection.

Task assignment should include planning of liaison and reporting channels. Assignments should clearly indicate who has overall responsibility and contractual authority for implementing the results of the measurements and observations.

TABLE 6-1
CHART USED FOR TASK ASSIGNMENT

Task	Responsible Party			
	State	Design Consultant	Instrumentation Specialist	Construction Contractor
Plan monitoring program				
Procure instruments and make factory calibrations				
Install instruments				
Maintain and calibrate instruments on regular schedule				
Establish and update data collection schedule				
Collect data				
Process and present data				
Interpret and report data				
Decide on implementation of results				

6.10 SELECT INSTRUMENTS

The preceding eight steps should be completed before instruments are selected. Instruments are described in Chapters 2 through 5 in this Module.

When selecting instruments, the overriding desirable feature is **reliability**. Inherent in reliability is maximum simplicity.

Lowest cost of an instrument should never be allowed to dominate the selection, and the least expensive instrument is not likely to result in minimum total cost. In evaluating the economics of alternative instruments, the overall cost of procuring, calibration, installation, maintenance, monitoring, and data processing should be compared.

6.11 SELECT INSTRUMENTATION LOCATIONS

The selection of instrument locations should reflect predicted behavior and should be compatible with the method of analysis that will later be used when interpreting the data. Numerical modeling methods are often helpful in identifying critical locations and preferred instrument orientations. A practical approach to selecting instrument locations entails three steps.

First, zones of particular concern are identified, such as structurally weak zones, most heavily loaded zones, or zones where highest pore water pressures are anticipated, and appropriate instrumentation is located. If there are no such zones, or if instruments are also to be located elsewhere, a second step is taken. A selection is made of zones, normally cross sections, where predicted behavior is considered representative of behavior as a whole. When considering which zones are representative, variations in both geology and construction procedures should be considered. These cross sections are then regarded as *primary instrumented sections*, and instruments are located to provide comprehensive performance data. There should usually be at least two such primary instrumented sections. Third, because the selection of representative zones may be incorrect, instrumentation should be installed at a number of *secondary instrumented sections,* to serve as indices of comparative behavior. Instruments at these secondary sections should be as simple as possible and should also be installed at the primary sections so that comparisons can be made. For example, instrumentation of a tieback wall might entail selection of two or three primary cross sections for installation of optical survey points, inclinometers, and load cells. Optical survey points would also be installed at a large number of secondary sections and used for monitoring both horizontal and vertical deformation of the wall. If in fact the behavior at a secondary section appears to be significantly different from the behavior at the primary sections, additional instrumentation may be installed at the secondary section as construction progresses.

When selecting locations, survivability of instruments should be considered, and additional quantities should be selected to replace instruments that may become inoperative. For example, Abramson and Green (1985) report on a survey of users, conducted to establish the required number of strain gages and load cells to compensate for losses occurring after installation. The survey indicates an average survivability rate for load cells of 75%, and 60% for strain gages.

Locations should generally be selected so that data can be obtained as early as possible during the construction process. Because of the inherent variability of soil and rock, it is usually unwise to rely on a single instrument as an indicator of performance.

6.12 PLAN RECORDING OF FACTORS THAT MAY INFLUENCE MEASURED DATA

Measurements by themselves are rarely sufficient to provide useful conclusions. The use of instrumentation normally involves relating measurements to causes, and therefore complete records and diaries must be maintained of all factors that might cause changes in the measured parameters. As discussed in Section 6.6, a decision may have been made to monitor various causal parameters, and these should always include construction details and progress. Visual observations of expected and unusual behavior should also be recorded. Records should be kept of geology and other subsurface conditions and of environmental factors that may, in themselves, affect monitored data, for example, temperature, precipitation, sun, and shade.

Details of each instrument installation should be recorded on installation record sheets, because local or unusual conditions often influence measured variables. Installation record sheets are discussed further in Chapter 12.

6.13 ESTABLISH PROCEDURES FOR ENSURING READING CORRECTNESS

Personnel responsible for instrumentation must be able to answer the question: *Is the instrument functioning correctly?* The ability to answer depends on availability of good evidence, for which planning is required. The answer can sometimes be provided by visual observations.

In critical situations, duplicate instruments can be used. A backup system is often useful and will often provide an answer to the question even when its accuracy is significantly less than that of the primary system. For example, optical survey can often be used to examine correctness of apparent movements at surface-mounted heads of instruments installed for monitoring subsurface deformation.

Data correctness can also be evaluated by examining consistency. For example, in a consolidation situation, dissipation of pore water pressure should be consistent with measured settlement, and increase of pore water pressure should be consistent with added loading. Repeatability can also give a clue to data correctness, and it is often worthwhile to take many readings over a short time span to disclose whether or not lack of normal repeatability indicates suspect data.

6.14 LIST THE SPECIFIC PURPOSE OF EACH INSTRUMENT

At this point in the planning, it is useful to question whether all planned instruments are justified. Each planned instrument should be numbered and its purpose listed. If no viable specific purpose can be found for a planned instrument, it should be deleted.

6.15 PREPARE BUDGET

Even though the planning task is not complete, a budget should be prepared at this stage for all tasks listed in Table 6-1, to ensure that sufficient funds are indeed available. A frequent error in budget preparation is to underestimate the duration of the project and the real data collection and processing costs. If insufficient funds are available, the instrumentation program may have to be curtailed or more funds sought on a timely basis. Clearly, an application for more funds must be supported by reasons that can be defended.

6.16 PREPARE INSTRUMENTATION SYSTEM DESIGN REPORT

Green (1995) recommends the preparation of an instrumentation system design report. This report should summarize the results of above planning steps 6.2 thru 6.15. It forces the designer to produce a definitive document that covers all these issues, at which point the report can be reviewed and checked to ensure that everything is consistent, that the plan is a good one and covers the need of the project. Reviewing the specifications for this is too late. The instrumentation system design report should include a section on the selected contract method, both for procurement of instruments (Section 6.17) and for field instrumentation services (Section 6.21), and the reasoning behind the selection.

6.17 WRITE INSTRUMENT PROCUREMENT SPECIFICATIONS

Attempts by users to design and manufacture instruments generally have not been successful, although joint efforts by user and manufacturer are sometimes undertaken. Instruments should therefore be purchased from established manufacturers, for which procurement specifications are usually needed.

6.17.1 Recommended Types of Specification

Many states and project design managers encourage the use of a low-bid procurement method. However, use of a low-bid method often results in some corner-cutting. Sherard (1982) wrote: "The common *or acceptable equivalent* clause, combined with competitive bidding, leads inevitably to excessive emphasis on economy, with the result that high-quality instruments cannot compete. This keeps the quality of the average instrument on the market just above the *acceptable* level, a highly undesirable situation." In discussions to Sherard's paper, other respected engineers agreed with his view. This is also the consensus of a series of articles that describe contract practices on six major projects (Dunnicliff et at. 1994).

Procurement of instrumentation materials should generally be made through a process different from procurement of routine construction items. If valid measurements are to be made, the manufacturer must pay extremely close attention to quality and details. The low-bid method should never be used unless regulations allow for no alternative. Instead, one of the following two methods is recommended:

- The state or the state's design consultant procures the instruments directly, negotiating prices with suppliers.

- The state or the state's design consultant enters an estimate of procurement cost in the construction contract bid schedule and subsequently selects appropriate instruments for the construction contractor to procure. Price is negotiated between the state and suppliers of instruments, the suppliers become "assigned suppliers," and the construction contractor is reimbursed at actual cost plus a handling fee.

6.17.2 Contents of Low-Bid Specifications

In cases where neither of the above methods can be used and the low-bid method with an "or equivalent" provision is unavoidable, a clear, concise, complete, and correct specification must be written. Unless the specification covers all salient features, unsatisfactory instruments may be supplied. Table 6-2 lists appropriate content.

TABLE 6-2
CONTENTS OF LOW-BID SPECIFICATIONS FOR PROCUREMENT OF INSTRUMENTATION MATERIALS

Part	Article
General	Acceptable equivalents Submittals Factory calibrations - general Quality assurance Delivery schedule Instruction manuals
Details for each instrument	Materials specifications Factory calibrations - details

6.17.3 Low-Bid Specifications. Substitutions for Specified Brand Names

When specifying instrumentation materials by referring to brand names, most states require the addition of "or acceptable equivalent" wording. When brand names are specified, the following wording can be used so that the state maintains appropriate control over acceptability of substitutions:

"Whenever any product is specified by brand name and model number, such specifications shall be deemed to be used for the purpose of establishing a standard of quality and facilitating the description of the product desired. The term acceptable equivalent shall be understood to indicate that the acceptable equivalent product is the same or better than the product named in the specifications in function, performance, reliability, quality, and general configuration. This procedure is not to be construed as eliminating from competition other suitable products of equal quality by other manufacturers. The Contractor may, in such cases, submit complete comparative data to the Engineer for consideration of another product. Substitute products shall not be ordered, delivered to the site, or used in the Work unless accepted by the Engineer in writing. The Engineer will be the sole judge of the suitability and equivalency of the proposed substitution."

The following wording should also be included:

"Within ___ days after the Notice to Proceed, submit manufacturers' product data and instruction manuals describing all specified instruments to the Engineer for review, including requests for consideration of substitutions, if any, together with product data and instruction manuals for requested substitutions."

6.17.4 Factory Calibration and Quality Assurance

If an instrument is not working **perfectly** before installation, it is not likely to work well after installation. Some manufacturers have comprehensive quality assurance programs and perform extensive factory calibrations. However, some do not.

The following wording can be used if regulations allow for no alternative to low-bid procurement, under a heading "Quality Assurance and Factory Calibration," in an attempt to maximize quality of instrumentation materials:

"A factory calibration shall be conducted on all instruments prior to shipment. Certification shall be provided to indicate that the test equipment used for this purpose is calibrated and maintained in accordance with the test equipment manufacturer's calibration requirements and that, where applicable, calibrations are traceable to the National Institute of Standards and Technology.

Each factory calibration shall include a calibration curve with data points clearly indicated, and a tabulation of the data. Each instrument shall be marked with a unique identification number. *[Details can be specified for each type of instrument, for example for vibrating wire piezometers the following is possible wording: Factory calibrations of vibrating wire piezometers shall be made against a pressure gage traceable to the National Institute of Standards and Technology. The accuracy of the pressure gage shall not be less than twice the specified accuracy of the piezometers. Calibrations shall be made to full scale in two complete cycles, recording the reading in 10 equal increments during two loading and two unloading cycles. The thermal factor of each piezometer shall be determined in a precision test chamber, at 0, 10, 20, and 30 degrees C. The calibration record shall include gage factor, thermal factor, and zero reading, with corresponding temperature and barometric pressure.]*

A final quality assurance inspection shall be made prior to shipment. During the inspection, a checklist shall be completed to indicate each inspection and test detail. A completed copy of the checklist shall be supplied with each instrument."

Where a low-bid method is not used, similar provisions can be incorporated in purchase documents.

6.18 PLAN INSTALLATION

Installation procedures should be planned well in advance of scheduled installation dates, following the guidelines given in Chapter 12.

Written step-by-step procedures should be prepared, making use of the manufacturer's instruction manual and the designer's knowledge of specific site geotechnical conditions. The written procedures should include a detailed listing of required materials and tools, and installation record sheets should be prepared, for documenting factors that may influence measured data. In cases where state personnel will install the instruments, written procedures are also needed. An example of an instrument installation procedure and an installation record sheet is given by Dunnicliff (1988, 1993).

Staff training should be planned. Installation plans should be coordinated with the construction contractor and arrangements made for access and for protection of installed instruments from damage. An installation schedule should be prepared, consistent with the construction schedule.

6.19 PLAN REGULAR CALIBRATION AND MAINTENANCE

Regular calibration and maintenance should be planned, following the guidelines given in Chapter 12.

6.20 PLAN DATA COLLECTION, PROCESSING, PRESENTATION, INTERPRETATION, REPORTING, AND IMPLEMENTATION

Written procedures for data collection, processing, presentation, interpretation, reporting and implementation should be prepared, following the guidelines given in Chapter 12.

The effort required for these tasks should not be underestimated. Many engineer's offices have files filled with large quantities of partially processed and undigested data because sufficient time or funds were not available for these tasks. The computer is a substantial aid but is no panacea.

Staff training should be planned. At this stage in the planning a verification should be made to ensure that remedial actions have been planned, that personnel responsible for interpretation of instrumentation data have contractual authority to initiate remedial action, that communication channels between design and construction personnel are open, and that arrangements have been made to forewarn all parties of the planned remedial actions.

6.21 WRITE SPECIFICATIONS FOR FIELD INSTRUMENTATION SERVICES

Field services include instrument installation, regular calibration and maintenance, and data collection, processing, presentation, interpretation, and reporting.

As for procurement of instrumentation materials (Section 6.17), many states and project design managers encourage use of a low-bid selection method for field instrumentation services. However, contractual arrangements for the selection of personnel may govern success or failure of a performance monitoring program, and a low-bid selection method often results in failure.

6.21.1 Recommended Type of Specification

Geotechnical instrumentation field work should not be considered a routine construction item because successful measurements require extreme dedication to detail, personal effort and motivation throughout all phases of the work. Again, the low-bid method should never be used unless regulations allow for no alternative. A "professional service" atmosphere is needed, and one of the following two methods is recommended:

- The state performs field work that requires specialist instrumentation skills. If necessary, the state retains the services of a consulting firm that specializes in instrumentation. Supporting work (work that does not require specialist instrumentation skills) is performed by the construction contractor.

- The state or the state's design consultant enters an estimate of specialist field service costs in the construction contract bid schedule. Subsequently, the state and construction contractor select an appropriate specialist consulting firm, which is retained as an "assigned subcontractor" by the construction contractor to perform field work that requires specialist skill. Charges for specialist work are negotiated between the state and consulting firm, and the construction contractor is reimbursed at actual cost plus a handling fee. Supporting work is performed by the construction contractor.

Klingler (1997) provides a case history of the first method, with a public agency owner, which can be used by state geotechnical personnel as a precedent when endeavoring to convince management personnel to support this method.

6.21.2 Contents of Low-Bid Specifications

In cases where regulations do not allow either of the above methods, and where the low-bid method is unavoidable, a clear, concise, complete, and correct specification should be written to maximize the quality of field services. Table 6-3 lists appropriate content. Guidelines on some of the "construction methods" articles are given in Chapter 12.

6.21.3 Low-Bid Specifications. Submittals of Field Procedures

If low-bid specifications are used, it is important for state personnel to review the construction contractor's planned field procedures. The following submittal wording is appropriate: the task of preparing such submittals also forces the construction contractor to plan field procedures well ahead of actual field work.

> "At least ___ days prior to commencing installation of the first of each type of instrument, submit to the Engineer for review the following items pertaining to that instrument type:
>
> 1. Detailed step-by-step procedures for installation, including:
>   a. The method for conducting pre-installation acceptance tests
>   b. The method to be used for cleaning the inside of casing or augers.
>   c. Specifications for proposed grout mixes, including commercial names, proportions of admixtures and water, mixing sequence, mixing methods and duration, pumping methods and tremie pipe type, size and quantity.
>   d. Drill casing or auger type and size.
>   e. Depth increments for backfilling boreholes with sand and granular bentonite.
>   f. Method for overcoming buoyancy of instrumentation components during grouting.
>   g. Method of sealing joints in pipes and inclinometer casing to prevent ingress of grout.
>   h. Method for conducting post-installation acceptance test.
>   i. Method for protecting instruments from damage.
>   j. Sample installation record sheet.

2. A bar chart indicating the proposed time sequence of instrument installation.

3. Detailed step-by-step procedures for:
   a. Calibrations during service life.
   b. Maintenance of readout units, field terminals and embedded components.
   c. Data collection, both for initial and subsequent readings.
   d. Data reduction, plotting and reporting."

TABLE 6-3
CONTENTS OF LOW-BID SPECIFICATIONS FOR FIELD INSTRUMENTATION SERVICES

Part	Article
General	Work included Related work Definitions Purpose of geotechnical instrumentation program Responsibilities of construction contractor Qualifications of construction contractor's instrumentation personnel (field and office, drillers, surveyors) Quality assurance Submittals (personnel, materials, field procedures, data, plans of action relating to hazard warning levels) Scheduling work Storage of instruments
Construction Methods	Pre-installation acceptance tests Installation-general (casing, grouting, construction contractor's additional instruments, installation records) Installation of...(one article for each instrument type) Post-installation acceptance tests Field calibration and maintenance Data collection (initial readings, other readings, schedule, records, construction contractor's additional readings, access for Engineer) Data reduction, processing, plotting and reporting (data format, detailed plot requirements, report content and schedule, causal data) Damage to instrumentation Disclosure of data Interpretation and implementation of data (construction contractor's responsibility, hazard warning levels, actions in event hazard warning levels are reached) Disposition of instruments
Compensation	Method of measurement Basis of payment Payment items

6.21.4 Low-Bid Specifications. Method of Payment

When the low-bid method is used, a lump sum payment method is often favored by states and project design managers. However, with geotechnical instrumentation work, numerous changes usually occur in the field, including instrument quantities, drilling depths, and reading schedules, and determination of equitable price adjustments to a lump sum bid is a very laborious process, often resulting in the state paying more than the change is worth.

A unit price payment method should therefore be used. Table 6-4 indicates possible payment items, both for field instrumentation services and for materials.

TABLE 6-4
POSSIBLE UNIT PRICE PAYMENT ITEMS

Item	Unit	Comments
Furnish...[instrument type] readout unit	Each	One item for each instrument type. Includes factory calibrations
Furnish and install... [instrument type]	Linear foot for borehole instruments. Each for others	One item for each instrument type. Includes all materials left in place, labor, tools and equipment, drilling, sampling, installation, installation of surface protection, and determination of as-built location
Read...[instrument type] and report data	Each	One item for each instrument type. Need to specify exactly what is meant by one reading. Includes reading; data reduction, processing, presentation, reporting; regular field calibration and maintenance; repair
General geotechnical instrumentation requirements	Lump Sum	Includes repairing or replacing damaged instruments, furnishing specified submittals, interpreting data, all other items of work for which no separate bid item is provided

6.22 UPDATE BUDGET

Planning is now complete, and the budget for all tasks listed in Table 6-1 should be updated in light of all planning steps.

CHAPTER 7.0
INSTRUMENTATION OF SOIL SLOPES AND EMBANKMENTS

7.1 INTRODUCTION

This chapter is a companion to Module 3 (Soil Slopes and Embankments), and includes a discussion of instrumentation for embankments on soft ground, cut slopes in soil, and landslides in soil. For each of these three topics, there are two sections. The first section indicates the general role of geotechnical instrumentation. The second section suggests the principal geotechnical questions that may arise and indicates the types of instruments that may be used to help provide answers to those questions. The sequence of geotechnical questions in the second section is intended to match the time sequence in which the question may be addressed during the design, construction, and performance process and does not indicate any rating of importance.

7.2 EMBANKMENTS ON SOFT GROUND

7.2.1 General Role of Instrumentation

In many cases, selection of soil parameters for the foundation soil is reliably conservative. The embankment is therefore designed with confidence that performance will be satisfactory, and "comfortable" factors of safety are used. In such cases, many projects will proceed without the use of instrumentation. However, some uncertainties always exist. Where design uncertainties are great, factors of safety small, or the consequences of poor performance severe, a prudent designer will include a performance monitoring program in the design.

In spite of a long record of embankment construction throughout the history of civil engineering, embankments that are designed with a factor of safety greater than unity fail embarrassingly often. On the other hand, some test embankments that are designed to fail intentionally, never do. Thus, it is not surprising that instrumentation plays a significant role in design and construction of embankments on soft ground.

The most frequent uses of instrumentation for embankments on soft ground are to monitor the progress of consolidation and to determine whether the embankment is stable. If the calculated factor of safety is likely to approach unity, instrumentation will generally be installed to provide a warning of any instability, thereby allowing remedial measures to be implemented before critical situations arise.

If uncertainties in the selection of soil parameters are unacceptably great, or if construction feasibility is in doubt, it may be appropriate to construct a test embankment. Instrumentation data provide an essential role in evaluating the performance of such a test.

7.2.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for embankments on soft ground. Table 7-1 summarizes possible instruments.

TABLE 7-1
SUITABLE INSTRUMENTS FOR MONITORING EMBANKMENTS ON SOFT GROUND

Geotechnical Question	Measurement	Suitable Instruments
What are the initial site conditions?	Groundwater pressure Vertical deformation	Open standpipe, vibrating wire, or pneumatic piezometers Surveying methods
What information can be provided by instrumenting a test embankment?	As for "What is the progress of consolidation?"	As for "What is the progress of consolidation?"
What is the progress of consolidation?	Vertical deformation of embankment surface and ground surface at and beyond toe of embankment Vertical deformation of original ground surface below embankment Vertical deformation and compression of subsurface Groundwater pressure	Surveying methods Settlement platforms Single-point and full-profile liquid level gages Horizontal inclinometers Probe extensometers with induction coil or magnet/reed switch transducers Open standpipe, vibrating wire or pneumatic piezometers. Consider push-in type.
Is the embankment stable?	Horizontal deformation	Surveying methods Inclinometers
What are the fill quantities?	Vertical deformation of original ground surface below embankment	Settlement platforms Full-profile liquid level gages Horizontal inclinometers

What Are the Initial Site Conditions?

Initial site conditions are determined by use of conventional site investigation procedures, often supplemented by in situ testing (Module 1 - Subsurface Investigations). However, performance monitoring instrumentation sometimes plays a role. For example, initial groundwater pressures and fluctuations must often be determined for design purposes. Piezometers can be installed well before the start of filling, to define the preconstruction groundwater pressure regime, including any perched or artesian water. If the piezometers are installed sufficiently early, seasonal variations can be defined. Settlement measurements may also be required to establish preconstruction settlement behavior.

What Information Can Be Provided by Instrumenting a Test Embankment?

Test embankments are sometimes constructed to resolve uncertainties in the selection of soil parameters, to examine alternative construction methods, or to demonstrate construction feasibility. The goal of instrumentation in these tests will generally be to provide an indication of incipient failure and/or to evaluate the progress of consolidation. In addition, instrumentation may be installed to permit a back-analysis for determining the engineering properties of the underlying soil. In all cases, the primary parameters of interest are vertical and horizontal deformations and pore water pressure.

Depending on the specifics of the case, instrumentation for a test embankment will include any or all of the monitoring methods discussed elsewhere in Section 7.2.2. The intensity of instrumentation will usually be greater than for a prototype embankment (one that is part of the constructed project), so that maximum information is gained from the test.

What is the Progress of Consolidation?

When evaluating the progress of consolidation beneath an embankment on soft ground, both settlement and pore water pressure measurements are normally made. Whenever piezometers are installed in the vicinity of the embankment, reference piezometers should also be installed, remote from the embankment, to monitor any variation in groundwater pressure that may result from other causes.

A predictive extrapolation can be made from a plot of pore water pressure versus time because the equilibrium piezometric level corresponding to full consolidation is known to be equal to the reference value remote from the embankment. Prediction from a plot of settlement versus time is less reliable, because the magnitude of ultimate settlement cannot be known with certainty.

Instruments for monitoring the progress of consolidation can be selected from the list given in Table 7-1. The list is not intended to be exclusive: it merely indicates instruments that have provided satisfactory data on projects with which the author has been involved. Selection among the various options will depend on the factors discussed in Chapters 2 and 3, including the feasibility of extending instrument pipes and leads up through the embankment without risk of damage, and the need for redundancy. In all cases predictions should be made of maximum vertical foundation compression and horizontal spreading, and a special effort must be made to select instruments that are both capable of surviving the deformations and capable of providing reliable data as deformation occurs. Installation procedures must also be planned in recognition of the future deformations.

Is the Embankment Stable?

If the possibility of lateral instability is minor, instrumentation will not usually be required during construction of an embankment on soft ground. However, in other cases a monitoring program is generally required to provide a forewarning of instability, thereby allowing remedial measures to be implemented before critical situations arise. Remedial measures may include a waiting period to allow the foundation material to increase in strength as pore water pressures dissipate and/or the construction of stabilizing berms or the removal of some fill.

When a monitoring program is required to provide an indication of incipient failure, horizontal deformation measurements will normally provide the most direct data. An inclinometer is the primary tool, supplemented with surface deformation monitoring by surveying methods.

What Are the Fill Quantities?

Soft clay foundations may experience substantial settlement during embankment construction. When embankment fill is paid for on a volume basis and measured to the actual base of the fill, a determination must be made of the elevation of the base of the fill. Settlement platforms are often used, but full-profile liquid level gages and horizontal inclinometers are suitable alternatives.

7.3 CUT SLOPES IN SOIL

7.3.1 General Role of Instrumentation

It is imperative that, prior to planning an instrumentation program for a cut slope in soil, an engineer first develop one or more working hypotheses for a potential behavior mechanism. The hypotheses must be based on a comprehensive knowledge of the locations and properties of stratigraphic discontinuities.

Instrumentation can be used to define the groundwater regime prior to excavating a slope. Results of measurements during excavation can be used as a basis for modification of the designed slope angle. Measurements of ground movement and groundwater pressure can assist in documenting whether or not performance during and after excavation is in accordance with predicted behavior. Measurements can also be used to document whether short- and long-term surface and/or subsurface drainage measures are performing effectively. If evidence of instability appears during or after construction, instrumentation plays a role in defining the characteristics of the instability, thus permitting selection of an appropriate remedy.

7.3.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for cut slopes in soil. Table 7-2 summarizes possible instruments.

What Are the Initial Site Conditions?

Initial site conditions are determined by use of conventional site investigation procedures, sometimes supplemented by in situ testing (Module 1 - Subsurface Investigations). However, performance monitoring instrumentation sometimes plays a role in defining initial site conditions. For example, groundwater pressures can have a large impact on the stability of cut slopes. Piezometers can be installed well before the start of excavation, to define the preconstruction groundwater pressure regime, including any perched or artesian water. If piezometers are installed sufficiently early, seasonal variations can be defined.

If there is evidence of instability prior to excavation of the slope, such as an old landslide, the available data should be analyzed to identify potential failure mechanisms. An instrumentation program can then be planned to test hypotheses and to determine whether adverse conditions are present.

Is the Slope Stable During Excavation?

A program to monitor stability during excavation is not usually required if the design is very conservative, if there is previous experience with design and construction of similar facilities under similar conditions, or if the consequences of poor performance will not be severe. However, under other circumstances a monitoring program will normally be required to demonstrate that the excavation is stable and that nearby structures are not affected adversely.

is stable and that nearby structures are not affected adversely.

TABLE 7-2
SUITABLE INSTRUMENTS FOR MONITORING CUT SLOPES IN SOIL

Geotechnical Question	Measurement	Suitable Instruments
What are the initial site conditions?	Groundwater pressure	Open standpipe, vibrating wire, or pneumatic piezometers
	Horizontal deformation	Surveying methods Inclinometers
Is the slope stable during excavation?	Surface deformation	Surveying methods Tiltmeters Metallic time domain reflectometry
	Subsurface deformation	Inclinometers Shear plane indicators In-place inclinometers Metallic time domain reflectometry
	Groundwater pressure	Vibrating wire or pneumatic piezometers
Is the slope stable in the long term?	As for "Is the slope stable during excavation?"	As for "Is the slope stable during excavation?"
	Precipitation	Rain gages Snow stakes
	Load in tiebacks	Load cells

Deformation and groundwater pressure are the primary parameters that assist in the evaluation of stability during excavation. Deformation measurements are usually the primary interest but, because high groundwater pressures can cause deformation, groundwater pressure measurements are also often needed so that cause and effect relationships can be established.

Is the Slope Stable in the Long Term?

The question applies primarily to cut slopes that have been unstable during excavation. However, there are cases where the question applies also to cut slopes with no history of instability, for example, when construction is planned near the toe.

In general, a choice must be made among three options. First, doing nothing and accepting the consequences of slope failure. Second, monitoring to provide a forewarning of instability, so that remedial measures can be implemented before critical situations arise. Third, stabilizing the slope, perhaps including a monitoring program to verify that stability has been achieved. The choice will be based on many factors, including the consequences of failure and the economics of stabilizing. When planning a monitoring program for either of the second two options, a two-step approach is recommended: first, measurements of deformation on the surface of the entire slope to indicate the existence of any instability and second, additional measurements at any location shown to be unstable.

If permanent tiebacks have been installed to stabilize the slope, special load cells can be installed for long-term monitoring.

7.4 LANDSLIDES IN SOIL

7.4.1 General Role of Instrumentation

If there is evidence of slope instability, its characteristics must be defined so that any necessary remedial measures may be taken. The question *how much ground is moving?* can be answered by use of instrumentation. The question *why is the ground moving?* will not be answered by instrumentation alone: the answer of course also requires a complete geotechnical investigation and analysis. Dunnicliff (1995b) presents an overview of monitoring methods.

Instrumentation also plays a role in monitoring the long-term stability of the slope after remedial measures have been taken.

7.4.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for landslides in soil. Table 7-3 summarizes possible instruments.

TABLE 7-3
SUITABLE INSTRUMENTS FOR MONITORING LANDSLIDES IN SOIL

Geotechnical Question	Measurement	Suitable Instruments
What are the post-landslide conditions?	Surface deformation	Surveying methods Tiltmeters Metallic time domain reflectometry
	Subsurface deformation	Inclinometers Shear plane indicators In-place inclinometers Metallic time domain reflectometry
	Groundwater pressure	Vibrating wire or pneumatic piezometers
Is the slope stable in the long term?	As for "What are the post-landslide conditions?"	As for "What are the post-landslide conditions?"
	Precipitation	Rain gages Snow stakes
	Load in tiebacks	Load cells

What Are the Post-Landslide Conditions?

There are two parts to this question: *how much ground is moving?* and, because the cause of movement is often high groundwater pressure, *what are the groundwater pressures?* Knowledge of answers to both parts usually allows remedial measures to be designed.

Is the Slope Stable in the Long Term?

After remedial measures have been taken, there will often be a need to monitor long-term stability.

7.5 STUDENT EXERCISE FOR PLANNING A MONITORING PROGRAM FOR A SOIL SLOPE

7.5.1 Introduction

The instructor will present the first step in the planning process, "Define the Project Conditions", and will indicate why a monitoring program is required. Students will then work in groups to consider the following steps:

- Step 3. Predict Mechanisms That Control Behavior
- Step 4. Define the Geotechnical Questions That Need to be Answered
- Step 6. Select the Parameters to be Monitored
- Step 7. Predict Magnitudes of Change
- Step 10. Select Instruments
- Step 12. Plan Recording of Factors That May Influence Measured Data
- Step 13. Establish Procedures for Ensuring Reading Correctness

Space is provided on the following pages for students to make notes during the exercise, and selected wording from Appendix A "Checklist for Planning Steps" is included to assist students.

An example of results of the student exercise is included in Appendix C. However, **it is important to recognize that entirely different results may apply to any real project situation.**

7.5.2 Step 1. Define the Project Conditions

A cut for a highway has been made in a previously stable slope. Subsurface conditions are interbedded sands and clays. A surface crack has been noticed above the top of the cut: see Figure 7-1.

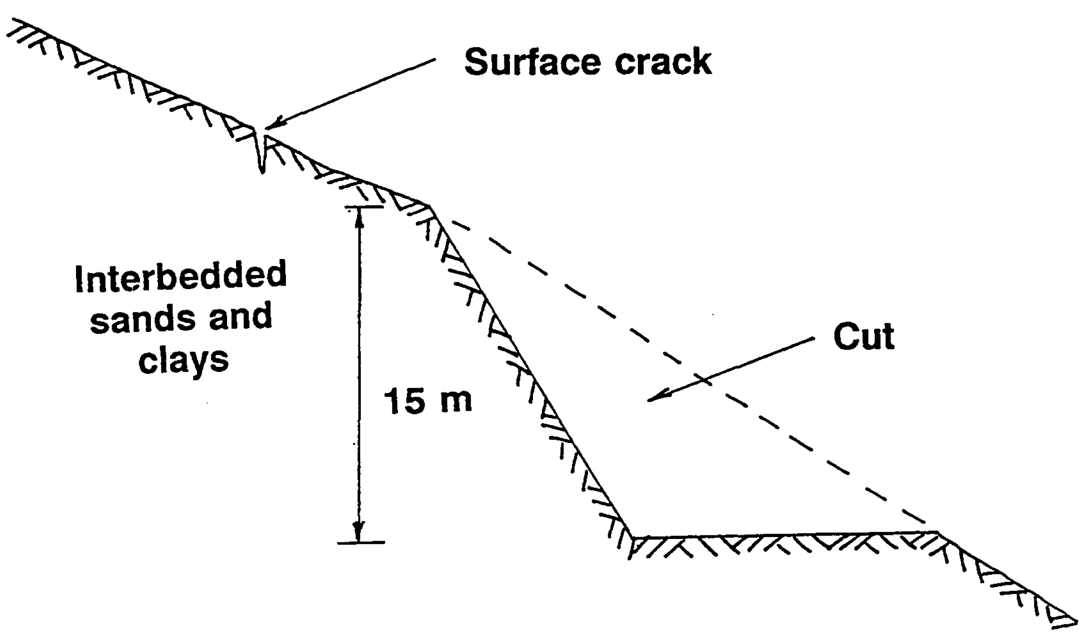


Figure 7-1: Soil Slope

7.5.3 Step 3. Predict Mechanisms That Control Behavior

Consider possible modes of deformation within the ground. Draw some sketches here, or write some words.

7.5.4 Step 4. Define the Geotechnical Questions That Need to be Answered

Refer to Sections 7.3.2 and 7.4.2 as needed.

7.5.5 Step 6. Select the Parameters to be Monitored

Appendix A lists the following:

(a) Groundwater pressure
(b) Total stress within soil mass
(c) Total stress at contact with structure or rock
(d) Vertical deformation
(e) Horizontal deformation
(f) Tilt
(g) Strain in soil or rock
(h) Load or strain in structural members
(i) Temperature
(j) Precipitation

Check those that might apply to this case.

7.5.6 Step 7. Predict Magnitudes of Change

Appendix A lists the following:

(a) Predict maximum value, thus instrument range
(b) Predict minimum value, thus instrument sensitivity or accuracy
(c) Determine hazard warning levels

Just think about your approach to estimating the needed measurement range and accuracy of no more than two of your selected parameters. Don't make any calculations (you can't do this with the information you have been provided): concepts only.

Parameter 1 : ____________________

- Range

- Accuracy

Parameter 2 : ____________________

- Range

- Accuracy

7.5.7 Step 10. Select Instruments

Refer to Table 7.2 as needed. Be as specific as you can.

7.5.8 Step 12. Plan Recording of Factors That May Influence Measured Data

Appendix A lists the following:

(a) Construction details
(b) Construction progress
(c) Visual observations of expected and unusual behavior
(d) Geology and other subsurface conditions
(e) Environmental factors

Check those that might apply to this case, and add some details if you can.

7.5.9 Step 13. Establish Procedures for Ensuring Reading Correctness

Appendix A lists the following:

(a) Visual observations
(b) Duplicate instruments
(c) Backup system
(d) Study of consistency
(e) Study of repeatability
(f) Regular in-place checks

Check those that might apply to this case, and add some details if you can.

CHAPTER 8.0
INSTRUMENTATION OF DEEP FOUNDATIONS

8.1 INTRODUCTION

This chapter is a companion to Module 8 (Deep Foundations), and includes a discussion of instrumentation for driven piles and drilled shafts. For each of these two topics, there are two sections. The first section indicates the general role of geotechnical instrumentation. The second section suggests the principal geotechnical questions that may arise and indicates the types of instruments that may be used to help provide answers to those questions. The sequence of geotechnical questions in the second section is intended to match the time sequence in which the question may be addressed during the design, construction, and performance process and does not indicate any rating of importance.

8.2 DRIVEN PILES

8.2.1 General Role of Instrumentation

The subsurface length of a driven pile cannot usually be inspected after driving; thus, its physical condition and alignment are unknown. Subsurface geotechnical conditions are rarely known with certainty, and therefore the design of driven piles involves assumptions and uncertainties that are often addressed by conducting instrumented full-scale tests. Tests may examine the behavior of the pile under load applied to the pile butt or under load caused by settlement of soil with respect to the pile.

Defects in piles can be created during driving, and inspection procedures are available for examining the condition and alignment after driving. Certain types of driven pile cause large displacements and changes of pore water pressure in the surrounding soil, and these may in turn have a detrimental effect on neighboring piles or on the stability of the site as a whole. Instrumentation can be used to quantify the consequences of pile driving and thus to assist in planning any necessary action.

8.2.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for driven piles. Table 8-1 summarizes possible instruments.

What is the Vertical Load-Movement Relationship of the Pile? (Determination of Relationship by Static Testing, Without Comprehensive Load Transfer Data)

Load at the pile butt should be measured using a load cell, equipped with a spherical bearing, because load determinations based on hydraulic jack pressures are often unreliable (Chapter 5). Vertical deformation at the pile butt is normally measured using dial indicators attached to a reference beam. A wire/mirror/scale arrangement is often used as backup. Measurements of horizontal deformation with dial indicators are recommended to indicate whether or not the applied load is axial and concentric. The reference beam should ideally be supported on ground that is not influenced by the test, but often this is not possible, and any movement of the beam should be measured throughout the test by surveying methods.

TABLE 8-1
SUITABLE INSTRUMENTS FOR MONITORING DRIVEN PILES

Geotechnical Question	Measurement	Suitable Instruments
What is the vertical load-movement relationship of the pile? (Determination of relationship by static testing, without comprehensive load transfer data)	Deformation at butt	Dial indicators with reference beam Wire/minor/scale Optical survey
	Load at butt	Load cell, preferably vibrating wire type
	Deformation at toe	Telltales
What is the minimum time period between pile driving and load testing?	Pore water pressure	Pneumatic piezometers at soil/pile contact
What is the vertical load-movement relationship of the pile? (Determination of relationship by static testing, with comprehensive load transfer data)	Deformation at butt	Dial indicators with reference beam Wire/mirror/scale
	Load at butt	Load cell, preferably vibrating wire type
	Deformation at toe	Telltales
	Stress along pile	Strain gages, preferably vibrating wire: arc welded, spot welded, embedment, sister bar Multiple telltales Series extensometers
	Load at toe	Contact earth pressure cell (toe load cell) Strain gages, as above Multiple telltales Series extensometers
What is the vertical load-movement relationship of the pile? (Determination of relationship by dynamic testing)	Strain and acceleration at pile butt, during driving	Pile Driving Analyzer

TABLE 8-1 (CONTINUED)
SUITABLE INSTRUMENTS FOR MONITORING DRIVEN PILES

Geotechnical Question	Measurement	Suitable Instruments
What is the horizontal load-movement relationship of the pile? (Determination of relationship by static testing)	Deformation at butt	Dial indicators with reference beam Wire/mirror/scale Optical survey
	Load at butt	Load cell, preferably vibrating wire type Cable tension meter
	Bending data along pile	Strain gages, as above In-place inclinometers
Will there be significant downdrag loading?	Surface settlement	Optical survey
	Groundwater pressure	Vibrating wire or pneumatic piezometers
	Stress along pile	Vibrating wire strain gages Multiple telltales Series extensometers Load cells, preferably vibrating wire type
	Relative settlement between soil and pile	Settlement platforms Multiple telltales in pile Series extensometers in pile Probe extensometers in soil
Has the capacity of the pile been reduced by defects created during driving?	Curvature of pile	Inclinometer Shear plane indicator
	Condition of pile	Integrity tests
Is pile driving causing excessive lateral deformation of the ground?	Deformation	Surveying methods Inclinometers in piles or soil Shear plane indicator in piles or soil
	Pore water pressure	Vibrating wire or pneumatic piezometers
Is pile driving causing excessive vibration?	Peak particle velocity	Seismographs

Most conventional static load tests in axial compression include measurements of load and deformation at the pile butt only. However, such measurements are insufficient for evaluating load tests of relatively short duration on long end-bearing piles because much of the test load is carried by skin friction. They are also insufficient for concrete piles if they are tested while the concrete is "green," as is often the case when available time for testing is very limited. In general, therefore, toe movement should be measured, using a telltale and dial indicator.

What is the Minimum Time Period Between Pile Driving and Load Testing?

When driving a displacement pile through saturated clay, silt, or fine sand, temporary excess pore water pressures will be developed. If a pile is load tested while excess pore water pressures exist, the result is likely to be misleading; therefore, pressures should be allowed to dissipate before testing.

A waiting period of several weeks may be necessary. If the test pile is selected from a group of piles, the excess pore water pressures may be significantly higher than for a single pile. Pore water pressures may also be significantly higher if the soil is excessively disturbed. In these cases, one or more piezometers should be installed at the soil/pile contact so that pore water pressures at the contact can be evaluated. Pneumatic piezometers are preferable to vibrating wire piezometers for this purpose, as there is minimal chance of zero drift during driving. Kraemer and Davidson (1995) report on a successful case history.

What is the Vertical Load-Movement Relationship of the Pile? (Determination of Relationship by Static Testing, With Comprehensive Load Transfer Data)

When comprehensive load transfer data are required along the lengths of driven piles during static vertical load tests, strain gages and/or multiple telltales and/or removable series extensometers are used. Multiple telltales and series extensometers cannot be the primary measurement system because they can be installed only after pile driving, and therefore cannot provide *residual stress* data. Residual stresses are stresses that remain in a pile after pile driving: without a knowledge of their magnitude, absolute load in a pile cannot be calculated from telltale or series extensometer data.

For steel piles, strain gages provide the primary measurements. For concrete piles, strain data can be misleading, because they may be greatly influenced by cracks that develop in the concrete during driving. Ideally, several load cells should be installed along the length of the pile, but practical and economic considerations usually prevent this approach. When comprehensive load transfer data are required for concrete piles, both embedment strain gages and multiple telltales (or series extensometers) should be used. Special arrangements must be made for converting measured strain to stress in a prestressed concrete pile. Two methods are possible during an axial load test and are described below. Preferably, both methods should be used, to maximize confidence in the data.

First, the top portion of the pile can be used as an *unconfined compression test specimen*. If possible, the length should be at least three times the diameter of the pile and should not include any length that may have been damaged during driving. Strain in the specimen is measured with strain gages, backed up with a pair of telltales if the specimen is long enough. Because change in moisture content can cause significant strain, the specimen must be under the same moisture conditions as the length of the pile in which the other strain measurements are made. If that length is below the water table, the specimen should also be either below the water table or kept wet by artificial means. The relationship between strain and stress is then determined during the load test, using strain data from the specimen and load data from the load cell at the pile butt. The method provides data for every load increment.

The second method for strain/stress conversion makes use of load cell data and measurements of subsurface strain, determined from strain gages, multiple telltales or series extensometers in the pile. During the load test, when all shaft resistance has been mobilized at the location of the strain gage, the pile acts as a column in compression, and the strain/stress relationship can be determined from subsequent data.

Toe load can, on occasion, be measured by installing special contact earth pressure cells (toe load cells) on the pile toe.

What is the Vertical Load-Movement Relationship of the Pile? (Determination of Relationship by Dynamic Testing)

Dynamic testing of driven piles, using the Pile Driving Analyzer, is discussed in Module 8 (Deep Foundations).

What is the Horizontal Load-Movement Relationship of the Pile? (Determination of Relationship by Static Testing)

Load measurements at the pile butt require use of a load cell or cable tension meter. Deformation measurements at the pile butt require horizontal and vertical dial indicators and/or a wire/mirror/scale arrangement. Any movement of the reference beam should be measured throughout the test by surveying methods.

When bending data are required during a horizontal (lateral) load test, strain gages are installed on or near both faces of the pile in the plane of loading. In-place inclinometers can be used to measure horizontal deformations along the pile during load testing.

Will There Be Significant Downdrag Loading?

When soil settles with respect to a pile, frictional forces between the soil and pile may cause a load increase in end bearing piles or increased settlement of friction piles. The loading is referred to as *downdrag* or *negative skin friction* (Module 8 - Deep Foundations).

Ongoing settlement prior to pile driving can be diagnosed by precise settlement surveys. Ongoing primary consolidation prior to pile driving can be diagnosed by using piezometers to measure groundwater pressure. The allowance for downdrag loading can be based on accepted design procedures but, when the allowance is substantial and a large quantity of piles are to be driven, a full-scale test to determine actual loading may be warranted. A full-scale test may also be warranted to prove the effectiveness of coating piles with a friction-reducing material such as bitumen. A full-scale test will normally entail driving several instrumented piles and placing a fill to cause the onset of downdrag loading. Primary parameters to be measured during the test are magnitude of load along the piles and relative settlement between the soil and piles.

For steel piles, adequate load data can be obtained by measuring strain, preferably by using vibrating wire strain gages, with multiple telltales or series extensometers as backup.

For driven prestressed concrete piles, the methods described above for converting strain to stress cannot be used, because no load is applied to the pile butt. Load determinations in concrete piles must therefore include at least one direct measurement of load, by incorporating a load cell in a pile splice and/or by attaching a contact earth pressure cell to the pile toe. Load data should be supplemented by installing vibrating wire embedment strain gages or sister bars. The relationship between load and strain is determined, at points where load is measured directly, by relating load to strain measured near those points. The same relationship is applied to strains measured elsewhere.

Relative settlement between the soil and pile can be measured simply by installing a settlement platform at the original ground surface alongside each pile and measuring settlement of the platform with respect to the pile butt. However, if comprehensive data are required to define the relationship between relative settlement of soil and loading along the length of the pile, multiple telltales or series extensometers are needed in or on the pile, along with probe extensometers in the nearby soil.

Has the Capacity of the Pile Been Reduced by Defects Created During Driving?

Many building codes impose allowable limits on the curvature of piles, because excessive bending reduces capacity and increases deformation of piles under long-term loading. Capacity is also reduced if piles are dog-legged. The curvature of piles can be determined by using an inclinometer. When used in pipe piles, the inclinometer can be fitted with long arms and guide wheels that ride on the inside wall of the pipe and can be lowered down the pile on orientation rods. When used in other types of pile, a steel or plastic guide pipe is required, attached to or embedded in the pile. For an approximate determination of curvature, a shear plane indicator can be used within a guide pipe.

The condition of a driven pile can also be determined by integrity testing.

Is Pile Driving Causing Excessive Lateral Deformation of the Ground?

When displacement piles are driven in soft clay, there will normally be a temporary increase of pore water pressure and a reduction of soil strength, perhaps leading to excessive lateral deformation and perhaps even a stability failure. Examples include pile driving on a slope that already has a low factor of safety and pile driving nearby an existing structure or nearby other piles. When this potential exists, an instrumentation program can be conducted to quantify displacements and pore water pressure changes, thus allowing adoption of a driving procedure or sequence that minimizes detrimental effects.

Is Pile Driving Causing Excessive Vibrations?

Vibrations due to pile driving can cause damage to adjacent structures. Earth-borne vibrations emanate from the source in three distinct modes: first there is the fastest traveling 'P' wave (or compressional wave), followed closely by the slightly slower 'S' wave (or shear wave) and then the Raleigh wave (or surface wave), which travels along the ground surface rather like the ripples in a pond.

Experience and studies have shown that the best criterion for damage to structures is the peak particle velocity. This is defined as the maximum velocity at which the soil particles or other materials vibrate locally. (It should not be confused with the propagation velocity of the vibration, which is equal to the speed of sound in the material). Limits for peak particle velocity should be set on a case-by-case basis, depending on the sensitivity to vibration of adjacent structures.

Vibration acceptance criteria are published by AASHTO, U. S. Bureau of Mines and by Wiss (1967).

8.3 DRILLED SHAFTS

8.3.1 General Role of Instrumentation

Many uncertainties exist during design of drilled shafts, and instrumentation plays a role in determining the load-movement relationship, by conducting load tests. Concrete integrity is often uncertain during construction, particularly when shafts are constructed in granular soils below the water table or in softer, squeezing clays, when concrete slump is inadequate, or when concrete placement practices are inferior. Instrumentation can be used to examine the integrity of the concrete.

8.3.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for drilled shafts. Table 8-2 summarizes possible instruments.

What is the Vertical Load-Movement Relationship of the Drilled Shaft?

The load-movement relationship of a drilled shaft will be uncertain during the design phase, and one or more load tests may be conducted either during the design phase or at the beginning of the construction phase. Application of sufficient load to large drilled shafts can be expensive; therefore, the number of load tests is limited, and instrumentation programs should be planned to provide comprehensive performance data. The cost of instrumentation is usually a minor part of the total cost.

Measurement methods at the shaft top are similar to those described for driven piles in Section 8.2.2. Stress along the shaft should be determined from strain gage measurements, with a preference for the sister bar type with vibrating wire transducers. Alternatively, spot welded vibrating wire strain gages can be installed on the reinforcing cage. Removable series extensometers can be used as an alternative to strain gages. Multiple telltales can be used as backup, and the longest telltale provides data for base settlement. Data for conversion of strain to stress are provided as described in Section 8.2.2 for driven piles: by gaging an *unconfined compression test specimen* within a sleeve or above the ground surface, supplemented by use of strain data after all shaft resistance has been mobilized at a particular depth. However, for a drilled shaft it is not normally reasonable to leave a length of shaft above the ground surface, equal to three shaft diameters, and end effects in the *specimen* must be accounted for by averaging between several strain gages installed at one or more cross sections.

An attempt can be made to determine load at the base, from strain gage, series extensometer or multiple telltale data. However, this is subject to inaccuracies in converting strain to stress, and use of multiple telltales and series extensometers involves an extrapolation. It is recommended that a hard data point be obtained by installing a special contact earth pressure cell (base load cell) at the base.

The vertical load-movement relationship of a drilled shaft can also be determined by using the Osterberg load cell, as described in Module 8 - Deep Foundations.

What is the Horizontal Load-Movement Relationship of the Drilled Shaft?

Measurement methods at the shaft top are similar to those described for driven piles in Section 8.2.2.

Horizontal (lateral) load tests should include, wherever possible, determinations of bending stress in the shaft. Vibrating wire strain gages can be used, either for sister bar or spot welded type. Alternatively, bending stresses can be determined from series extensometer measurements, using one instrument near each face of the shaft in the plane of loading. The profile of bending can be determined by using an in-place inclinometer, and these data allow calculation of bending stresses if horizontal deformation is large enough.

The understanding of the behavior under horizontal load can be improved if the inclination of the top of the shaft is measured, using a tiltmeter.

What is the Integrity of the Concrete?

The integrity of the concrete can be determined by integrity testing.

TABLE 8-2
SUITABLE INSTRUMENTS FOR MONITORING DRILLED SHAFTS

Geotechnical Question	Measurement	Suitable Instruments
What is the vertical load-movement relationship of the drilled shaft?	Deformation at top	Dial indicators with reference beam Wire/mirror/scale Optical survey
	Load at top	Load cell, preferably vibrating wire type
	Stress along shaft	Vibrating wire sister bars Spot welded vibrating wire strain gages Multiple telltales Series extensometers
	Load at base	Contact earth pressure cell (base load cell) Strain gages, as above Multiple telltales Series extensometers
	Load at base or in shaft	Osterberg load cell
What is the horizontal load-movement relationship of the drilled shaft?	Deformation at top	Dial indicators with reference beam Wire/mirror/scale Optical survey
	Load at top	Load cell, preferably vibrating wire type Cable tension meter
	Bending data along shaft	Strain gages, as above Series extensometers In-place inclinometers Tiltmeters
What is the integrity of the concrete?	Condition of shaft	Integrity tests

8.4 STUDENT EXERCISE FOR PLANNING A MONITORING PROGRAM FOR A DRIVEN PILE

8.4.1 Introduction

The instructor will present the first step in the planning process, "Define the Project Conditions", and will indicate why a monitoring program is required. Students will then work in groups to consider the following steps:

Step 3.	Predict Mechanisms That Control Behavior
Step 4.	Define the Geotechnical Questions That Need to be Answered
Step 6.	Select the Parameters to be Monitored
Step 7.	Predict Magnitudes of Change
Step 10.	Select Instruments
Step 12.	Plan Recording of Factors That May Influence Measured Data
Step 13.	Establish Procedures for Ensuring Reading Correctness

Space is provided on the following pages for students to make notes during the exercise, and selected wording from Appendix A "Checklist for Planning Steps" is included to assist students.

An example of results of the student exercise is included in Appendix D. However, **it is important to recognize that entirely different results may apply to any real project situation.**

8.4.2 Step 1. Define the Project Conditions

A large number of prestressed concrete piles are to be driven for bridge abutments. There is little previous experience with driving similar piles in the area, and the designers believe that they don't have enough information to be able to estimate pile capacity and length.

They therefore decide to perform a static axial load test, prior to construction, to provide input to design of the abutment foundations. Provisionally, 400 mm (16 inch) octagonal prestressed concrete piles have been selected. See Figure 8-1.

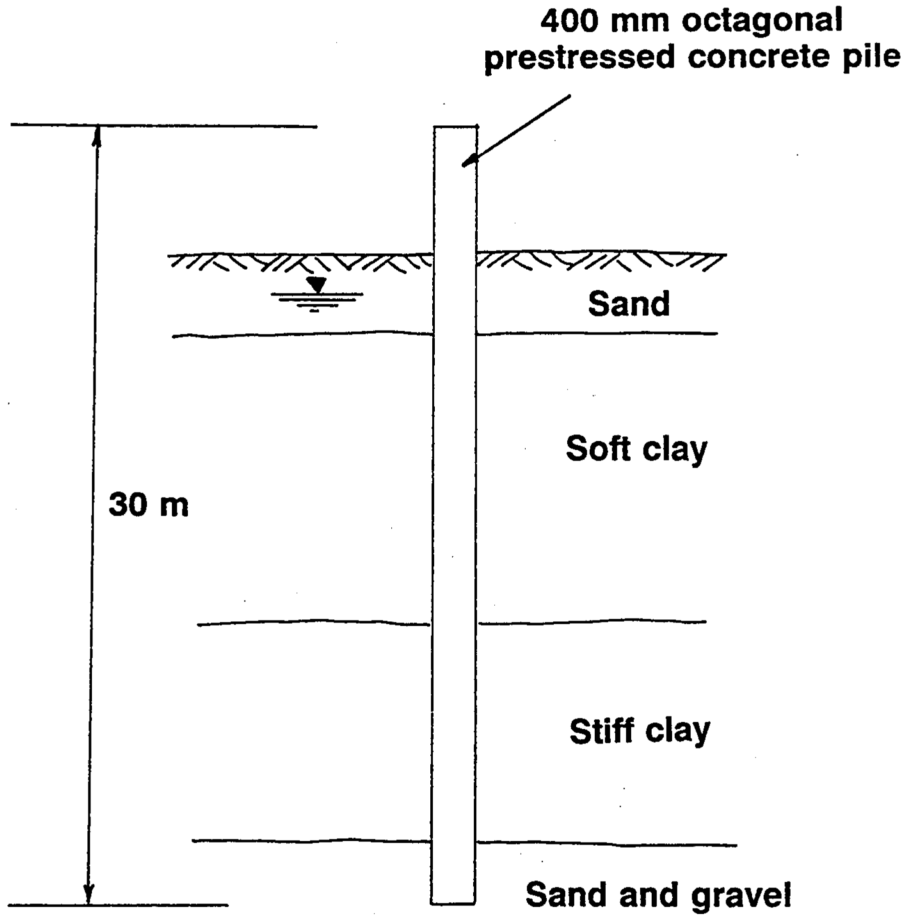


Figure 8-1: Driven Pile

8.4.3 Step 3. Predict Mechanisms That Control Behavior

Consider what happens to loads and deformations down the pile. Draw some sketches, or write some words (Also use the space on page 8-9 if needed).

8.4.4 Step 4. Define the Geotechnical Questions That Need to be Answered

Refer to Section 8.2.2 as needed.

8.4.5 Step 6. Select the Parameters to be Monitored

Appendix A lists the following:

- (a) Groundwater pressure
- (b) Total stress within soil mass
- (c) Total stress at contact with structure or rock
- (d) Vertical deformation
- (e) Horizontal deformation
- (f) Tilt
- (g) Strain in soil or rock
- (h) Load or strain in structural members
- (i) Temperature
- (j) Precipitation

List the parameters that might apply to this case. Indicate where (in general) each parameter would be monitored, e.g. at butt, along pile, at toe.

8.4.6 Step 7. Predict Magnitudes of Change

Appendix A lists the following:

(a) Predict maximum value, thus instrument range
(b) Predict minimum value, thus instrument sensitivity or accuracy
(c) Determine hazard warning levels

Just think about your approach to estimating the needed measurement range and accuracy of no more than two of your selected parameters. Don't make any calculations (you can't do this with the information you have been provided): concepts only.

Parameter 1 : ____________________

- Range

- Accuracy

Parameter 2 : ____________________

- Range

- Accuracy

8.4.7 Step 10. Select Instruments

Refer to Table 8.1 as needed. Be as specific as you can. Would you consider a datalogger?

8.4.8 Step 12. Plan Recording of Factors That May Influence Measured Data

Appendix A lists the following:

(a) Construction details
(b) Construction progress
(c) Visual observations of expected and unusual behavior
(d) Geology and other subsurface conditions
(e) Environmental factors

List any factors that might apply to this case.

8.4.9 Step 13. Establish Procedures for Ensuring Reading Correctness

Appendix A lists the following:

(a) Visual observations
(b) Duplicate instruments
(c) Backup system
(d) Study of consistency
(e) Study of repeatability
(f) Regular in-place checks

List any procedures that might apply to this case.

CHAPTER 9.0
INSTRUMENTATION OF EARTH RETAINING STRUCTURES

9.1 INTRODUCTION

This chapter is a companion to Module 6 (Earth Retaining Structures), and includes a discussion of instrumentation for internally braced excavations (with cross-lot bracing), externally braced excavations (with tiebacks), slurry walls, mechanically stabilized earth walls and soil nailing walls. For each of these five topics, there are two sections. The first section indicates the general role of geotechnical instrumentation. The second section suggests the principal geotechnical questions that may arise and indicates the types of instruments that may be used to help provide answers to those questions. The sequence of geotechnical questions in the second section is intended to match the time sequence in which the question may be addressed during the design, construction, and performance process and does not indicate any rating of importance.

Dunnicliff (1988, 1993) provides an annotated listing of case histories. Annotated case histories related to mechanically stabilized earth walls are given by FHWA (1989a, 1989b).

9.2 INTERNALLY BRACED EXCAVATIONS

9.2.1 General Role of Instrumentation

The design of internally braced excavations is based for the most part on empirical procedures and past experience. The consequences of poor performance can be severe and may on occasion be catastrophic. A monitoring program may not be required if the design is very conservative, if there is previous experience with design and construction of similar facilities under similar conditions, or if the consequences of poor performance will not be severe. However, under other circumstances a monitoring program will normally be required to demonstrate that the excavation is stable and that nearby structures are not affected adversely. Depending on the specific needs of each case, the monitoring program may apply to the wall and bracing, to the ground beneath or surrounding the excavation and/or to adjacent structures or utilities.

9.2.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for internally braced excavations. Table 9-1 summarizes possible instruments.

What Are the Initial Site Conditions?

Initial site conditions are determined by use of conventional site investigation procedures, sometimes supplemented by in situ testing (Module 1 - Subsurface Investigations). However, performance monitoring instrumentation sometimes plays a role. For example, initial groundwater pressures and fluctuations must often be determined for design purposes. Piezometers can be installed well before the start of excavation to define the preconstruction groundwater pressure regime, including any perched or artesian water. If the instruments are installed sufficiently early, seasonal variations can be defined.

A preconstruction conditions survey will normally be made, to observe and document the conditions of structures that may be influenced by the excavation, using photographs, written descriptions, and/or videotapes of existing defects in the structures. The preconstruction conditions survey is frequently

TABLE 9-1
SUITABLE INSTRUMENTS FOR MONITORING INTERNALLY BRACED EXCAVATIONS

Geotechnical Question	Measurement	Suitable Instruments
What are the initial site conditions?	Groundwater pressure	Open standpipe, vibrating wire, or pneumatic piezometers
	Vertical deformation	Surveying methods
	Widths of cracks in buildings	Crack gages
Is the bracing being installed correctly?	Load in bracing	Calibrated hydraulic jack
Is the excavation stable, and are nearby structures being affected adversely by ground movements?	Settlement of ground surface, structures, and top of supporting wall	Surveying methods
	Horizontal deformation of ground surface, structures, and exposed part of supporting wall	Surveying methods Convergence gages
	Change in width of cracks in structures and utilities	Crack gages
	Subsurface horizontal deformation of ground	Inclinometers Fixed borehole extensometers In-place inclinometers
	Subsurface settlement of ground and utilities	Subsurface settlement points Probe extensometers Fixed borehole extensometers
	Load in bracing	Vibrating wire strain gages Mechanical strain gages Calibrated hydraulic jack and load cell (lift-off test)
	Groundwater pressure	Open standpipe, vibrating wire or pneumatic piezometers
	Bottom heave	Magnet/reed switch probe extensometers

TABLE 9-1 (CONTINUED)
SUITABLE INSTRUMENTS FOR MONITORING INTERNALLY BRACED EXCAVATIONS

Geotechnical Question	Measurement	Suitable Instruments
Is an individual brace being overloaded?	Load in bracing	Vibrating wire strain gages Mechanical strain gages Calibrated hydraulic jack and load cell (lift-off test)
What is the magnitude and distribution of load in the support system?	Load in bracing	As above
	Stress in sheeting, walers and slurry walls	Vibrating wire strain gages
	Total stress on slurry walls	Special earth pressure cells and piezometers at soil/wall contact
Is the groundwater table being lowered?	Groundwater pressure	Open standpipe, vibrating wire or pneumatic piezometers
Is excessive bottom heave occurring?	Bottom heave	Magnet/reed switch probe extensometers
	Subsurface horizontal deformation	Inclinometers

supplemented by use of surveying methods to define the elevations of reference points on the structures and on the ground surface. The widths of any existing cracks should also be measured.

Is the Bracing Being Installed Correctly?

Specifications for cross-lot bracing normally require that ground deformation is minimized by preloading the struts. Preloading involves loading the strut to some fraction of the design load by using hydraulic jacks and then wedging the strut between opposite walls of the excavation. Contractor performance can be monitored by observing hydraulic jack pressures.

Is the Excavation Stable and Are Nearby Structures Being Affected Adversely by Ground Movements?

Ground movements may be caused by lateral deformation of the supporting wall and also by bottom heave. The movements may lead to an unstable excavation and may also affect nearby structures or utilities adversely. Instrumentation can often be used to provide a forewarning of adverse behavior, thereby allowing remedial measures to be implemented before critical situations arise. A planned instrumentation program, which indicates that the construction will be watched carefully, can give reassurance to building owners and thus can expedite the legal approval and public acceptance of a project.

Monitoring usually consists of deformation measurements on the surface and alongside utilities. However, if these measurements indicate adverse behavior, there will often be inadequate data for defining the cause of the problem and for developing an economical solution. A well-planned program for monitoring stability and for providing forewarning of adverse behavior should therefore include instrumentation that indicates **causes** of surface deformation. Inclinometers are primary tools for this purpose, supplemented by other instruments as indicated in Table 9-1.

Is an Individual Brace Being Overloaded?

Monitoring of individual braces is not normally required if the design of the support system is very conservative, if there is previous experience with design and construction of similar facilities under similar conditions, or if the consequences of poor performance will not be severe. However, under other circumstances, loads developed in selected representative braces should be monitored.

Surface-mounted vibrating wire strain gages are usually the preferred instruments, and mechanical strain gages are sometimes used as backup. Load cells are not favored for this application, because insertion of a load cell usually alters the method of strut installation, thereby creating nontypical loading conditions on the strut. Strut load can also be determined by temporary insertion of a calibrated hydraulic jack between the waler and a reaction member welded to the strut and by performing a lift-off test, but this method is cumbersome and accuracy is not great.

What is the Magnitude and Distribution of Load in the Support System?

A change in distribution of load with time can be an indicator of impending instability, but the support system will not normally be instrumented for this purpose. The question more normally implies an interest in improving the state of the art of design, rather than in performance monitoring to benefit the project under construction.

Determination of load in internal bracing requires either lift-off testing or use of strain gages, as discussed above.

Stresses in sheeting and walers can be determined by use of strain gages, but because stresses may vary widely from point to point, data may be misleading. Attempts to measure total stress acting on a sheet pile wall by attaching contact earth pressure cells to the sheeting have generally not been successful.

Stresses in slurry walls can be determined by use of strain gages, either vibrating wire gages installed on the reinforcing cage or sister bars with vibrating wire transducers installed in the concrete. Earth pressure cells and piezometers can be installed at the interface between the slurry trench and the soil, using special transducer housings and hydraulic jack installation procedures, for subsequent monitoring of total pressure and pore water pressure on the completed wall.

Is the Groundwater Table Being Lowered?

When dewatering has been implemented to predrain the ground around the excavation, its effectiveness may be monitored by using piezometers.

Prolonged lowering of the groundwater table may cause serious consolidation settlements over a large area when excavations are made in compressible soils. Lowering of the groundwater table can be diagnosed by installing piezometers in any permeable layers and, if economy permits, also with depth along several vertical sections behind the supporting wall.

Is Excessive Bottom Heave Occurring?

When excavating in soft clay, the possibility must be considered that the bottom may fail by heaving. Heave gages can be installed below the bottom of the excavation and time-heave plots used to indicate if and when critical soil movements start to occur. Measurements of horizontal deformation within inclinometer casings installed alongside and below the supporting wall can also be used as indicators of any instability caused by bottom heave.

9.3 EXTERNALLY BRACED EXCAVATIONS

9.3.1 General Role of Instrumentation

The general role of instrumentation for externally braced excavations is the same as for internally braced excavations (Section 9.2.1). However, it is possible to make regular visual inspections of internal bracing, but external bracing cannot be **seen.** Although confidence in the performance of an externally braced excavation is increased by conducting a proof test on every tieback anchor, if an anchor subsequently fails, the failure may be progressive and catastrophic. In general, therefore, instrumentation plays a role in three phases of external bracing that are not applicable to internal bracing: testing of *test anchors* during the design phase or at the start of construction, *performance* and *proof testing* of anchors during construction, and subsequent *monitoring* of selected representative anchors. The third phase may be omitted if a conservative design has been used.

9.3.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for externally braced excavations. Table 9-2 summarizes possible instruments.

What Are the Initial Site Conditions?

This question has been discussed for internal bracing in Section 9.2.2. Monitoring methods are similar for externally braced excavations.

What is a Suitable Design for Tieback Anchors?

The load-movement relationship of a tieback anchor depends on the soil or rock properties and the size and shape of the grouted body of the anchor. The relationship is generally not governed by the properties of the tendon, and computations of allowable loads for anchors are therefore only rough approximations. When design uncertainties are unacceptable, *test anchors* can be installed and tested either during the design phase or at the start of construction. The tests include measurement of load and deformation at the anchor heads.

If the objective of test anchors includes determination of the pattern of load transfer in the grouted zone, strain gages should be used. Spot welded vibrating wire gages are typically used on solid bar tiebacks. Options for stranded tiebacks are vibrating wire "strand gages" and electrical resistance gages, designed specifically for measurements on stranded cables.

TABLE 9-2
SUITABLE INSTRUMENTS FOR MONITORING EXTERNALLY BRACED EXCAVATIONS

Geotechnical Question	Measurement	Suitable Instruments
What are the initial site conditions?	As in Table 9-1	As in Table 9-1
What is a suitable design for tieback anchors?	Load in tieback Deformation at head Load transfer in grouted zone	Load cell, preferably vibrating wire type Long-range dial indicators Vibrating wire or resistance strain gages
Are the tiebacks being installed correctly?	Load in tieback Deformation at head	Load cell, preferably vibrating wire type Long-range dial indicators
Is the excavation stable, and are nearby structures being affected adversely by ground movements?	As in Table 9-1, except for load in bracing Load in tieback	As in Table 9-1, except for load in bracing Load cells, preferably vibrating wire type Calibrated hydraulic jack and load cell (lift-off test) Vibrating wire strain gages
Is an individual tieback being overloaded?	Load in tieback	As for question immediately above
What is the magnitude and distribution of load in the support system?	Load in tieback Stress in sheeting, walers and slurry walls, total stress on slurry walls	As above As in Table 9-1
Is the groundwater table being lowered?	As in Table 9-1	As in Table 9-1
Is excessive bottom heave occurring?	As in Table 9-1	As in Table 9-1
Is long-term performance of the bracing satisfactory?	Load in tiebacks Surface deformation	Load cells, preferably vibrating wire type Calibrated hydraulic jack and load cell (lift-off test) Vibrating wire strain gages Surveying methods

Are the Tiebacks Being Installed Correctly?

Specifications for external bracing normally require that each tieback anchor is proof tested to a value in excess of the working load used in design. A *proof test* either verifies that the carrying capacity is within acceptable limits or indicates that an anchor is defective. The test normally involves loading the anchor incrementally to a load greater than the design load, holding for a creep test, and locking-off at a lesser load. Load and displacement at the anchor head are monitored throughout the test, using a load cell and dial indicator as described above for test anchors. The load cell is positioned immediately above or below the jack and is removed after completion of the proof test.

In addition to proof testing each anchor, it is customary to conduct a *performance test* on a specified percentage of anchors. A performance test normally entails loading and unloading the anchor incrementally in cycles, increasing the load at the end of each cycle to the same maximum load as for the proof test, holding for a longer creep test, and locking-off as usual. Instrumentation is the same as for proof testing.

Is the Excavation Stable, and are Nearby Structures Being Affected Adversely by Ground Movements?

This question has been discussed for internal bracing in Section 9.2.2. With the exception of load in the bracing, monitoring methods are similar for externally braced excavations.

Is an Individual Tieback Being Overloaded?

For an externally braced excavation, anchor loads subsequent to lock-off can be determined by lift-off testing. If several measurements are required on an anchor, at different times, it will usually be more economical to install a load cell. Surface-mounted strain gages, preferably of the spot welded vibrating wire type, are an alternative to load cells if solid bar tiebacks are used.

What is the Magnitude and Distribution of Load in the Support System?

Determination of total load in external bracing requires either lift-off testing or use of load cells or strain gages, as discussed above.

Is the Groundwater Table Being Lowered?

This question has been discussed for internal bracing in Section 9.2.2. Monitoring methods are similar for externally braced excavations.

Is Excessive Bottom Heave Occurring?

This question has been discussed for internal bracing in Section 9.2.2. Monitoring methods are similar for externally braced excavations.

Is Long-Term Performance of the Bracing Satisfactory?

Tieback anchors are now a proven and accepted method for temporary support, and there is an increasing trend to install permanent tiebacks for long-term support. This trend leads to an increased role for instrumentation on critical projects where there is a need to demonstrate long-term satisfactory performance.

Long-term performance monitoring can consist of monitoring load in and deformation of selected anchors, or monitoring deformation on a larger scale, or both. Monitoring long-term deformation on a larger scale than at selected anchors normally requires surveying methods, with electronic distance measurements playing a primary role. Monitoring of selected anchors will generally follow the procedures outlined above for examining whether an individual anchor is overloaded. However, two additional issues must be taken into account when monitoring anchors on a long-term basis: longevity of load cells and interruption to corrosion protection. Recommendations for addressing these two issues are given by Dunnicliff (1988, 1993).

9.4 SLURRY WALLS

9.4.1 General Role of Instrumentation

Instrumentation plays a role in determining the feasibility of excavating for slurry walls, and in controlling their construction.

9.4.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for slurry walls. Table 9-3 summarizes possible instruments.

What Is a Suitable Design for Slurry Trench Excavations?

If soil conditions are well defined and if the performance of slurry trench excavations has already been proven in similar soils, feasibility is not in doubt. However, in other cases, either the safe and economical construction method or even the feasibility of constructing a slurry trench may be in doubt, and a full-scale instrumented test may be warranted.

Is Construction of Slurry Trenches Adequately Controlled?

Observations and measurements are generally required for construction control of slurry trenches. Construction contractors will generally measure and control the level and properties of the slurry, including density, pH, viscosity, sand content, and filtration. They may also monitor verticality, but this is not common practice, and when verticality is critical to use or appearance, inspection personnel should initiate a measuring program. Five methods are available for measuring verticality. First, a tiltmeter can be attached to the kelley bar, cable, or bucket. Second, a short length of pipe, with outside diameter slightly less than the width of the trench, can be lowered within the trench on the end of a wire, and the lateral position of the wire with respect to the guide walls can be monitored as the pipe is lowered. Third, if a pipe is used as an end-stop in the panel, it can first be traversed along the length of the panel: if it will not pass easily along the length, the trench is not likely to be vertical. Fourth, an ultrasonic monitoring instrument can be used to map out the profile of both sides of the trench. Fifth, an inclinometer can be used, but this method is time-consuming and its use is normally limited to test trenches.

Vertical movements of the ground surface, guide walls, and adjacent structures should be monitored as panels are excavated, and rate of movement is a key factor in evaluating trench stability. Trench stability can also be evaluated by comparing the quantity of excavated material with the depth of the trench: if material is removed, but the depth does not increase, caving is occurring.

The integrity of concrete in a slurry trench can be tested by using one of the various integrity tests.

TABLE 9-3
SUITABLE INSTRUMENTS FOR MONITORING SLURRY WALLS

Geotechnical Question	Measurement	Suitable Instruments
What is a suitable design for slurry trench excavations?	Stability in relation to time, panel length, slurry level and density. Also deformation of trench during concreting	Surveying methods Inclinometers Special gages to monitor convergence across trench Probe extensometers Vibrating wire or pneumatic piezometers Settlement platforms
	Shape and alignment of trench	Remote reading calipers Inclinometers
Is construction of slurry trenches adequately controlled?	Verticality of trench	Tiltmeter Pipe and wire End stop Ultrasonic "drilling monitor" Inclinometer
	Vertical movement of ground surface, guide walls and adjacent structures	Surveying methods
	Integrity of concrete	Integrity testing

9.5 MECHANICALLY STABILIZED EARTH WALLS

9.5.1 General Role of Instrumentation

Mechanically stabilized earth walls are a relatively recent technology, and many types are in current use (Module 6 - Earth Retaining Structures). One of the steps towards acceptance by the geotechnical engineering profession has been to monitor performance with instrumentation, as a way to demonstrate stability. As advances in technology increase, the need for performance monitoring remains. FHWA (1989a, 1989b, 1996) present guidelines on monitoring mechanically stabilized earth walls.

9.5.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for mechanically stabilized earth walls. Table 9-4 summarizes possible instruments.

What Information Can be Provided by Instrumenting a Test Wall?

Test walls are sometimes constructed to resolve uncertainties in the selection of soil parameters and stabilizing materials, to examine alternative construction methods, or to demonstrate construction feasibility. The goal of instrumentation in these tests will generally be to provide an indication of incipient failure and/or to monitor stresses and deformations to permit an evaluation of performance.

Depending on the specifics of the case, instrumentation for a test wall will include any or all of the monitoring methods discussed elsewhere in Section 9.5.2.

Is the Wall Stable?

If the possibility of lateral instability is minor, instrumentation will not usually be required during construction of a mechanically stabilized earth wall. However, in other cases a monitoring program is generally required to provide a forewarning of instability, thereby allowing remedial measures to be implemented before critical situations arise.

Primary monitored parameters are surface and subsurface deformation, and drainage behavior of the backfill.

TABLE 9-4
SUITABLE INSTRUMENTS FOR MONITORING MECHANICALLY STABILIZED EARTH WALLS

Geotechnical Question	Measurement	Suitable Instruments
What information can be provided by instrumenting a test wall?	As for remaining questions in Table 9-4	As for remaining questions in Table 9-4
Is the wall stable?	Deformation of the face and overall surface of the wall	Surveying methods
	Local deterioration of facing elements	Crack gages
	Drainage behavior of the backfill	Visual observation of outflow points during rainfall Open standpipe, vibrating wire or pneumatic piezometers
	Horizontal deformation within wall	Probe extensometers Fixed embankment extensometers Inclinometers
	Vertical deformation within wall	Probe extensometers Horizontal inclinometers Liquid level gages

TABLE 9-4 (CONTINUED)
SUITABLE INSTRUMENTS FOR MONITORING MECHANICALLY STABILIZED EARTH WALLS

Geotechnical Question	Measurement	Suitable Instruments
What is the magnitude and distribution of stress in the wall?	Stress distribution at base of wall or in backfill	Embedment earth pressure cells
	Lateral earth pressure at back of facing elements	Contact earth pressure cells Strain gages at connections Load cells at connections
	Stress in reinforcement	Electrical resistance strain gages Vibrating wire strain gages Multiple telltales
What is the performance of structures constructed on or adjacent to the wall?	Depends on structure	Numerous possible instruments (depends on structure), e.g. Surveying methods Crack gages Tiltmeters Convergence gages
What is the long-term performance of the wall?	Deformation of the face, overall surface of the wall, and within wall	As for "Is the wall stable?"
	Local deterioration of facing elements	As for "Is the wall stable?"
	Drainage behavior of the backfill	As for "Is the wall stable?"
	Stress in reinforcement	As for "What is the magnitude and distribution of stress in the wall?"

What is the Magnitude and Distribution of Stress in the Wall?

A change in distribution of stress with time can be an indicator of impending instability, but the wall system will not normally be instrumented for this purpose. The question more normally implies an interest in improving the state of the art of design, rather than in performance monitoring to benefit the project under construction.

The accuracy of stress measurements within the backfill, using embedment earth pressure cells, is severely hampered by the issues discussed in Section 4.2.3. The accuracy of measurements of lateral earth pressure at the back of the facing elements is also hampered by the difficulty in installing the sensitive face absolutely flush with the back surface of the facing elements (Section 4.3.6), and also because hydraulic earth pressure cells are very sensitive to ambient temperature changes (Section 4.3.6): such measurements have not usually been successful.

Stress in steel reinforcement is usually measured with spot welded vibrating-wire strain gages, although spot welded electrical resistance strain gages have sometimes been used with success. The steel should be gaged at both the top and the bottom, so that axial stresses, unaffected by bending, can be calculated. On very small diameter steel reinforcement, such as grid reinforcement, the only strain gage option is small bonded electrical resistance strain gages, and these must be installed in a very controlled environment, with great care over detailed issues such as gage circuitry, surface preparation, bonding, waterproofing and connections (Chapter 5).

Measurement of stress in geotextile sheet reinforcement requires electrical resistance strain gages with a long gage length (e.g. 100 mm), with elongation characteristics that are compatible with the geotextile. Details are given by Fowler and Leshchinsky (1990), Leshchinsky and Fowler (1990), and Koerner (1996). Multiple telltales can also be used.

What is the Performance of Structures Constructed on the Wall?

Structures constructed on mechanically stabilized earth walls may include approach slabs for bridge abutments or footings.

What is the Long-Term Performance of the Wall?

Regular visual observations play a critical role in long-term performance monitoring. If instrumentation is installed for this purpose, a special effort must be made to maximize longevity of all the components.

9.6 SOIL NAILING WALLS

9.6.1 General Role of Instrumentation

The general role of instrumentation for soil nailing walls is similar to that for mechanically stabilized earth walls (Section 9.5.1): to examine performance of new technology. Thompson and Miller (1990) present an instrumented case history.

9.6.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for soil nailing walls. Table 9-5 summarizes possible instruments.

What Information Can be Provided by Instrumenting a Test Wall?

The applicability of this question is the same as for mechanically stabilized earth walls, as discussed in Section 9.5.2. Pullout tests can be conducted to determine nail capacity (Elias and Juran, 1991).

Is the Wall Stable?

Again, the applicability is as discussed in Section 9.5.2.

TABLE 9-5
SUITABLE INSTRUMENTS FOR MONITORING SOIL NAILING WALLS

Geotechnical Question	Measurement	Suitable Instruments
What information can be provided by instrumenting a test wall?	As for remaining questions in Table 9-5	As for remaining questions in Table 9-5
	Nail capacity	Load cell, preferably vibrating wire type Dial indicator
Is the wall stable?	Deformation of the overall surface of the wall	Surveying methods
	Drainage behavior of the ground	Visual observation of outflow points during rainfall Open standpipe, vibrating wire or pneumatic piezometers
	Horizontal deformation within wall	Probe extensometers Inclinometers
	Vertical deformation within wall	Probe extensometers
What is the magnitude and distribution of stress in the soil nails?	Stress in nails	Vibrating wire strain gages
What is the performance of structures constructed on the wall?	As in Table 9-4	As in Table 9-4
What is the long-term performance of the wall?	Deformation	As for "Is the wall stable?"
	Stress in nails	Vibrating wire strain gages

What is the Magnitude and Distribution of Stress in the Soil Nails?

Stress in the nails is usually measured with spot welded vibrating wire strain gages. The nails should be gaged at both the top and the bottom so that axial stresses, unaffected by bending, can be calculated.

What is the Performance of Structures Constructed on the Wall?

The applicability is as discussed in Section 9.5.2.

What is the Long-Term Performance of the Wall?

Again, the applicability is as discussed in Section 9.5.2.

9.7 STUDENT EXERCISE FOR PLANNING A MONITORING PROGRAM FOR A MECHANICALLY STABILIZED EARTH WALL

9.7.1 Introduction

The instructor will present the first step in the planning process, "Define the Project Conditions", and will indicate why a monitoring program is required. Students will then work in groups to consider the following steps:

- Step 3. Predict Mechanisms That Control Behavior
- Step 4. Define the Geotechnical Questions That Need to be Answered
- Step 6. Select the Parameters to be Monitored
- Step 7. Predict Magnitudes of Change
- Step 10. Select Instruments
- Step 12. Plan Recording of Factors That May Influence Measured Data
- Step 13. Establish Procedures for Ensuring Reading Correctness

Space is provided on the following pages for students to make notes during the exercise, and selected wording from Appendix A "Checklist for Planning Steps" is included to assist students.

An example of results of the student exercise is included in Appendix E. However, **it is important to recognize that entirely different results may apply to any real project situation.**

9.7.2 Step 1. Define The Project Conditions

A mechanically stabilized earth wall is to be constructed. Based on restricted geometry, and the fact that ideal backfill material is not available, the designers believe that a monitoring program is necessary to verify that field performance is in accordance with design assumptions. See Figure 9-1.

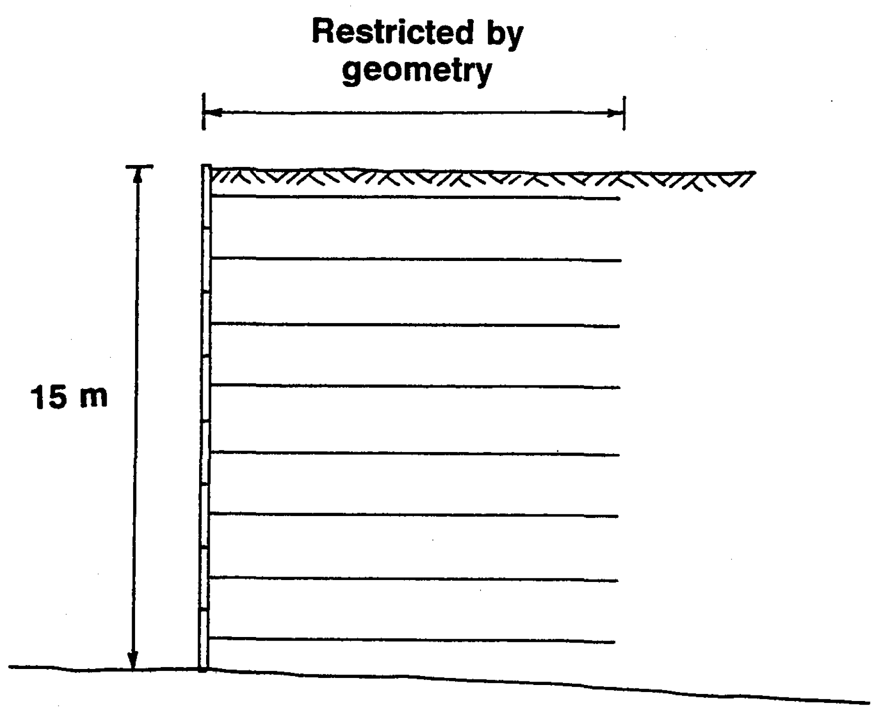


Figure 9-1: Mechanically Stabilized Earth Wall

9.7.3 Step 3. Predict Mechanisms That Control Behavior

Consider possible modes of deformation. Draw some sketches here, or write some words.

9.7.4 Step 4. Define the Geotechnical Questions That Need to be Answered

Refer to Section 9.5.2 as needed.

9.7.5 Step 6. Select the Parameters to be Monitored

Appendix A lists the following:

(a) Groundwater pressure
(b) Total stress within soil mass
(c) Total stress at contact with structure or rock
(d) Vertical deformation
(e) Horizontal deformation
(f) Tilt
(g) Strain in soil or rock
(h) Load or strain in structural members
(i) Temperature
(j) Precipitation

List the parameters that might apply to this case.

9.7.6 Step 7. Predict Magnitudes of Change

Appendix A lists the following:

(a) Predict maximum value, thus instrument range
(b) Predict minimum value, thus instrument sensitivity or accuracy
(c) Determine hazard warning levels

Just think about your approach to estimating the needed measurement range and accuracy of no more than two of your selected parameters. Don't make any calculations (you can't do this with the information you have been provided): concepts only.

Parameter 1 : ____________________

- Range

- Accuracy

Parameter 2 : ____________________

- Range

- Accuracy

9.7.7 Step 10. Select Instruments

Refer to Table 9-4 as needed. Be as specific as you can.

9.7.8 Step 12. Plan Recording of Factors That May Influence Measured Data

Appendix A lists the following:

(a) Construction details
(b) Construction progress
(c) Visual observations of expected and unusual behavior
(d) Geology and other subsurface conditions
(e) Environmental factors

Check those that might apply to this case, and add some details if you can.

9.7.9 Step 13. Establish Procedures for Ensuring Reading Correctness

Appendix A lists the following:

(a) Visual observations
(b) Duplicate instruments
(c) Backup system
(d) Study of consistency
(e) Study of repeatability
(f) Regular in-place checks

Check those that might apply to this case, and add some details if you can.

CHAPTER 10.0
INSTRUMENTATION OF GROUND IMPROVEMENT TECHNIQUES

10.1 INTRODUCTION

This chapter is a companion to Module 4 (Ground Improvement Techniques), and includes a discussion of instrumentation for preloading, vertical drains, staged construction and grouting. For each of these topics, there are two sections. The first section indicates the general role of geotechnical instrumentation. The second section suggests the principal geotechnical questions that may arise and indicates the types of instruments that may be used to help provide answers to those questions. The sequence of geotechnical questions in the second section is intended to match the time sequence in which the question may be addressed during the design, construction, and performance process and does not indicate any rating of importance.

10.2 PRELOADING, VERTICAL DRAINS AND STAGED CONSTRUCTION

10.2.1 General Role of Instrumentation

Instrumentation of embankments on soft ground is discussed in Chapter 7. When preloading vertical drains, or staged construction is employed, instrumentation will normally be used as a tool to indicate the progress of consolidation. Instrumentation data allow schedules to be determined for removal of preload and for placement of stages during staged construction and also allow an evaluation of the effectiveness of vertical drains.

10.2.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for preloading, vertical drains and staged construction. They are the same questions that may lead to the use of instrumentation for embankments on soft ground, as discussed in Chapter 7, and possible instruments are the same as those summarized in Table 7-1.

What are the Initial Site Conditions?

This question is discussed in Section 7.2.2.

What Information Can be Provided by Instrumenting a Test Embankment?

Instrumentation for test embankments on soft ground is discussed in Section 7.2.2. When preloading vertical drains or staged construction is used, a test embankment is sometimes constructed to determine the necessary height of the preload, the spacing and/or type of vertical drains, or the necessary height of each construction stage.

What is the Progress of Consolidation?

This question is discussed in Section 7.2.2. When planning a program to monitor the performance of vertical drains, a decision must be made on whether to install instruments before or after installation of drains. Installing instruments well before drains ensures that baseline data are obtained, but runs the risk of damage to instruments as drains are installed. The author suggests installing up to half of the instruments well before installing drains, and the remainder afterward. Ladd (1991) provides guidelines on the interpretation of field data when vertical drains are used.

Is the Embankment Stable?

This question is discussed in Section 7.2.2.

What are the Fill Quantities?

Again, this question is discussed in Section 7.2.2.

10.3 GROUTING

10.3.1 General Role of Instrumentation

The role of instrumentation for grouting depends on the type and purpose of the grouting. Instrumentation to control flows and pressures at the grout pump is outside the scope of this Module.

Instrumentation for permeation grouting may entail monitoring of groundwater pressure to examine the effectiveness of the grouting, vertical deformation of the ground to examine possible uplift during grouting, and/or monitoring of permeability before and after grouting. Monitoring of permeability is outside the scope of this Module.

Instrumentation for compaction grouting may entail monitoring of surface and subsurface deformations, to examine the effectiveness of the grouting.

Instrumentation for jet grouting may entail monitoring of groundwater pressure, to examine the effects of high short-term pressures on the behavior of the ground.

Instrumentation for fracture grouting may entail monitoring of possible uplift during grouting.

10.3.2 Principal Geotechnical Questions

The following are possible geotechnical questions that may lead to the use of instrumentation for grouting. Table 10-1 summarizes possible instruments.

What are the Groundwater Pressures?

This question may arise during permeation and jet grouting. Selection among piezometers depends on the issues discussed in Chapter 2, paying particular attention to response time requirements. For jet grouting, high-range vibrating wire piezometers are the instruments of choice.

What are Vertical Ground Deformations?

Where uplift is a concern, during permeation or fracture grouting, measurements need to be taken rapidly so that measured deformations can be used to control the grouting operation. Surveying methods can be used, either to determine uplift of points on the ground surface or of subsurface settlement points, provided that enough surveying personnel are available for rapid data collection. If more rapid data are required, fixed borehole extensometers with electrical transducers can be used. For rapid determination of ground surface uplift, a single point fixed borehole extensometer can be used, with the downhole anchor set below any zone of possible uplift. Tiltmeters can also be used to indicate the rotational component of uplift at the ground surface.

When measurements of vertical deformation are required to examine the effectiveness of compaction grouting, surveying methods and probe extensometers are typically used.

What are Horizontal Ground Deformations?

When measurements of horizontal deformation are required to examine the effectiveness of compaction grouting, surveying methods, inclinometers and in-place inclinometers are typically used.

TABLE 10-1
SUITABLE INSTRUMENTS FOR MONITORING GROUTING

Geotechnical Question	Measurement	Suitable Instruments
What are the groundwater pressures?	Groundwater pressure	Permeation grouting: open standpipe, vibrating wire or pneumatic piezometers Jet grouting: vibrating wire piezometers
What are vertical ground deformations (uplift)?	Vertical deformation	Surveying methods Subsurface settlement points Probe extensometers Fixed borehole extensometers Tiltmeters
What are horizontal ground deformations?	Horizontal deformation	Surveying methods Inclinometers In-place inclinometers

10.4 OTHER GROUND IMPROVEMENT TECHNIQUES

Other ground improvement techniques discussed in Module 4 (Ground Improvement Techniques) include dynamic compaction, lightweight fill materials and shredded tire fills.

The effectiveness of dynamic compaction is usually evaluated by visual observations and by monitoring ground surface settlement. If a numerical evaluation is required, a comparison can be made between "before" and "after" data obtained by using one of the in situ testing methods described in Module 1 (Subsurface Investigations). Geotechnical instruments installed in boreholes are rarely useful, because of the likelihood of damage. However vibrating wire piezometers can be used to measure pore water pressure in situations where this is a relevant parameter.

Instrumentation for lightweight fills should follow the guidelines given in Chapter 7 for instrumentation of embankments on soft ground.

Monitoring of elevated temperatures in shredded tire fills requires use of thermistors, thermocouples or resistance temperature devices (RTDs). Table 10-2 provides a comparison among these three options.

TABLE 10-2
COMPARISON AMONG TEMPERATURE MEASURING INSTRUMENTS

Feature	Thermistor	Thermocouple	RTD
Readout	Digital ohmmeter or multimeter	Thermocouple reader	Wheatstone bridge with millivolt scale
Sensitivity	Very high	Low	Moderate
Linearity	Very poor	Fair	Fair
Accuracy	High	Moderate	Very high (but may be reduced by lead wire effects)
Stability	Excellent	Good	Excellent
Type of lead wire	Two-conductor	Special (bimetal)	Three-conductor
Repairability of lead wire	Straightforward	Less straightforward (can cause errors)	Straightforward
Temperature range	Wide	Wide	Wide
Rapidity of response	Rapid	Rapid	Rapid
Applicability for instrument temperature corrections	Preferred	Possible	Possible
Suitability for automatic data acquisition	Fair	Excellent	Good

CHAPTER 11.0
INSTRUMENTATION OF ROCK SLOPES

11.1 INTRODUCTION

This chapter is a companion to Module 5 (Rock Slopes), and includes a discussion of instrumentation for cut slopes in rock, and landslides in rock. For each of these two topics, there are two sections. The first section indicates the general role of geotechnical instrumentation. The second section suggests the principal geotechnical questions that may arise and indicates the types of instruments that may be used to help provide answers to those questions. The sequence of geotechnical questions in the second section is intended to match the time sequence in which the question may be addressed during the design, construction, and performance process and does not indicate any rating of importance.

11.2 CUT SLOPES IN ROCK

11.2.1 General Role of Instrumentation

The general role of instrumentation is identical to the role for cut slopes in soil, as discussed in Section 7.3.1.

11.2.2 Principal Geotechnical Questions

The principal geotechnical questions are identical to those that arise for cut slopes in soil, as discussed in Section 7.3.2. However, because stability normally depends on conditions along discontinuities in the rock mass, rather than within intact rock blocks, and also because critical deformations are often smaller for cut slopes in rock, the list of suitable instruments is not the same as for cut slopes in soil. Table 11-1 summarizes possible instruments.

When planning to monitor the stability of rock slopes, it is important to recognize that if the slope is subject to a brittle failure mode, movement will be sudden. In such cases, geotechnical instrumentation may not be appropriate to forewarn of instability. It may be more appropriate to develop an area-wide correlation between precipitation intensity and slope instability, and to use precipitation measurements to warn of potential problems.

Deformation monitoring will often be limited to surface measurements. Surface measurements should extend beyond the uppermost limit of any possible movement zone to an area that is known to be stable, so that possible surface strain in advance of cracking can be monitored. Any toe heave should also be monitored. Tension cracks at the crest of the slope may be the first sign of instability. If cracks appear at the crest of the slope or elsewhere, their widths and vertical offsets should be monitored. Crack measurements give clues to the behavior of the entire slope, and often the direction of movement may be inferred from the pattern of cracking, particularly by the matching of the irregular edges of the cracks. If necessary, crack gages can be connected to alarms, set to trigger after a predetermined deformation has occurred.

Acoustic emission techniques can sometimes be used by experienced personnel over a wide area in shallow drillholes to determine deformation trends and locations.

Monitoring the tilt of critical blocks can provide an assessment of stability if the deformation has a rotational component. Tiltmeters with electrolytic level transducers provide the most precise data, and the high precision allows trends to be determined in a minimum time period.

TABLE 11-1
SUITABLE INSTRUMENTS FOR MONITORING CUT SLOPES IN ROCK

Geotechnical Question	Measurement	Suitable Instruments
What are the initial site conditions?	Groundwater pressure	Open standpipe, vibrating wire, or pneumatic piezometers Multi-point piezometers
	Horizontal deformation	Surveying methods Fixed borehole extensometers
Is the slope stable during excavation?	Surface deformation	Surveying methods Crack gages Tiltmeters Metallic time domain reflectometry
	Subsurface deformation	Fixed borehole extensometers In-place inclinometers Acoustic emission monitoring Metallic time domain reflectometry
	Groundwater pressure	Vibrating wire or pneumatic piezometers Multi-point piezometers
Is the slope stable in the long term?	As for "Is the slope stable during excavation?"	As for "Is the slope stable during excavation?"
	Precipitation	Rain gages Snow stakes
	Load in tiebacks	Load cells

Because critical deformations are often small, the required accuracy of deformation measurements is generally greater than for soil. Fixed borehole extensometers, installed from the face of the slope following excavation of a rock bench, may therefore be selected for monitoring subsurface horizontal deformation of slopes in rock in preference to inclinometers.

In-place inclinometers can provide real-time monitoring of subsurface deformation, and these instruments can be connected to alarms if required.

Metallic time domain reflectometry (MTDR) (Chapter 3) shows promise for monitoring subsurface deformation. It also shows promise for locating scarps and providing an early warning of movement by installing MTDR cables in horizontal trenches along crests of slopes.

The heterogeneous nature of most rock masses results in a need for comprehensive monitoring of joint water pressure along, above, and below each possible failure plane. Patton (1983) presents a strong case for use of the movable probe type of multi-point piezometer, because it (1) provides a large number of measuring points, (2) does not alter existing groundwater pressures, (3) allows the transducer to be calibrated at any time, (4) provides redundancy in field data, and (5) reduces the problems of creating multiple seals when more than one conventional piezometer is installed in a single borehole. Patton comments: *In my opinion most existing field piezometer installations for [rock] slope stability investigations are deficient in the number of piezometers (probably by a factor of 5 to 10) unless the geology and hydrology of the slope are very simple. However, simple geology and hydrology cannot be demonstrated without a significant number of drillholes to document the geologic and piezometric data.*

If permanent tiebacks have been installed to stabilize the slope, special load cells can be installed for long-term monitoring.

11.3 LANDSLIDES IN ROCK

11.3.1 General Role of Instrumentation

The general role of instrumentation is identical to the role for landslides in soil, as discussed in Section 7.4.1. Dunnicliff (1995b) presents an overview of monitoring methods.

11.3.2 Principal Geotechnical Questions

The principal geotechnical questions are identical to those that arise for cut slopes in soil, as discussed in Section 7.4.2. However, as discussed in Section 11.2.2, the list of possible instruments is not the same as for cut slopes in soil. Table 11-2 summarizes possible instruments.

TABLE 11-2
SUITABLE INSTRUMENTS FOR MONITORING LANDSLIDES IN ROCK

Geotechnical Question	Measurement	Suitable Instruments
What are the post-landslide conditions?	Surface deformation	Surveying methods Crack gages Tiltmeters Metallic time domain reflectometry
	Subsurface deformation	Fixed borehole extensometers In-place inclinometers Acoustic emission monitoring Metallic time domain reflectometry
	Groundwater pressure	Vibrating wire or pneumatic piezometers Multi-point piezometers
Is the slope stable in the long term?	As for "What are the post-landslide conditions?"	As for "What are the post-landslide conditions?"
	Precipitation	Rain gages Snow stakes
	Load in tiebacks	Load cells

CHAPTER 12.0
GENERAL GUIDELINES ON EXECUTION OF MONITORING PROGRAMS

12.1 INTRODUCTION

This chapter presents general guidelines on calibration, maintenance and installation of instruments, and on collection, processing presentation, interpretation and reporting of instrumentation data.

12.2 CALIBRATION

An instrument reading is useful only if the correct calibration is known. Transducer zero shifts and scale span changes can occur because of normal wear and tear, misuse, creep, moisture ingress, and corrosion. If these changes are not accounted for, the entire monitoring program can become worthless. To maximize their effectiveness, all instruments must be calibrated properly.

Calibration consists of applying known pressures, loads, displacements, or temperatures to an instrument, under controlled environmental conditions, and measuring the response.

Instrument calibrations are generally required at three stages: prior to shipment of instruments to the user (factory calibrations), when instruments are first received by the user (pre-installation acceptance tests) and during service life. These are discussed in turn in the following sections.

12.2.1 Factory Calibrations

Instruments should be calibrated at the factory before shipment to the user. Experience indicates that factory calibrations are often minimal and may be incomplete and insufficient. The responsibility for this shortcoming is not with manufacturers, but with users who opt for low-bid procurement procedures.

Chapter 6 indicates that instrumentation procurement documents should include factory calibration and quality assurance requirements, and includes suggested wording.

12.2.2 Pre-Installation Acceptance Tests

Instruments often receive rough handling while in transit from the manufacturer to the user and must be checked by the user to ensure correct functioning before installation. Such checks are called pre-installation acceptance tests.

Whenever possible, pre-installation acceptance tests should include a verification of calibration data provided by the manufacturer, by checking two or three points within the measurement range, with transducers and readout unit at the various temperatures anticipated in the field during service life. Tests at extreme anticipated temperatures are important and may reveal malfunctions that, if not corrected, would result in faulty data.

When comprehensive pre-installation acceptance tests are not possible, simple tests should be performed to verify that instruments appear to be working correctly. These are referred to as function checks. Transducers should be connected to readout units and tilted, pressurized, squeezed, or pulled to induce changes of magnitude consistent with the calibrations supplied. Each electrical connector should be unmade and remade several times. The zero reading should agree with the reading supplied by the manufacturer. All electrical transducers intended for burial should be immersed in water for as long as possible to check the waterproofing.

Table 12-1 indicates possible items in pre-installation acceptance tests.

TABLE 12-1
POSSIBLE ITEMS IN PRE-INSTALLATION ACCEPTANCE TESTS

Category	Item
Data supplied by manufacturer	Examine factory calibration curve and tabulated data, to verify completeness Examine manufacturer's final quality assurance inspection checklist, to verify completeness
Documentation	Check, by comparing with procurement document, that model, dimensions, and materials are correct Check that quantities received correspond to quantities ordered
Calibration checks	Check two or three points, if practicable Check zero reading, e.g. of vibrating wire piezometers
Function checks	Connect to readout and induce change in parameter to be measured Make and remake connectors several times, to verify correct functioning Immerse in water, if applicable, and check
Electrical	Perform resistance and insulation testing, in accordance with criteria provided by the instrument manufacturer
Miscellaneous	Check cable length Check tag numbers on instrument and cable Verify that all components fit together in the correct configuration Check all components for signs of damage in transit

Any instrument that fails a pre-installation acceptance test or function check should be returned to the manufacturer for replacement or repair, with a description of failure characteristics. In addition to verifying calibrations and detecting faulty instruments, these tests and checks provide an opportunity for the user to learn how to operate the instruments correctly.

The following recommendations (Dunnicliff, 1996b) relate to pre-installation acceptance tests of vibrating wire piezometers:

1. Temperature transients cause false readings. "Temperature transient" refers to different parts of the instrument being at different temperatures, such that strains occur through the instrument until an equilibrium temperature is reached. To observe this effect, hang a vibrating wire piezometer outside the window on a cold or a hot day, and take a reading every minute. If a vibrating wire piezometer is lowered, on a hot or a cold day, down a water-filled borehole, it will take between 20 minutes and 2 hours, depending on manufacturer, to achieve a uniform temperature and hence to give a correct reading. Gripping a piezometer in the hand can also cause temperature transients. Temperature transient errors cannot be eliminated by using the manufacturer's temperature correction factors, because these assume uniform temperature.

2. When a vibrating wire piezometer is in air, it measures atmospheric pressure, just like a barometer. When the piezometer is submerged in water that is open to the atmosphere, it measures both water pressure and atmospheric pressure. The pre-installation reading therefore depends on barometric pressure. Manufacturers provide a pre-shipment reading and corresponding barometric pressure with each piezometer, and also a temperature correction factor. A pre-installation reading should be established, and this should be compared with the manufacturer's pre-shipment value, correcting for barometric pressure and temperature. If the two differ significantly, the manufacturer should be contacted.

3. If changes **have** occurred between factory and site, they are much more likely to be changes in "zero" reading rather than changes in slope of the calibration plot.

4. Although changes in slope of the calibration plot are unlikely, some users choose to verify that this has not occurred. Three methods are possible, in priority order:

   - Connect the piezometer to a known air pressure source, and check a few points within the range. This can be done in a laboratory. Alternatively some manufacturers provide a small portable pressure chamber with a precise electrical pressure transducer for this purpose.

   - Lower the piezometer within a water-filled pipe, and check that a few readings agree with the level of water in the pipe. If an installed inclinometer casing is available, this can be used. Remember that the water level varies as the piezometer is lowered or raised, because of the volume of the cable below the water level.

   - Lower the piezometer down the borehole in which it is to be installed, and check as above, again remembering the changing water level. Also, an allowance must be made for the likelihood of borehole water not being clean, hence having a specific gravity greater than one.

   In all three methods, time must be allowed for thermal equilibrium.

5. In general, it is difficult to duplicate factory conditions in the field. Pre-installation acceptance tests should be regarded more as function checks rather than check calibrations.

6. If a piezometer with a dry filter is placed in water, surface tension in the filter will affect the reading. Hence, for checking, either the filter should be removed or the filter and cavity should be completely saturated.

7. The same problem can arise if the piezometer is placed in a sand-filled bag.

It should be noted that almost all sensors are affected by temperature transients–this issue is by no means limited to vibrating wire piezometers.

12.2.3 Calibrations During Service Life

Calibrations or function checks of readout units are required during service life. These calibrations are usually performed by personnel responsible for data collection.

Portable readout units are especially vulnerable to changes in calibration, often resulting from mishandling and lack of regular maintenance. They can sometimes be checked and/or recalibrated by following the pre-installation acceptance test procedure. When this is insufficient, calibrations can often be made at local commercial calibration houses, using equipment traceable to the National Institute of Standards and Technology. Certain instruments such as inclinometers can only be function-checked by the user or at calibration houses and must be returned to the manufacturer for any needed complete calibration, adjustment, and repair.

Calibration frequency, of course, depends on the application and use environment, but as a general rule the user should arrange for regular calibrations on a frequent rather than infrequent schedule.

Many users have experienced the dilemma of discovering that changes in calibration have occurred, therefore being unsure of data correctness since the last calibration date. Frequent calibrations minimize this dilemma.

12.3 MAINTENANCE

Regular maintenance required during service life is usually performed by personnel responsible for data collection. They should always be on the lookout for damage, potential for damage, and deterioration or malfunction and should initiate repair or replacement without delay. Maintenance requirements are usually given in the manufacturer's instruction manual.

12.3.1 Readout Units

Readout units should be kept clean and dry, following the manufacturer's instructions.

12.3.2 Field Terminals

Regular inspections should be made of field terminals to be sure that they are clean, dry, and functioning, that protective plugs, caps, and covers are in place, and that instrument numbers are clearly visible. Terminal accessibility should be verified and the integrity of enclosures and barricades checked. The inspections should include locations where embedded components exit from the ground: these locations are often susceptible to damage.

12.3.3 Embedded Components

Various embedded and normally inaccessible components require maintenance. For example, probe extensometer access pipes and inclinometer casings may require occasional flushing to remove accumulated debris. When embedded components include retrievable parts, these can be removed for maintenance and reinstalled. Examples include transducers for in-place inclinometers, and electrical transducers suspended in standpipe piezometers..

12.4 INSTALLATION

Peck (1988, 1993) states: "Equipment that has an excellent record of performance can be rendered unreliable if a single essential but apparently minor requirement is overlooked during the installation. The best of instruction manuals cannot provide for every field condition that may affect the results. Therefore, even slavish attention to instructions cannot guarantee success. The installer must have a background in the fundamentals of geotechnics as well as knowledge of the intricacies of the device being installed. Sometimes the installer must consciously depart from the installation manual."

As indicated in Section 6.18, planning for installation of instruments should include preparation of detailed written installation procedures. If instruments are to be installed by the state, these procedures will be used directly. If they are to be installed by the construction contractor, an abbreviated version will be included in the specifications, and the procedures will be used when reviewing the contractor's submittal of proposed installation methods.

The various factors to be considered when preparing detailed installation proeedures are described in the following sections. Detailed installation procedures should be in step-by-step form and should include a complete "shopping list" of required materials and equipment, including spare parts for replacing components likely to be damaged during installation.

12.4.1 Pre-Installation Acceptance Tests

As discussed in Section 12.2.2, pre-installation acceptance tests should always be made before installing instruments.

12.4.2 Installation at the Ground Surface

Surface-installed instruments should usually be protected with robust cover plates, welded, bolted, or otherwise attached to the surface. Exposed tubes and cables are extremely vulnerable to damage and should usually be protected by conduit. Instrument terminals must be protected from damage caused by construction activities, vandalism, and the environment. Strong metallic cover plates, boxes, or pipes, with locks if necessary, may be required as protection from construction activities and vandals.

12.4.3 Installation in Boreholes: Conformance

Ideally, the presence of a measuring instrument should not alter the value of the parameter being measured. If in fact the instrument alters the value, it is said to have poor *conformance*. When either changes in the length of the borehole or local shear displacements are anticipated during instrument service life, downhole components must be selected so that conformance is maintained and so that components are not damaged as deformation occurs. For example, when inclinometer casings or probe extensometers are installed vertically in a soft clay foundation for monitoring deformations beneath an embankment, telescoping couplings will normally be required in the casing or access pipe.

12.4.4 Installation in Boreholes: Drilling Methods

Various methods are used to drill boreholes for instruments. The drilling method selected depends on the soil or rock type to be drilled, type of instrument to be installed, availability of drilling equipment and trained personnel, requirements for borehole diameter, depth, inclination, straightness, and wall roughness, access for drilling equipment, and the need for sampling. Various drilling methods, together with their advantages, limitations and applicability for borehole installations, are given by Dunnicliff (1988, 1993). They include wash boring, auger drilling with hollow-stem augers, rotary drilling, core drilling and rotary percussion drilling.

If grout is to be used while installing downhole components, a test should be made to ensure that grout will not leak from the borehole. An estimate of grout-tightness can sometimes be made from knowledge of stratigraphy and previous boring experience at the site. A more positive test is to perform a falling head permeability test, with water starting 3 m above the ground surface. The author has used a flow acceptability criterion of not more than 0.5 liters of water per minute per 3 m of borehole, for a duration of 5 minutes. If the flow exceeds this amount, the borehole should be pre-grouted, the grout allowed to take an initial set, the borehole redrilled and the test repeated. If the upper length of the borehole is cased, for example when installing an inclinometer casing through soil with a base fixity zone in bedrock, a packer should be set immediately below the drill casing prior to performing the test.

12.4.5 Installation in Boreholes: Installation of Downhole Components

The method of installing downhole components depends on the type of instrument and drilling method, but several general guidelines can be given.

Prior to installing any instrument through drill casing or augers, all material adhering to the inside of the casing or augers, and all cuttings, should be removed thoroughly.

Whenever withdrawing drill casing during instrument installation in a borehole, care should be taken to minimize the length of unsupported borehole and the rate of casing withdrawal. Collapse of the borehole must not be allowed to occur, and backfill material must not be allowed to build up inside the casing such that the instrument is lifted as the casing is withdrawn. The casing must be withdrawn without rotation. The casing should not be omitted unless it can be shown that instrument installation without the casing will not cause collapse of the borehole or in any way adversely affect instrument installation. If casing is omitted, the instrument should be installed in the borehole in a continuous operation, starting when instrumentation materials are first placed in the borehole, and not be interrupted before the borehole is completely backfilled to the ground surface. These same issues apply to installation through hollow-stem augers.

When installing in downward boreholes, if the instrument is heavy a 6 mm manila rope or 3 mm airplane cable should be attached securely to the bottom and used for lowering the instrument, thereby avoiding excessive tension in the components and allowing for recovery if problems arise. If the instrument is very light, it may be necessary to add weight before installation. Instruments such as inclinometer casings can often be installed in downward boreholes under neutral buoyancy by ensuring that the borehole is water-filled and filling the casing with water as it descends.

12.4.6 Installation in Boreholes: Backfilling Boreholes with Grout

If a grout backfill is to be used, an estimate of grout density should be made, the volume and weight of downhole components determined, and any weight necessary for overcoming buoyancy should be added to the **bottom** of the instrument before installation. Typical grout density ranges from 1.1-1.6 Mg/m^3.

Any required flexible tubes or rigid pipes for backfilling the borehole with grout or granular material should be inserted in the borehole as installation of downhole components proceeds. Typically, 25 mm diameter plastic tubes or pipes are required. If rigid pipe is used, joints should be flush on both the inside and outside so that the pipe does not "hang up" on other downhole components. Schedule 80 rigid PVC pipe, with square threads or threads of the type shown in Figure 2-3, is usually the best choice.

When selecting a mix for grout backfill, the first task is to define the required engineering properties. As a goal, the grout should ensure conformance between the instrument and the surrounding soil or rock and should not alter the value of the parameter being measured. For example, grout in boreholes for piezometers should satisfy permeability criteria and, if compression is anticipated along the axis of the borehole, also compressibility criteria (see Section 2.8.4). When probe extensometers relay on grout to ensure conformance, the grout should satisfy criteria for compressibility and shear strength. Grout for fixed borehole extensometers in soft ground should not have significant compressive or tensile strength. Grout for inclinometer casing should satisfy criteria for maximum and minimum strength. For these reasons there is no universally suitable grout, and each installation must be considered individually.

Dunnicliff (1988, 1993) lists some grout mixes that have been used for borehole installations. However, they should not be used as a "cook-book." Trial mixes should be made for each application and judgment often made by visual observation and supplemented with simple tests such as pressing with a thumb or use of a Torvane. The properties of grout are often dependent on the sequence of adding ingredients, and the sequence should be standardized. As a general rule, liquids should be mixed-first, followed by the finest through the coarsest materials. When using cement/bentonite grout, bentonite should be added to the water first, because if bentonite is added to a cement and water mix an ion exchange takes place and the expansion of the bentonite is reduced significantly. Whenever quoting proportions of grout mixes, it is essential to indicate both the mixing sequence and whether proportions are by weight or by volume: there is significant confusion in the technical literature because this is not always done. Properties of grout are often also dependent on the chemical constituents of the mixing water, and water for trial mixes should be from the same source as the field mix.

Even if the "perfect" grout mix can be determined, it probably will not set as a uniform column throughout the borehole. In the view of the author, the various uncertainties relating to in-place properties of grouts should color our reliance on grout where properties and uniformity are critical. For example, when a probe extensometer is to be installed to monitor substantial vertical compression, reliance on grout may be unwarranted, and conformance should be ensured by using a positive anchorage at each measuring point.

12.4.7 Installation in Boreholes: Backfilling Boreholes with Sand or Pea Gravel

Rounded grains are less likely to bridge than angular grains. Two backfilling methods are possible. First, sand or pea gravel can be poured down the annular space. The material should be saturated with water before pouring, should be poured slowly to avoid bridging in the borehole, and should be tamped thoroughly in 600 mm maximum layers. Second, sand or pea gravel can be tremied to the bottom of the borehole. A 25 mm Schedule 80 PVC pipe with flush threaded couplings is inserted within the borehole as downhole components are installed, and a 45 degree Y-branch is fitted to the pipe near its upper end. A water supply is connected to the branch, and water is circulated down the pipe until it spills out of the top of the borehole. Sand or pea gravel is poured slowly into the top of the pipe and washed to the bottom of the borehole, and the pipe is gradually raised as backfilling progresses.

The second method generally results in more complete backfilling than the first.

12.4.8 Installation in Boreholes: Backfilling Boreholes with Granular Bentonite

Section 2.8.3 includes recommendations for selection of sealing material above piezometers, and indicates that pit-run granular bentonite is the preferred material. Section 2.8.5 provides information on use of a sounding hammer when backfilling boreholes with granular bentonite, and indicates that the maximum number of piezometers in a single borehole should be two, with the leads separated by at least 20 mm, and at least 6 m of granular bentonite seal between the two piezometers. If this is not possible, each piezometer should be installed in an individual borehole.

12.4.9 Installation in Fill

When deformations are anticipated during instrument service life, instrument components must be selected so that conformance is maintained and so that they are not damaged as deformation occurs. For example, probe extensometers and inclinometer casings normally require either axially compressible access pipe or telescoping couplings in rigid access pipe.

When pipes, tubes, and cables are installed in fill containing large-sized particles, finer material may be required around them.

Pipes, tubes, and cables should be checked for integrity and correct functioning before installation and again before backfilling. If they are installed in groups, they should be marked individually at approximately 6 m intervals so that they can be identified if they are damaged and repairs are attempted.

Armoring or conduit is often the preferred method for protecting tubes and cables from damage. Use of conduit alleviates stringent criteria on maximum particle size of backfill and greatly reduces the likelihood of damage when the fill deforms. If no conduit is provided, the tubes or cables must accommodate all strains experienced by the fill, but if they are intentionally left slack within conduit, a greater length is available for accommodating local deformation. The benefit of this arrangement for accommodating tensile strains is self-evident, but less so for compressive strains. However, when electrical cables are installed without conduit and subjected to compressive strains, conductors can break, protrude through the cable insulation, and create a short circuit. Rigid PVC pipe is most frequently used as conduit when installing horizontal leads in fill, but corrugated flexible polyethylene pipe is an alternative in cases where it is adequately robust.

Many specifications require that instrument leads are installed in a meander from one side of the trench to the other to create slack, but this arrangement appears to be of marginal value. It cannot provide slack to accommodate tensile strain along the leads, because any such benefit assumes that leads can move laterally with respect to the fill surrounding them. Perhaps some benefit is gained when deformation occurs transverse to the leads, particularly during compaction, but a better solution appears to be selection of leads that can accommodate deformation and use of conduit where possible.

Special arrangements must be made for tubes and cables where they emerge from the ends of conduit or from boreholes. If there is any possibility of shear deformation or kinking at the ends of conduit, cushioning must be provided between the leads and conduit at the exit points. Rubber hose, such as automobile radiator hose or thick-walled garden hose, can be used, or the space can be packed with burlap or soft rags. Where leads emerge from vertical boreholes to pass horizontally within fill or at the bottom of fill, there is generally a concern for shearing at the top of the boreholes. If conduit is not used, the leads should be cushioned with screened sand and routed along a curve as shown in Figure 12-1. If leads emerge from a vertical borehole and enter pipe conduit, the conduit should be anchored to the borehole by attaching a long-radius elbow and short length of pipe and inserting the short length down the borehole, as shown in Figure 12-1.

12.4.10 Protection From Damage

When embedded components that terminate at the ground surface are subject to damage by construction activities, special precautions must be taken. For example, inclinometer casings and probe extensometers will often require protective barricades, marked clearly to warn operators of construction equipment. When vandalism is the overriding issue, terminals should if possible be buried and made unobtrusive, since a strong protective box often encourages a vandal to look for a stronger vandal. Burying of course necessitates a foolproof method of locating the terminal. All vertical pipes should be provided with a cap to prevent intrusion of debris.

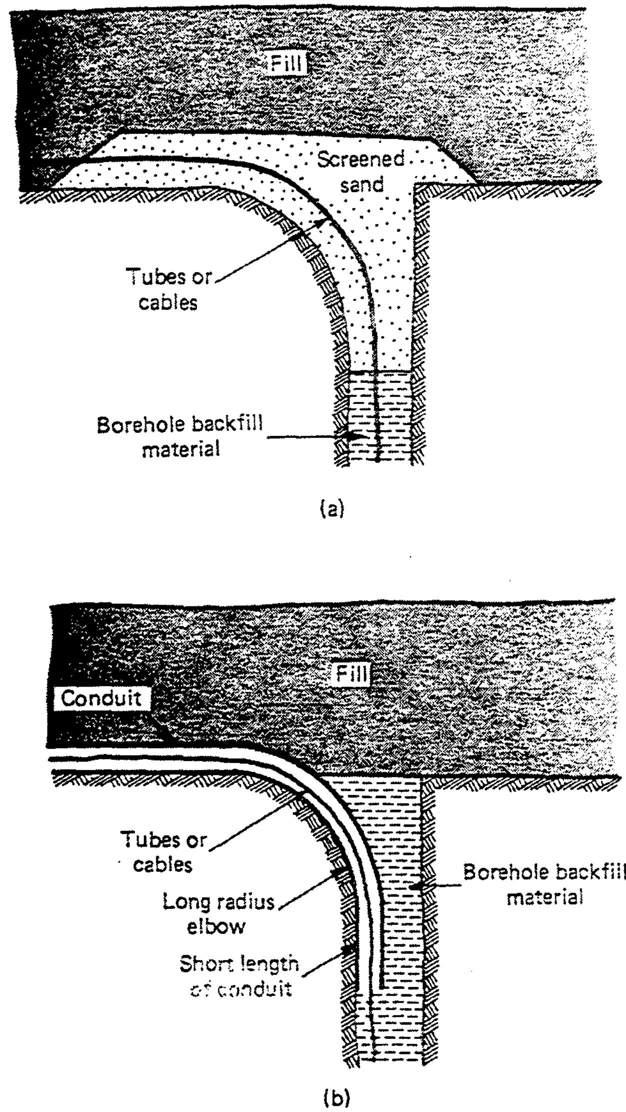


Figure 12-1 Typical Arrangements for Protecting Tubes and Cables Where They Emerge from a Borehole: (a) Without Conduit and (b) with Conduit. (after Dunnicliff, 1988, 1993)

12.4.11 Post-Installation Acceptance Tests

Post-installation acceptance tests should always be performed to ensure, to the extent practicable, that installations have been completed satisfactorily. For example, groove tracking tests and spiral survey measurements can be made in inclinometer casings, and verification of correct spider magnet locations can be made in probe extensometers with magnet/reed switch transducers. Repeatability of readings can also provide acceptability criteria; a minimum of three sets of instrument readings can be made, repeatability examined, and an evaluation made of whether data are within expected tolerances.

12.4.12 Installation Records

As discussed in Section 6.12, "installation record sheets" should be prepared during the planning phase. These records serve two purposes. First, when installation personnel are required to enter data in blank spaces on a field form, they are more likely to follow the installation procedure with care. Second, "as-built" data are required both for record purposes and for use during evaluation of data. A major task during evaluation of data is development of a relationship between measurements and causes, and details of installation may be one of those causes. For example, if installation personnel are unable to set a borehole extensometer anchor correctly, this fact may explain later measurements. If more or less backfill than normal is used in a borehole installation, this fact may be evidence that is helpful when evaluating data. If an instrument is installed in adverse weather conditions, less reliance on data may be justified.

Installation record sheets should include a record of appropriate items from Table 12-2.

TABLE 12-2
POSSIBLE CONTENT OF INSTALLATION RECORD SHEETS

Category	Content
Heading	Project name Instrument type and number, including readout unit Personnel responsible for installation Date and time of start and completion
Planned data	Planned location in plan and elevation Planned orientation Planned lengths, widths, diameters, depths, and volumes of backfill Spaces for necessary measurements or readings required during installation to ensure that all previous steps have been followed correctly, including post-installation acceptance tests
As-built data	As-built location in plan and elevation As-built orientation As-built lengths, widths, diameters, depths, and volumes of backfill Plant and equipment used, including diameter and depth of any drill casing used A log of appropriate subsurface data Type of backfill used Post-installation acceptance test
Weather	Weather conditions
Notes	A space for notes, including problems encountered, delays, unusual features of the installation, and any events that may have a bearing on instrument behavior

12.4.13 Installation Schedule

A deadline should be determined before which each instrument must be installed. The deadlines will be based on the need to establish preconstruction conditions, construction activity, availability of installation personnel, instrument delivery dates, and other relevant factors.

Instruments should be installed as early as possible to provide vital initial readings for establishing reliable base conditions. Unfortunately, this requirement is often not fulfilled, and important baseline data are not obtained. Early installation will also provide information on faulty instruments so that they can be repaired or replaced before construction begins.

12.4.14 Coordination of Installation Plans

Detailed planning for installation should be coordinated among all parties involved, including the instrument supplier, state personnel, and the construction contractor.

When state personnel are responsible for installation, a cooperative working relationship with the construction contractor is essential, and the state's field personnel should make a special effort to establish a cooperative relationship.

12.4.15 Installation Report

On completion of installation, an installation report will usually be required to provide a convenient summary of information needed by personnel responsible for data collection, processing, presentation and interpretation. The installation report should contain at least the following information:

- Plans and sections sufficient to show instrument numbers and locations.
- Appropriate surface and subsurface stratigraphic and geotechnical data.
- Descriptions of instruments and readout units, including manufacturer's literature and photographs.
- Details of calibration procedures.
- Details of installation procedures (photographs are often helpful).
- Initial readings.
- A copy of each installation record sheet.

12.5 COLLECTION OF INSTRUMENTATION DATA

Responsibility for collection of instrumentation data will have been determined during the planning phase and should preferably be under the direct control of the state or instrumentation specialists selected by the state. The construction contractor may be responsible for support work, such as provision of access to instrumentation locations, and will sometimes be responsible for optical survey work associated with data collection.

12.5.1 Role of Automatic Data Acquisition Systems

Until a few years ago, almost all data were recorded by hand: the *long way*. The advent of automatic data acquisition systems (ADASs) has changed the state-of-the-practice, for better and for worse, and users should be aware of the limitations as well as the advantages of these systems. These remarks should not be taken as a vote against the use of ADASs, but as a plea for an honest appraisal of their suitability before they are selected.

ADASs can be separated into two basic categories: dedicated systems and flexible systems. A dedicated system, often referred to as a datalogger, is a device designed for connection to one or two types of transducer. Dataloggers manufactured by geotechnical instrumentation manufacturers are generally dedicated systems, designed for extended field use with that manufacturer's transducers. Flexible systems are applicable for use with a large variety of transducer types and a large number of transducers and may be capable of handling digital data. Flexible systems generally have more sophisticated data handling, storage, and processing capability than dedicated systems.

Descriptions of ADASs, and suggestions for maximizing success when they are used, are given by Davidson et al. (1991) and by Dunnicliff and Davidson (1991). Problems that have been experienced with flexible systems include:

- Manufacturers have experienced a "learning curve," resulting in unexpected hardware and software problems.

- Users have generally been civil engineers, with limited ability to cope with electronic aspects of the new technology. Some users have developed a full capability with all aspects of ADASs, and are able to plan, install and provide associated field services for the entire system. However, many (typically most state highway departments and most geotechnical consulting firms) do not have this capability.

- Lightning strikes and power surges have destroyed several systems.

- No automatic system can replace human engineering judgment. When ADASs are used, there is a real possibility that people will not be assigned to go to the field, to make visual observations and record other causal data.

- Users have over-relied on the ADAS, assuming all is well, when in fact it has sometimes been malfunctioning.

Some of these same problems apply also to dedicated systems.

Recent experience indicates that the following are likely to be worthwhile:

- Special care must be taken to make visual observations and record all causal data.

- Automatically recorded data should be evaluated on a frequent schedule, to examine whether recorded data are correct.

- The ADAS should be housed within a secure structure to prevent vandalism.

- Each instrument or transducer should maintain the ability to be read prior to entering the automated network. Connections should be arranged for convenient connection to the front of the ADAS panel without disturbing automatic readings, so that values entering the ADAS can be independently checked for quality assurance purposes.

- Proven performance in a similar application is strongly recommended. Off-the-shelf standard components are being introduced by manufacturers, and this has improved reliability and reduced reliance on risky customized systems.

- To the extent possible, the user should assign responsibilities to people with experience and understanding of ADASs. Manufacturers tell of a full range of customers, from sophisticated users to first-timers who are baffled by anything electronic.

- The system must have transient voltage suppression to protect against effects of lightning strikes and power surges. Hall (1995) provides guidelines on transient protection.

- Remote communication to the ADAS is a very great asset. This is most easily and cheaply done by telephone modem (cellular modem is also available) or hard-wire if the site office is close by. With this capability the ADAS is dialed up from the comfort of an office using a PC. System status and recorded data are quickly and easily accessed, enabling rapid quality assurance and allowing more time for data review and thinking. However, use of remote communications emphasizes the above for special care to be taken to make visual observations and record all causal data.

- The user should discuss, with the manufacturer, the need for both a primary and backup power source, to ensure uninterrupted data collection. Also the need for internal voltage regulators to ensure that instruments give stable readings over a wide range of input voltages from external power supplies.

- When using an ADAS to record data during a short-term test, such as a pile load test, double-check all circuits for correct functioning before starting the test. Before the start of the test, also connect a manual backup (portable readout unit with jumper cable and switch box) to the ADAS panel, and verify that data are consistent with automatically recorded data. Spot check the automatic data regularly during the test, by reading with the portable readout unit.

- Flexible systems should be designed utilizing an integrated "systems" approach. Mixing and matching components generally does not provide satisfactory results.

- For flexible systems, arrangements should be made with the ADAS manufacturer for technical support after shipment, relating as a minimum to questions of wiring, noise and performance characteristics.

12.5.2 Field Data Records for Data Collected Manually

When data are collected manually, field data records are required. Readings can be recorded either in a field book or on field data sheets.

Field books contain previous readings and therefore facilitate immediate comparisons. They are easier to handle and to keep dry than field data sheets, but loss of a field book can be a serious issue. The author prefers the use of field data sheets, specially prepared for each project and instrument.

Field data sheets should include project name and instrument type, and spaces are required for date, time, observer, readout unit number, instrument number, readings, remarks, data correctness checks, visual observations, and other causal data including weather, temperature, and construction activities. Data collection personnel are less likely to write down all these important factors when entering data on a blank page in a field book. The need for comparing readings immediately with the previous set of readings is handled by taking a copy of previous readings into the field or by transcribing key data to a column on the new data sheets. Special paper, available from suppliers of weatherproof field books, can be used to allow writing in wet conditions. One or more field data sheets will be used for each date, with later transcription of data to one calculation sheet for each instrument. Raw data should be copied and the copy and original stored in separate safe places to guard against loss. An example of a field data sheet is given in Figure 12-2. Other typical field data sheets are given by Dunnicliff (1988, 1993).

12.5.3 Initial Readings

Most instrumentation data are referenced to initial data, and engineering judgments are usually based on changes rather than on absolute values. Correct initial readings are therefore vital.

Typically, a minimum of two readings will be taken immediately after installation, as part of the post-installation acceptance test (Section 12.4.11). Daily readings should then be taken until data are stable, at which time the formal initial readings should be made. Initial values should be based on a minimum of two readings, and repeatability between these readings should satisfy the expected tolerance. If the tolerance is exceeded, the transducer or readout unit may be faulty, installation may be faulty, real changes may be occurring, or stabilization may not be complete. The cause must be evaluated and the problem remedied. After the formal initial readings have been taken, daily readings should be continued for a few days to ensure that data are indeed stable.

CURIOUS GEORGE PROJECT
FIELD DATA SHEET FOR PROBE EXTENSOMETER

Extensometer No.__________________ Date______________________________
Readout Unit No.__________________ Start Time__________________________
Survey Tape No.__________________ Finish Time_________________________
Observer____________________________

Meas. Point No.	Tape Reading (m)				Remarks
	Individual			Best	

Additional Remarks (Data correctness checks, visual observations, causal data, weather, construction activities. Continue on back of sheet if necessary.)

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Figure 12-2: Typical Field Data Sheet for a Probe Extensometer, with Deepest Measuring Point in Bedrock.

12.5.4 Data Collection Frequency

The frequency of data collection should be related to construction activity, to the rate at which the readings are changing, and to the requirements of data interpretation. Too many readings overload the processing and interpretation capacity, whereas too few may cause important events to be missed and prevent timely actions from being taken. Good judgment in selecting an appropriate frequency is vital if these extremes are to be avoided.

When construction commences and approaches the instrument location, readings should be taken frequently; for example, once a week, once a day, once a shift, or even more frequently in relation to construction activity (such as before and after each blast, during pile driving, or during the placement or removal of a preload). It is often wise to increase the frequency of readings during heavy precipitation. As construction activity moves away from the instrument location or ceases altogether, and when readings have stabilized and remained constant, the frequency may then be decreased.

12.6 PROCESSING AND PRESENTATION OF INSTRUMENTATION DATA

The first aim of data processing and presentation is to provide a rapid assessment of data in order to detect changes that require immediate action. The second aim is to summarize and present the data in order to show trends and to compare observed with predicted behavior so that any necessary action can be initiated.

Responsibility for processing and presentation of instrumentation data will have been determined during the planning phase and should preferably be under the direct control of the state or instrumentation specialists selected by the state.

12.6.1 Electronic Data Processing and Presentation

The advent of electronic data processing and plotting programs has made an enormous increase in the efficiency of data processing and presentation. Computers and datalogging devices (hardware) and electronic programs (software) are used by technical personnel who collect, process, plot, and analyze instrumentation data. The use of electronic hardware and software reduces the time between data collection and analysis, allowing construction decisions to be made much faster.

Some of the available software programs for processing and presentation of data include: Lotus 1-2-3, dBase, Oracle, Gtilt, DigiPro, Excel, Easyplot, Freelance, various seismograph programs, computer aided drafting and design (CADD), and graphic data system (GDS).

Advantages of electronic data processing include (Bobrow, 1997):

- Rapid collection, input, reduction, and output (tables and plots) of large amounts of data
- Creates a variety of plots to analyze data; can modify plot format rapidly and repeatedly
- Easily links one software program to another, thus enabling more in-depth data analysis
- Can view reduced data on the computer screen, thus reduce the amount of paper used
- Quickly provide copies of the reduced data to outside agencies using electronic transfer via modems
- Can store large amounts of data in a small space
- Multiple methods can be used to compare reduced data
- Users unfamiliar with instrument locations can access a graphical interface (e.g., GDS, CADD) to view a location plan
- Graphical interfaces integrated with data storage systems allow viewing of the reduced data on an instrument location plan

- A variety of instrument reports (e.g., status, location, strata at installation location) can be generated easily
- Instrumentation data can be readily integrated with other project information
- Lower cost compared with manual methods

Precautions when using electronic data processing included (Bobrow, 1997):

- Time must be spent during the setting up phase to learn the use of hardware and software
- Reduced data needs to be 'spot' checked to ensure correct functioning of software prior to releasing the reduced data for plotting and analysis
- Need for a person who can troubleshoot problems that may occur with the computer or other electronic hardware and software
- For some software programs, need for a person who is familiar with program writing
- Because of the increased dataprocessing power, there is a temptation to collect more data than necessary

12.6.2 Plots of Data

After calculations have been made, plots of data should always be prepared. Various types of plots are described below.

Routine Plots of Data Versus Time

These plots are updated immediately after converting data from raw readings to engineering units. They will be used by data collection and processing personnel to assist in an assessment of data quality, to show data trends, and as base material for data interpretation.

Plots to Assist with Predictions

Frequently, the routine plots of data versus time are adequate for predicting future trends. For example, when monitoring performance of an embankment on soft ground, plots of pore water pressure versus time may be adequate for predicting the length of time required for an adequate degree of consolidation to occur. On other occasions, for example, when monitoring deformation in rock, trends may not stand out clearly on plots of deformation versus time, and it is often helpful to plot velocity or even acceleration versus time. In many cases the velocity of deformation is much more important than the absolute magnitude, because an increasing velocity (acceleration) will generally indicate potential hazard.

Plots for Comparing Observed and Predicted Behavior

When comparison between observed and predicted behavior is part of data interpretation, the two sets of data should generally be plotted against time and on the same axes.

Plots for Comparison of Measurements and Observations

Comparisons between two sets of different data often form part of data interpretation. For example, when monitoring both settlement and pore water pressure in soft ground beneath an embankment, there should be consistency between the two sets of data, and data can be evaluated by plotting both sets on the same axes. When both a load cell and hydraulic jack are used to measure load during a pile load test or tieback proof test, one can be plotted against the other, or the ratio of the two can be plotted against the load as determined by the load cell.

Plots to Examine Cause and Effect Relationships

Data interpretation nearly always includes a study of cause and effect relationships. Causal information includes construction details and progress, geology and other subsurface conditions, and environmental factors such as temperature, precipitation, sun, and shade. Major causal information and visual observations should be added to the routine plots of data versus time, either by noting the occurrence of relevant events or by plotting causal data to the same time scale. Figure 12-3 shows an example of such a plot.

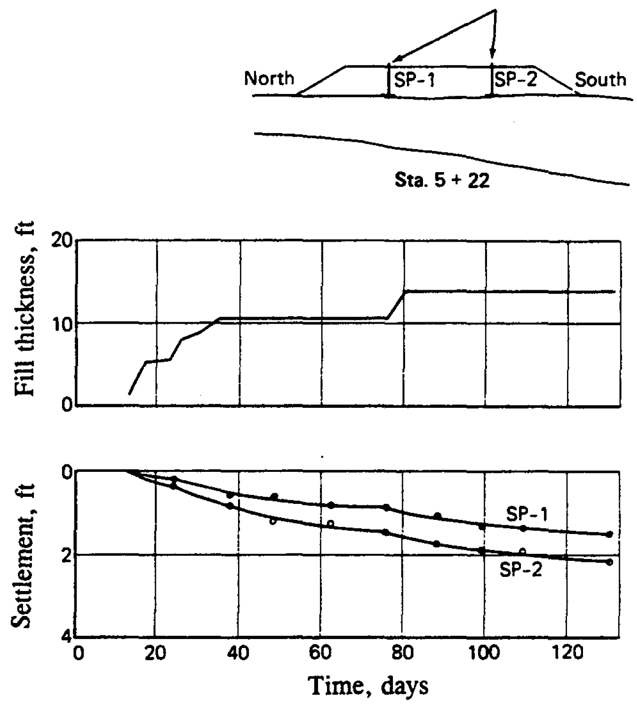

Figure 12-3: Plot to Examine Cause and Effect Relationships: Settlement of Soft Ground Below an Embankment. (After Dunnicliff, 1988, 1993)

Summary Plots

Many of the plots described above may show too much detail for submittal to senior personnel or for use in a report, and often simplified summary plots are required for presentation of data to personnel who may not have the time or technical background to understand or digest all the measurements. Summary plots should not contain all the instruments or all the data. In most cases, they should consist only of a few selected instruments that show the significant trends most clearly and enough data points to show the trends and any significant fluctuations. The plots should be arranged to show which factors are influencing the data trends. In some cases, both the predicted behavior and the hazard warning levels can be included on the plots.

12.7 INTERPRETATION OF INSTRUMENTATION DATA

Monitoring programs have failed because the data generated were never used. If there is a clear sense of purpose for a monitoring program, the method of data interpretation will be guided by that sense of purpose. Without a purpose there can be no interpretation.

A first data interpretation step has been described in Section 12.6: a rapid assessment of data in order to detect changes that require immediate action. The essence of subsequent data interpretation steps is to correlate the instrument readings with other factors (cause and effect relationships) and to study the deviation of the readings from the predicted behavior. By its very nature, interpretation of data is a people-intensive activity, and no technique has yet been developed for automatic interpretation of data.

12.7.1 Schedule

Interpretation should not be delayed until a large quantity of data has been collected and processed, because the tasks of data collection, processing, and interpretation should influence each other. The process should be iterative, allowing the needs of any one task to alter any other.

12.7.2 Interpretation and Reinterpretation

Interpretation and reinterpretation is an ongoing process. When early data are available, preliminary interpretations will usually be made. However, these interpretations will usually be revised or refined as more data become available, and as a clearer understanding of real behavior is developed.

12.7.3 Questionable Data

When faced with data that on first sight do not appear to be reasonable, there is a temptation to reject the data as false. However, such data may be real and may in fact carry an important message. A significant question to ask is: *Can I think of a hypothesis that is consistent with the data?* The resultant discussion, together with the use of appropriate procedures for ensuring reading correctness, will often lead to an assessment of data validity.

Details recorded on installation record sheets (Section 12.4.11) are often helpful when evaluating questionable data, because difficulties encountered during installation may cause abnormal data.

12.7.4 Communication

An open communication channel should be maintained between design and construction personnel, so that discussions can be held between design engineers who raised the questions that caused instrumentation to be used and field engineers who provide the data.

12.8 REPORTING OF CONCLUSIONS

After each set of data has been interpreted, conclusions must be reported in the form of an interim monitoring report and submitted to personnel responsible for implementation of data. In addition, a final report of the monitoring program is often required.

12.8.1 Interim Monitoring Reports

The conclusions from data interpretation should be communicated to all parties who have a role in data implementation. The initial communication may be verbal but should be confirmed in an interim monitoring report. Reporting will be on a regular schedule to allow timely implementation and will include the following:

- Updated summary plots.

- A brief commentary, drawing attention to all significant changes that have occurred in the measured parameters since the previous interim monitoring report, together with probable causes.
- Recommended action.

12.8.2 Final Report of Monitoring Program

A formal report is often prepared to document key aspects of the monitoring program and to support any remedial actions. The report also forms a valuable bank of experience and should be distributed to all personnel who have been involved in the design and execution of the monitoring program, so that any lessons learned may be incorporated into subsequent designs. Contents of the final report should be selected from the following list:

- Summary of the report.
- Introduction, including a brief description of the project and the reason for using geotechnical instrumentation.
- A summary of the instrumentation system design report (Section 6.16). Alternatively the entire report can be included as an attachment.
- Any project design and construction information that is relevant to the monitoring program
- Description of instruments and readout units.*
- Plans and sections sufficient to show instrument numbers and locations.
- Appropriate surface and subsurface stratigraphic and geotechnical data.*
- Observed behavior, including summary plots and factors that influence measured data.
- Analysis of observed behavior, including comparisons between measurements and predictions, a discussion of significant changes and probable causes, and comparisons with published information.
- Conclusions, discussion, and recommendations, including a statement of any remedial actions taken.

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* For a major monitoring program, brief material can be included in the body of the report and details in appendixes.

12.9 STUDENT EXERCISE ON EVALUATION OF DATA

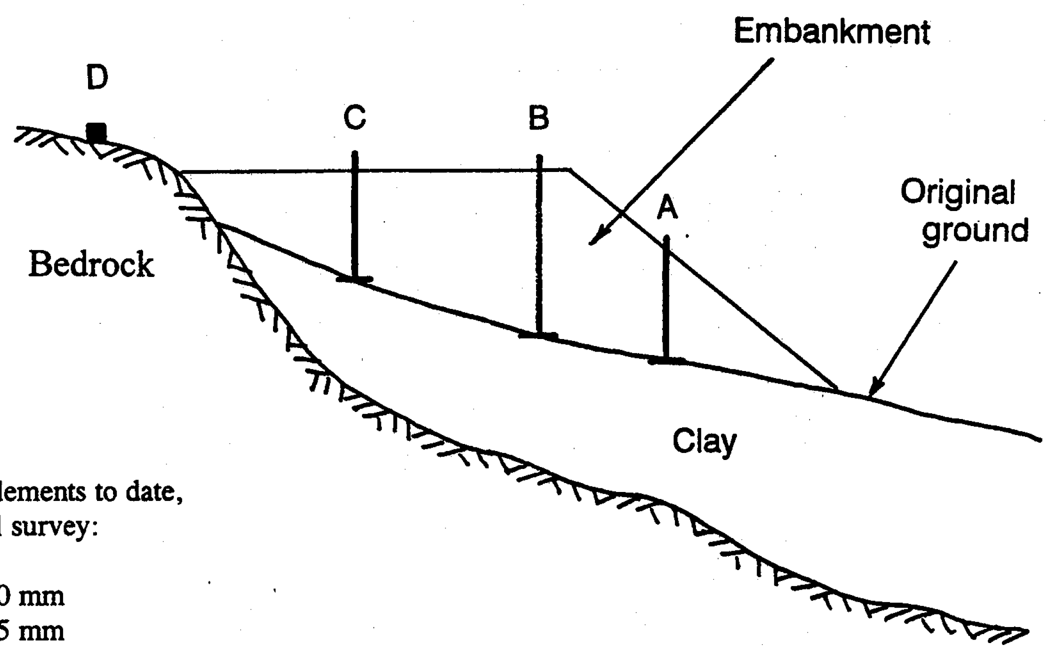
12.9.1 Introduction

Various measurement data will be presented, and questions asked. Students will be asked to think about the data for a few minutes, and to answer the questions orally. Any necessary discussion will follow.

Suggestions for data evaluations are included in Appendix F.

12.9.2 Exercise No. 1. Embankment on Soft Ground, With Settlement Platforms

See Figure 12-4, showing three settlement platforms (Figure 3-13) and one surface survey marker on bedrock.



Total settlements to date,
by optical survey:

A 50 mm
B 75 mm
C 50 mm
D -25 mm (heave)

Figure 12-4: Embankment on Soft ground, With Settlement Platforms

- Does the data look reasonable?
- If not, what is unreasonable about the data?
- Can you think of a hypothesis that is consistent with these data?

12.9.3 Exercise No. 2. Inclinometer on Slope

See Figure 12-5, showing an inclinometer casing (Figures 3-19 and 3-20) installed in a soil slope.

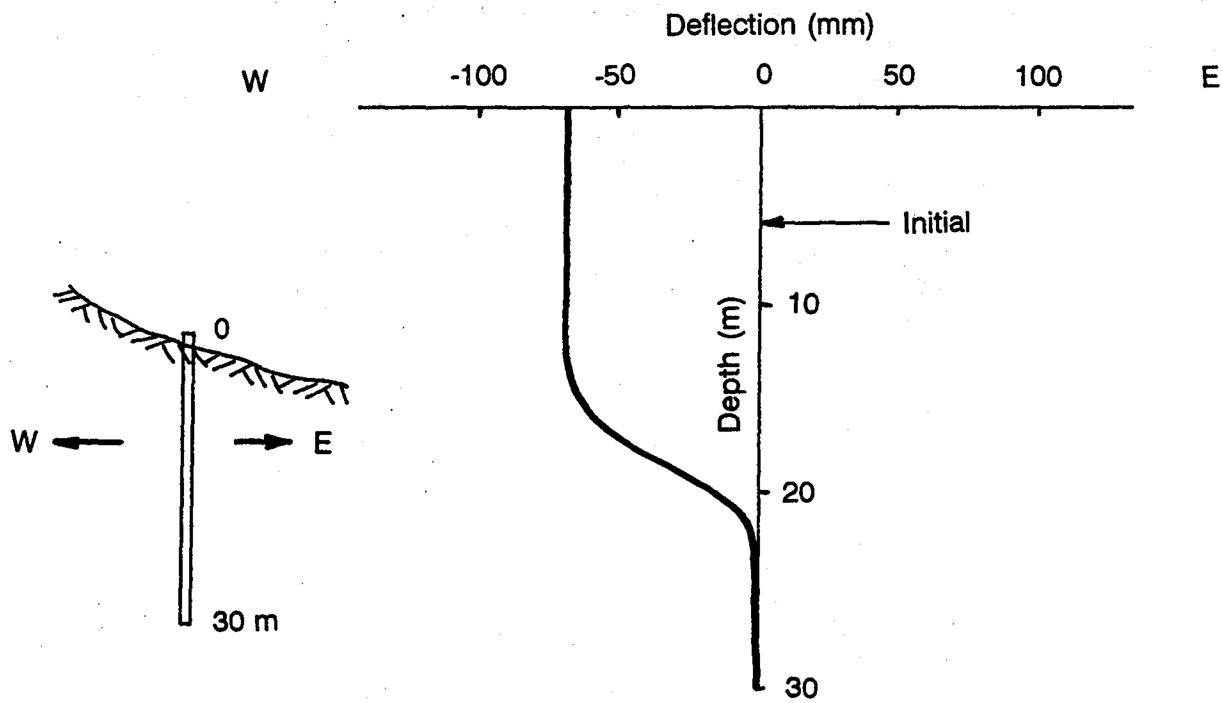


Figure 12-5: Inclinometer on Slope

- Does the data look reasonable?
- If not, what can be done about the data?

12.9.4 Exercise No. 3. Inclinometer Data

See Figure 12-6, showing inclinometer (Figures 3-19 and 3-20) data.

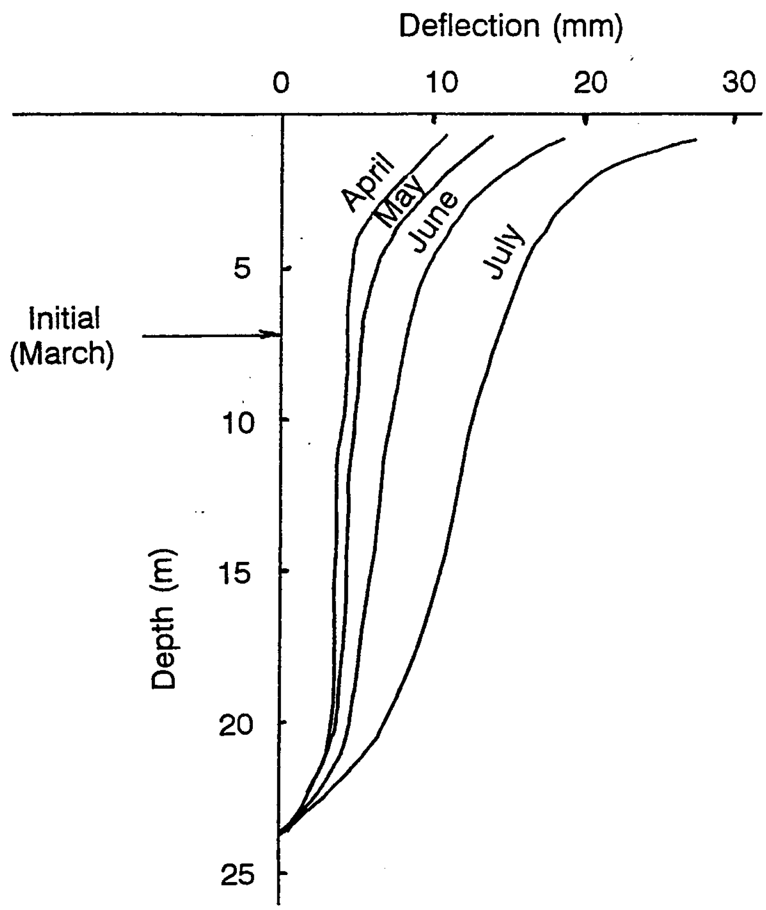


Figure 12-6: Inclinometer Data

- What does this plot tell you?
- What should you do about it?

12.9.5 Exercise No. 4. Probe Extensometer Data

See Figure 12-7, showing probe extensometer (Figures 3-9 and 3-11) data.

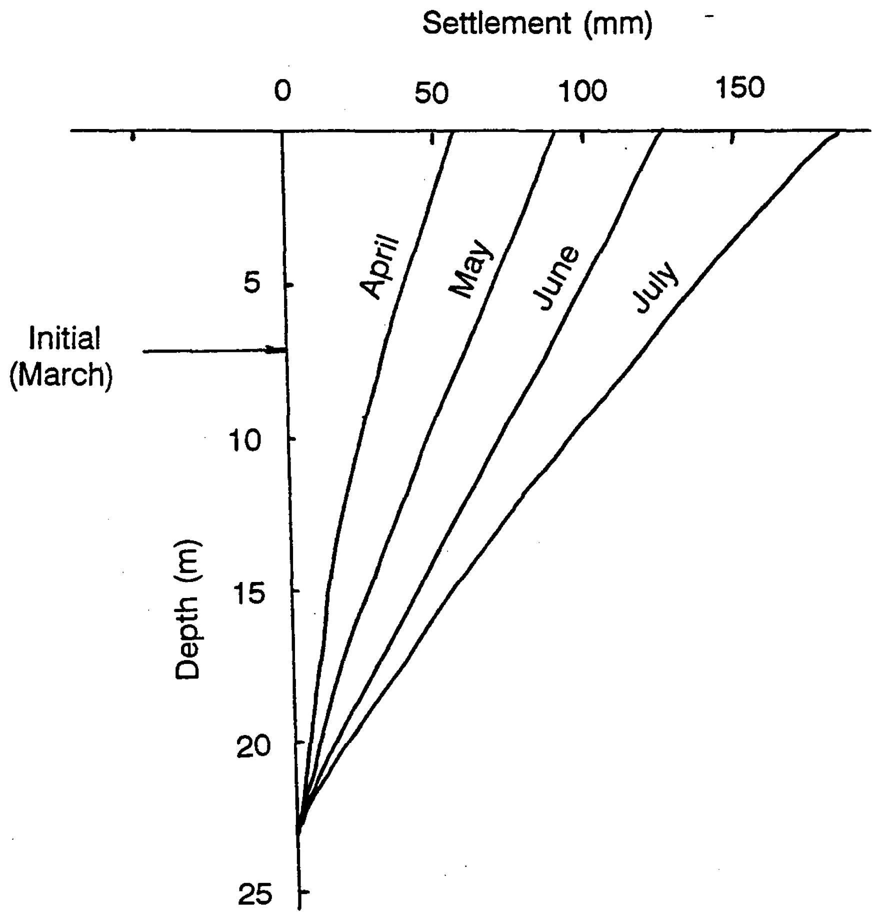


Figure 12-7: Probe Extensometer Data

- What does this plot tell you?
- What should you do about it?

12.9.6 Exercise No. 5. Full Profile Settlement Gage

See Figure 12-8, showing settlement data from a full profile settlement gage beneath a highway embankment.

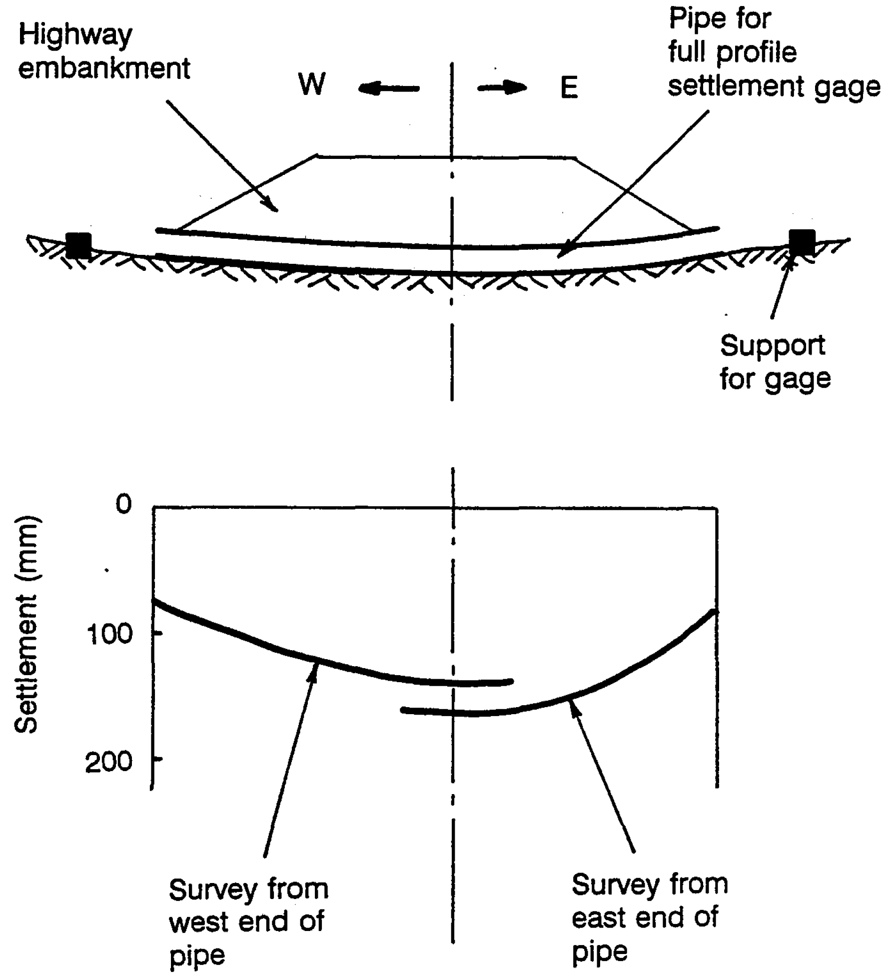


Figure 12-8: Full Profile Settlement Gage

- Does the data look reasonable?
- If not, what can be done about the data?

12.9.7 Exercise No. 6. Optical Survey Data

Surveyors have made elevation measurements on a surface monument located at the surface of a highway embankment on soft ground. The soft ground is known to be consolidating under the weight of the embankment. No fill has been placed during the period.

The surveyors have provided you with the data in the first two columns below, and you have calculated settlement as in the last column (the calculation is mathematically correct):

Day	Elevation (m)	Settlement (m)
0	100.00	0
1	99.89	0.11
2	99.83	0.17
4	**99.73**	**0.27**
10	99.74	0.26
14	99.68	0.32
27	**99.69**	**0.31**
60	99.67	0.33
100	99.65	0.35
300	99.64	0.36

For a settlement analysis you need to know the incremental settlement between day 4 and day 27 (don't question why), and you use data from the above table: 0.31 - 0.27 = 0.04 m. Can you see anything wrong with this procedure? Can you think of a better procedure?

12.9.8 Exercise No. 7. Embankment on Soft Ground, With Borros Anchors

See Figure 12-9, showing three Borros anchors (Figure 3-15) and one settlement platform (Figure 3-13). All pore water pressure in the clay has dissipated. The figures are millimeters of settlement, measured by optical levelling on to the tops of the pipes.

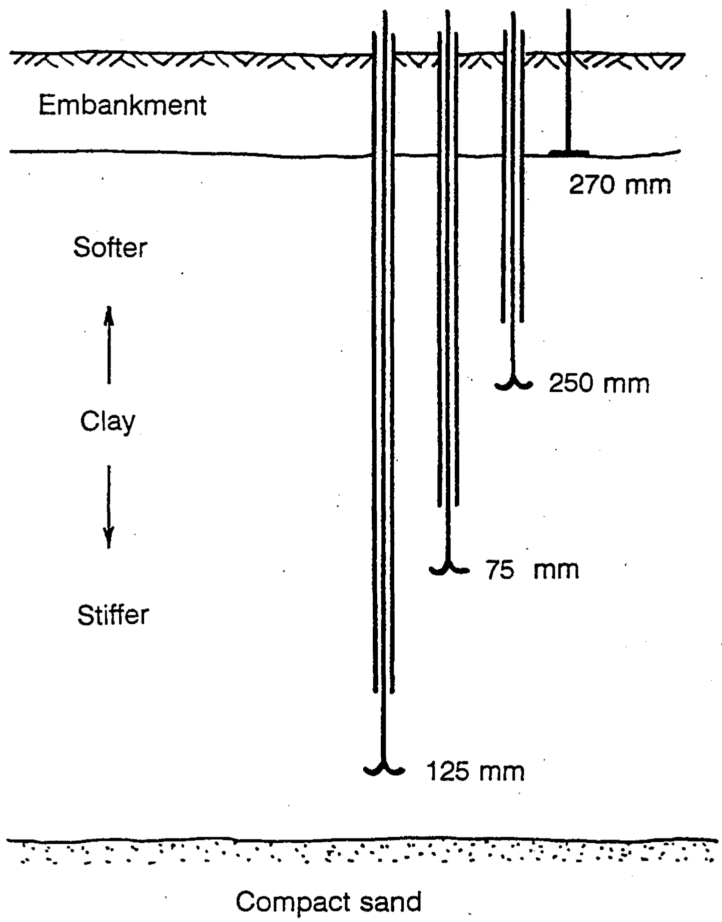


Figure 12-9: Embankment on Soft Ground, With Borros Anchors

- Does the data look reasonable?
- If not, what is unreasonable about the data?
- Can you think of hypothesis that is consistent with these data?
- What can you do about it?

12.9.9 Exercise No. 8. Data With Large Scatter

Your field crew has provided you with manual data (i.e. not obtained with a datalogger) which shows obviously excessive scatter. What are some possible causes?

12.9.10 Exercise No. 9. Data With No Change

Your field crew has provided you with manual data (i.e. not obtained with a datalogger) which shows no change with respect to previous data. You **know** that there must be changes. What are some possible causes?

12.9.11 Exercise No. 10. Vibrating Wire Piezometer

Readings of a vibrating wire piezometer (Figure 2-9) have been taken as it is lowered down a water-filled borehole during installation. Readings are compared with the static water head. The agreement between the two sets of data is much worse than the manufacturer's accuracy claim. What might have happened?

12.9.12 Exercise No. 11. Inclinometer Data

See Figure 12-10, showing two sets of A-Axis inclinometer data. Horizontal movement to the west is highly unlikely, and you have reason to believe that any horizontal movement below 20 m is also highly unlikely.

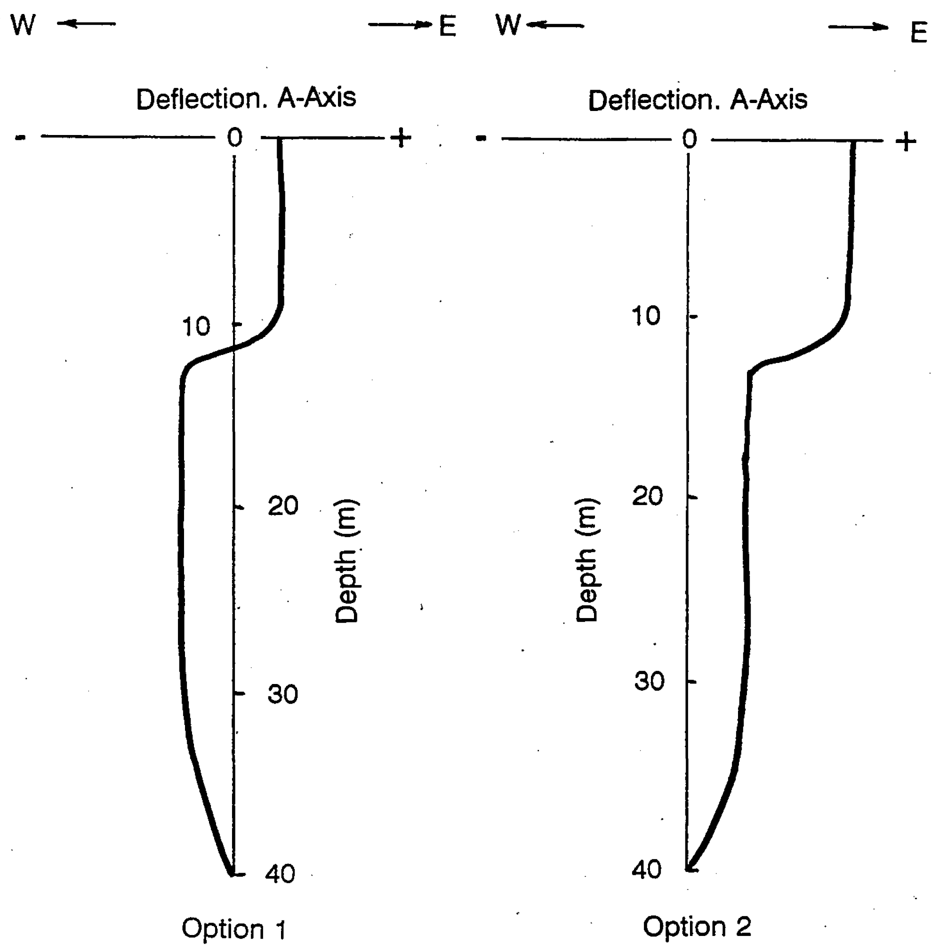


Figure 12-10: Inclinometer Data

- Do you have a possible explanation for the data? (Both options are incorrect, for the same reason.)

12.9.13 Exercise No. 12. Sondex Probe Extensometer

Sondex probe extensometer (Figure 3-9) data show heave with respect to the initial reading. You know that this is impossible. Do you have an explanation?

12.9.14 Exercise No. 13. Pneumatic Piezometer

Data from a pneumatic piezometer (Figure 2-6) do not appear to be correct. What hypotheses do you have, and what can you do about the problem?

CHAPTER 13.0
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APPENDIX A
CHECKLIST FOR PLANNING STEPS

Systematic planning of a monitoring program should proceed through the steps described in Chapter 6. The steps are summarized in this appendix in checklist form. Step numbers are consistent with headings in Chapter 6.

1. **Introduction** (Not a Planning Step)

2. **Define the Project Conditions**
   (a) Project type
   (b) Project layout
   (c) Subsurface stratigraphy and engineering properties
   (d) Groundwater conditions
   (e) Status of nearby structures or other facilities
   (f) Environmental conditions
   (g) Planned construction method
   (h) Knowledge of crisis situation

3. **Predict Mechanisms that Control Behavior**

4. **Define the Geotechnical Questions that Need to Be Answered**

5. **Define the Purpose of the Instrumentation**
   (a) Benefits during design
      - definition of initial site conditions
      - proof testing
      - fact-finding in crisis situations
   (b) Benefits during construction
      - safety
      - observational method
      - construction control
      - providing legal protection
      - measurement of fill quantities
      - enhancing public relations
      - advancing the state of the art
   (c) Verifying satisfactory performance after construction is complete

6. **Select the Parameters to Be Monitored**
   (a) Groundwater pressure
   (b) Total stress within soil mass
   (c) Total stress at contact with structure or rock
   (d) Vertical deformation
   (e) Horizontal deformation
   (f) Tilt
   (g) Strain in soil or rock
   (h) Load or strain in structural members
   (i) Temperature
   (j) Precipitation

7. **Predict Magnitudes of Change**
   (a) Predict maximum value, thus instrument range
   (b) Predict minimum value, thus instrument sensitivity or accuracy
   (c) Determine hazard warning levels

8. **Devise Remedial Action**
   (a) Devise action for each hazard warning level, ensuring that labor and materials will be available
   (b) Determine who will have contractual authority for initiating remedial action
   (c) Ensure that communication channel is open between design and construction personnel
   (d) Determine how all parties will be forewarned of planned remedial actions

9. **Assign Tasks for Design, Construction, and Operation Phases**
   (a) Complete Table 6-1
   (b) Assign supervisory responsibility for tasks by instrumentation specialist
   (c) Plan liaison and reporting channels
   (d) Plan who has overall responsibility and contractual authority for implementation

10. **Select Instruments**
   (a) Plan for high reliability:
   - maximum simplicity
   - don't allow lowest cost to dominate selection
   - maximum durability in installed environment
   - minimum sensitivity to climatic conditions
   - good past performance record
   - consider transducer, readout unit, and communication system separately
   - is reading necessarily correct?
   - can calibration be verified after installation?

   (b) Discuss application with manufacturer
   (c) Recognize any limitations in skill or quantity of available personnel
   (d) Consider both construction and long-term needs and conditions
   (e) Ensure good conformance
   (f) Ensure minimum interference to construction and minimum access difficulties
   (g) Determine need for automatic data acquisition system
   (h) Plan readout type and arrangements, consistent with required reading frequency
   (I) Plan need for spare parts and standby readout units
   (j) Evaluate adequacy of lead time
   (k) Evaluate adequacy of time available for installation
   (l) Question whether the selected instrument will achieve the objective

11. **Select Instrument Locations**
   (a) Identify zones of primary concern
   (b) Select primary instrumented sections
   (c) Select secondary instrumented sections
   (d) Plan quantities to account for less than 100% survival
   (e) Arrange locations to provide early data
   (f) Arrange locations to provide cross-checks

12. **Plan Recording of Factors that May Influence Measured Data**
   (a) Construction details
   (b) Construction progress

(c) Visual observations of expected and unusual behavior
(d) Geology and other subsurface conditions
(e) Environmental factors

13. Establish Procedures for Ensuring Reading Correctness

(a) Visual observations
(b) Duplicate instruments
(c) Backup system
(d) Study of consistency
(e) Study of repeatability
(f) Regular in-place checks

14. List the Specific Purpose of Each Instrument

15. Prepare Budget

Include costs, being particularly careful to make a realistic estimate of project duration, for

(a) Planning monitoring program
(b) Making detailed instrument designs
(c) Procuring instruments
(d) Making factory calibrations
(e) Installing instruments
(f) Maintaining and calibrating instruments on a regular schedule
(g) Establishing and updating data collection schedule
(h) Collecting data
(I) Processing and presenting data
(j) Interpreting and reporting data
(k) Deciding on implementation of results

16. Prepare Instrumentation System Design Report

(a) Steps 2-15
(b) Selected contract method for instrument procurement (See Section 6.17)
- negotiated procurement by state
- assigned suppliers
- low-bid

(c) Selected contract method for field instrumentation services (See Section 6.21)
- specialist work by state personnel
- specialist work by consulting firm under contract to state
- assigned subcontractor
- low-bid

17. Write Instrument Procurement Specifications

(a) Use method selected in Step 16(b)
(b) Write specifications, if needed

18. Plan Installation

(a) Prepare step-by-step installation procedure well in advance of scheduled installation dates, including list of required materials and tools
(b) Prepare installation record sheets
(c) Plan staff training
(d) Coordinate plans with contractor

(e) Plan access needs
(f) Plan protection from damage and vandalism
(g) Plan installation schedule

19. Plan Regular Calibration and Maintenance

(a) Plan pre-installation acceptance tests
(b) Plan calibrations during service life
- readout units
- embedded components

(c) Plan maintenance
- readout units
- field terminals
- embedded components

20. Plan Data Collection, Processing, Presentation, Interpretation, Reporting, and Implementation

(a) Plan data collection
- prepare preliminary detailed procedures for collection of initial and subsequent data
- prepare field data sheets
- plan staff training
- plan data collection schedule
- plan access needs

(b) Plan data processing and presentation
- determine need for automatic data processing
- prepare preliminary detailed procedures for data processing and presentation
- prepare calculation sheets
- plan data plot format
- plan staff training

(c) Plan data interpretation
- prepare preliminary detailed procedures for data interpretation

(d) Plan reporting of conclusions
- define reporting requirements, contents, frequency

(e) Plan implementation
- verify that all Step 8 items are in place

21. Write Specifications for Field Instrumentation Services

(a) Use method selected in Step 16(c)
(b) Write specifications, if needed

22. Update Budget

Include costs for all tasks listed in Step 15

APPENDIX B
OVERHEAD VIEWGRAPHS USED DURING COURSE TO ILLUSTRATE AN EXAMPLE OF A SYSTEMATIC APPROACH TO PLANNING MONITORING PROGRAMS
(Embankment on Soft Ground)

The following overhead viewgraphs are used during the course to illustrate a systematic approach to planning monitoring programs.

6.2 DEFINE THE PROJECT CONDITIONS

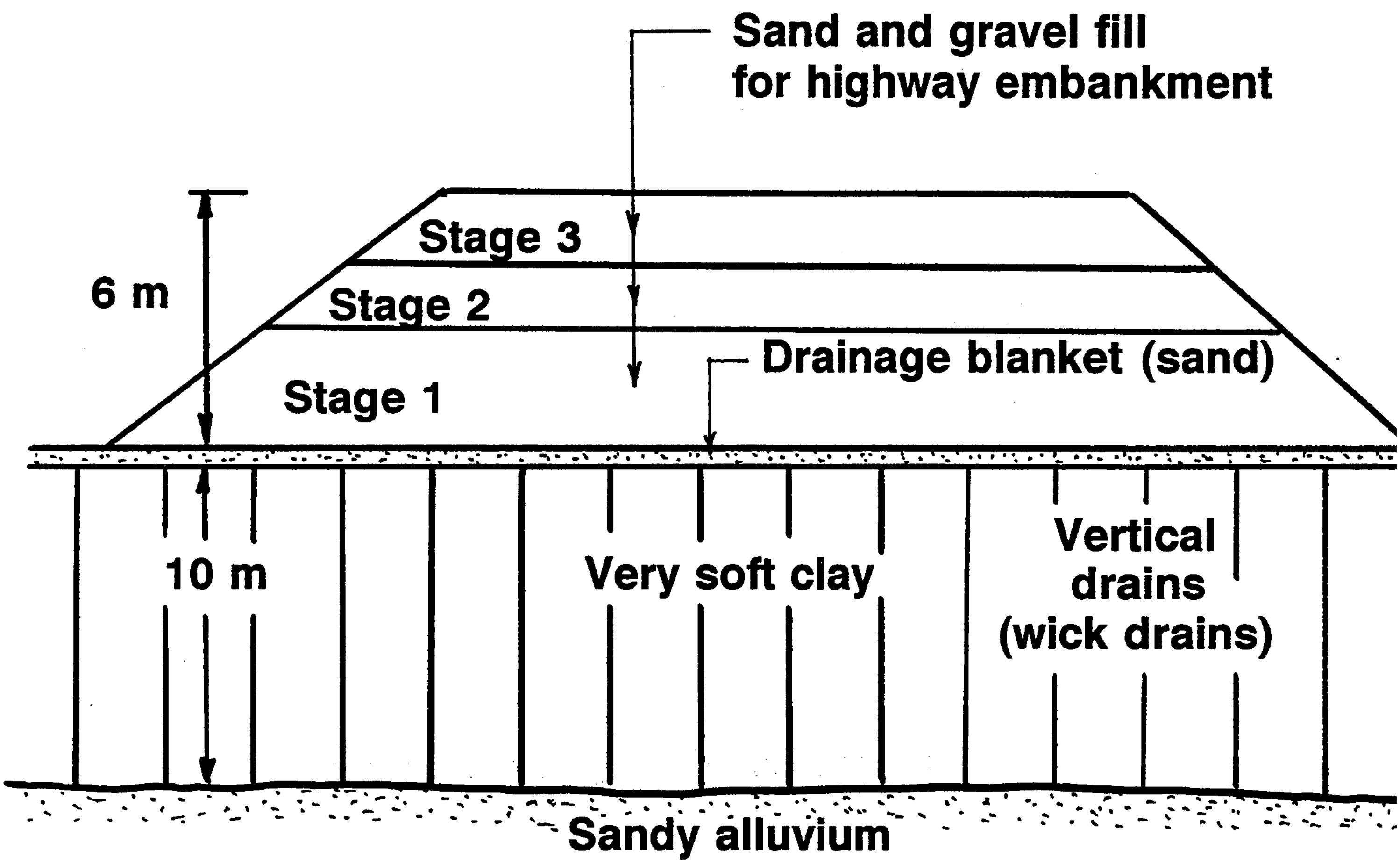


FIGURE B-1

6.3 PREDICT MECHANISMS THAT CONTROL BEHAVIOR

- Rotational slide along arc
- Settlement (consolidation of very soft clay)
- Lateral bulging of very soft clay

FIGURE B-2

6.4 DEFINE THE GEOTECHNICAL QUESTIONS THAT NEED TO BE ANSWERED

- Is the embankment stable?
- What is the progress of consolidation?

FIGURE B-3

6.5 DEFINE THE PURPOSE OF THE INSTRUMENTATION

- To provide warning of any instability
- To evaluate effectiveness of vertical drains
- To indicate when more fill can be placed

FIGURE B-4

6.6 SELECT THE PARAMETERS TO BE MONITORED

- ◇ Groundwater pressure ✔
- ◇ Deformation (vertical & horizontal) ✔
- ◇ Earth pressure
- ◇ Load and strain in structural members

FIGURE B-5

6.7 PREDICT MAGNITUDES OF CHANGE

- Instrument range
- Instrument sensitivity or accuracy
- Hazard warning level
  - green
  - yellow
  - red

FIGURE B-6

TABLE 6-1
CHART USED FOR TASK ASSIGNMENT

Task	Responsible Party			
	State	Design Consultant	Instrumentation Specialist	Construction Contractor
Plan monitoring program				
Procure instruments and make factory calibrations				
Install instruments				
Maintain and calibrate instruments on regular schedule				
Establish and update data collection schedule				
Collect data				
Process and present data				
Interpret and report data				
Decide on implementation of results				

FIGURE B-7

6.9 ASSIGN TASKS

The party with the greatest vested interest in the data should be given direct line responsibility for producing it accurately.

Wally Baker

FIGURE B-8

6.10 SELECT INSTRUMENTS

1. Pore Water Pressure
   - Open standpipe - *maybe*
   - Vibrating wire - *probably yes*
   - Pneumatic - *okay, but prefer v-w*
   - Push-in type

2. Vertical Deformation
   - Surface monuments w/surveying
   - Probe extensometers, magnet/reed switch
   - Settlement platforms - *maybe*
   - Liquid level gages - *probably no*

3. Horizontal Deformation
   - Surface monuments w/surveying
   - Inclinometers

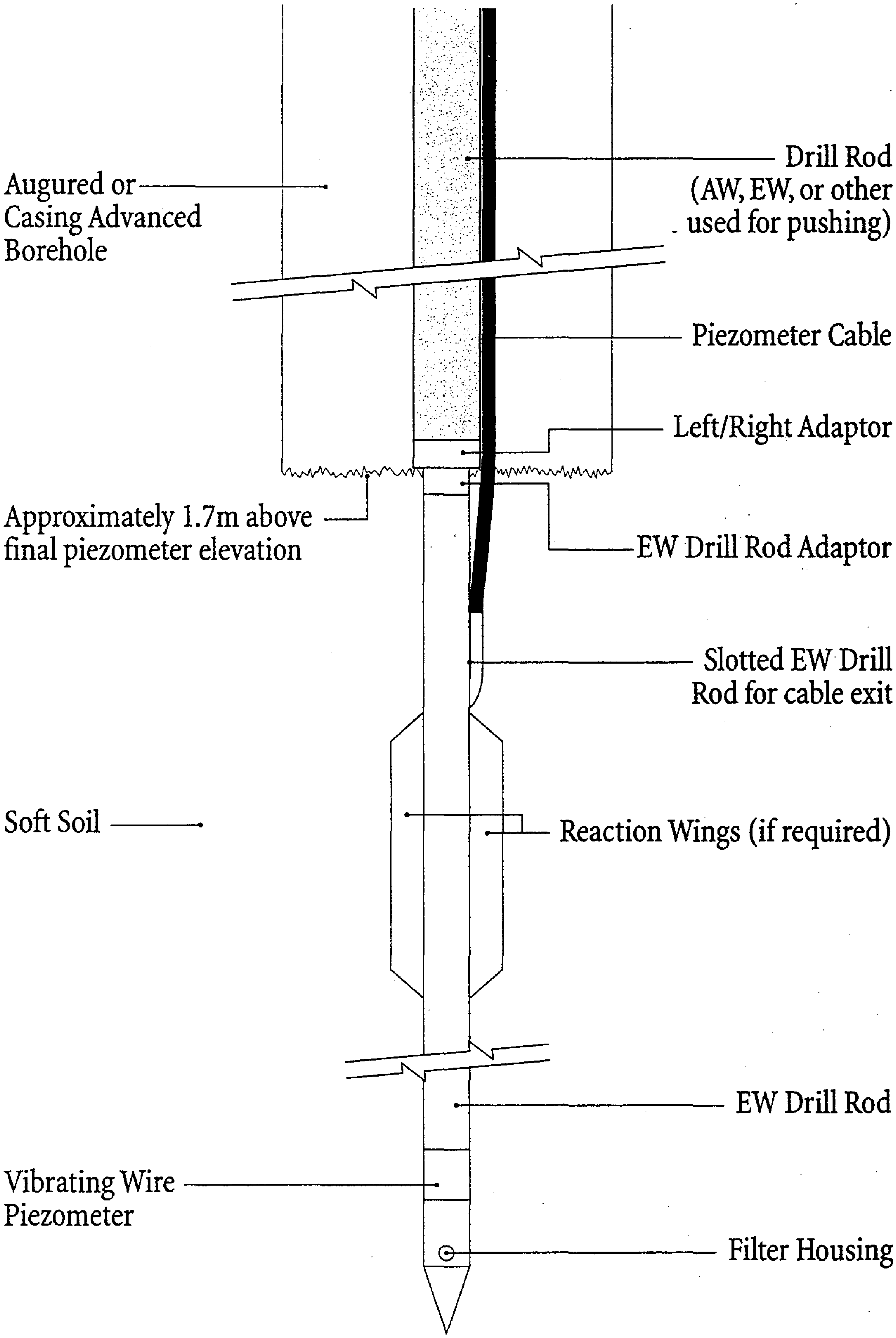
FIGURE B-9

(Use only in clays w/o sand grains)

OPEN STANDPIPE PIEZOMETER: PUSH-IN TYPE

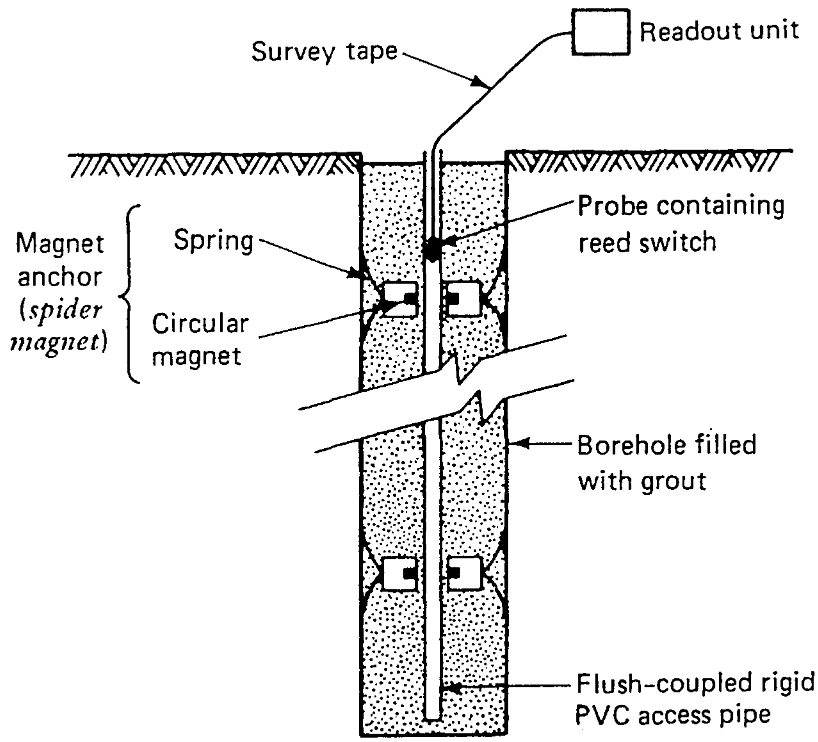
(Geonor model M-206)

FIGURE B-10



VIBRATING WIRE PIEZOMETER: PUSH-IN TYPE

FIGURE B-11



MAGNET/REED SWITCH PROBE EXTENSOMETER

FIGURE B-12

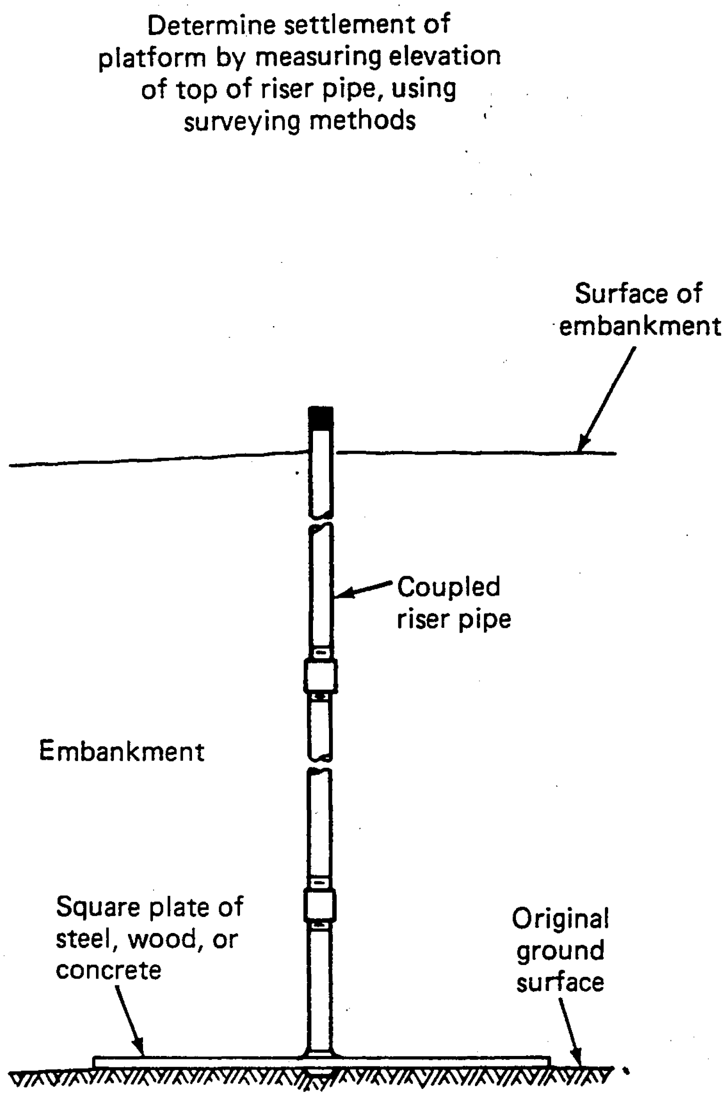


FIGURE B-13

SETTLEMENT PLATFORM

$P = H\gamma$; thus, determine elevation of cell

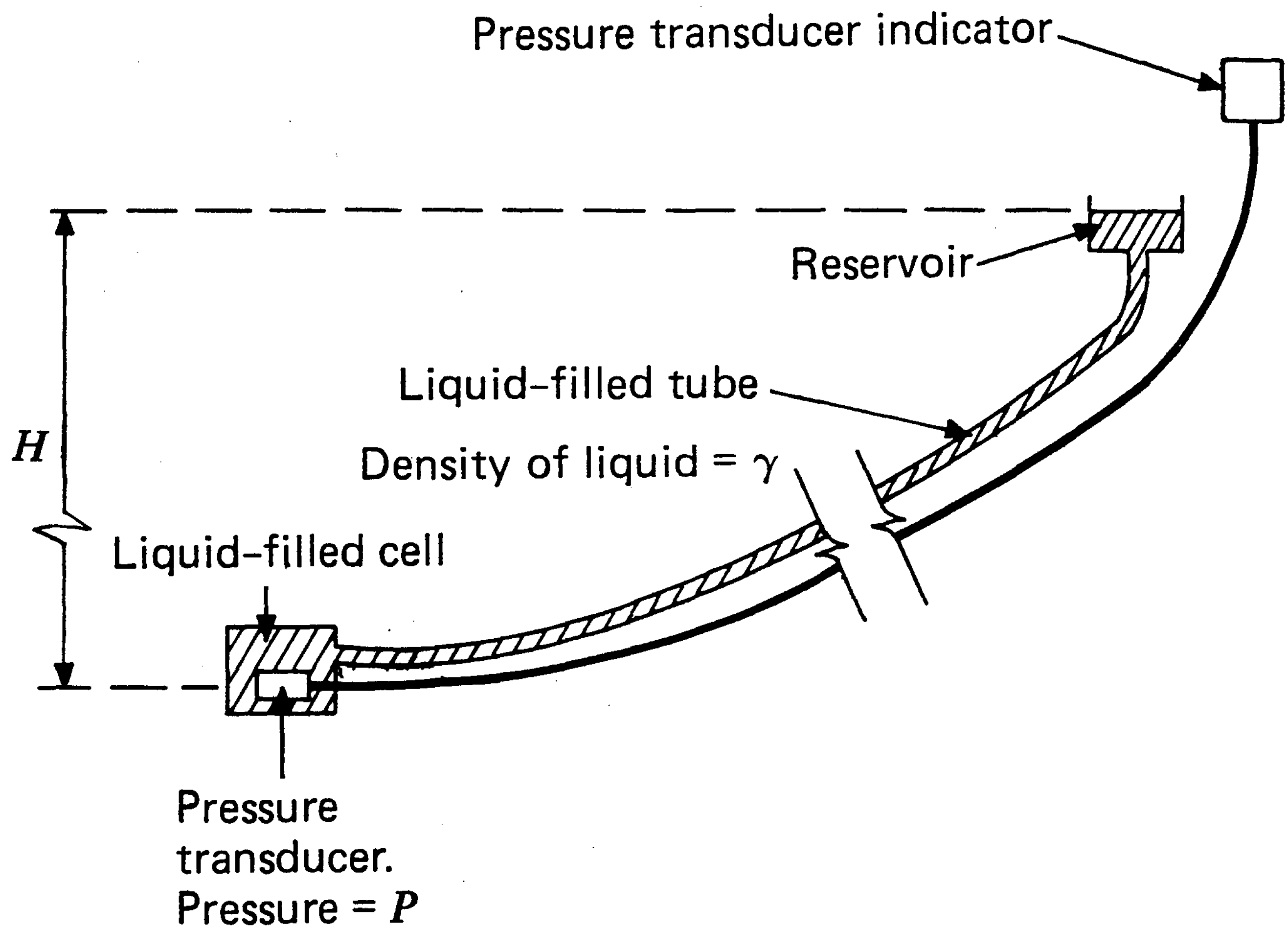
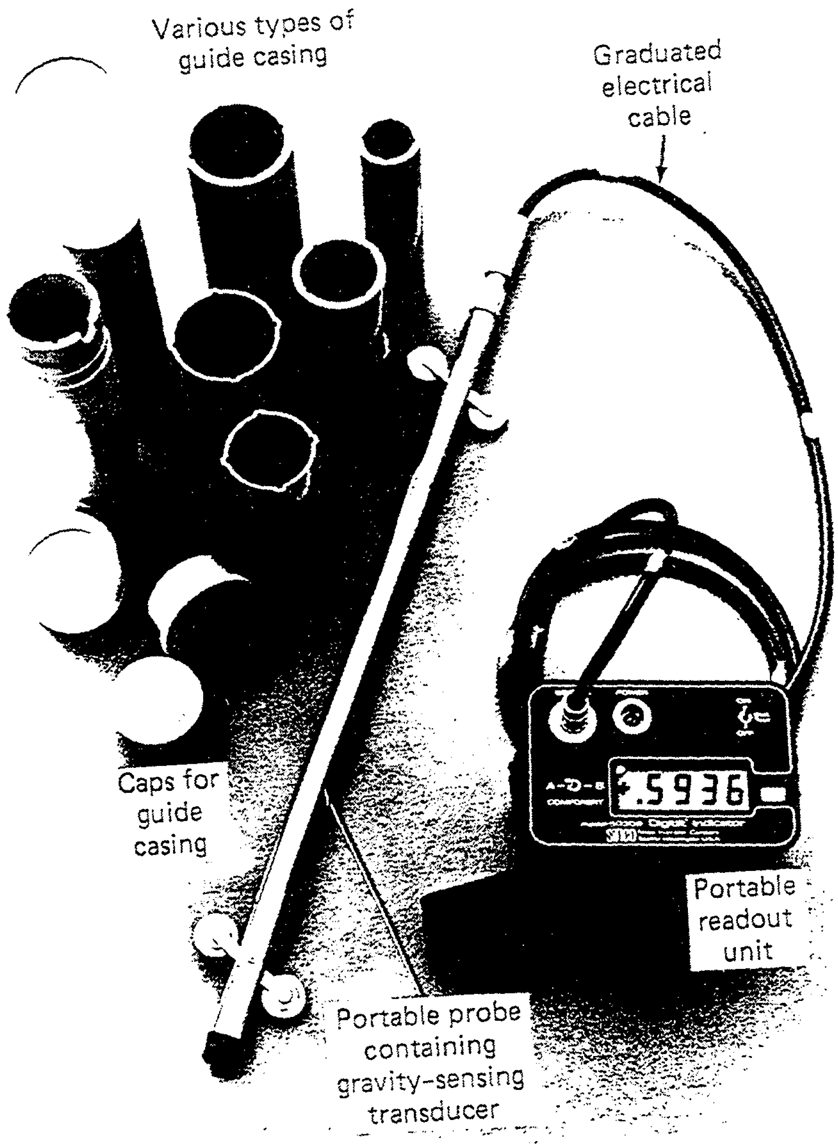


FIGURE B-14

LIQUID LEVEL GAGE



INCLINOMETER

FIGURE B-15

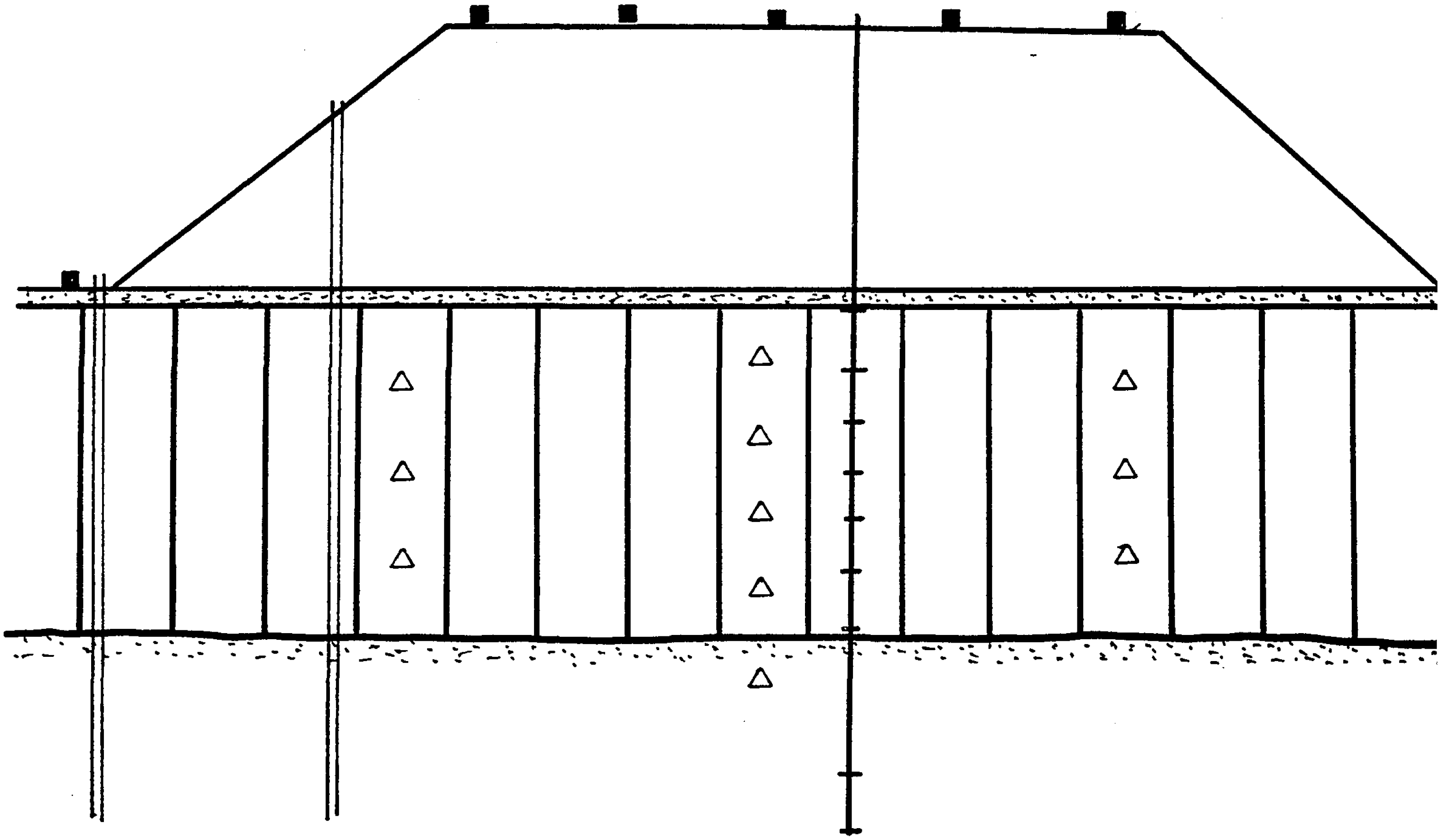
6.11 SELECT INSTRUMENT LOCATIONS

1. Identify zones of particular concern

2. Primary instrumented sections

3. Secondary instrumented sections

FIGURE B-16



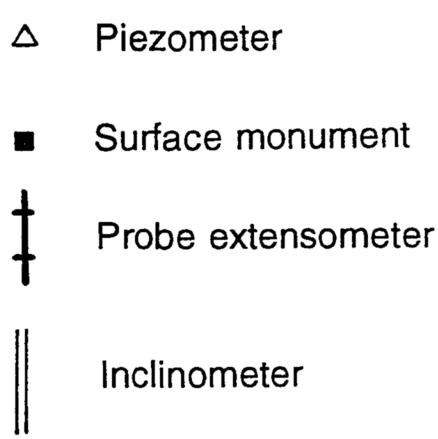
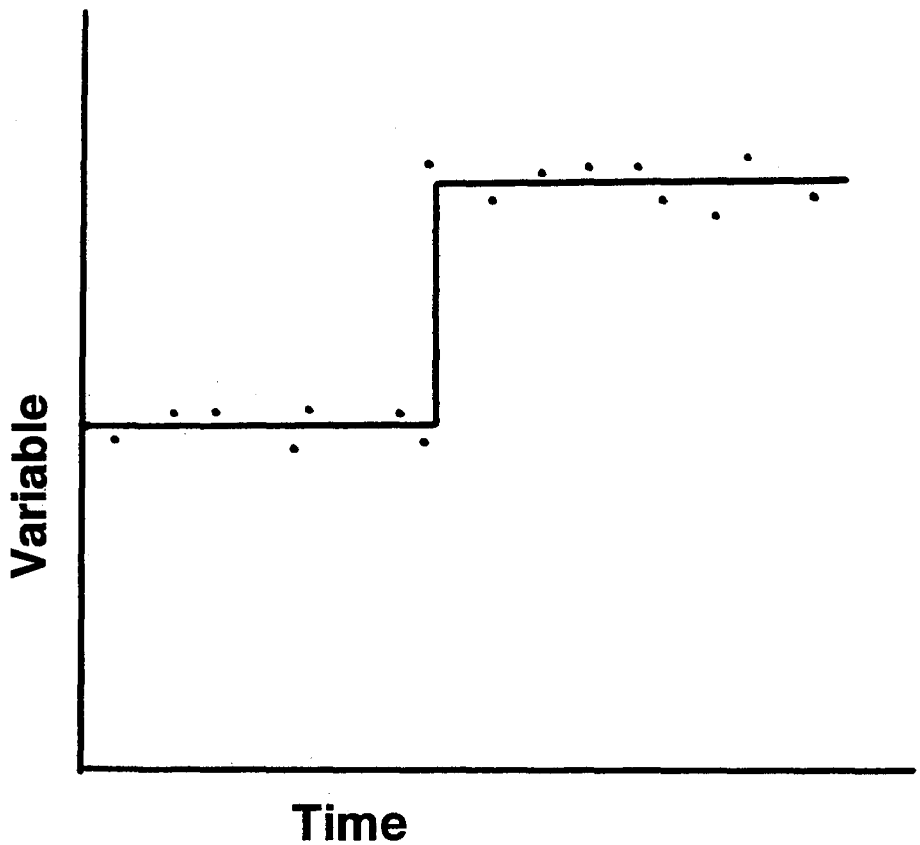


FIGURE B-17

POSSIBLE INSTRUMENT LOCATIONS

6.12 PLAN RECORDING OF FACTORS THAT MAY INFLUENCE MEASURED DATA



PLOT OF DATA

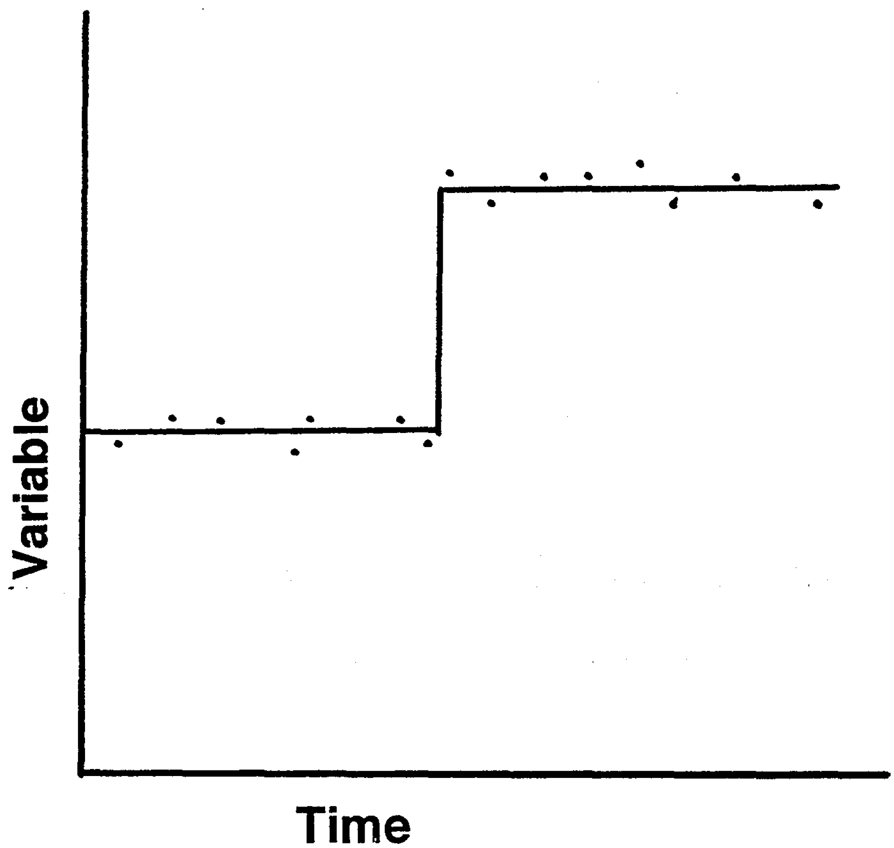
FIGURE B-18

6.12 PLAN RECORDING OF FACTORS THAT MAY INFLUENCE MEASURED DATA

- Fill level
- Temperature
- Rainfall
- Barometric pressure, maybe
- Visual observations of unexpected and unusual behavior

FIGURE B-19

6.13 ESTABLISH PROCEDURES FOR ENSURING READING CORRECTNESS



PLOT OF DATA

FIGURE B-20

6.13 ESTABLISH PROCEDURES FOR ENSURING READING CORRECTNESS

- Visual observations
- Consistency between instruments
- Consistency between measured parameters
- Regular calibrations of readout units
- Study of repeatability, over both short and longer term
- Consistency of reaction of instruments to new lift of fill
- Measured pore water pressures, under no load change, should extrapolate to initial pressures

FIGURE B-21

6.16 PREPARE INSTRUMENTATION SYSTEM DESIGN REPORT

- Report on all previous steps
- Selected contract method for instrument procurement
- Selected contract method for field instrumentation services

FIGURE B-22

6.17 WRITE INSTRUMENT PROCUREMENT SPECIFICATIONS

- Negotiated procurement by state
- Assigned suppliers
- Low-bid

FIGURE B-23

6.18 PLAN INSTALLATION

- Written step-by-step procedures, with post-installation acceptance tests

- Listing of tools and materials

- Installation record sheets

FIGURE B-24

6.19 PLAN REGULAR CALIBRATION

- Factory calibrations
- Pre-installation acceptance tests
- Calibrations during service life

FIGURE B-25

6.19 PLAN REGULAR MAINTENANCE

- Readout units
- Field terminals
- Embedded components

FIGURE B-26

6.21 WRITE SPECIFICATIONS FOR FIELD INSTRUMENTATION SERVICES

- Specialist work by state personnel
- Specialist work by consulting firm under contract to state
- Assigned subcontractor
- Low-bid

FIGURE B-27

APPENDIX C
EXAMPLE OF RESULTS OF STUDENT EXERCISE FOR PLANNING A MONITORING PROGRAM FOR A SOIL SLOPE

Introduction

A student exercise for planning a monitoring program for a soil slope is included in Section 7.5. This appendix indicates one possible result of the exercise, but **it is important to recognize that entirely different results may apply to any real project situation.**

Predict Mechanisms That Control Behavior (Step 3)

Possibilities:

- Shallow surface ravelling
- Deep-seated failure

(Look for evidence of other surface movement.)

Define the Geotechnical Questions That Need to be Answered (Step 4)

- Why has the surface crack appeared?

and/or

- Is there a stability problem?

Select the Parameters to be Monitored (Step 6)

- Groundwater pressure
- Vertical deformation (surface)
- Horizontal deformation (surface and subsurface)
- Tilt (perhaps)
- Precipitation

Predict Magnitudes of Change (Step 7)

- Groundwater pressure. Range to accommodate largest piezometric head. Accuracy doesn't need to be better than ± several cm.
- Surface deformation, in general. Range not an issue (we know how we're going to measure deformation). Accuracy perhaps ± 5 mm (no need for ± 1 mm).
- Surface deformation, crack width. Range large, perhaps 100 mm, extendable by resetting. Accuracy high, to document trend. Perhaps ± 1 mm.
- Subsurface horizontal deformation. Range large. Accuracy high.
- Tilt. Range not large. Accuracy high.

Select Instruments (Step 10)

- Open standpipe or vibrating wire piezometers (select appropriate range)
- Surveying methods, conventional third order

- Crack gages. Perhaps Figure 3-1 gage
- Inclinometers
- Tiltmeters (perhaps)
- Precipitation gages (or rely or nearby weather station)
- Tiltmeters and/or in-place inclinometers if an automatic deformation monitoring system is required

Plan Recording of Factors That May Influence Measured Data (Step 12)

- Records of any subsequent excavation
- Visual observations of expected and unusual behavior
- Precipitation (already planned)

Establish Procedures for Ensuring Reading Correctness (Step 13)

- Visual observations of surface deformation
- Study of repeatability
- Crack gage backup by survey tape
- Falling head tests in open standpipe piezometers
- Manufacturer's check and re-calibration of inclinometer
- Check-sums for inclinometer data
- Statistical closure procedures for surveying

REMEMBER

You have only worked some of the planning steps.
For a real project situation, you must work all relevant steps.

Possible Layouts

Some possible layouts of instruments for monitoring cut slopes in soil and rock are shown in Figures C-1 and C-2 respectively. Again, **the instrument selection and layout must be selected based on the needs of each individual project.**

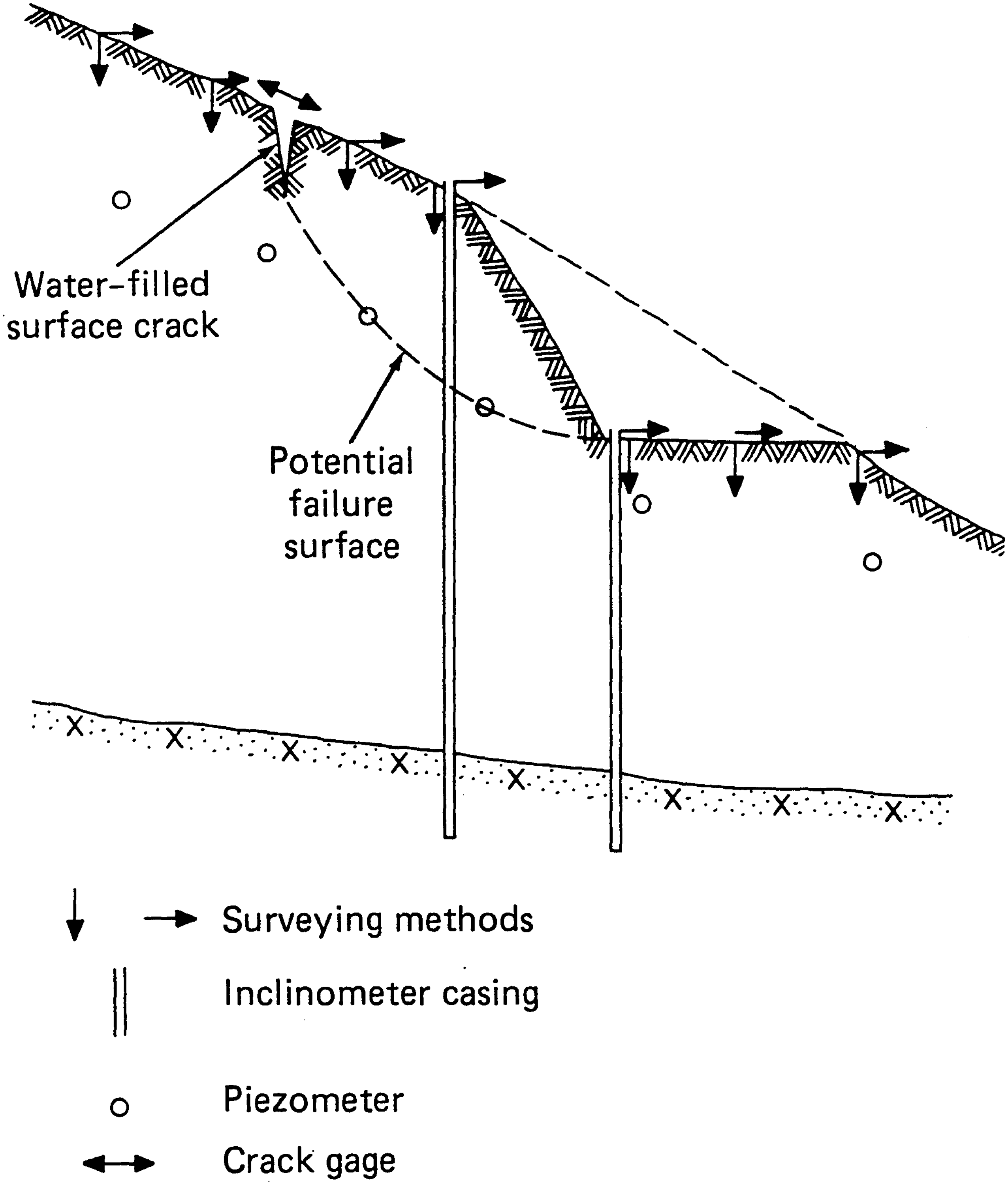


Figure C-1. EXAMPLE: MONITORING CUT SLOPE IN SOIL

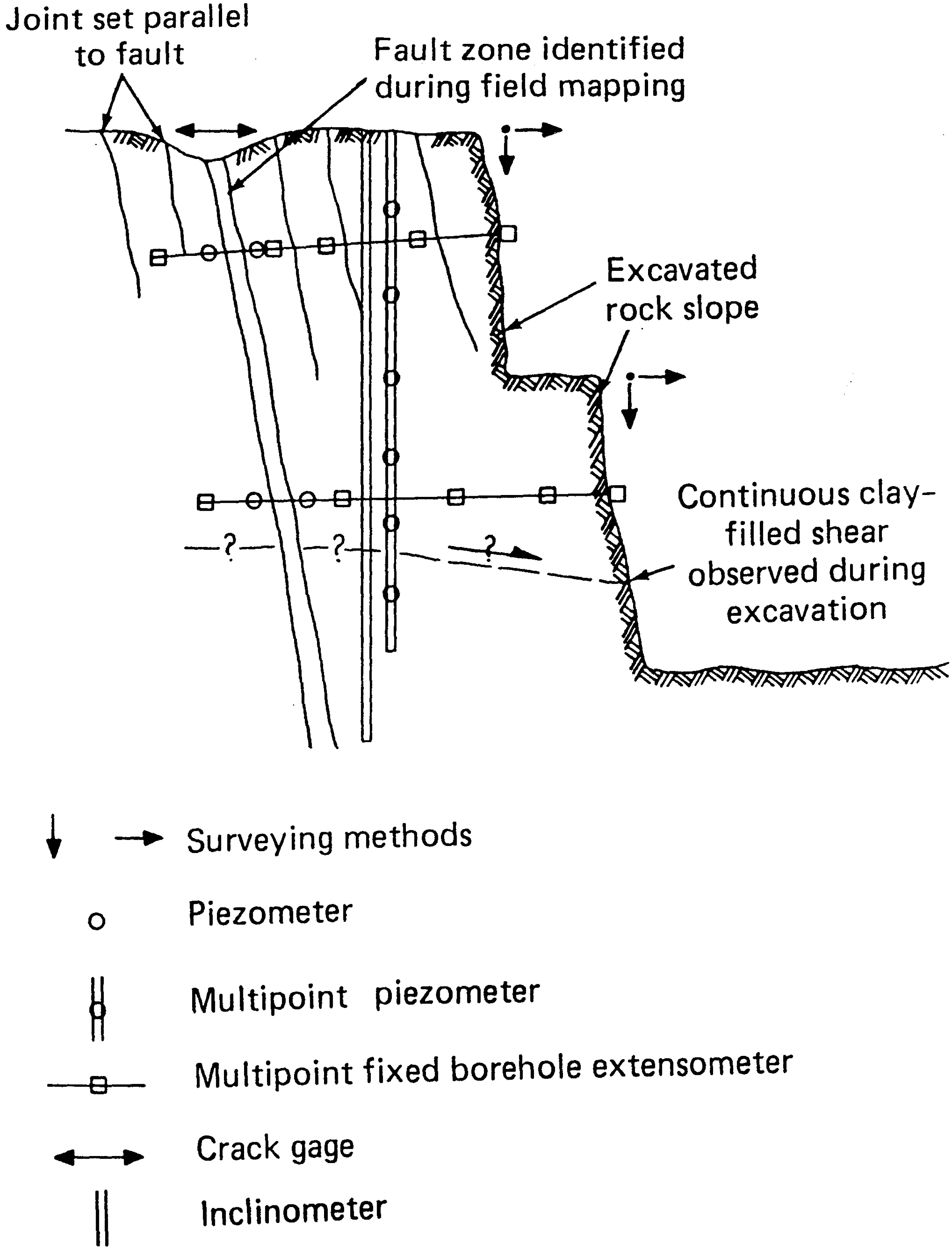


Figure C-2. EXAMPLE: MONITORING CUT SLOPE IN ROCK

APPENDIX D
EXAMPLE OF RESULTS OF STUDENT EXERCISE FOR PLANNING A MONITORING PROGRAM FOR A DRIVEN PILE

Introduction

A student exercise for planning a monitoring program for a driven pile is included in Section 8.4. This appendix indicates one possible result of the exercise, but **it is important to recognize that entirely different results may apply to any real project situation.**

Predict Mechanisms That Control Behavior (Step 3)

See Figure D-1.

Define the Geotechnical Questions That Need to be Answered (Step 4)

- What is the load-movement relationship of the pile? (i.e. When we load it, how much does it go down? What is its capacity?)
- How and where is the load transferred from the pile to the soil? (Leading to: is the pile too short, or unnecessarily long?)

Select the Parameters to be Monitored (Step 6)

- Deformation at butt (horizontal and vertical)
- Load at butt
- Deformation at toe
- Stress along pile
- Load at toe

Predict Magnitudes of Change (Step 7)

- Deformation at butt. Range typically 50 mm. Accuracy typically $\pm$ 0.2 mm (don't confuse this with how we measure it: measurement accuracy is typically better than this).
- Load at butt. Range is the intended maximum applied total load. Accuracy enough to give adequately accurate data at each load increment: perhaps 0.2% of applied total load.
- Deformation of toe. Range less than at butt. Accuracy similar to butt.
- Stress along pile. Range is the stess that corresponds to maximum applied load. Accuracy for stress data will be controlled by conversion of strain to stress, hence strain measurement accuracy need not be high.
- Load at toe. Range: estimate for each specific case. Accuracy, similar reasoning as for load at butt.

Select Instruments (Step 10)

- Deformation at butt
  - Dial indicator with reference beam
  - Wire/mirror/scale
  - Optical survey

- Load at butt
  - Load cell, preferably vibrating wire type
  - Datalogger, if one has been selected for strain gages (see later)
- Deformation at toe
  - Telltale
- Stress along pile
  - Vibrating wire strain gages, sister bar type; and/or series extensometers (see Figure D-2)
  - Multiple telltales not a good choice, because of low accuracy (see Figure D-3)
  - Datalogger
  - Remember need for strain/stress conversion data (see Figure D-4)
- Load at toe
  - Use strain gages and/or series extensometers, as above, because toe load cells are likely to create too much complexity

Plan Recording of Factors That May Influence Measured Data (Step 12)

- Visual observations of expected and unusual behavior
- Temperature

Establish Procedures for Ensuring Reading Correctness (Step 13)

- Visual observations
- No need for additional procedures for deformation at butt, because two backup methods are planned
- For load at butt, use pressure gage on hydraulic jack as backup. If using a datalogger for the load cell readings, check regularly with a portable readout unit. Use the same precaution for other uses of a datalogger
- For strain gages and/or series extensometers, review repeatability, and also that pattern of strain along the pile is consistent

REMEMBER

**You have only worked some of the planning steps.
For a real project situation, you must work all relevant steps.**

Possible Layouts

Some possible layouts of instruments, for use during static axial load testing of piles and drilled shafts, are shown in Figures D-5 through D-8. Again, **the instrument selection and layout must be selected based on the needs of each individual project.**

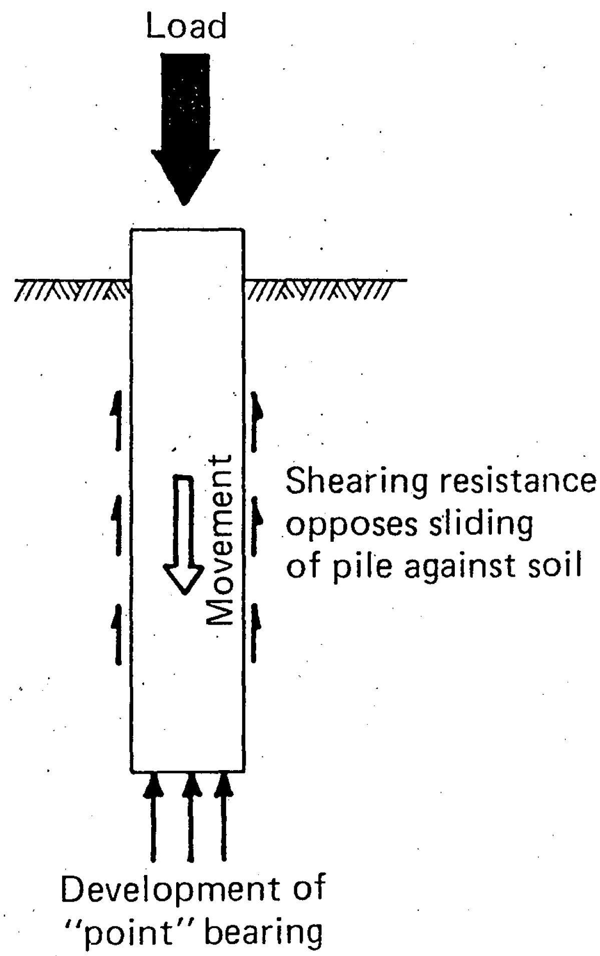


Figure D-1. POSSIBLE BEHAVIOR MECHANISMS FOR A DRIVEN PILE

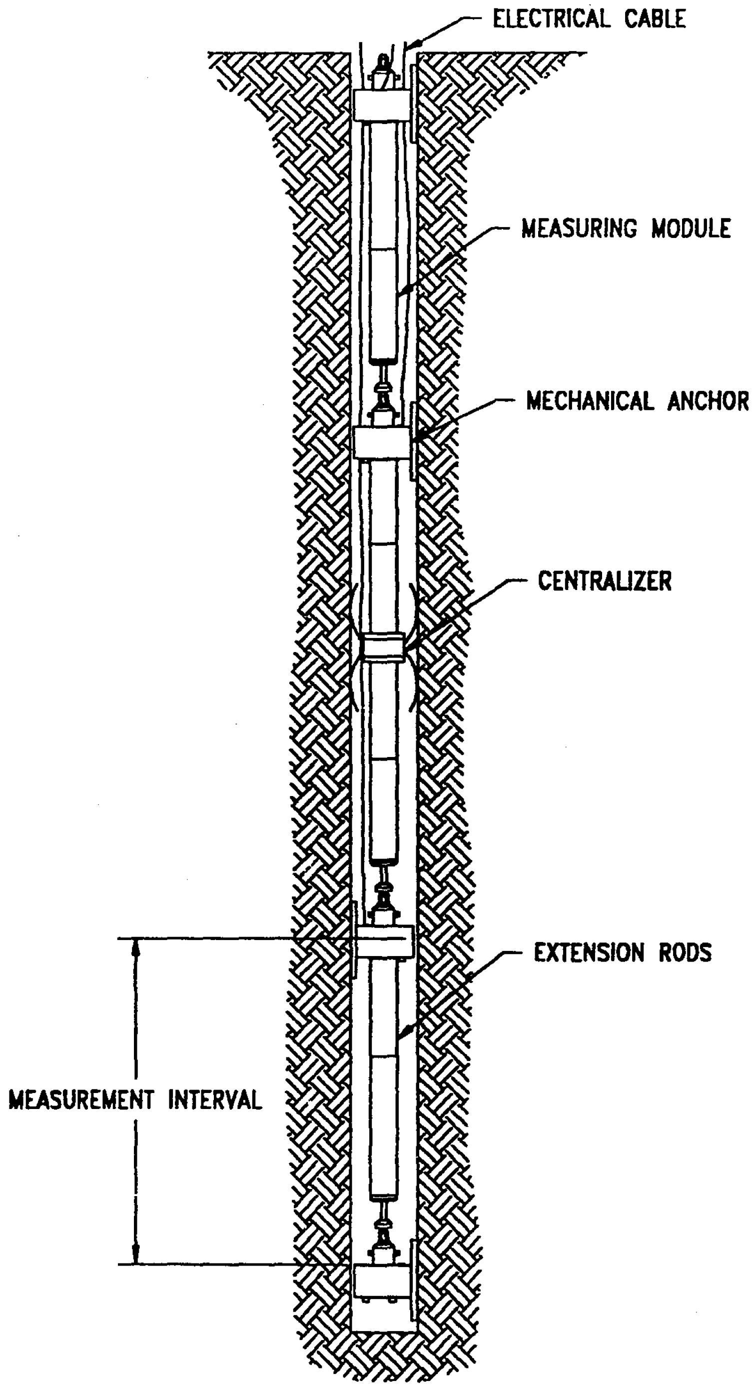
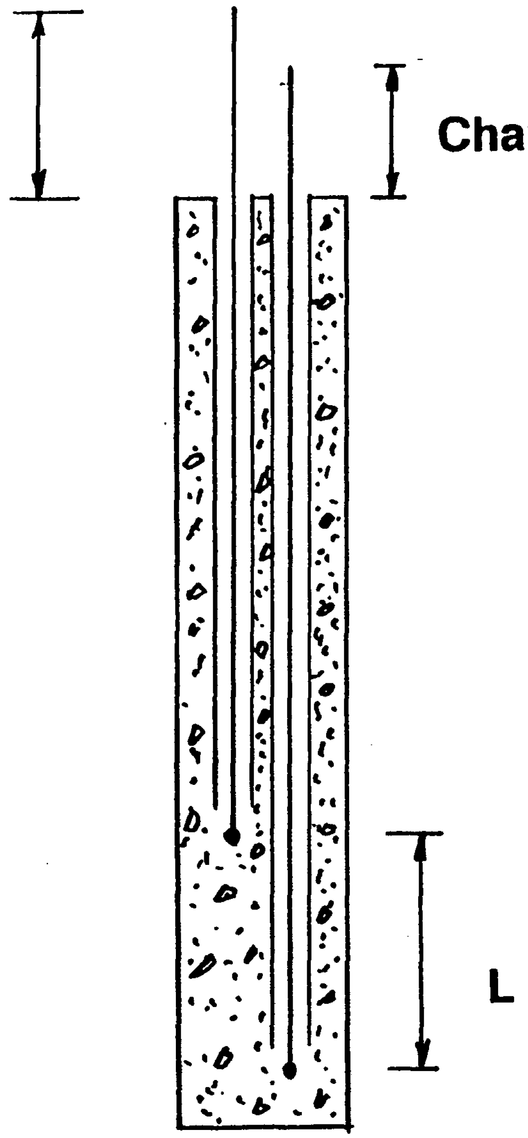


Figure D-2. SERIES EXTENSOMETER



$$\Delta L_1 - \Delta L_2 = \Delta L$$

$$\frac{\Delta L}{L} = \text{strain}$$

$$\text{Stress} = E \times \text{strain}$$

Figure D-3

MULTIPLE TELLTALES FOR STRESS DETERMINATION

(Stress determination is very sensitive to errors in measuring ΔL_1 and ΔL_2, hence accuracy is low)

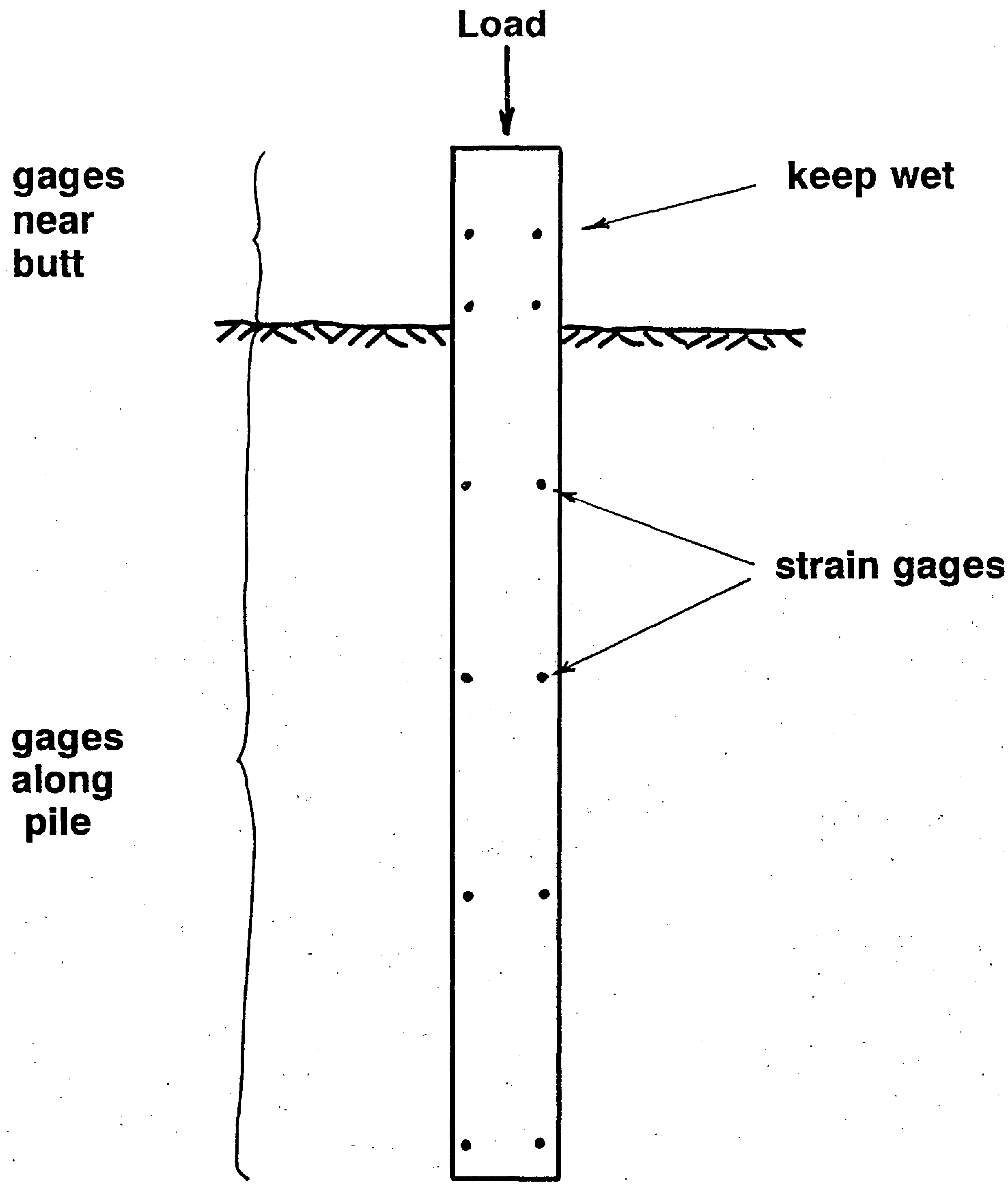
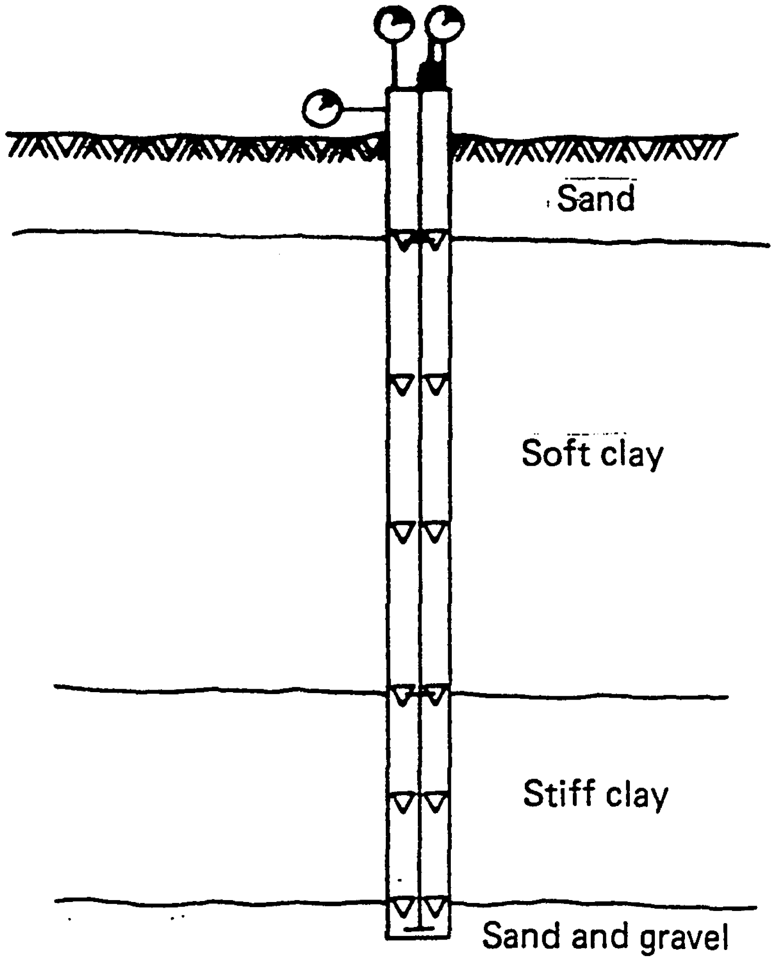


Figure D-4

STRAIN/STRESS CONVERSION IN <u>CONCRETE</u> PILE



Dial indicator, mounted on reference beam (wire/mirror/scale arrangement as backup)

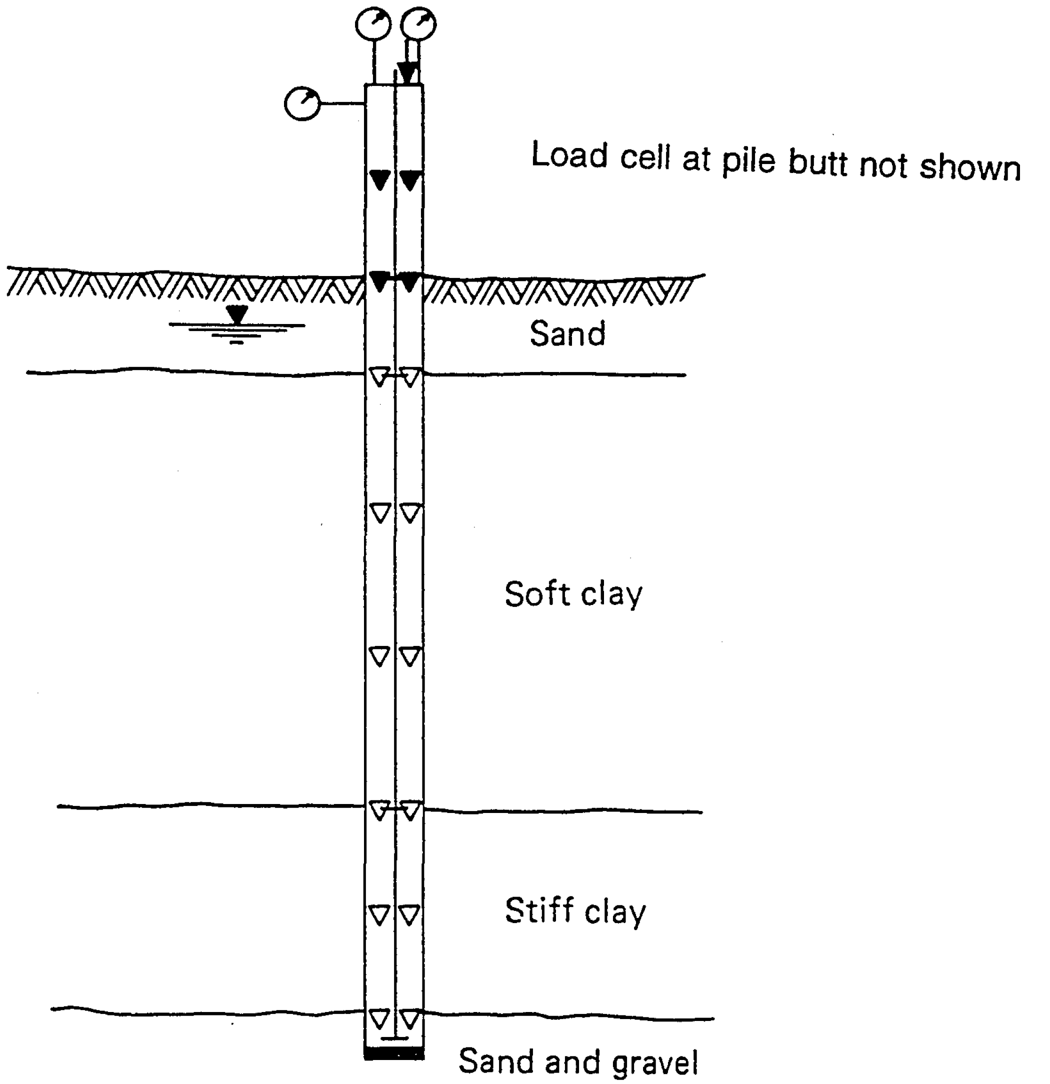
Surveying methods (also on reference beam)

Multiple telltales, with dial indicators mounted on pile butt. Duplicate telltales at each depth

▽ Surface-mounted strain gage

Load cell at pile butt not shown

Figure D-5. EXAMPLE: STATIC AXIAL LOAD TEST OF A STEEL PILE WITH COMPREHENSIVE LOAD TRANSFER DATA



Dial indicator, mounted on reference beam (wire/mirror/scale arrangement as backup)

Surveying methods (also on reference beam)

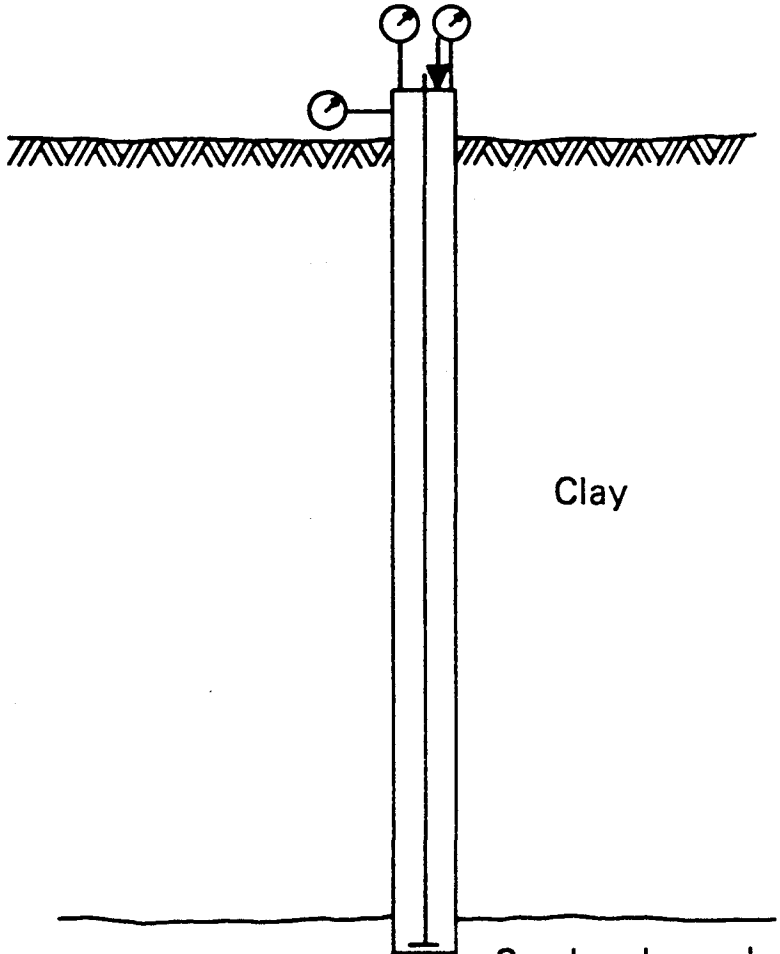
Multiple telltales, with dial indicators mounted on pile butt. Duplicate telltales at each depth

Embedment strain gage

Embedment or surface-mounted strain gage

Contact earth pressure cell

Figure D-6. EXAMPLE: STATIC AXIAL LOAD TEST OF A CONCRETE PILE WITH COMPREHENSIVE LOAD TRANSFER DATA



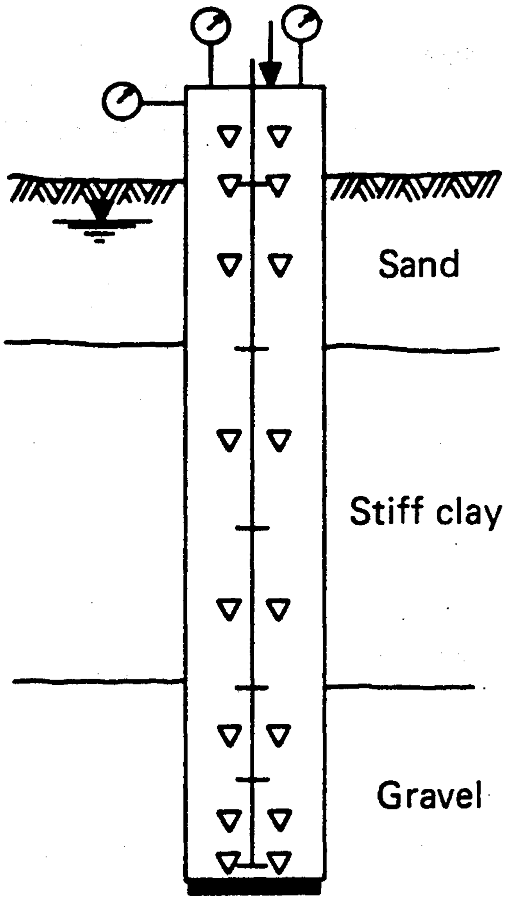
Dial indicator, mounted on reference beam
(wire/mirror/scale arrangement as backup)

Surveying methods (also on reference beam)

Telltale, with dial indicator mounted on pile butt

Load cell at pile butt not shown

Figure D-7. STATIC AXIAL TEST: NO COMPREHENSIVE LOAD TRANSFER DATA



Load cell at shaft top not shown

Multiple telltales, with dial indicators mounted on shaft top. Duplicate telltales at each depth. Or series extensometer

Dial indicator, mounted on reference beam (wire/mirror/scale arrangement as backup)

V-w sister bar or spot welded strain gage

Contact earth pressure cell (base load cell or Osterberg load cell)

Surveying methods (also on reference beam)

Figure D-8. EXAMPLE: AXIAL LOAD TEST OF A DRILLED SHAFT

APPENDIX E
EXAMPLE OF RESULTS OF STUDENT EXERCISE FOR PLANNING A MONITORING PROGRAM FOR A MECHANICALLY STABILIZED EARTH WALL

Introduction

A student exercise for planning a monitoring program for a mechanically stabilized earth wall is included in Section 9.7. This appendix indicates one possible result of the exercise, but **it is important to recognize that entirely different results may apply to any real project situation.**

Predict Mechanisms That Control Behavior (Step 3)

Possibilities:

- Shallow-seated horizontal deformation
- Deep-seated failure
- Settlement of backfill, leading to shear or tensile failure of connections between facing elements and reinforcing

Define the Geotechnical Questions That Need to be Answered (Step 4)

- Is the wall stable?
- What is the magnitude and distribution of stress in the wall? (Perhaps: this question more usually applies to a test wall built prior to construction of the project, but may be relevant here because of the concern for failure of connections.)
- What is the long-term performance of the wall?

Select the Parameters to be Monitored (Step 6)

- Deformation at face
- Deformation of top of wall
- Local deterioration of facing elements
- Drainage behavior of the backfill
- Horizontal deformation within wall
- Vertical deformation within wall
- Stress in reinforcement (perhaps)
- Precipitation

Predict Magnitudes of Change (Step 7)

- Deformation at face and top of wall. Range not an issue (we know how we're going to measure deformation). Accuracy perhaps $\pm$ 5 mm (no need for $\pm$ 1 mm)
- Local deterioration of facing elements. Range large. Accuracy not high
- Drainage behavior of the backfill by monitoring pressure. Range to accommodate largest piezometric head. Accuracy doesn't need to be any better than $\pm$ several cm
- Drainage behavior of the backfill, by monitoring flow. Range large. Accuracy not high
- Horizontal deformation within wall. Range large. Accuracy high
- Vertical deformation within wall. Range large. Accuracy high, but not highest possible
- Stress in reinforcement. Range up to yield stress. Accuracy not high

Select Instruments (Step 10)

- Surveying methods, conventional third order
- Crack gages at face. Perhaps Figure 3-2 or 3-3 gage
- Vibrating wire piezometers
- Inclinometers
- Probe extensometers
- Horizontal inclinometers (perhaps)
- Strain gages on reinforcement (perhaps)
- Precipitation gages (or rely on nearby weather station)

Plan Recording of Factors That May Influence Measured Data (Step 12)

- Records of construction
- Visual observations of expected and unusual behavior
- Precipitation (already planned)

Establish Procedures for Ensuring Reading Correctness (Step 13)

- Visual observations of surface deformation
- Study of repeatability
- Manufacturer's check and re-calibration of inclinometer
- Check-sums for inclinometer data
- Statistical closure procedures for surveying
- Check length of tape for probe extensometer

REMEMBER

**You have only worked some of the planning steps.
For a real project situation, you must work all relevant steps.**

Possible Layouts

Some possible layouts of instruments for monitoring mechanically stabilized earth walls are shown in Figures E-1 and E-2. Again, **the instrument selection and layout must be selected based on the needs of each individual project.**

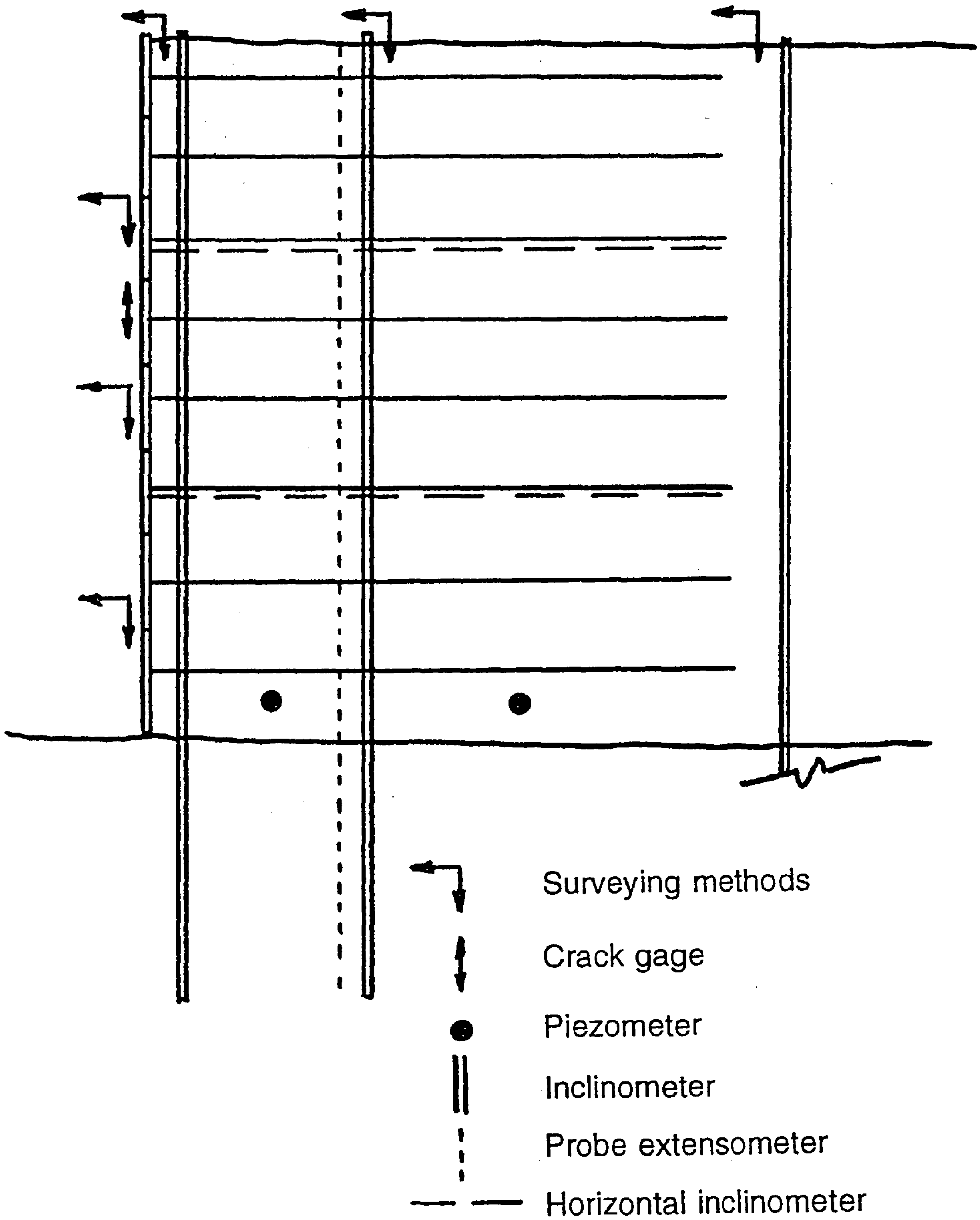
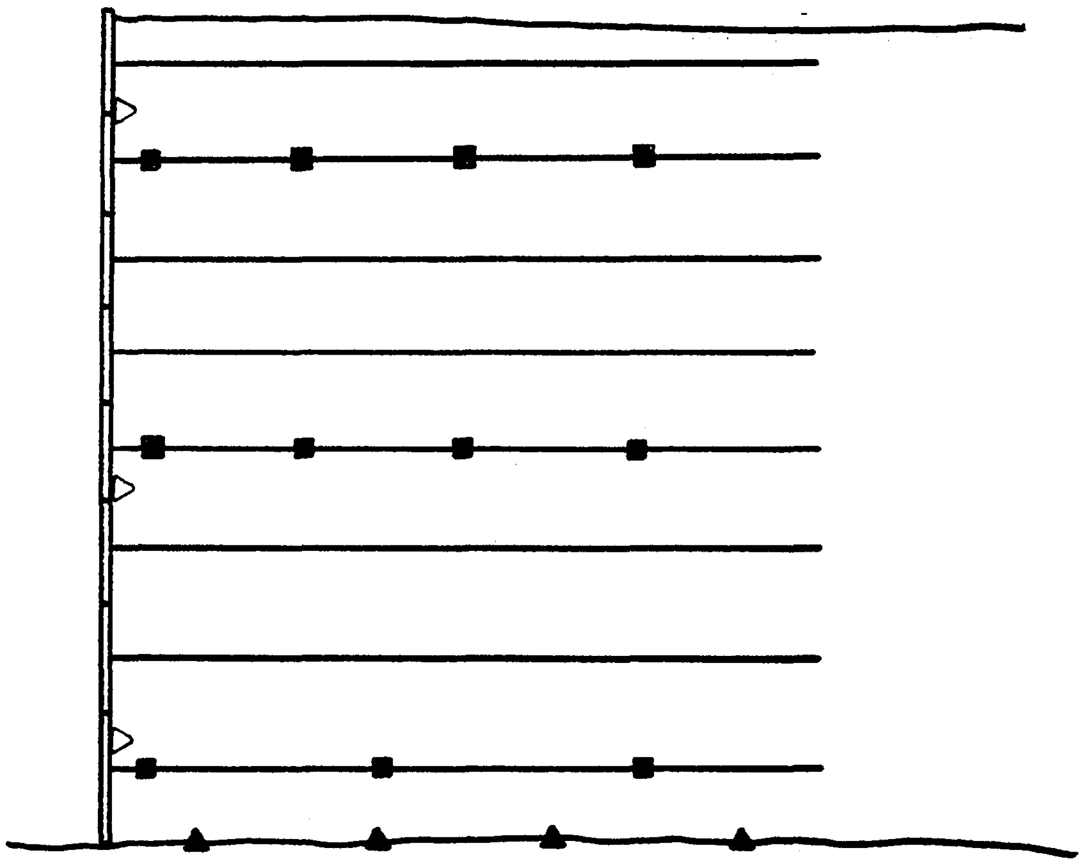


Figure E-1. EXAMPLE: POSSIBLE INSTRUMENTATION FOR COMPREHENSIVE STABILITY MONITORING OF MECHANICALLY STABILIZED EARTH WALL



■ Vibrating wire strain gages

▷ Contact earth pressure cells (??)

▲ Embedment earth pressure cells (?)

Figure E-2. EXAMPLE: POSSIBLE INSTRUMENTATION FOR DETERMINING MAGNITUDE AND DISTRIBUTION OF STRESS IN MECHANICALLY STABILIZED EARTH WALL

APPENDIX F
SUGGESTIONS FOR STUDENT EXERCISE ON EVALUATION OF DATA

Introduction

Various student exercises on evaluation of data are included in Section 12.9. The following are suggestions for evaluation.

Exercise No. 1. Embankment on Soft Ground

- We would not expect the bedrock to heave
- Has the benchmark settled? Check it
- Always make provision for a method of checking benchmarks

Exercise No. 2. Inclinometer on Slope

Repeat the reading. It appears that the operator may have inserted the inclinometer 180° off the correct orientation.

Exercise No. 3. Inclinometer Data

Because the data plots are not asymptotic to the "initial" line at the bottom of the casing (i.e. the plots do not show zero deflection over a depth range at the bottom of the casing), we know that horizontal movement is occurring all the way to the bottom. Unless the casing is in bedrock below 23.5 m, we must suspect that horizontal movement is occurring below this depth.

Keep track of the horizontal movement of the top of the casing by survey methods, and compute data from the top instead of the bottom. Learn a lesson: next time install casings to a depth significantly below the possible zone of horizontal movement ("base fixity").

Exercise No. 4. Probe Extensometer Data

As with Exercise No. 3, movement (in this case settlement) appears to be occurring below 23.5 m depth. Keep track of settlement of the top by optical levelling and compute data from the top instead of the bottom. Learn a lesson: next time install the extensometer to a depth significantly below the possible zone of vertical movement "base fixity").

Exercise No. 5. Full Profile Settlement Gage

Overlapping data should agree. Check optical levelling on to the gage supports.

Exercise No. 6. Optical Survey Data

If you plot the points first on a time scale, preferably using a semi-log plot as on Figure F-1, you will be able to smooth out scatter and get a better estimate of incremental settlement. In this case a better estimate of incremental settlement between days 4 and 27 would be 0.07 m (as opposed to the earlier estimate of 0.04 m, i.e., nearly twice as much).

Always plot data before working with it. However, be cautious about discarding data just because some points do not fall on a preconceived plot: they may be the points that tell you what you need to know.

Exercise No. 7. Embankment on Soft Ground

- Total settlement cannot increase with depth (if a point in the clay settles, all points above must settle by at least the same amount, unless local strain such as arching is occurring). Hence the 75/125 mm relationship looks unreasonable.
- We would expect more consolidation in the softest layers. Hence the 270/250 mm relationship may not be reasonable
- Settlement platform data are likely to be more reliable than Borros anchor data
- A hypothesis: outer Borros anchor pipes are supposed to protect inner pipes from downdrag. Has downdrag on the outer pipe caused the upper and lower anchors to be forced downward through the soil? We have no certain way of knowing whether this hypothesis is right, but it seems to be a believable one
- Check whether greasy garden hoses (Figure 3-15) were used. Check whether all anchors drove the full distance during actuation. Check whether inner and outer couplings may have interfered with each other. Check whether the gap between each outer pipe and anchor was opened sufficiently. All these factors should be on the installation record sheet (this is an example of the importance of such records)
- Drive on the outer pipe – does inner pipe go down?
- Maybe you can salvage something, perhaps that the 75 mm reading is valid. But maybe not. Maybe you will have to rely only on the settlement platform. Maybe you will learn to keep better installation records next time

Exercise No. 8. Data With Large Scatter

Possibilities are:

- Instrument malfunction. Do whatever you have to do to discover whether this is true: repeat readings by the crew; repeat readings by yourself; inspect the readout unit and installed components for signs of damage; re-calibrate the readout unit; compare data with data from other instruments, using the same readout unit
- The field crew is either not properly trained or not sufficiently motivated. Training is always possible. Motivation can be created by making sure that all members of the field crew understand the importance of their work
- Is the field crew working on a low-bid basis? Avoid this whenever possible

Exercise No. 9. Data With No Change

- Same as for Exercise No. 8
- Did the crew merely sit in a coffee shop and submit previous data?

Exercise No. 10. Vibrating Wire Piezometer

Possibilities are:

- Perhaps you have a "bad" piezometer. But before concluding this, consider the following
- Most such piezometers take between 20 minutes and 2 hours to achieve a uniform temperature (the thin metal parts "catch up" quickly, while thicker parts lag). When the temperature in the piezometer varies (this is called a "temperature transient"), the reading will be wrong. To observe this effect,

hang a vibrating wire piezometer outside the window on a cold or a hot day, and take a reading every minute. Alternatively, grip a piezometer in your hand and see how the reading changes. Temperature transient errors cannot be eliminated by using the manufacturer's temperature correction factors, because these assume uniform temperature. Almost all sensors are affected by temperature transients - this issue is by no means limited to vibrating wire piezometers.

- Perhaps you forgot that, as the piezometer is lowered, the added volume of submerged cable causes the water level in the borehole to rise

<u>**Exercise No. 11. Inclinometer Data**</u>

- Did the operator wait the required 10 minutes with the probe at the bottom of the borehole for warm-up? Failure to do this can cause the early data in a survey to be incorrect, hence creating data such as in the figure. Option 1 shows a negative offset during warm-up, from 40 to 27 m. Option 2 shows a positive offset. In both cases you will see poor check-sums.
- For many years, most manufacturers of inclinometers have recommended that, after lowering an inclinometer probe to the bottom of a casing, it should be left there for at least ten minutes to achieve temperature stabilization, with the power on. There are two components of this: electrical warm-up and the need to achieve a uniform temperature throughout the mechanical components
- The advent of memory-equipped readout units, which include a displayed "ready" symbol to indicate that readings are stable, appears to have lead to some incorrect reading techniques. Such "ready" symbols usually merely indicate that both A and B axis readings are stable. The "ready" signal is **NOT** intended to monitor the warm-up period and should not be used for that purpose
- A correct plot is shown on Figure F-2

<u>**Exercise No. 12. Sondex Probe Extensometer**</u>

- If readings have been taken by relying on graduations on the electrical cable, data may be wrong. The cable can (and does) stretch. If no vertical movement has occurred, and the cable has stretched, data will appear as apparent heave
- Never rely on the cable graduations. Reply on a survey tape. The best arrangement is a survey tape with electrical conductors bonded to its edges, as shown for the electrical dipmeter in Figure 2-5. Most magnet/reed switch probe extensometers (Figure 3-12) have such a tape

<u>**Exercise No. 13. Pneumatic Piezometer**</u>

- Is there water in the tubes? Sources are broken tubing, leaking connections, broken diaphragm, incorrect gas. Connections should never be planned in buried tubing: reserve them for repair only. There is no solution for broken tubing, leaking buried connections or a broken diaphragm. Gas <u>must</u> be dry (usually dry nitrogen): air should never be used. If the problem is wet gas, discuss with the manufacturer how to blow the water out – there may be a maximum allowable gas pressure
- Is it becoming harder to circulate gas? If yes, this could indicate pinched tubing. Some commercial tubing pinches very readily. The most problem-free tubing is unplasticized nylon 11 with a polyethylene sheath, but this costs more than most conventional tubing and may not be available from all suppliers of pneumatic piezometers
- Try reading the piezometer under a condition of no gas flow (Section 2.4 and Figure 2-6): this may overcome errors caused by poor control of gas flow rate (Dunnicliff, 1988, 1993)
- Discuss with the manufacturer whether you can reverse inlet and outlet connections, in an attempt to get a good reading
- Discuss with the manufacturer whether it is permissable to do a pressure test on the system (pressure applied to both tubes), to check for gas-tightness. There may be a maximum allowable pressure.

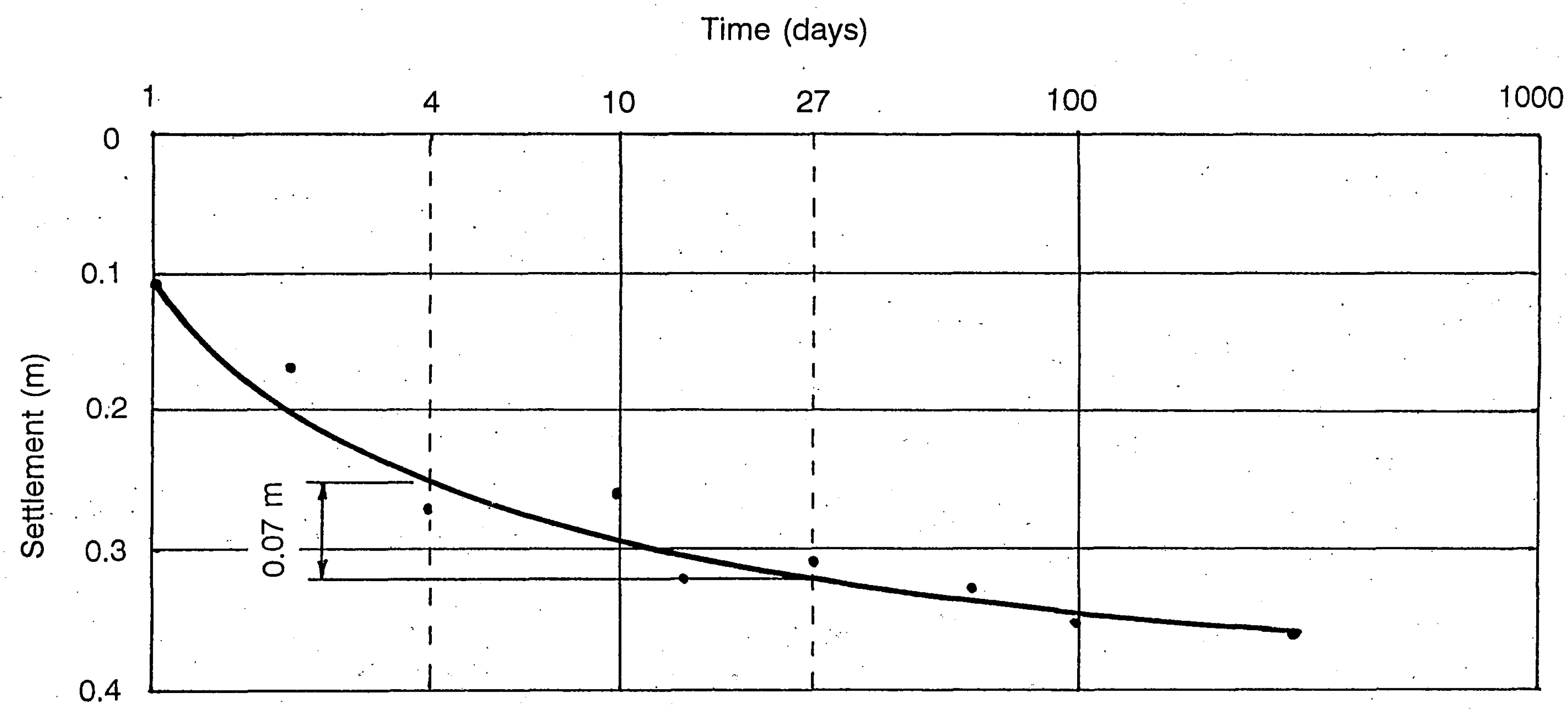


FIGURE F-1. OPTICAL SURVEY DATA

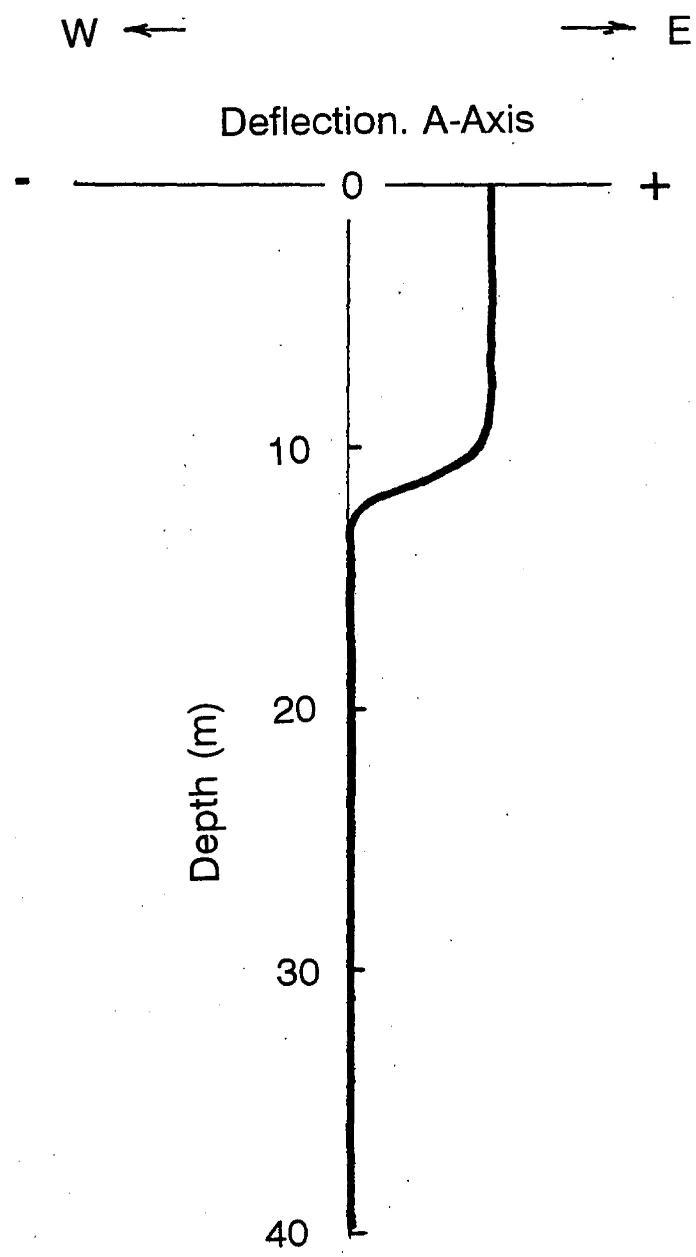


FIGURE F-2. CORRECT INCLINOMETER DATA