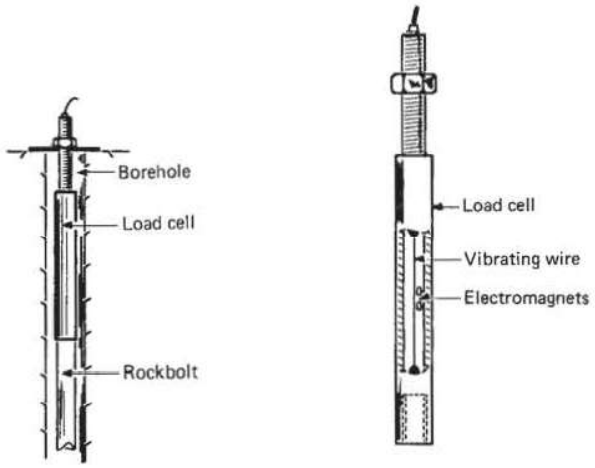


Figure 13.7. Schematic of vibrating wire rockbolt load cell (after Bellier and Debreuille, 1977).

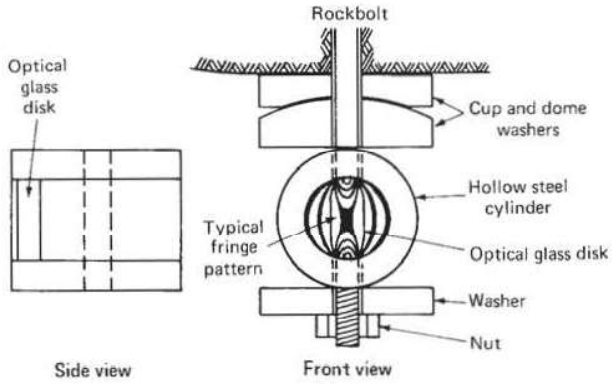


Figure 13.8. Schematic of photoelastic rockbolt load cell.

bolt of similar cross-sectional area, and miniature vibrating wire transducers are fixed within the core at required measuring points, using set screws or spot welds. The instrument manufactured by Irad Gage and Geokon is a complete expansion shell type rockbolt with a vibrating wire transducer mounted in a central longitudinal hole in the bolt near the free face. The gage is available with a contact point on the head of the bolt so that readings can be made with a hand-held probe, but difficulties are sometimes experienced with the contact, and hard-wiring to either a nearby or a remote terminal is preferable. When using any of the three versions of this instrument, individual calibrations are required.

Specially fabricated vibrating wire load cells can be used for measurement of load at the tips of drilled shafts and are classified in this book as contact earth pressure cells. They are described in Chapter 10.

Fellenius and Haagen (1969) describe a vibrating wire load cell capable of withstanding pile-driving forces. A patented feature was adopted to ensure that dynamic load was not transmitted directly through the wire clamping points during the large number of impacts while pile driving.

13.2.5. Photoelastic Load Cells

Photoelastic load cells (Roberts and Hawkes, 1979) have been used, primarily in Europe, for monitoring loads in tiebacks, rockbolts, piles, mine pillars, scaffolding, cables, and chains. They consist of a disk of optical glass, normally enclosed within the end of a hollow steel cylinder. When the cylinder is loaded diametrically, a stress is applied to the glass disk, and strain in the disk produces light and dark areas when the disk is illuminated with polarized light and observed through a hand viewer. The light and dark areas are referred to as *photoelastic interference fringes*. The change in the number of fringes is directly proportional to the change in load on the steel cylinder. Figure 13.8 shows a schematic of a photoelastic load cell, designed for use on rockbolts, that is also suitable for measuring loads in stranded tendon tiebacks (Liu and Dugan, 1972). Figure 13.9 shows the version manufactured by Perard Torque Tension Ltd.

Although a versatile technique, use of photoelastic load cells has been hampered by awkwardness of reading and lack of wide commercial availability, most cells being manufactured on a custom basis. It is suspected that most users prefer a more direct numerical reading method, thus reducing the commercial market for photoelastic cells. However, they are relatively inexpensive, robust, and reliable.

13.2.6. Calibrated Hydraulic Jacks

Calibrated hydraulic jacks (Figure 13.10) 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.

Reliance on the measured pressure in the jack

Figure 13.9. Photoelastic load cell (courtesy of Perard Torque Tension Ltd., Worksop, England).

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.

Figure 13.10. Calibrated hydraulic jack: 150 ton capacity, 6 in. stroke, with center hole (courtesy of Richard Dudgeon, Inc., Stamford, CT).

Davisson (1966) reports on pile load testing using calibrated jacks equipped with swivel heads, where actual load was determined by using high-quality electrical resistance load cells. Jack fluid pressure was also recorded. Error in using the jack calibration was erratic and ranged up to 10%. Davisson concludes: "Even under relatively good conditions, using a swivel head, we must conclude that the probable error is approximately 5%, and varies from zero to 10% erratically. Without the swivel head, the error is probably higher. The foregoing behavior probably explains the [erratic] load–settlement curves often observed for pile tests." Davisson recommends use of both a swivel head and a load cell. Fellenius (1980, 1984) reports on the use of a load cell and a calibrated hydraulic jack during pile load testing and indicates that during loading the jack typically overregisters by 10–25%, and during unloading it typically underregisters by 5%. Figure 13.11 shows typical errors.

Whenever comparisons are made between the two methods of load measurement, it should be remembered that load cells can be in error by 5–10% because of load misalignment, off-center loading, and end effects; therefore, the disagreement between hydraulic jack and load cell does not necessarily result entirely from inaccuracies in hydraulic jack data. Load cell inaccuracy should be minimized by following the guidelines given in Section 13.2.8.

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

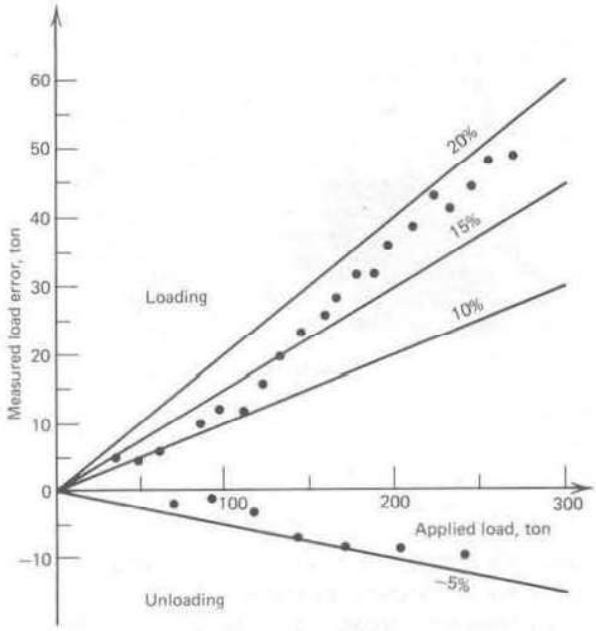


Figure 13.11. Typical error in load determined from fluid pressure in hydraulic jack (after Fellenius, 1984). Reprinted from *Geotechnical News*, Vol. 2, No. 4, December 1984.

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.

The following is a useful procedure for checking whether a hydraulic jack is functioning correctly:

1. Calibrate the pressure gage against a dead weight tester.
2. Measure the jack piston diameter (or circumference) and if possible check against the manufacturer's specification or general assembly drawing.
3. Determine the pressure/load calibration from steps 1 and 2.
4. Compare this calibration with the loads indicated by a load cell or testing machine, on loading and unloading cycles. The pressure/load calibration determined in step 3 should lie within the hysteresis loop from this step 4, usually nearer the readings made during the unloading cycle. If it does not, something is wrong.

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.

13.2.7. Cable Tension Meters

Vibrating wire, electrical resistance, and photoelastic load cells can be manufactured in a configuration suitable for measuring tension in cables and chains. Figure 13.12 shows a version with an electrical resistance transducer. Two additional devices are manufactured specifically for measuring tension in cables. Both are portable devices for use on cables that are already under tension, and measurements are made without detensioning the cable.

First, the Fulmer tension meter (Hanna, 1985)

Figure 13.12. Cable tension meter with electrical resistance transducer (courtesy of Strainsert, West Conshohocken, PA).

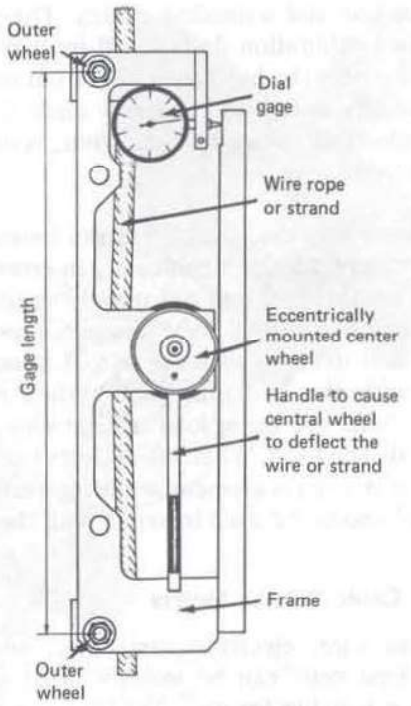


Figure 13.13. Fulmer tension meter (after Hanna, 1985).

depends on the relationship between the tension in the cable and the normal force required to cause a transverse deflection. As shown in Figure 13.13, a deflecting force is applied via an eccentric wheel, causing the frame of the meter to bend in proportion to the applied force. Bending of the frame is measured with a dial indicator and calibrated to tension in the cable. Calibration charts are required for each cable type and size. Accuracies of $\pm 5\%$ on cable and $\pm 3\%$ on wire are claimed. Second, the Roctest cable tensiometer (Figure 13.14) depends on the relationship between the tension in the cable and the travel velocity of a wave generated by impact. An impactor and geophone are attached to the cable 5 ft (1.5 m) apart, and the time for wave travel is measured. Given the dimensions and mechanical properties of the cable, tension can then be determined from a calibration chart. Accuracy is claimed to be better than $\pm 2\%$ when the tension is at least 15% of the yield strength of the cable, but accuracy decreases rapidly when tension is lower.

13.2.8. Guidelines on Use of Load Cells

General guidelines on instrumentation use are given in Part 4. The following additional guidelines apply specifically to load cell use.

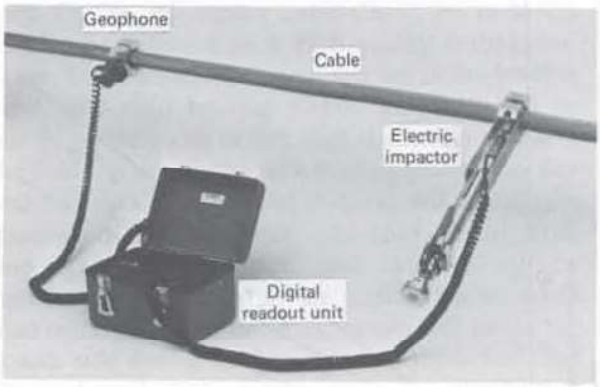


Figure 13.14. Cable Tensiometer (courtesy of Roctest Ltd., Montreal, Canada).

Cell Type, Configuration, and Capacity

Table 13.1 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. Many general-purpose load cells for measuring compressive loads have a solid center, the loads being applied to the cell through a raised center section incorporating a spherical seat to allow for initial lack of fit in the loading setup. This arrangement is not possible with hollow-core load cells, and lack of fit and eccentric loading must be minimized by using swivel bearing washers and/or soft washers. Vibrating wire and electrical resistance load cells must be gaged at several intervals around the periphery so that strains can be averaged to compensate for eccentric loading. End effects can be minimized by using large height/diameter ratios, but if the height is large the cell becomes cumbersome and difficult to use. Most commercially available hollow-core cells have a height approximately equal to the outside diameter of the hollow-core load member and appear to be free from significant end effects. However, there are many load cells on the market that have much smaller height/diameter ratios, their compact design being claimed as an advantage.

The working capacity of the selected load cell should be equal to at least the yield load of the structural member, unless the application or previous experience indicates that substantially less load is expected. The size of the load-bearing member within the cell should be selected so that its elastic limit is at least twice the working capacity. Unwanted capacity of course reduces sensitivity and should be avoided.

Pretreatment of Cell

Residual stresses are usually created in the load-bearing portion of the cell during fabrication. During subsequent stressing, these residual stresses may readjust such that measured strains do not reflect applied stresses, and measurements will be in error. The load-bearing member should preferably be stress relieved by heat treatment before gaging.

Calibration

Load cell calibrations should model field conditions as far as possible, using similar bearing plates, washers, and range of possible off-center loading and load misalignment. Specific factors relating to calibration of hydraulic jacks are discussed in Section 13.2.6. Littlejohn (1981) recommends that, when basic characteristics of a load cell are being established, consideration should be given to the following rigorous tests:

(i) Routine calibration using centric loading and rigid flat platens at 20°C, say.
(ii) As in (i) but using (a) concave inclined platens, (b) convex inclined platens, and (c) 0.3 mm [0.012 in.] thick shims with irregular spacing to simulate uneven bedding [Figure 13.15].
(iii) Eccentric loading between rigid flat platens, with eccentric distance up to 10% of cell diameter.
(iv) Inclined platens up to 1 degree with centric loading.
(v) On completion of the appropriate series of tests, the cell should finally be subjected to a repeat routine calibration (i).

It is emphasized that these rigorous tests are not suggested for routine calibration but as a means of evaluating an unproven design. Littlejohn (1981) makes recommendations for routine calibration of load cells, including the following:

- For routine calibration the load cell should be delivered to the laboratory at least one day before the test to permit sufficient time for the cell to attain the correct ambient temperature (20°C). The cell should be subjected to centric loading between rigid flat platens using a testing machine with an absolute accuracy better than 0.5%.
- Bearing in mind that the load cell may not have been used for some time, it may be prudent to load cycle the cell two or three times over its full loading range until the zero and maximum readings are consistent. [This is particularly important for electrical resistance load cells.] The load increments and decrements should not exceed 10% of the rated capacity of the cell, and short pauses at these intervals need only be long enough to take careful readings.
- To measure the specific effects of temperature, a centric loading test using rigid flat platens should be carried out at temperatures above and below ambient (20°C), say 40°C and 0°C, respectively.
- [When used to monitor load while stressing a structural member], load cells should be calibrated every 200 stressings or after every 60 days use, whichever is the more frequent.

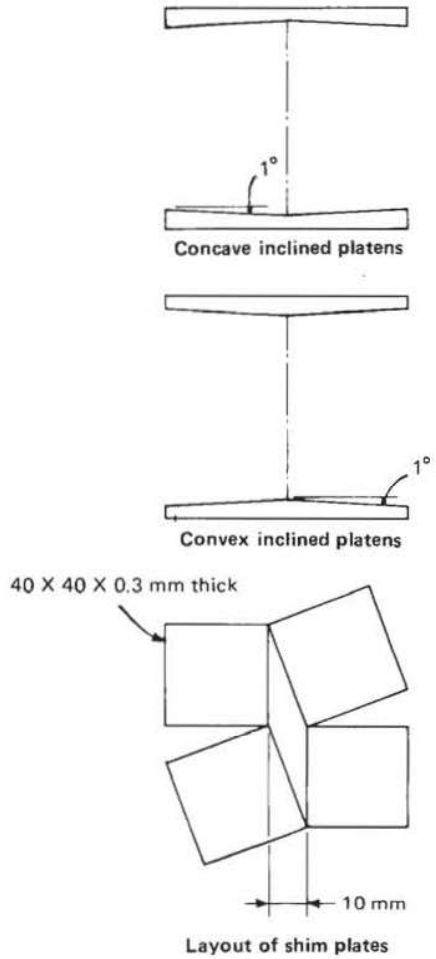


Figure 13.15. Typical types of platen to simulate uneven loading (after Littlejohn, 1981). Reprinted with permission from *Ground Engineering* (1981), Vol. 14, No. 3, p. 29.

Prior to accepting load cells for applications subjected to dynamic forces at some point in time, such

as measurements of static load within a driven pile, full-scale tests should be made to ensure that zero readings will not be changed by the dynamic forces.

When load cells are used for long-term measurements, provision should be made, if possible, for periodic checks on the zero reading. In many applications a *lift-off* test can be made by measuring the load required to free the cell from its bearing surface, using a hydraulic jack with a second load cell in series. Special arrangements for periodic check-calibrations of load cells installed on permanent tiebacks are discussed in Chapter 19.

Installation

Accuracy of load cell data is highly dependent on the design of the mounting arrangements, and good load cells can fail to provide adequate data if insufficient attention is given to these arrangements. Bearing plates should be sufficiently smooth, flat, and stiff. Load should be axial and concentric, and features to reduce the effect of misalignment should be adequate. Cells and any swivel bearings or soft washers should be aligned with the load-bearing member to within ⅛ in. (3 mm) if possible. Centralizer bushings are often useful for centering hollow-core load cells around tiebacks and rockbolts. A quick-set mortar pad can be used to provide a firm and square bearing surface for a rockbolt load cell. Keil and Hellwig (1987) present practical experience compiled over many years of load cell installation and indicate the most effective mounting methods available.

Where load cells are used on steel tunnel sets or on struts across braced excavations, the standard rib or strut will usually require modification to accommodate the cell. The cell may break the continuity of the structural member, and arrangements should be made to ensure that the cell will not kick out or allow buckling or shear deformations of the member. The load cell should be installed concurrently with the structural member rather than inserted later, thereby simplifying erection and alignment and ensuring that the total load history will be recorded.

Where load cells are used for temporary measurements in permanent supports, arrangements must be made for jacking to remove the load cell and insert permanent spacers, thereby allowing reuse of the cell. Of course, cells should be calibrated before reuse.

13.3. SURFACE-MOUNTED STRAIN GAGES

Strain gages in general use for measurements on structural surfaces are described in Sections 13.3.1–13.3.4 and compared in Table 13.2.

13.3.1. Surface-Mounted Mechanical Strain Gages

Portable gages with dial indicators are the most common mechanical gages for measurement of surface strain in geotechnical applications. Scratch gages are occasionally used. Telltales are also used as mechanical strain gages.

Portable Gages with Dial Indicators

Portable gages with dial indicators, also called *mechanical extensometers,* are available in various configurations. The *Demec* (*de*mountable *mec*hanical) gage (Base, 1955; Morice and Base, 1953) was developed by the Cement and Concrete Association in England and is shown on Figure 13.16. 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 200 mm (approximately 8 in.) gage length is typically used for field measurements. 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. Under ideal laboratory conditions a measurement accuracy of ±5 microstrain is possible, but under field conditions ±25–50 microstrain accuracy is more common. Details of the instrument, with procedures for disk installation, reading, and data calculation are given by ISRM (1984). Operating guidelines are also given by Cording et al. (1975). If maximum accuracy is required, extreme care must be taken during installation and reading. Cording et al. (1975) describe a temperature-compensated standard bar (Figure 13.17) in which the lengths of invar and steel are inversely proportional to their respective linear coefficients of thermal expansion. The separation of the reference points therefore does not vary with temperature, and the bar can be used to examine temperature and zero drift effects on the portable gage.

Table 13.2. Surface-Mounted Strain Gages

Gage Type	Advantages	Limitations	Typical Gage Length	Typical Range (microstrain)	Sensitivity (microstrain)	Approximate Accuracy (microstrain)
Portable dial indicator (e.g., Figure 13.16)	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	2–80 in. (50–2000 mm)	Up to 50,000	3–50	± 5–200
Scratch (e.g., Figure 13.18)	Inexpensive Self-recording Waterproofing not required	Requires access to structure Requires skill to read Strains must be dynamic and large	3–48 in. (75–1200 mm)	Up to 6000	30–300	±25–200
Multiple telltales	Simple Inexpensive	Requires access to telltales (but can be converted to remote reading device)	Unlimited	Unlimited	Depends on application	±25–400
Vibrating wire (e.g., Figure 13.20)	Remote readout Lead wire effects minimal Readout can be automated Factory waterproofing Arc welded or bolted version is reusable	Limited range Cannot be used to measure high-frequency dynamic strains Special manufacturing techniques required to minimize zero drift Need for lightning protection should be evaluated	2–14 in. (50–350 mm)	3000	0.2–2	±5–50
Electrical resistance (weldable and bonded foil) (e.g., Figures 13.22, 13.23, 13.24)	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	0.01–6 in. (0.25–150 mm)	20,000	1–4	±1–100

A gage with accuracy similar to that of the Demec gage is described by Kovari et al. (1977). The *Whittemore gage*, used primarily for strain measurements during laboratory testing of concrete, is significantly less accurate than the Demec gage.

Kovari et al. (1977) also describe a *curvometer*, designed to monitor the change in curvature of a straight or curved beam at a given point, so that bending strains can be determined. The device consists of a mechanical reference bar, with a dial gage mounted perpendicular to the bar at its midpoint. Use of the curvometer to monitor bending strains in

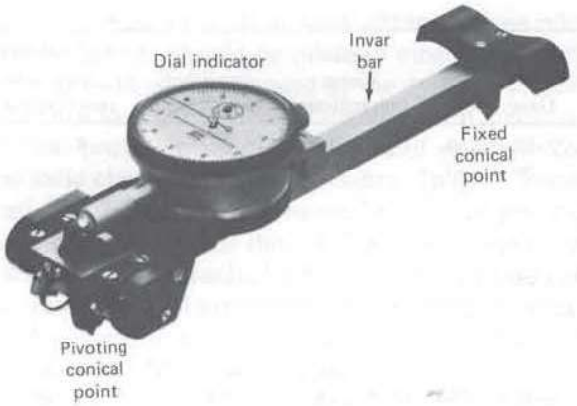


Figure 13.16. Demec strain gage (courtesy of W. H. Mayes & Son, Windsor Ltd., Windsor, England).

steel ribs and concrete segments for tunnel linings is described by Robinson et al. (1985).

Scratch Gages

Scratch gages provide a record of strain by scribing a line on a target. The target is attached to one end of the gage length, and a pointer attached to the other end bears on the target. A mechanical linkage causes the target to advance or rotate as strain occurs, so that the line scribed on the target by the pointer does not overlap a previously scribed line. The scribed lines are usually examined under a microscope, and irregularities are converted to strain.

The *Prewitt scratch gage*, shown in Figure 13.18, has a disk-shaped brass target. The two ends of the gage are clamped, bolted, or glued to the structure. The rotator device contacts the rim of the disk and causes it to rotate when compressive strain occurs, and a new disk can be inserted at any time. Scribe lines are interpreted by using a calibrated microscope or special optical comparator. The device is only suited to recording dynamic strains, and use is limited to situations in which strains are sufficiently large and dynamic so that the disk is turned automatically by the rotator device. The author attempted to use the gage for recording dynamic strains in struts across a braced excavation as a train moved along a railroad immediately adjacent to the excavation, but the disk did not rotate.

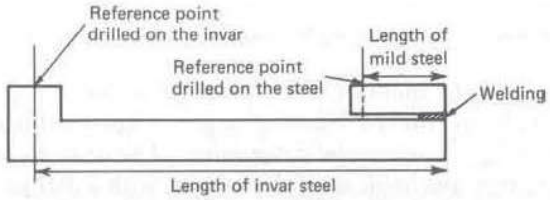


Figure 13.17. Temperature-compensated standard bar for portable strain gage with dial indicator (after Cording et al., 1975).

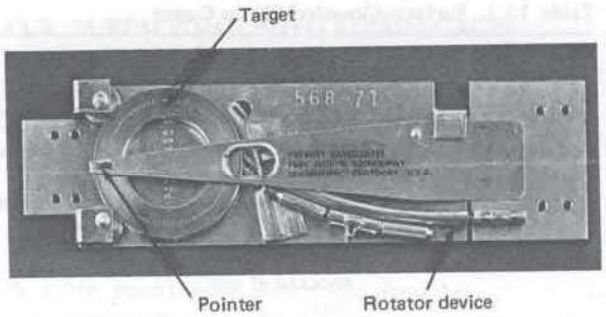


Figure 13.18. Prewitt scratch gage (courtesy of Eastlex Machine Corporation, Lexington, KY).

The scratch gage manufactured by Leigh Instruments Ltd. records strain data on metallic tape, stored in an interchangeable cassette attached to the gage. The primary application has been on aircraft structural components, and data are processed using a video display unit.

Telltales

A *telltale* is an unstressed wire, rod, or pipe, mounted alongside or within a structural member to indicate change in length of the member. Although a telltale is generally considered as an instrument for measuring deformation, change in length can be converted to strain, and the device is also classified in this book as a mechanical strain gage.

Figure 13.19 shows a telltale arranged to monitor load in a tieback anchor. This arrangement is also referred to as a *telltale load cell*. The telltale rod is inserted within a pipe sleeve, and one end is attached to the tieback near the lower end of the stressing length. The distance d between the upper end of the telltale rod and the anchor locking plate is measured, normally using a portable mechanical gage. The change in distance d is equal to ΔL, the change in stressing length L. Because the stressing length is acting as an elastic member under tension, strain is equal to $\Delta L/L$, and load change can be calculated since cross-sectional area and modulus of the tieback anchor are known. In practice, the relationship between ΔL and load change is determined in the field while proof testing the anchor, using a hollow-core load cell in series with the

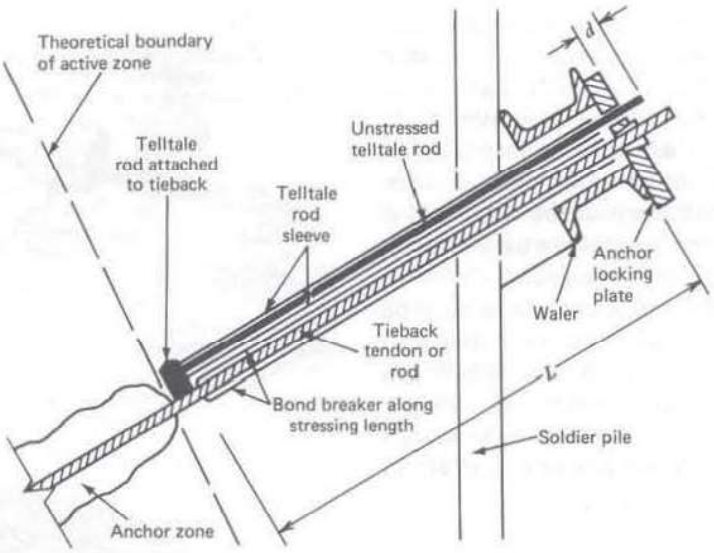


Figure 13.19. Schematic of telltale arranged for load measurement in a tieback anchor (*telltale load cell*).

stressing jack. During proof testing, the telltale rod can be shortened so that deformation measurements are made with respect to the waler. Alternatively, the rod can be extended outward through the jack and hollow-core load cell and deformation measurements made with respect to the pulling head, applying a correction for travel of the jack piston. The hollow-core load cell is also used to determine load at lock-off, and load changes are referenced to that datum. The fact that load is derived from deformation measurements sometimes raises concerns that slippage of the anchor or movement of the soldier pile will create errors in load measurement, but this is not so. The arrangement can be used to monitor change of load resulting from all causes and thus is an alternative to a hollow-core load cell at the anchor head. A similar arrangement, installed on the surface of a structure, can be used to monitor change of load.

A pair of telltales can be used to monitor strain. The arrangement is essentially the same as a multipoint fixed borehole extensometer (Section 12.7). For example, a second sleeved telltale can be attached to the tieback shown in Figure 13.19, within the anchor zone. The average strain between the two attachment points is given by:

$$\frac{\Delta L}{L} = \frac{(\text{change in } d_2) - (\text{change in } d_1)}{L},$$

where: d_1 = distance d for first telltale,
d_2 = distance d for second telltale,
L = distance between attachment points of the two telltales.

Therefore, average strain between the two attachment points can be monitored by measuring changes in the distances d_1 and d_2, and strains can be converted to approximate values of stress by using an estimate of the composite modulus of the tieback and grout within the anchor zone. By using multiple telltales, attached to the tieback at different points within the anchor zone, the pattern of load transfer in the tieback can be estimated. It is necessary, however, to assume the shape of the load transfer plot between each pair of 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 as much as ±0.02 in. (±0.5 mm). 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 12.106. Multiple telltales can also be used for determining the pattern of load transfer in driven piles and drilled shafts and for monitoring strain in other structures.

13.3.2. Surface-Mounted Vibrating Wire Strain Gages

The vibrating wire transducer is described in Chapter 8, and a schematic of a surface-mounted gage is shown in Figure 8.22. Chapter 8 also includes a discussion of techniques for minimizing errors caused by zero drift and by corrosion of the vibrating wire, and guidelines on cable selection. The surface-mounted gage is available in the two basic configurations illustrated in Figure 13.20.

The configuration designed for installation by arc welding (Figure 13.20a) or bolting is generally between 3 and 14 in. (75–350 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. Light tapping of the end blocks with a hammer will improve the initial stability of the gage. Most versions have O-ring seals between the protective tube and the gage end blocks to provide waterproofing and to allow the tube to remain unstressed. All versions permit adjustment of the initial wire tension, and some contain internal springs such that the vibrating wire is set by the manufacturer in midrange. This arrangement promotes gage stability, allows for easy acceptance testing on receipt from the manufacturer, and eases the task of setting the initial tension. Gage range is typically 3000 microstrain, corresponding to a stress change of approximately 90,000 lb/in.^{2} (620 MPa) in steel, and the initial tension should

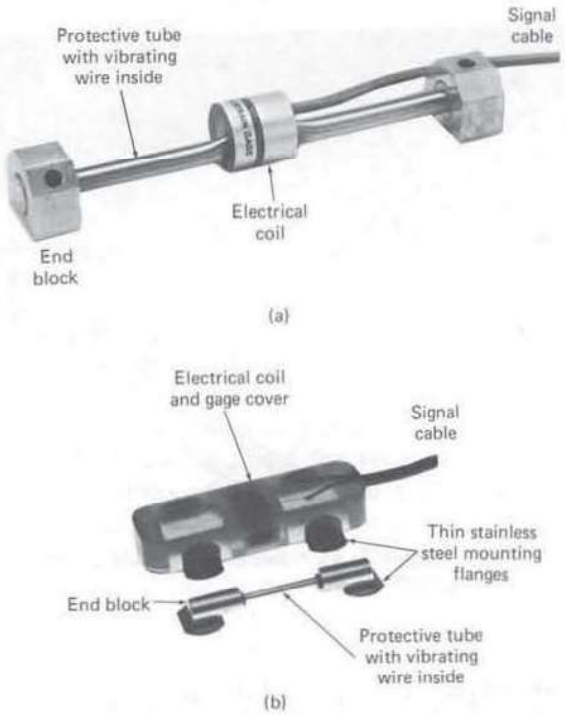


Figure 13.20. Surface-mounted vibrating wire strain gages: (a) gage installed by arc welding (gage can also be fitted with different end blocks for installation by bolting) and (b) gage installed by spot welding (courtesy of Geokon, Inc., Lebanon, NH).

therefore be set with care so that the wire will not become too slack or too tight under the anticipated tensile or compressive strains. Some versions allow the electrical coil assembly to be detached from the protective tube to allow for easier repair in the event of damage to the signal cable. After attachment of all gage components, a robust protective cover should be attached, and the signal cable should be protected, for example, by conduit.

The Norwegian Geotechnical Institute in Oslo has recently developed a method of installing the gage on the inside of a closed-ended pipe pile, using welded end blocks, thereby minimizing the chance of damage during pile driving. Four holes are drilled in the pile, on a straight line parallel to its axis. A setting tool is used to insert the gage within the pile and to locate end block stems through the first and last holes. The stems are held in place temporarily with a positioning jig and welded to the pile. The other two holes are used to set the initial wire tension, by using a screwdriver and allen wrench.

The arc welded gage is also available with two vibrating wires. The wires are at different distances

Figure 13.21. Capacitive discharge spot welder for installing weldable strain gages (courtesy of Eaton Corporation, Los Angeles, CA).

from the surface of the structure, so that bending strains can be monitored by attaching the gage to one surface only.

The spot welded gage is typically 2 in. (50 mm) long. The version shown in Figure 13.20b 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 capacitive discharge spot welder (Figure 13.21) to weld the mounting flange to the structure, with weld points at approximately 0.06 in. (1.5 mm) spacing. The surface is first prepared by removing surface paint, rust, and scale using a disk sander, with progressively finer grits, to expose the bare metal and remove pits. When welding, the weld current must be controlled carefully, following the manufacturer's recommendations. It is useful to practice welding and to demonstrate the suitability of the selected weld current by using test strips of the same material as the mounting flange. After welding, the weld points must be sealed with a waterproof coating to prevent later corrosion and gage slippage. Where access is available, the electrical coil and gage cover can be placed on the gage temporarily whenever a reading is required. However, they are normally spot welded in place and covered with a mastic compound as corrosion protection. A robust protective cover is usually installed over the gage, and the signal cable is normally installed in conduit.

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.

Most manufacturers attempt to match the linear coefficient of thermal expansion of the vibrating wire to that of structural steel, so that the gage is insensitive to temperature change when installed on steel. However, remaining differences in coefficients may be significant enough to require correction, particularly where the structural member is exposed to large temperature variations. Temperature should therefore be measured at the time of each reading, and calculated stresses should be corrected on the basis of correction factors, developed as discussed in Section 13.3.9. Most manufacturers provide gages with integral thermistors for temperature measurement, or alternatively the resistance change of the plucking coil can be calibrated to temperature change and used for temperature measurement.

Where gages or signal cables are installed in regions of high thunderstorm activity, lightning protection (Chapter 8) should be considered. If gages will be subjected to vibration, the user should ensure that zero drift will not be caused. Guidelines are given in Chapter 8.

13.3.3. Surface-Mounted Electrical Resistance Strain Gages*

Five basic types of electrical resistance strain gage are described in Chapter 8: bonded wire, unbonded wire, bonded foil, semiconductor, and weldable. Chapter 8 also includes a description and comparison of Wheatstone bridge circuits for use with electrical resistance strain gages, a discussion of cable selection, data acquisition and communication systems, gage and cable integrity testing, and general guidelines for the use of electrical transducers and data acquisition systems.

Both bonded foil (Figure 13.22) and weldable

*Written with the assistance of Given A. Brewer, Consultant, Brewer Engineering Laboratories, Teledyne Engineering Services, Marion, MA.

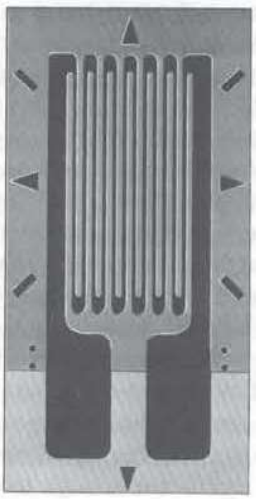


Figure 13.22. Bonded foil electrical resistance strain gage (courtesy of Measurements Group, Inc., Raleigh, NC).

(Figures 13.23 and 13.24) gages are used for surface strain measurements in geotechnical field applications. Bonded wire, unbonded wire, and semiconductor gages are used for special applications only and will not be discussed further in this chapter. When one is selecting between weldable and bonded foil 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.

Environmental Conditions

When instruments must be installed in the field, weldable gages are usually preferable to bonded gages. Weldable gages can be installed successfully by relatively inexperienced personnel, but installation of bonded gages in the field requires personnel highly experienced in field procedures. Even with experienced personnel, installation of bonded gages is limited to warm-weather conditions. Success depends on many painstaking steps, including surface

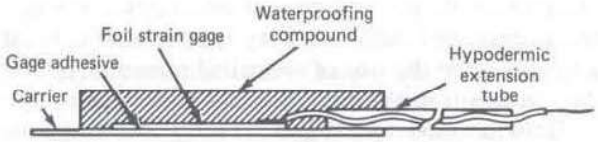


Figure 13.23. Weldable electrical resistance strain gage with bonded foil transducer (courtesy of HITEC Products, Inc., Ayer, MA).

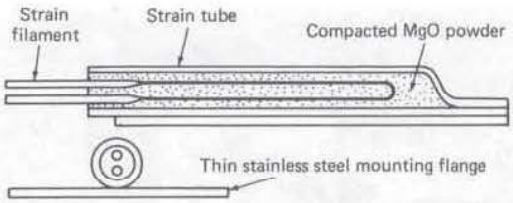


Figure 13.24. Weldable electrical resistance strain gage with strain filament encased in a small tube (courtesy of Eaton Corporation, Los Angeles, CA).

preparation, bonding, waterproofing, connection, and physical protection, and under field conditions success is difficult to attain. Installation steps are discussed later in this section. Successful installation is easier to achieve in a controlled indoor environment, but this also requires great experience. When bonded gages are installed by experts, they can be stable and reliable, but there are many case histories of unsuccessful installations by geotechnical personnel, and a *do-it-yourself* approach is not likely to lead to success.

Accuracy

When bonded foil strain gages for field use are installed in a controlled indoor environment by experts, they can have a short-term accuracy of ± 1 microstrain. However, this accuracy will be downgraded severely if skill is inadequate or if installation and field conditions are other than ideal. Accuracy of weldable gages depends on the resistance element. When a bonded foil transducer is used (Figure 13.23), typical accuracy is ± 5 microstrain; for the arrangement shown in Figure 13.24, typical accuracy is ± 15 microstrain.

Cost

A typical weldable strain gage costs 10–15 times as much as a bonded gage. However, installation costs are generally significantly less, and cost comparison depends on the application.

Size

Typical weldable strain gages are about 1 in. (25 mm) long, but special gages as small as 0.2 in. (5 mm) long are available. Bonded gages range in length from 0.01 to 6 in. (0.25–150 mm) and may therefore be preferred when available space is very limited.

Selection and Installation of Bonded Gages

Key factors when using bonded gages in a field environment include gage selection, selection of bridge circuit, and installation procedure.

When one is selecting the type of gage, the following must be considered: gage material, backing material, resistance value, strain limits, fatigue life, temperature range, temperature compensation, static or dynamic application, and use environment. Gages with integral terminals are preferred, so that the user solders lead wires to a tab rather than to the gage wires themselves.

Guidance on selection of a bridge circuit is given in Chapter 8. Many factors influence this decision, but the primary consideration is that gages must detect the desired strains and properly cancel or eliminate the unwanted strains. In all cases, the lead wire shield should be isolated from the structural member to which the gages are attached.

Selection of the installation procedure for bonded gages depends primarily on usage and gage type. Installation typically involves the following five steps:

- *Surface Preparation*. The surface usually must be degreased, sanded, etched, neutralized, and cleaned with a solvent and a chemical cleaner.
- *Bonding*. Bonding entails extreme care in handling the gage, application of a layer of suitable bonding cement with careful control over glue thickness, careful positioning, clamping under a controlled pressure, and usually curing under a controlled temperature. When bonded in humid environments, the surface of the structure must be heated to prevent formation of a moisture film. The *easy-to-use* cyanoacrylate contact cements, which cure at room temperature and do not require application of controlled pressure throughout the bonding period, are usually not suitable. When they are used, they should be purchased only from strain gage manufacturers and applied by experienced personnel.
- *Connection*. Interconnect wires are soldered to create the required Wheatstone bridge circuit, connections cleaned with a solvent, the gage and connection area coated with a protective and vapor barrier, the connectors heat treated, and lead wires soldered in place. Rosin core solder should be used.
- *Waterproofing*. The type, number of coatings, vapor barrier and lead wire sealer depend on the field environment and duration of the monitoring program. For a one to two week outdoor monitoring program a single layer of waterproofing compound may be sufficient. For a one to two month outdoor monitoring program the layer should be covered by aluminum foil and in turn covered by a second layer of waterproofing compound. For longer term programs, more comprehensive waterproofing is required, and a multi-layered patented procedure has been proved (Brewer, 1972): waterproofing compound, allowed to cure; waterproofing compound extending beyond the first layer; stainless steel foil spot welded through unset waterproofing compound; second layer of waterproofing compound allowed to cure; third layer of waterproofing compound extending beyond the stainless steel foil.
- *Physical Protection*. Protection will generally be a steel cover plate or box or a fiberglas coating.

As the installation of gages proceeds, they should be tested functionally and checked for insulation, and if possible a leak detection test should be made after waterproofing is complete. Gage resistance testing, insulation testing, and leak detection testing are described in Chapter 8. All data should be recorded and maintained as initial test records. Strain gage manufacturers will normally provide standard installation procedures but will not necessarily have sufficient experience with field installations. If in doubt, the guidance of field specialists should be sought.

When using bonded strain gages for measuring surface strains on concrete or rock, moisture will cause gage deterioration unless special precautions are taken. All loose surface material should first be removed and the surface cleaned with a wire brush and a chemical cleaner. A room-temperature curing epoxy should be applied liberally, thus ensuring the absence of air bubbles, and allowed to cure. The epoxy should then be sanded, to create a smooth surface, and checked for absence of air bubbles; the gage should be bonded to the epoxy and waterproofed as described above.

Bonded strain gages can be attached to a spring steel strip to form a *clip-on* arrangement that is reusable. Fang and Koerner (1977) describe a clip-

on gage for measurement of crack openings in structures while the structures were subjected to vibration.

Selection and Installation of Weldable Gages

Weldable gages are restricted to installation on steel, and the installation procedure is similar to the weldable vibrating wire gage procedure described in Section 13.3.2. Gages with integral leads and factory waterproofing are most suitable. Quarter bridge gages are available with integral three-wire leads. Half bridge gages are also available, with one active arm and one dummy.

13.3.4. Miscellaneous Surface-Mounted Strain Gages

Cragg (1984a) describes various strain gages for use on geotextiles in reinforced embankments. Simple gages were used by the U.S. Department of Agriculture, Forest Service (Steward et al., 1977; Vischer, 1975), consisting of different lengths of electrical wire attached to a standard gage length on the geotextile. Wire lengths were chosen such that breakage occurred at strain levels in the geotextile of 1, 3, 5, 10, and 20%, and an ohmmeter was used to monitor circuit continuity. Similar gages, using platinum wire and a copper tube, were used in Holland (Leeuwen and Volman, 1976), by clamping the wire in the tube so that a force of 2 lb (9 N) was needed to break the connection. Neither of these gages provided continuous strain readings. Cragg (1984a) and Rowe et al. (1984a) describe a hydraulic strain gage developed to provide continuous strain readings. The gage (Figure 13.25) is similar to a large-diameter, liquid-filled hypodermic syringe, connected to a small-diameter sight tube. A small movement of the large-diameter piston produces a large movement of the liquid level in the sight tube. The magnification factor is about 12. The gages were fitted into wood blocks that were both clamped and glued to the geotextile, to monitor strains over a gage length of 8 in. (200 mm). Dummy sight tubes were installed to correct for temperature differences between the gages and the remote monitoring station. The gages are applicable where strain is expected to be up to about 5% and have been used successfully for monitoring tensile strains up to 20%. Performance data are given by Rowe et al. (1984a, 1984b). The soil strain gage (Section 12.6.4) was also used, but Cragg (1984b) comments that because the gage is sensitive to tilting out of the initial coplanar arrangement, it should not be used for monitoring strains in geotextiles of less than 1%.

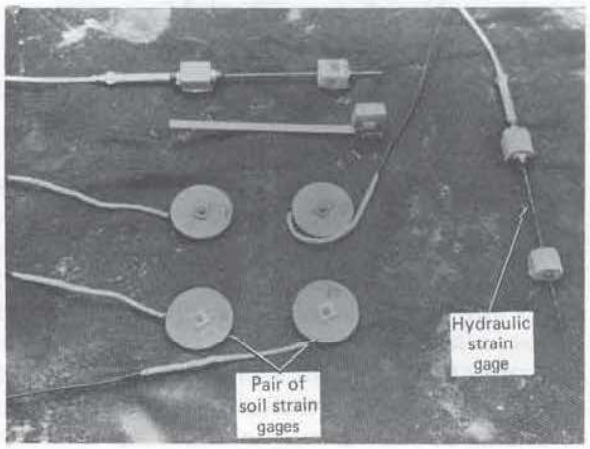


Figure 13.25. Hydraulic strain gages and soil strain gages attached to woven geotextile fabric (after Rowe et al., 1984a).

A demountable electrical gage, called a *demountable extensometer,* is manufactured by Cambridge Insitu in England. The gage is similar to the Demec mechanical strain gage (Section 13.3.1) but with an electrical output. Precision is claimed to be ±5 microstrain but, as for the Demec gage, under field conditions ±25–50 microstrain is likely to be more common.

A special gage has been used successfully to monitor axial and bending strains in timber piles. The arrangement consists of electrical resistance strain gages bonded to the outside of a 0.25 in. (6 mm) aluminum alloy tube, which is then embedded in epoxy in longitudinal grooves routed along the piles.

Photoelastic strain gages (Roberts and Hawkes, 1979) depend on the principle that polarized light incident on certain types of plastics and glass will produce patterns of light and dark areas, referred to as *photoelastic interference fringes*. The number and position of these fringes changes in response to the strain in the material to which the gage is attached, and the magnitude and direction of strain can be determined from an analysis of the fringe patterns. Photoelastic materials are used extensively for model studies, and bonded photoelastic strain gages are available for surface mounting. However, these gages are subject to substantial errors because of creep or deterioration of the bonding cement and are rarely used for geotechnical applications.

Various electrical crack gages (Section 12.3.2) can be used for measurement of surface strain.

13.3.5. Selection of Strain Gage Length, Cable, and Connectors

The selection of strain gage length for measurements on steel depends on the purpose of measurements, gage type, available space, and access. In general, unless very localized measurements are required, gages should be as large as possible. For measurements on concrete the gage length should be long enough to average out strain differences in the constituents of the concrete, as aggregate will generally strain less than the cement matrix. If possible, the gage length should be at least five times the maximum size of aggregate.

Selection of cable and connectors should follow the guidelines given in Chapter 8.

13.3.6. Locations of Surface-Mounted Strain Gages

The location of strain gages depends on the purpose of the measurements, available access, and on protection requirements during and after installation. If stresses at particular points are of interest, the gages should be located at those points. However, measurements are often required at locations that are not influenced by uneven or eccentric loadings or by the proximity of irregularities such as holes in the structural member. For example, if end effects are to be avoided, gages should not be located near the ends of the structural member or near irregularities. When gaging struts across braced excavations, gages are normally located a minimum distance from the end of the strut equal to three times the depth of the strut and a similar distance from any interbracing connections and points of future welding or flame cutting. On tunnel ribs, gages should preferably be a minimum distance from connections, tie-rod holes, and blocking points equal to three times the depth of the rib.

Figure 13.26 shows typical strain gage locations on the cross section of various structural steel members. If the member is small and not subjected to bending (e.g., a threadbar tendon for tiebacks), a single gage may be sufficient, although two are usually more prudent because in most applications the member will be subjected to both axial and bending stresses. If axial stress alone is required, a minimum of two gages must be used, equidistant from and on opposite sides of the neutral axis. If maximum stress is required and if the member may be subjected to bending about any axis, more than two gages must be used. On a pipe pile or pipe strut, a minimum of three gages spaced 120 degrees apart is recommended for determination of maximum stress. Four gages spaced 90 degrees apart provide a worthwhile redundant measurement. On a driven H-pile, four gages are recommended, back-to-back on each side of the web and as near the flanges as possible. Gages at these locations can usually be protected adequately, using welded channels, angles, or corner strips. On large I-beams, four gages can be located as for an H-pile, but the accuracy of strain estimates in the flanges is often limited by the relative flimsiness of the web. It is usually better to install four gages on the inner faces of the flanges as far from the web as possible, providing a good compromise between a protected location and the zone of maximum stress. For sheet piles in tension, for example, for cofferdams, a pair of gages midway between the clutches is appropriate. For sheet pile walls in bending, pairs of gages on each of the outer portions of the Z-shape are recommended.

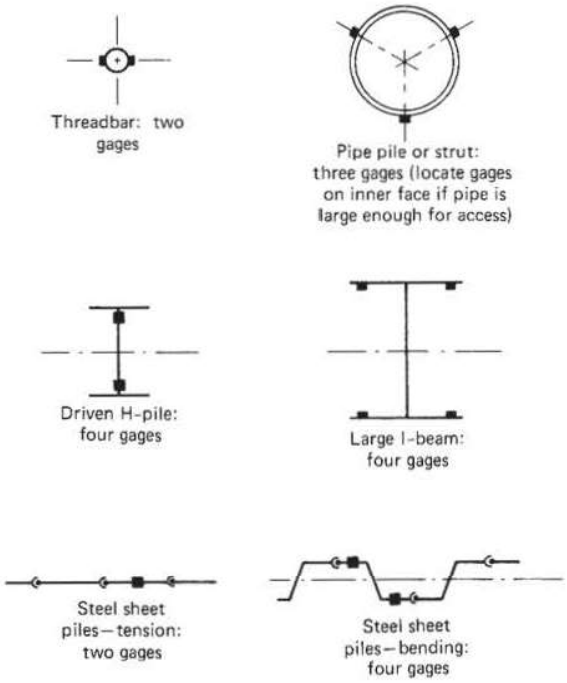


Figure 13.26. Typical locations of strain gages on structural steel members.

It should be remembered that the stress at any point in the cross section is the sum of the axial stress and the stress caused by bending, that stresses at the outer edges of the member can therefore be very much higher than stresses at the neutral axis, and that failure can consequently be initiated at the outer edges. Thus, two strain gages are rarely sufficient for providing a forewarning of failure.

Where a structural member is subjected to bending and only one surface is accessible—for instance, a steel tunnel lining or the outside of sheet piling—bending strains can be monitored by installing the type of vibrating wire gage with two vibrating wires at different heights above the surface of the member.

13.3.7. Installation of Surface-Mounted Strain Gages

General guidelines on the installation of instruments are given in Chapter 17. Methods of strain gage installation have been outlined in the above sections describing the various types of gage. The following additional guidelines apply to the installation of all types of surface-mounted strain gage.

Wherever possible, gages should be installed when the structural member is in an environment conducive to maintaining the quality of work, remote from pressures of construction progress, such as in a steel storage yard or warehouse. The work area should be protected from adverse weather conditions. Gages should be installed a minimum of 4 days before erection of the structural member to allow time for stabilization prior to taking initial readings.

Most strain gages and cables require protection from water and from mechanical damage caused by normal construction activities, vehicular traffic, and vandalism. Gages usually require protective cover plates, welded, bolted, or epoxied to the structure. Cover plates should not stiffen the structure and should not be tightened or welded so as to distort the structure and cause spurious strains at the gage location. Use of a stiff cover plate, attached to the structure at one end only, with mastic between the structure and cover plate, usually overcomes this problem. Cutting or welding can create significant stress changes in steel and may damage electrical cables; therefore, welding for attachment of cover plates should be minimized.

Gage cables are usually vulnerable to damage and must be protected by careful selection of location, by placement in conduit, or by embedment.

13.3.8. Initial Reading

General guidelines on initial readings are given in Chapter 18. Selection of initial readings depends on the purpose of the measurements. As an example, if actual stress in the structural member is required, initial readings should be made when the member is under no load, perhaps when supported evenly on a flat surface in a steel storage yard or warehouse. However, if gage readings are to be used for determination of load imposed on the structural member, initial readings should be made after erection of the member.

13.3.9. 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.

Causes of Strain in Concrete

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 prestressed concrete piles, creep strains are particularly significant.

*Written with the assistance of Stanley L. Paul, Associate Professor, Department of Civil Engineering, University of Illinois at Urbana–Champaign, IL.

The importance of creep and shrinkage is related to the duration of the measurement program. For a program lasting a few days, their influence may be small, but for a duration of several weeks or months, they will be much more important. Creep and shrinkage are influenced by mix design parameters, type of aggregate, fineness of the cement, and conditions during and after set, such as moisture content and temperature. The importance of creep and shrinkage is also related to stress magnitude. When stresses are greater than about 60% of the ultimate strength of the concrete, their importance increases substantially. A general discussion of the influences of factors that cause strain in concrete, and the conditions under which they are important, is given by Neville (1981).

Methods of Converting Strain to Stress

To avoid repetition in the later section describing embedment strain gages, the scope of this subsection includes strain measurements made with surface-mounted and also embedment strain gages.

The behavior of a concrete member is often predicted by measuring strains on the surface of the member or on reinforcement within the member. Stresses are then calculated by using the method of transformed areas, requiring knowledge of cross-sectional areas and moduli of steel and concrete, and these stresses are used to calculate axial loads and moments. This approach suffers from two major limitations. First, significant error may result from uncertainty in the modulus of the concrete. The modulus may vary from point to point because of variations in concrete properties that are caused by air voids, trapped moisture, and variation in compaction. This is particularly true of cast-in-place concrete. Second, the approach does not account for strains other than those caused by stress. A special effort must therefore be made to determine the relationship between strain and stress. The options are as follows:

1. 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 or shaft butt, 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 or shaft butt should therefore be kept wet if the subsurface gages are below the water table.
2. 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 tip of a pile or drilled shaft, or a load cell can sometimes be installed at a splice in a driven pile.

If neither of the above options is possible, three other approaches can be considered:

1. Use of creep curves developed for each particular project (e.g., England and Illston, 1965; Jones, 1961). These curves enable creep strains to be estimated from the age of the concrete at the time of loading and the duration and magnitude of loading. These strains are then subtracted from measured strains to obtain strains that result from stress.
2. Use of averaged empirical data that relate creep and shrinkage to environmental and mix parameters (e.g., ACI, 1982).
3. If the cross-sectional area is large enough, measure stress directly by using *concrete stress cells,* as described in Section 13.6.

It should be noted that the *no-stress* method, often used in large mass concrete structures, is not recommended. In this method a strain gage is installed in a cube of concrete and a gap is formed around the four sides and the top, so that the cube is in contact with surrounding concrete only at its base. Strain at the gage is assumed to result from all causes other than stress and is subtracted from strain readings of

the active gages. However, this does not account for strain of the active gages caused by creep under stress. It also does not account correctly for shrinkage, because the size of the cube is so different from the structure that migration of moisture is entirely different.

Temperature Effects

Temperature effects on the gages themselves have been discussed in earlier sections describing the various types of strain gage. However, as discussed in Chapter 14, temperature change can cause a real stress change if the structural member is subjected to end restraint, and it may be necessary to apply a correction factor to account for temperature effects. Two cases will be considered: thermally matched strain gages and gages that indicate actual length change.

First, consider a strain gage that is thermally matched to the structural member: for example, a temperature-compensated electrical resistance strain gage (one for which the change in resistance caused by thermal strain of the member is compensated for by change in resistance caused by temperature change in the strain gage material) or a thermally matched vibrating wire strain gage (one for which the thermal coefficients of the vibrating wire and structural member are equal) attached to steel. In these cases the gage will respond only to strain changes that result from stress changes, and the stress changes can be calculated from the change in strain gage reading multiplied by modulus of elasticity. This rule is independent of end restraint conditions and applies for all causes of stress in an elastic member: change in applied loading, stress caused by temperature change in the member, and a combination of both. Figure 13.27 shows the example of thermally matched strain gages installed on a steel strut. For both conditions illustrated, and for any intermediate condition, stress changes can be calculated from the change in the strain gage reading, without the need for temperature corrections.

Second, consider a strain gage that is used to indicate actual length change, as would be the case with a mechanical strain gage referenced to the temperature-compensated standard bar illustrated in Figure 13.17. The thermal characteristics of the gage are now not matched with the member. Actual length change results both from temperature change and stress change, and the measurement must be

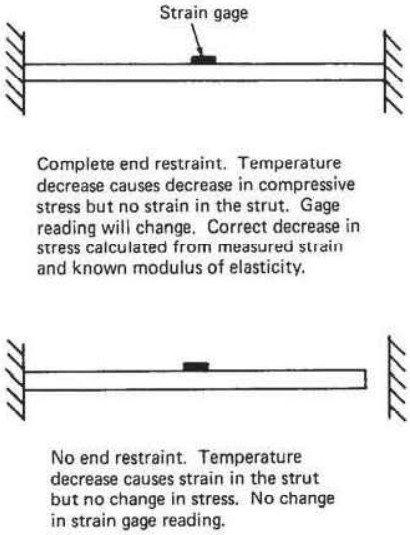


Figure 13.27. Behavior of strain gages as temperature changes: gages thermally matched to structural steel member.

corrected for any strain caused by temperature change:

$$\text{total strain} = \frac{\Delta\sigma}{E} + \varepsilon\Delta T$$

where: $\Delta\sigma$ = stress change,
ΔT = temperature change,
E = modulus of elasticity of steel,
ε = linear coefficient of thermal expansion of steel.

Therefore, in this case it is necessary to measure temperature of the steel and apply a correction when calculating stress change. As an example of the importance of such corrections, the author has used a mechanical strain gage for stress determination in 24 in. (610 mm) diameter, 0.5 in. (13 mm) wall thickness pipe struts across an internally braced excavation. Design load was 520 kips (2.3 MN). An error of 2°C in temperature measurement (corresponding to an error of 23 microstrain) caused an error of 25 kips (110 kN) in the calculated strut load.

In cases where the user is uncertain whether the gage is thermally matched, whether the gage indicates actual length change, or whether an intermediate condition exists, a temperature correction factor can be developed by making strain measurements at various temperatures in the laboratory (Cording et al., 1975). The gage should be mounted on the same type of material as it will be mounted

on in the field, and the test must be conducted in a way that avoids bending and other strains that are not related to temperature. If the gages are mounted on concrete, the moisture conditions must be maintained the same as in the field. Corrections to field readings can then be made by measuring temperature in the field and applying the correction factor. Alternatively, a temperature correction factor can sometimes be obtained by use of the *dummy gage* procedure, in which a gage is mounted on a specimen of the same material as the structural member and the specimen placed near the active strain gages in the field so that it will experience the same temperature but no stress. The dummy gage is read at the same time as the active strain gages, and its change in strain is used to correct the active gage readings. The dummy gage also serves as an indicator of any strain caused by factors other than stress and temperature. Several dummy gages should be used to obtain average corrections, since gages are unlikely to have identical characteristics.

An additional temperature error can occur if the strain gage and the structural member are at different temperatures. If possible, readings should be taken when temperature is uniform throughout the member. If the member is subjected to direct sun, readings should be taken on cloudy days whenever practicable. Shading or other insulation is often worthwhile and should extend to 1 ft (300 mm) on either side of the gage. White paint over gage covers and the nearby area can reduce the problem. When readings must be taken on sunny days, they are best taken in the early morning.

Zero Drift

A discussion of the zero drift potential of vibrating wire transducers is included in Chapter 8.

Zero drift of vibrating wire and electrical resistance strain gages can be examined by either of two methods. First, the dummy gage procedure, described above, can be used. When using this procedure, readings should ideally be taken with the dummy gage at the same temperature as the structure, so that temperature strains are not a factor. However, this is often not possible, and attempts must be made to correct for temperature strains by measuring temperature of the dummy gages and applying temperature correction factors developed in the laboratory. Second, a *check gage* can be placed in a constant-temperature environment.

If the gaged structural member is used for temporary ground support, *no-load* gage readings should be taken after removal of the structural member and compared with the initial no-load readings. This procedure of course does not provide data for correcting gage readings at the time they are taken, nor does it provide data on any variation of drift with time, but it can give an idea of any total drift that has occurred. The data may not be exact, because any permanent yielding of the member will affect the measurements.

In cases where access for mechanical readings is available and where accuracy of a mechanical strain gage is adequate, an estimate of zero drift at any time can be made by making duplicate measurements with a mechanical strain gage.

Zero drift can be assumed to be common to a gage type, model, and batch number and can be measured or estimated and applied to all gages. However, any abnormal behavior of individual gages owing to their own peculiarities will invalidate this approach.

Although zero drift can be examined by use of the above procedures, the overriding issue is to minimize the problem by gage selection and installation methods that promote low drift characteristics.

Summary of Guidelines for Converting Measured Strain to Stress

The following outline summarizes the guidelines given in this section.

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.
- Use measurement systems with low zero drift characteristics. If zero drift is judged to be a possible problem, use *dummy gage* procedure, *check gage* at constant temperature, and/or check the readings with a mechanical gage.

Measurements on Steel

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

Measurements in or on Concrete

Use one of the following six options.

- Subject specimen of same concrete to same influences as prototype, in known stress field, and measure strain. Preferably, the specimen should be in the field, but sometimes measurements are possible in the laboratory. Concrete must have the same moisture conditions as active gages in the field. Use the strain/stress relationship determined from the specimen and any necessary temperature correction factors.
- Interpose a load-measuring device across at least one cross section, relating load to measured strains.
- Use creep curves developed for each particular project.
- Use averaged empirical data.
- Use concrete stress cells.
- Use combinations of the above options.

13.4. 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 Sections 13.4.1–13.4.4 and compared in Table 13.3.

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.

13.4.1. Embedment Strain Gages: Mechanical Type

The mechanical type of embedment strain gage is identical in principle to the multiple telltales described in Section 13.3.1. They are sometimes used for monitoring strain when load transfer data are required while load testing driven piles and drilled shafts.

13.4.2. 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 13.3.2, except that large end flanges (Figure 13.28) replace the end blocks. Most gages are supplied with a preset tension, which can be specified by the user.

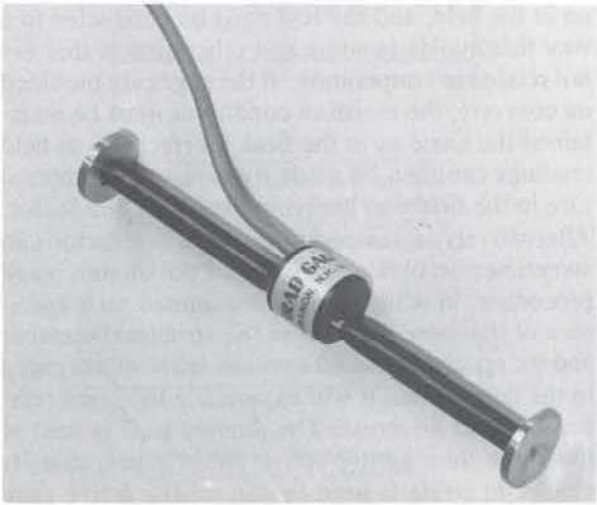


Figure 13.28. Vibrating wire strain gage for embedment in concrete (courtesy of Irad Gage, a Division of Klein Associates, Inc., Salem, NH).

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*, presumably because the gaged bar exists alongside and behaves similarly to the reinforcing steel in the structural member. It is also referred to as a *rebar strainmeter*. Figure 13.29 shows a sister bar suitable for installation in a drilled shaft, to provide strain data during a load test. Guidelines for minimizing the *inclusion effect* of sister bars are given in Section 13.4.7.

13.4.3. Embedment Strain Gages: Electrical Resistance Types

Embedment strain gages with electrical resistance transducers can be grouped into five categories:

Bonded Foil or Weldable Resistance Gages Attached to Central Portion of Length of Steel Bar

These gages are also referred to as *sister bars*. Two or four foil or weldable gages are installed on the central portion of a length of steel bar, such as reinforcing steel, and connected in a half or full bridge

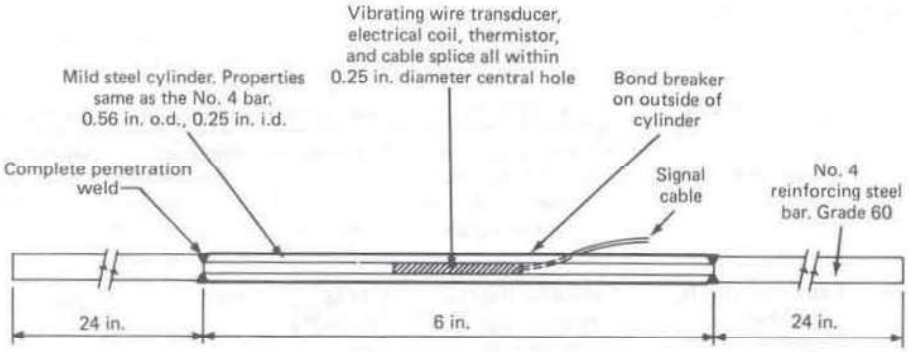
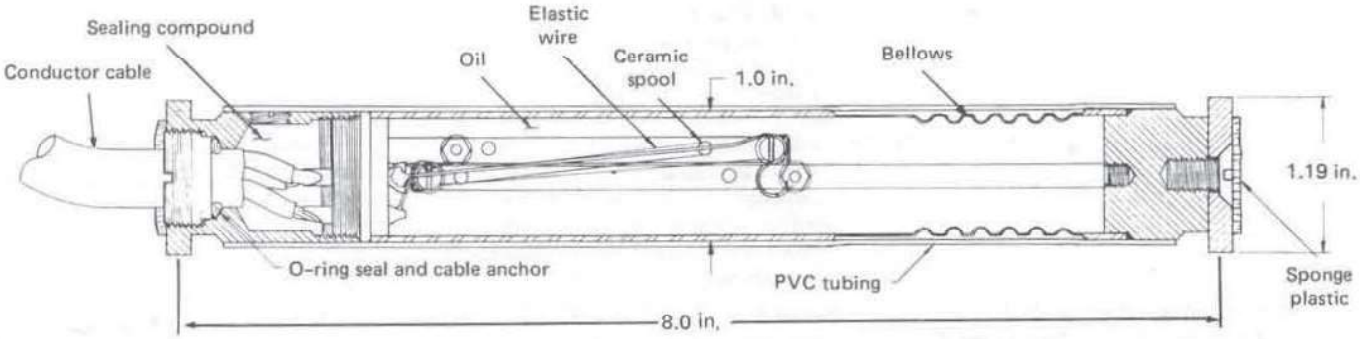


Figure 13.29. Schematic of *sister bar* with vibrating wire transducer.

network. Guidelines on minimizing the *inclusion effect* of sister bars are given in Section 13.4.7.

Unbonded Gages

The primary unbonded strain gage transducer for geotechnical applications is the Carlson *elastic wire strain meter*, shown in Figure 13.30. The instrument is stabilized by heat treating the resistance wire during manufacture, by filling with oil so that the resistance wire is totally immersed at all times, and by hermetically sealing to prevent the entrance of moisture. Accuracy is maximized and temperature effects are reduced by using two separate lengths of elastic wire, one of which increases in resistance with strain, while the other decreases, and by



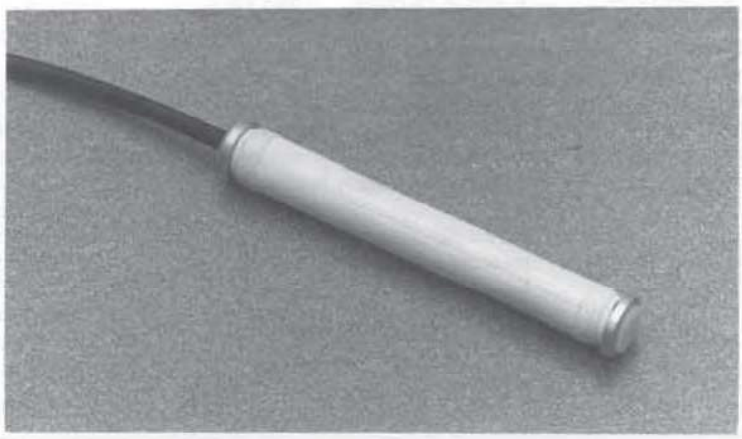


Figure 13.30. Carlson elastic wire strain meter (courtesy of Carlson Instruments, Campbell, CA).

Table 13.3. Embedment Strain Gages

Gage Type	Advantages	Limitations	Typical Gage Length	Typical Range (microstrain)	Sensitivity (microstrain)	Approximate Accuracy (microstrain)
Multiple telltales	Simple Inexpensive	Requires access to telltales (but can be converted to remote reading device)	Unlimited	Unlimited	Depends on application	±25–400
Vibrating wire; type similar to arc welded surface-mounted gage (e.g., Figure 13.28)	Lead wire effects minimal No conformance problem Remote readout Readout can be automated Factory waterproofing	Cannot be used to measure high-frequency dynamic strains Special manufacturing techniques required to minimize zero drift Need for lightning protection should be evaluated	5–10 in. (130–250 mm)	3000	0.2–2	±5–50
Vibrating wire; *sister bar* type (Figure 13.29)	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 manufacturing techniques required to minimize zero drift 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	Debonded length, plus 100× bar diameter	3000	0.2–2	±5–50
Bonded foil or weldable resistance gage; *sister bar* type (similar to Figure 13.29)	Robust Easy to install Suitable for monitoring dynamic strains Remote readout Readout can be automated Factory waterproofing	Special design features required to minimize inclusion effects Low electrical output Lead wire effects Errors owing to moisture, temperature, and electrical connections are possible Need for lightning protection should be evaluated	Debonded length, plus 100× bar diameter	20,000	1–4	±1–100

Table 13.3. *(Continued)*

Gage Type	Advantages	Limitations	Typical Gage Length	Typical Range (microstrain)	Sensitivity (microstrain)	Approximate Accuracy (microstrain)
Unbonded resistance (e.g., Figure 13.30)	Long good performance record No conformance problem Provides temperature measurement Remote readout Readout can be automated Factory waterproofing Suitable for monitoring dynamic strains but only up to about 25 Hz	Low electrical output Lead wire effects Errors owing to moisture, temperature, and electrical connections are possible Need for lightning protection should be evaluated	4–20 in. (100–500 mm)	3000	1.5–6	±20–75 (±2% full scale)
Mustran cell (Figure 13.31)	Robust Modulus matched Remote readout Readout can be automated	Large size Not available commercially Low electrical output Lead wire effects Errors owing to moisture, temperature, and electrical connections are possible Need for lightning protection should be evaluated	6–10 in. (150–250 mm)	1000	1–4	±10–50
Plastic encased gage (e.g., Figure 13.32)	Low cost Robust Easy to install No conformance problem Remote readout Readout can be automated Suitable for monitoring dynamic strains	Unstable in long term Low electrical output Lead wire effects Errors owing to moisture, temperature, and electrical connections are possible Need for lightning protection should be evaluated	0.5–10 in. (13–250 mm)	20,000	1–4	±10–50
Eaton Corporation gage (Figure 13.33)	No conformance problem Remote readout Readout can be automated Factory waterproofing Suitable for monitoring dynamic strains	Must be cast in a briquette before installation Low electrical output Lead wire effects Errors owing to moisture, temperature, and electrical connections are possible Need for lightning protection should be evaluated	2–6 in. (50–150 mm)	20,000	1–4	±10–50

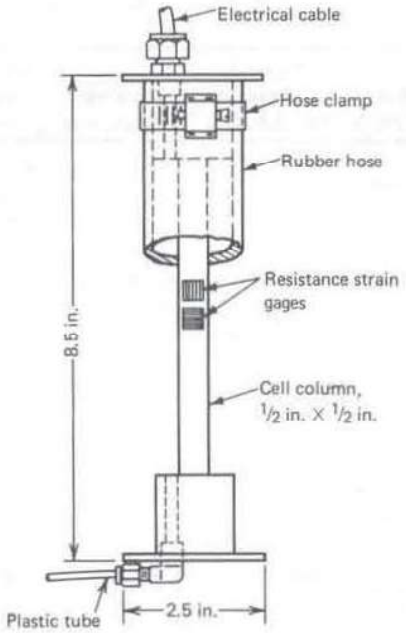


Figure 13.31. Mustran cell (after Barker and Reese, 1969).

measuring the ratio of the two resistances. It has a more than 40 year good performance record and is the first choice of many users for measuring strain within concrete. Variations of the Carlson elastic wire strain meter are the *joint meter*, for measuring movements across joints in concrete, and the *miniature strain meter*, for use in model studies. Details of the instruments, including installation and reading procedures, are given by Carlson (1975). Wiring diagrams are given by the Corps of Engineers (1980).

Mustran Cell

The *Mustran cell* was developed by personnel of the University of Texas under sponsorship of the Center for Highway Research (Barker and Reese, 1969), primarily for measuring strains in drilled shafts. The name is derived from *mu*ltiplying *st*rain t*ran*sducer. Strain gages are bonded to a steel rod (Figure 13.31) with square cross section. The rod has a large flange on each end and is surrounded by a length of rubber hose. The design attempts to match the stiffness of the cell to the stiffness of the displaced concrete, thereby ensuring conformance. The relationship depends on the size of the column and the diameter of the rubber hose. The plastic tube shown in Figure 13.31 is used for maintaining a nitrogen pressure, to minimize the possibility of false readings owing to moisture intrusion. The cell is pressure tested and filled with dessicant prior to installation.

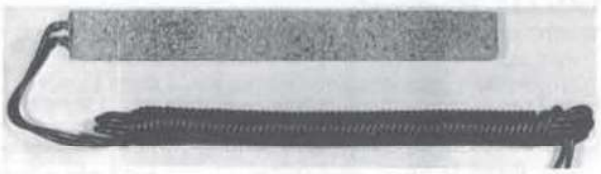


Figure 13.32. Polyester mold strain gage for embedment in concrete (courtesy of Texas Measurements, Inc., College Station, TX).

Plastic Encased Gages

A *plastic encased gage* consists of a standard wire-type resistance strain gage, to which lead wires are attached, and hermetically sealed between thin plastic plates. The plastic is either given an irregular surface or coated with a coarse grit to promote bonding to the concrete. Quarter bridge gages are the most common, with either two or three lead wires, but gages molded into rosette forms are available for monitoring two-dimensional strains. The gage works well under dynamic loading conditions (Hirsch et al., 1970) but is suitable for short-term measurements only. Attempts by the author to use the version shown in Figure 13.32 for long-term tests in precast concrete piles showed severe gage drift, presumably caused by creep in the polyester, and usage appears to be limited to tests of 1 or 2 days duration.

Eaton Corporation Gage

The embedment strain gage manufactured by Eaton Corporation (formerly manufactured by Ailtech) consists of a thin resistance element placed within a tube in a similar way to the surface-mounted gage shown in Figure 13.24, and wired as a quarter bridge three-wire system (Figure 13.33). The annular space between the tube and resistance element is filled with magnesium oxide powder to ensure frictional contact with the concrete. The gage is very delicate and must be precast in a briquette prior to installation.

13.4.4. Miscellaneous Embedment Strain Gages

Various *electrical crack gages* (Section 12.3.2) can be packaged for measurement of strain in concrete. The *soil strain gage* (Section 12.6.5) can also be

Figure 13.33. Strain gage for embedment in concrete (courtesy of Eaton Corporation, Los Angeles, CA).

used. The *sliding micrometer* (Section 12.5.4) is reported to be capable of strain measurements in concrete to an accuracy of ±2 microstrain.

13.4.5. Guidelines on Use of Embedment Strain Gages

Criteria for the selection of strain gage length, cable, connectors, locations, and initial readings are all similar to the criteria given in Sections 13.3.5, 13.3.6, and 13.3.8 for surface-mounted strain gages. Additional guidelines for the use of embedment strain gages are given in the following sections.

13.4.6. Installation of Embedment Strain Gages

General guidelines on the installation of instruments are given in Chapter 17. The following additional guidelines apply to the installation of embedment strain gages.

Embedment strain gages are usually installed by tying them with soft iron wire to nearby reinforcing bars. In general, the tie wires should be aligned at 90 degrees to the gage axis, so that any movement of reinforcing bars during concrete placement will not cause a direct pull on the gage and resultant damage. Tie wires on vibrating wire gages fitted with end flanges should be wrapped around the protective tube, leaving the flanges free. Care should be taken to keep concrete vibrators away from gages, and concrete should not be poured directly on to a gage. When the gage is delicate or the risk of damage large, gages can be precast in briquettes of concrete identical to that used in the structure, and if necessary the larger aggregate can be removed. Briquettes **must** be cast not more than 24–48 hours ahead of the main concrete pour to ensure conformance.

The need for protection to gage cable depends on the cable type, and if heavy duty or armored cable is used, additional protection is usually unnecessary. Cables are normally routed alongside reinforcing steel to the exit point and tied at frequent intervals with plastic cable ties. A blockout box can often be attached to the inside of the form at the exit point, and a coil of cable stored within the box. After the form is stripped, the cable can be uncoiled and either terminated or extended to its terminal location.

13.4.7. Relationship Between Strain and Stress*

Relationships between strain and stress for surface measurements are discussed in Section 13.3.9 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; that is, there will be poor conformance.

The influence of the gage on measured strain depends on both the stiffness of the gage relative to the concrete and on geometry. If the stiffness of the gage is equal to that of the concrete, conformance requirements will be satisfied and the strain measurements will be correct. If the stiffnesses do not match, the measurement error depends on the geometry of the gage and structural member, and the gage will be more accurate if its stiffness is less than that of the concrete. Loh (1952) indicates that when the stiffness of the gage is less than that of the concrete, the error will not be excessive provided that the length of the gage is greater than five times its diameter. This conclusion applies primarily to measurements of compressive stress, where bond between the concrete and gage is not an issue, and to the case where the mass of the surrounding concrete is large relative to the size of the gage.

*Written with the assistance of Stanley L. Paul, Associate Professor, Department of Civil Engineering, University of Illinois at Urbana–Champaign, IL.

The use of *sister bars,* which are generally very long relative to their diameter, is based on a somewhat different approach. The bar is long enough on each side of the gaged portion at the midpoint, so that adequate bond is created between the bar and concrete, and therefore at the gaged portion the strain in the bar and concrete are equal. Also, the bar should be small relative to the size of the structural member, so that the response of the member is not influenced by the presence of the bar. To satisfy bond length requirements, the bond length at **each** end of the bar shown in Figure 13.29 should be equal to or greater than about 50 times the diameter of the bar. For such a long bar, the inclusion effect is small, even when the gage is stiff relative to the concrete. Threaded connections should not be included in the bar, as the threads tend to create a zone of the bar with lower stiffness than the unthreaded portion. The central portion of the bar around the strain gage should be debonded from the concrete for a minimum length of 6 in. (150 mm) to ensure an adequate gage length. This debonded length should not be included in the length calculated for bond requirements, and hooks can be provided at each end of the bar to assist with developing full bond length. If the cross section at the gage is different from that elsewhere, strain will not be uniform along the bar and measurements will be misleading. A goal in the design of sister bars should therefore be to create a uniform cross section throughout the debonded length and the length required for bond. This goal is satisfied by the configuration shown in Figure 13.29 and essentially also by the sister bar described by DiBiagio (1983). The length of the cylindrical part of the gage shown in Figure 13.29 results from the need to avoid damaging the transducer, thermistor, electrical coil, and cable splice while welding the cylinder to the reinforcing bar. The diameter of the sister bar does not need to be the same as the reinforcing steel in the structural member: it should be the minimum required for gage mounting.

When the size of the structural member is large relative to the size of a sister bar, the author believes that sister bars of the type described above should be given first consideration when selecting an embedment strain gage. They are robust, reliable, and easy to install. They should preferably have vibrating wire transducers. Despite this recommendation, and despite the belief that the inclusion effect is small, there is a need to document the accuracy of this type of gage, and practitioners who are in a position to research this issue and to publish the results are encouraged to do so.

13.5. DETERMINATION OF EXISTING STRESS

Where measurement is required of existing stress in in-place steel struts, piles, or tunnel supports, strain gages can be attached and a stress relief technique used, whereby the structural member is unloaded and the strain measured. If the member can be unloaded completely, this is a reliable method.

For those cases where the member cannot be unloaded completely, two methods are possible. First, a small portion of the member including the strain gages can be removed by cutting around the gages to stress relieve the material and the gages. However, fabrication stresses in the steel member and stresses associated with the cutting operations may complicate the results considerably, and the results may be misleading. This point is clearly illustrated by DiBiagio (1977), who describes laboratory tests to examine the effect of cutting and welding. Vibrating wire strain gages were mounted on each face of an unloaded steel plate that had previously been burned out of a larger plate. The plate was sequentially reduced in size by cutting with a metal saw, with an oxygen-acetylene torch, and again with a metal saw. Measured apparent stress changes ranged from a tensile stress of 2600 lb/in.^{2} (18 MPa) to a compressive stress of 11,700 lb/in.^{2} (80 MPa) with respect to the starting conditions. The second method for use on steel is the *hole-drilling strain gage method,* also referred to as the *blind hole drilling method* (Measurements Group, 1985). A rosette of electrical strain gages is bonded to the structural surface and a short blind hole drilled in the center of the rosette, using a milling guide (Figure 13.34). Strains created by the drilling are analyzed to determine existing stress. However, it seems reasonable to assume that this method may also suffer from inaccuracies caused by fabrication stresses and stresses associated with the drilling operation.

Attempts have been made to determine existing stress in concrete by using a stress relief *overcoring* procedure. The procedure was developed for determination of in situ stress in rock and entails setting a borehole deformation gage or strain cell (Chapter 11) within a small-diameter borehole and overcoring the gage with a larger-diameter borehole. Gage

Figure 13.34. ***Hole-drilling strain gage method*** **for determination of existing stress (courtesy of Measurements Group, Inc., Raleigh, NC).**

measurements provide strain relief data, which are converted to stress by an analysis using an assumed or measured modulus. The U.S. Bureau of Mines overcoring procedure is described by Hooker and Bickel (1974) and Hooker et al. (1974). Although the overcoring procedure is used satisfactorily in rock, it has questionable value for measurements in concrete. While drilling to relieve stresses, cooling water is used on the bit, and strains resulting from concrete swell usually mask out strains resulting from stress relief. This problem does not usually arise in rock, because the rock will generally be below the water table. The method can, however, be used to determine approximate relative stress magnitudes in concrete. If a part of the same concrete is under a known stress, for example, zero stress, an approximate absolute stress determination can be made by making several overcoring tests also at this location and using the results as a "calibration" to account for strain resulting from swell at other locations.

13.6. CONCRETE STRESS CELLS

Problems of converting strain to stress in concrete have been discussed earlier in this chapter. An alternative approach is to measure stress directly, by using *concrete stress cells*, which are similar to the earth pressure cells described in Chapter 10.

Carlson (1984) indicates that the requirements for a concrete stress cell include the following:

1. It must not change appreciably the existing stress.
2. It must be embedded so as to be in close and intimate contact with the concrete.
3. Its composite modulus of elasticity must be at least one-half that of the concrete.

Concrete stress cells are subjected to a condition that does not exist for earth pressure cells. As concrete cures, its temperaure increases, and the temperature of the stress cell is therefore increased. The possibility therefore exists for the cell to expand, pushing the weak concrete away from the cell and, when the concrete takes its initial set and cools, for the cell to cool, contract, and uncouple itself from the surrounding concrete. In this condition the cell would not be responsive to subsequent stress changes in the concrete. The two approaches adopted for solving this problem are described in Sections 13.6.1 and 13.6.2.

Concrete stress cells have one notable weakness: they cannot be used to measure tensile stress. Carlson (1984) recommends that, whenever tensile stress is anticipated, an embedment strain gage should be installed alongside each stress cell. When the stress changes from compression to tension, the stress cell provides a way of knowing when the stress is zero, an all important reference point for interpretation of strain measurements.

13.6.1. Carlson Stress Meter

The *Carlson stress meter*, shown in Figure 13.35, consists of a mercury-filled circular cell with a Carlson elastic wire strain meter mounted on one face. The instrument has a composite elastic modulus of at least one-half that of concrete.

The problem of uncoupling between the meter and concrete during curing is overcome by minimizing the thickness of the mercury film. Carl-

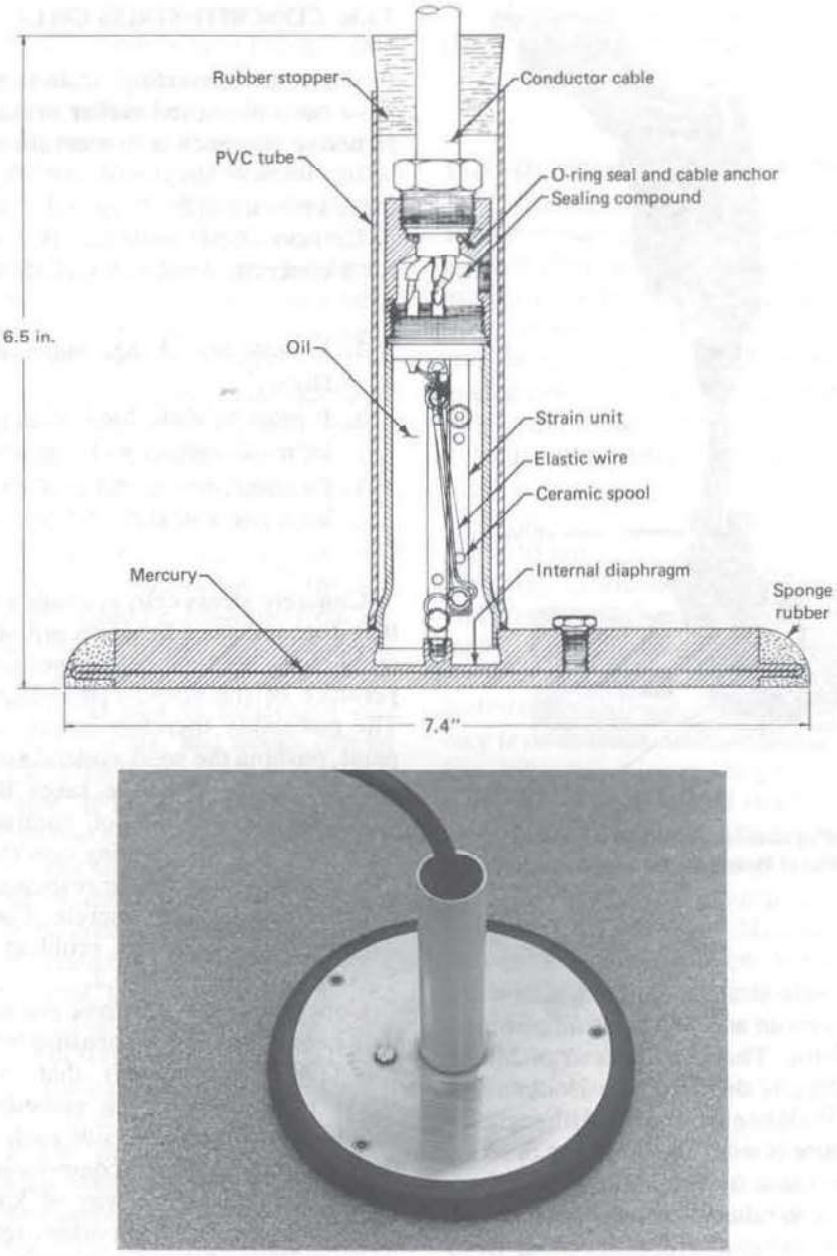


Figure 13.35. Carlson concrete stress meter (courtesy of Carlson Instruments, Campbell, CA).

son (1984) indicates that the mercury film should not be more than 0.011 in. (0.28 mm) thick and that even the finest machining is not adequate for preventing tiny grooves into which the mercury can squeeze under pressure. The surface tension of mercury is so great that it tends to span over even the slightest groove or imperfection, and the inner steel surfaces should be polished to a tolerance of 6×10^{-6} in. (150×10^{-6} mm) RMS. After assembly, each meter should be tested for compressibility under a pneumatic pressure, using an interferometer to measure compressibility at the low end of the range. Pirtz and Carlson (1963) describe comprehensive laboratory tests indicating that the meters, used individually, are suitable for use in concrete containing 6 in. (150 mm) aggregate in applications where accuracy of 10–15% is satisfactory. Carlson (1978) describes additional laboratory

tests, concluding that the worst error registered during the tests was 12% of the major principal stress, and this only happened when the load was applied more quickly than would be the case in a field structure.

The standard installation procedure for measuring stress in a horizontal plane involves installing the meter on the surface of previously cured concrete and later covering with fresh concrete. Great care must be taken during installation to ensure good contact between the meter and surrounding concrete, using a technique described by Carlson (1975). The technique includes preparing the surface, placing a mortar pad containing steel filings to increase the modulus, painting a film of grout on the mortar pad, pressing the meter into place with a rotary motion to squeeze the mortar outward, weighting the meter temporarily with a metal tripod, placing the overlying concrete, and removing the metal tripod.

An alternative installation procedure (Green, 1985) was used at the Mount Baker Ridge Tunnel in Seattle, where restricted access and potential damage precluded use of the standard installation procedure. Meters were cast, with the active face vertical, within a 12 in. diameter by 12 in. long (300 mm by 300 mm) briquette, cast 24–48 hours ahead of the main concrete pour. Laboratory tests on the briquette, in a concrete testing machine, indicated an almost 1:1 ratio between applied and measured stress, and field data appear very reasonable. The small size of the briquette apparently allowed temperature to dissipate rapidly enough so that the meters did not become uncoupled from the concrete. The young age of the concrete in the briquette at the time of the main concrete pour is vital to the success of this procedure.

13.6.2. Glötzl Cell

The Glötzl cell, shown in Figure 13.36, is similar to the hydraulic type of earth pressure cell, with pneumatic transducer, described in Chapter 10. For measurements greater than 300 lb/in.^{2} (2 MPa), the transducer is pressurized with oil rather than gas. The cell is provided with a post-stressing tube, long enough to protrude from the concrete pour. If a length longer than 30 ft (10 m) is required, a check valve is inserted in the post-stressing tube near the cell. After the concrete has set, the tube is pinched repeatedly until the pressure measurement indicates that the cell has regained contact with the concrete. Thereafter, provided that the thickness of the mercury film is restricted to approximately the same as in the Carlson stress meter, and the composite modulus of elasticity of the cell is at least one-half that of the concrete, the cell will function correctly.

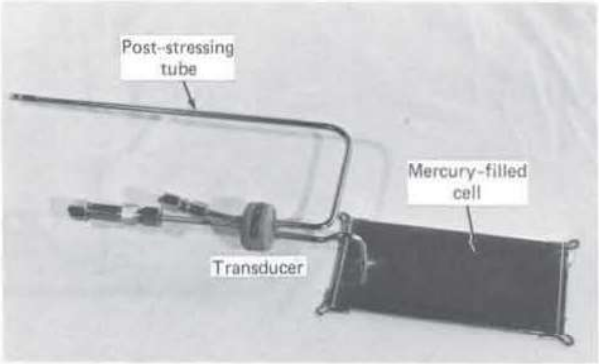


Figure 13.36. Glötzl concrete stress cell (courtesy of Glötzl GmbH, Karlsruhe, West Germany, and Geo Group, Inc., Wheaton, MD).

The cell is installed by wiring with soft iron wire to nearby reinforcing steel, with care to avoid entrapment of air as concrete is placed. For measurements in concrete containing aggregate larger than 1 in. (25 mm), it is advisable to surround the cell with a special concrete mix from which the coarser particles of aggregate have been removed. Alternatively, the cell can be installed in a briquette of concrete as described previously for the Carlson stress meter, ensuring that the briquette is cast within 24–48 hours of the main concrete pour.

13.6.3. Other Concrete Stress Cells

A post-stressing tube arrangement, similar to the Glötzl cell, has been adopted by manufacturers of concrete stress cells with vibrating wire transducers.

Interfels manufactures a concrete stress cell with post-stressing tube and a hydraulic/mechanical transducer, and changes in cell pressure create movement within a measuring unit that is monitored with a dial indicator.

CHAPTER 14

MEASUREMENT OF TEMPERATURE*

14.1. APPLICATIONS

Temperature measurements in geotechnical engineering fall into three general categories: first, where temperature itself is a primary parameter of interest; second, where temperature change causes a real deformation or stress change in the ground or in a structure; and third, where a transducer itself is sensitive to temperature change. Examples in each category are given in Sections 14.1.1–14.1.3.

14.1.1. Temperature as a Primary Parameter of Interest

Temperature measurements are often made while supporting excavations by ground freezing, while limiting the temperature of concrete during curing, during construction of projects in permafrost, during frost heave and frost penetration studies, and while studying ground response to high temperatures during studies for underground disposal of nuclear waste.

Birman et al. (1971) describe thermal monitoring in an earth dam to detect leakage and to monitor changes in the leakage pattern. Highways are frequently damaged during the "mud season," in spring when a subsurface frozen layer remains although the surface is unfrozen. Traffic speeds are often restricted until complete thaw occurs, using measurements of temperature to indicate the extent of freezing.

Where exposed rock cuts or tunnels are subjected to low temperatures, freezing at the surface can cause ice dams to form, holding unfrozen water in the rock mass with resulting buildup of joint water pressure and decrease in stability. Temperature measurements will show the depth and extent of the freezing problem. Finally, ambient temperature should be measured when temperature change may cause changes in other measured parameters, so that cause–effect relationships can be established.

14.1.2. Temperature Change Causing Real Deformation or Stress Change

A temperature increase causes most materials to expand and, if expansion is resisted, temperature increase will cause a compressive stress increase. Thus, steel struts across an open cut excavation can be subjected to substantial compressive stresses when heated by the sun: the greater the end restraint the greater is the stress. The same phenomenon is illustrated by thermal expansion of railroad tracks.

The coefficient of linear expansion is the change in length, per unit of length, for a temperature change of 1 degree. A bar, free to move, will increase in length with an increase in temperature and will decrease in length with a decrease in tempera-

*Written with the assistance of Ronald T. Atkins, Chief, Technical Services Division, U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, NH, and J. Barrie Sellers, President, Geokon, Inc., Lebanon, NH.

Table 14.1. Linear Coefficients of Thermal Expansion

Material	Linear Coefficient of Thermal Expansion × 10^{-6} per Degree, at 20°C[a]	
	°C	°F
METALS AND ALLOYS		
Aluminum, wrought	23.1	12.8
Brass	18.8	10.4
Copper	16	9
Invar	1.4	0.8
Iron, cast, gray	10.6	5.9
Iron, wrought	12.0	6.7
Steel, mild	11.7	6.5
Steel, stainless, 18-8	17.8	9.9
Super-invar	0.4	0.2
STONE AND MASONRY		
Brick masonry	6.1	3.4
Concrete	10–13	5.5–7.2
Granite	8.0	4.4
Limestone	7.6	4.2
Marble	8.1	4.5
Sandstone	9.7	5.4
Slate	8.0	4.4

[a] Coefficients of many materials increase very substantially with increasing temperature.

Source: After American Institute of Steel Construction.

ture. The change in length will be $\varepsilon \Delta T l$, where:

ε = linear coefficient of thermal expansion,
ΔT = change in temperature,
l = length.

If the ends of the bar are fixed, a change in temperature will cause a change in the unit stress of $E\varepsilon\Delta T$, where:

E = modulus of elasticity.

As an example of stress magnitude when end restraint is total, a 1°C temperature increase causes a compressive stress increase of 340 lb/in.^{2} (2.3 MPa) in mild steel. Linear coefficients of thermal expansion are given in Table 14.1.

Sellers et al. (1972) give an example of temperature-induced structural distress in a 60 ft (18 m) wide mine opening in competent marlstone. During the winter the roof rock contracted, loosened, and sagged downward, and several roof falls occurred. No such falls occurred during summer months.

14.1.3. Transducer Sensitive to Temperature Change

The third category for temperature measurements, where a transducer itself is sensitive to temperature change, can be illustrated by hydraulic systems such as earth pressure cells and hydraulic load cells. Temperature changes cause expansion or contraction of the liquid in the cell—thus an indicated pressure or load change—and temperature corrections must be applied if these changes are significant. Similarly, strain gages and extensometers will, if their thermal expansion and contraction characteristics are not matched with the monitored structure, show extraneous sensitivity to temperature.

Three approaches are possible for taking account of temperature sensitivity: matching thermal coefficients of instrument and structure, use of low coefficient of expansion materials such as invar or super-invar, and application of temperature corrections. The first two approaches are often only partially successful, owing to design and/or cost limitations. The third approach requires measurement of temperature and use of temperature sensitivity relationships either provided by the instrument manufacturer or developed by users for their site-specific instrument–structure interaction. It also requires ensuring that the temperature transducer is in good thermal contact with the instrument to be corrected.

14.2. MERCURY THERMOMETER

The familiar mercury thermometer is useful for spot measurements at accessible locations. However, mercury thermometers are fragile, are not suitable for remote readout, and are limited to temperatures above about −30°C. They have a much larger thermal mass than most other devices for measurement of temperature. They are therefore a poor choice where measurements of rapidly changing temperature are required or where their use might alter the thermal characteristics of the material to be monitored. Maximum and minimum thermometers are available for indication of extreme temperatures.

14.3. BIMETAL THERMOMETER

Bimetal thermometers use a bimetallic element in which two strips of different metals are bonded together back to back. The different coefficients of

Figure 14.1. Bimetal thermometer with clock-driven recorder (courtesy of Omega Engineering, Inc., Stamford, CT).

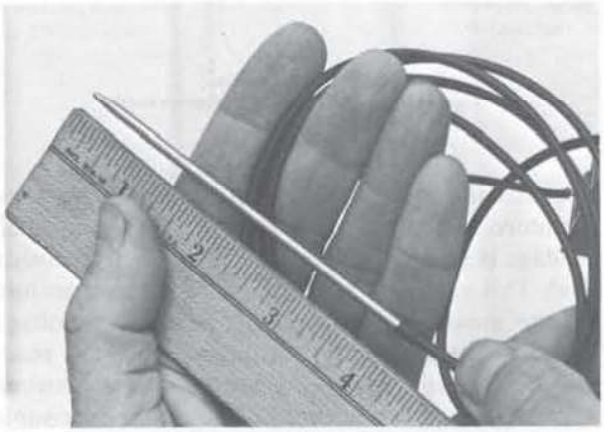
thermal expansion of the two metals cause the strip to bend when cooled or heated. The strip is usually wound into a spiral, so that temperature changes cause the spiral to wind or unwind and twist a pointer over a circular scale. These devices are rugged, but accuracy is usually low if the temperature range is wide. However, narrow range bimetal thermometers with ½–1% full-scale accuracy are available. Figure 14.1 shows a clock-driven 7 day recorder, suitable for ambient temperature measurements. Accuracy is 2% full scale.

Johnston (1966) describes a self-contained ground temperature recorder in which a bimetallic strip moves a pointer on pressure-sensitive recorder paper. The device is battery powered and has a range of 11°C and an accuracy of about ±0.3°C. The instrument is 3.75 in. (95 mm) in diameter by 11 in. (280 mm) long and is designed for installation in a borehole for recording ground temperature over a 1 year period without attention or servicing.

14.4. THERMISTOR

The name is derived from *therm*ally sensitive res*istor*. A thermistor is composed of semiconductor material that changes its resistance very markedly with temperature. Lead wires are used to connect the thermistor to a measuring instrument. Readout may be in resistance units (ohms), requiring the use of commercially available digital multimeters and resistance–temperature tables, or directly in temperature units by using a **matched** thermistor and readout instrument. Any device for reading thermistors must apply only a very small current, since the temperature of the thermistor will be changed significantly when as little as 0.001 watt is applied. The old type of volt–ohm–milliammeter (VOM) causes too much self-heating and should not be used.

(a)



(b)

Figure 14.2. Thermistors: (a) suitable for attachment to another instrument for monitoring its temperature so that a temperature correction can be applied and (b) for use as a portable probe or for permanent embedment (courtesy of Atkins Technical, Inc., Gainesville, FL).

Thermistors are most often used in a glass bead configuration, although they are available as disks, washers, rods, and in sizes as small as pin heads. Two configurations are shown in Figure 14.2. Thermistors are capable of highly accurate temperature measurements. For this reason they are often used where accuracy of ±0.1°C or better is required over a wide temperature range. Robertson et al. (1966) present comprehensive guidance for use of thermistors.

14.5. THERMOCOUPLE

A *thermocouple* is composed of two wires of dissimilar metals, with one end of each wire joined

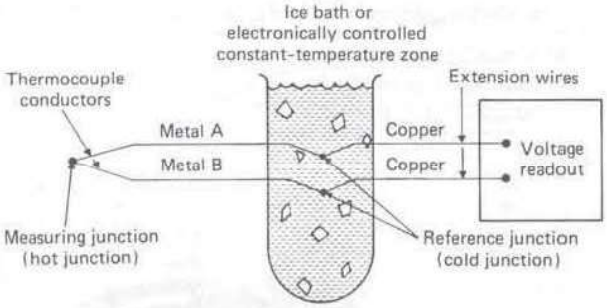


Figure 14.3. Schematic of thermocouple.

together to form a measuring junction. At any temperature above absolute zero (−273°C), a small voltage is generated between the wires at the other end. This voltage is proportional to the temperature of the measuring junction. However, the voltage cannot be read directly, since connecting any readout device will create two more contacts between dissimilar metals and thus unwanted thermocouple connections. To solve this problem, thermocouples must be "referenced" as shown in Figure 14.3, with the readout device connected to similar metals so that no thermocouple is formed. Modern thermocouple readout devices have internal electronic reference junctions, allowing direct connection to the thermocouple leads. They must be matched to a specific type of thermocouple: for example, an iron–constantan thermocouple cannot be read with a readout device designed for a copper–constantan thermocouple.

Thermocouples are widely used in applications requiring modest accuracy (±0.5°C over a wide temperature range) and in large installations requiring data logging equipment. Readout instruments read directly in temperature units and are available as battery operated units for field applications. Figure 14.4 shows an example.

Thermocouple manufacturers should be consulted for guidance when selecting thermocouple components. Factors that must be considered include type of thermocouple, proper installation for the wires, readout equipment, temperature range, length of service required, lead length, and the ambient temperature to which readout equipment will be subjected in the field. Copper–constantan (Type T) thermocouples are the most frequent choice for geotechnical applications. Properly selected and installed systems are capable of good long-term performance, meeting specification limits for 5 years and longer. Other sources that provide comprehensive guidance on the use of thermocouples are ASTM (1974) and ISA (1974).

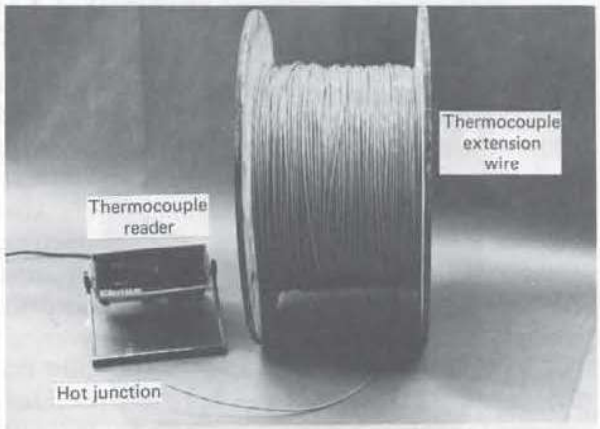


Figure 14.4. Thermocouple (courtesy of Conax Buffalo Corporation, Buffalo, NY).

14.6. RESISTANCE TEMPERATURE DEVICE (RTD)

Resistance temperature devices depend on the principle that change in electrical resistance of a wire is proportional to temperature change. The wire is usually mounted on a postage stamp-sized backing or wound on a small-diameter coil. Wire material is usually platinum, copper, nickel, or a copper–nickel alloy. Platinum has the greatest accuracy, linearity, stability, and repeatability but copper–nickel alloys have a much greater sensitivity. Readout devices normally contain a Wheatstone bridge circuit, since for a 100 ohm RTD a 10°C temperature change normally creates a resistance change of only 2 or 3 ohms. Figure 14.5 shows an example of an RTD.

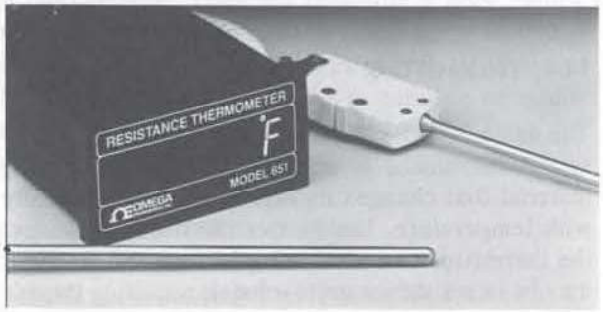


Figure 14.5. Resistance temperature device (courtesy of Omega Engineering, Inc., Stamford, CT).

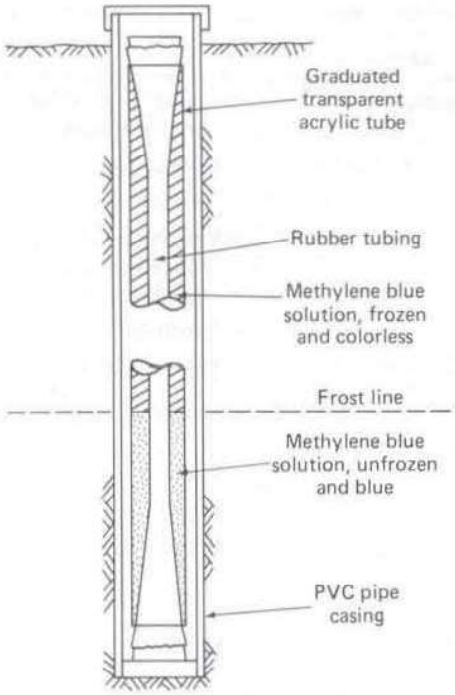


Figure 14.6. Schematic of Gandahl frost gage, used to measure depth to frost line.

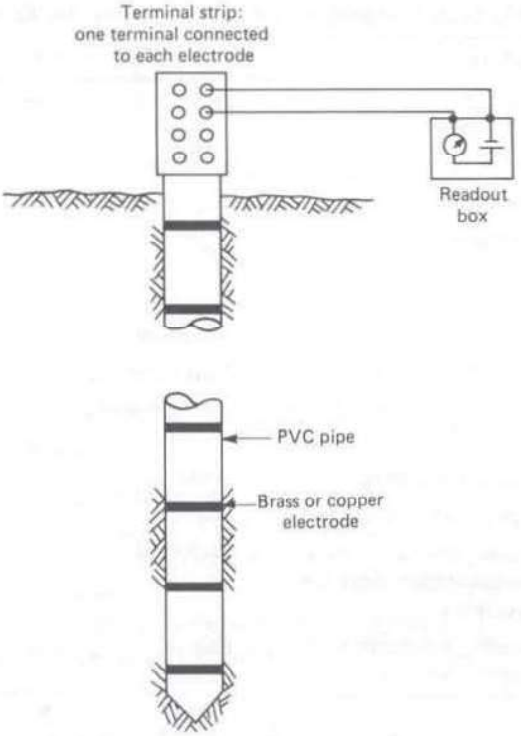


Figure 14.7. Schematic of Bergau frost gage, used to measure depth to frost line.

RTDs have not been used widely for measurement of temperature in geotechnical applications. However, their excellent stability and accuracy makes them well suited to long-term installations (15–20 years) where high accuracy is a requirement.

14.7. FROST GAGES

Two gages are described by Gandahl and Bergau (1957) for measuring the depth to the frost line. The Gandahl gage, Figure 14.6, consists of a transparent acrylic tube containing distilled water colored with methylene blue. A PVC pipe casing is installed in the ground, usually to a depth of between 4 and 8 ft (1.2–2.4 m) and the acrylic tube is left in place within the PVC pipe. To read the gage, the acrylic tube is withdrawn. Unfrozen methylene blue solution is blue but becomes colorless when frozen; thus, the depth to the frost line can be determined from the color transition, using graduations on the acrylic tube.

Although a simple and inexpensive device, the Gandahl gage will be in error if the freezing point of the groundwater is depressed to below 0°C, which is generally the case in highway applications where salt has been used on the pavement. The Bergau gage, shown in Figure 14.7, overcomes this problem. The electrical conductivity of frozen soil is much lower than that of unfrozen soil. Therefore, by attaching electrodes to a nonconductive pipe and installing the pipe in the ground, conductivity measurements between adjacent electrodes will indicate the depth to the frost line. The electrode spacing depends on the required accuracy. The gage must be read using alternating current, since direct current causes polarization of ions in the soil, leading to erroneous readings. The PVC pipe can be either driven into the ground while the soil is unfrozen or installed in an augured hole.

Atkins (1979) describes the design and evaluation of two resistivity gages for determining frost penetration and indicates their superiority to temperature transducers when salt has been used to depress the freezing point or when measurements are required during spring thaw. During spring thaw, subsurface temperatures often become nearly isothermal at 0.0°C, making it difficult to establish the frost line by measuring temperature. Atkins recommends use of both temperature and resistivity gages when

Table 14.2. Comparison Among Transducers for Remote Measurement of Temperature in Geotechnical Environments

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

determining maximum frost depth, since one complements and increases the confidence level in the other.

Other frost gages are described by Mackay (1973) and Wilen and MacConnell (1973).

14.8. OTHER TRANSDUCERS FOR MEASUREMENT OF TEMPERATURE

Vibrating wire transducers are available for measurement of temperature and may be the preferred type if other vibrating wire instruments are being used and an indicator is available, or if a frequency signal is required. However, they are more expensive than thermistors, thermocouples, and RTDs and appear to offer no other inherent advantages.

Some pressure or strain measuring instruments based on the vibrating wire principle use the resistance of the plucking coil as a measure of temperature. The Carlson type of unbonded resistance strain gage (Chapter 8) can also be used to measure temperature.

14.9. COMPARISON AMONG TRANSDUCERS FOR REMOTE MEASUREMENTS

Apart from vibrating wire and unbonded resistance strain gage transducers, the choice among transducers for remote or continuous measurement of temperature is restricted to thermistors, thermocouples, and RTDs. The three are compared in Sections 14.9.1–14.9.6, and comparisons are summarized in Table 14.2. All three are suitable for use in geotechnical environments, the choice depending on the application.

14.9.1. Readout

RTDs and thermistors provide a direct measurement of absolute temperature, whereas thermocouples provide a measure of the temperature differential between a measuring junction and a reference junction. In practice, however, direct temperature readout devices can be purchased for all three transducers. Thermistors can be read out using commonly available digital multimeters set to measure resistance. Battery operated 4½ digit multimeters are normally used with low readout currents (a few microamperes) when set to measure resistance. Battery operated thermocouple readout units are available with sufficient ruggedness for field use. RTDs are read out using devices containing a Wheatstone bridge circuit and with a millivolt scale.

14.9.2. Sensitivity, Linearity, Accuracy, and Stability

Typical sensitivities are: thermistors, 200 ohms/°C; thermocouples, 0.04 mV/°C; RTDs, 0.2 ohm/°C.

Thermistors have by far the greatest output signal and sensitivity but the worst linearity.

RTD transducers have the greatest accuracy, but system accuracy may be reduced substantially by lead wire effects. The overall system accuracy of thermistors and RTDs is similar and in the region of ±0.1°C. Thermocouple accuracies of about ±1°C are normal, but ±0.5°C accuracies are possible by using "special limits of error" wire.

Thermistors and RTDs have excellent long-term stability. Some thermistor manufacturers claim that thermocouples exhibit drift with time, but available evidence suggests that this is likely only where they are in an oxidizing environment and unprotected. If thermocouples are protected by heat-shrink polyolefin so that no metal oxidization can occur, they can be expected to remain within their accuracy limits for at least 5 years in most geotechnical applications.

14.9.3. Lead Wires

The high sensitivity and high nominal resistance of thermistors allows them to be used with two-conductor lead wires, since ambient temperature changes cause negligible changes in contact or transmission line resistance. For long lead wires where high accuracy is required, the resistance of the wires should be subtracted from the resistance measurement before converting to temperature. RTDs use a three-conductor cable, for the three-wire quarter bridge configuration. Thermocouples use special thermocouple wire between the transducer and the reference junction location.

Thermistors and RTDs may be procured preencapsulated in epoxy resin with lead wires attached. All lead wires must be suitable for outdoor use and should be selected in accordance with guidelines given in Chapter 8 for selection of cable.

Field repairs are possible to the lead wires for all three transducers. For thermistors and RTDs the leads may be repaired by using commercially available splice kits with crimp connectors, as described in Chapter 8. Field repair of thermocouple lead wires is also possible, though less straightforward because errors can be caused. Care must be taken to ensure that the two thermocouple materials are not interchanged at the splice and that close metal-to-metal contact is established. Each conductor should be repaired by gently twisting the two parts together for about 1 inch (25 mm), followed by the crimp procedure described in Chapter 8. Although a high-quality repair will often cause no error in thermocouple data, the addition of two junctions within the circuit creates the possibility of errors, and splices should be avoided whenever possible.

14.9.4. Temperature Range and Rapidity of Response

All three devices are suitable for the range of temperatures normally encountered and rapidity of response required in geotechnical environments.

14.9.5. Applicability for Instrument Temperature Corrections

When a transducer is sensitive to temperature change, and data are to be corrected in accordance with measured temperature, a temperature transducer is required at the instrument location. Thermistors are usually preferred, partly because of their small size and partly because two conductors in the multiconductor cable attached to the instrument can readily be used for the thermistor. Good thermal contact between the instrument and thermistor must be ensured.

14.9.6. Suitability for Automatic Data Acquisition

If automatic data acquisition equipment is to be used, thermocouples are the instruments of choice. Data loggers can readily be used with thermocouples since the necessary reference junctions are available from many commercial sources and, because the calibration is common to all thermocouples of the same type, it can be programmed into the data logger so that readout is in degrees.

It is less straightforward to read thermistors with an automatic data acquisition system because resistance changes are so very large. A few manufacturers of data loggers will supply a special input circuit designed to read thermistors directly, but the selection of compatible thermistors is limited, and readout may be in millivolts rather than in degrees. However, data can be logged automatically by using a data controller that has an autoranging digital voltmeter and that has programmed nonlinear scaling functions.

RTDs can be read with a data logger since their resistance change is relatively small. However, either each RTD must have its own bridge circuit or a single bridge circuit must be switched to each RTD, posing a switching problem for the data log-

ger. Special RTD readout circuits are available from some data logger manufacturers, and these special circuits usually have readout in degrees.

14.10. INSTALLATION OF TRANSDUCERS FOR MEASUREMENT OF TEMPERATURE

When planning installation of transducers for measurement of temperature, care must be taken to ensure good thermal contact between the transducer and the material to be sensed.

When installing transducers in a borehole to measure subsurface ground temperature, the borehole must be backfilled completely, and the backfilling procedure should depend on the required measurement accuracy. When maximum accuracy is required, for example, when measuring the depth of frost penetration or when measurements are used to make decisions on limiting roadway traffic in the spring, great care must be taken to minimize disturbance to the thermal regime and to the pathway for moisture movement. In such cases an accuracy of about $\pm 0.1°C$ is required, and the borehole should be backfilled with material removed from the borehole. Either the material can be placed and tamped in layers, or it can be mixed with water and poured or tremied into the borehole. When a lesser accuracy is acceptable, the borehole can be backfilled with cement grout or other material that ensures complete backfilling.

When transducers are attached to a structural surface, thermal contact is sometimes obtained simply by taping the transducers to the surface. Some manufacturers supply special rugged versions that can be welded to a metal surface for more permanent contact. Good thermal contact can also be created by drilling a small hole in the structure and using a heat-conducting epoxy to cement the transducer in place. The same epoxy can also be used to cement transducers directly to a structural surface, but this arrangement is less rugged.

General guidelines for installation of cables are given in Chapter 17.

Part 4

General Guidelines on the Execution of Monitoring Programs

Part 4 is addressed to the same audience as Part 3: the people who are involved in the details of instrumentation use. Chapter 15 presents a recipe for reliability of performance monitoring, including both *instrument ingredients* and *people ingredients*. Guidelines are given in Chapters 16–18 for calibration, maintenance and installation of instruments, and for collection, processing, presentation, interpretation and reporting of instrumentation data. These general guidelines are applicable to all instrumentation and supplement the more instrument-specific guidelines given in Part 3.

CHAPTER 15

A RECIPE FOR RELIABILITY OF PERFORMANCE MONITORING

The overriding desirable feature for performance monitoring is **reliability.** In suggesting a recipe for reliability, the ingredients can be divided into two types: *instrument ingredients* and *people ingredients*.

15.1. INSTRUMENT INGREDIENTS IN A RECIPE FOR RELIABILITY

As a general goal, the three major ingredients described in Sections 15.1.1–15.1.3 should be sought when selecting reliable instruments.

15.1.1. Simplicity

Inherent in reliability is maximum simplicity. In general, transducers can be placed in the following order of decreasing simplicity and reliability: optical, mechanical, hydraulic, pneumatic, electrical.

15.1.2. Self-Verification

This term means that instrument readings can be verified in place, as in the following:

- Checking a fixed borehole extensometer fitted with electrical transducers by inserting a dial indicator at the head.
- Water pressure can be increased or decreased manually within open standpipe or twin-tube hydraulic piezometers and the system then allowed to regain equilibrium. Equal readings before and after the change provide a degree of self-verification. Rates of increase or decrease should be consistent with the permeability of the ground around the piezometer.
- Use of duplicate transducers. For example, a vibrating wire and a pneumatic transducer packaged within the same housing to create a piezometer with two independent methods of reading.
- When taking readings with an inclinometer, a degree of self-verification is obtained when *check-sums* remain reasonably constant for all depth intervals in a given data set.
- Addition of an arrangement to a fixed borehole extensometer whereby each rod can be disconnected temporarily from its anchor, so that a check can be made for free-sliding.

15.1.3. Durability in the Installed Environment

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

sion and electrolytic breakdown. Guidelines on maximizing the longevity of transducers and connecting linkages between transducers and readout units are given in Chapter 8. Chapter 8 also includes guidelines on lightning protection when electrical systems are used.

Above-ground field terminals must be able to survive adverse environmental conditions and physical abuse.

The three goals outlined above are ideals that cannot always be achieved. Simplicity is not always appropriate, but there is no excuse for unnecessary complexity. Self-verification is a rare luxury, but it is often possible to achieve a similar goal by using the procedures, described in Chapter 4, for ensuring reading correctness. Good instrument design can usually create adequate durability in the installed environment. In summary, we must often settle for a little less than the best recipe, but we should try for the best.

15.2. PEOPLE INGREDIENTS IN A RECIPE FOR RELIABILITY

As a general goal, people responsible for performance monitoring programs should provide five major ingredients, described in Sections 15.2.1–15.2.5.

15.2.1. Thorough Planning

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. Twenty steps in the planning process are described in Chapter 4. All twenty steps should be taken with great care. Paramount among the steps is the requirement to prevent cost from dominating the selection of an instrument.

15.2.2. Initial Calibrations and Inspections

Instruments should be calibrated, inspected, and tested before shipment to the user. On receipt by the user, acceptance tests should be made to ensure correct functioning. Guidelines are given in Chapter 16.

15.2.3. Installation Care

No instrument will be reliable if it is not installed correctly. Guidelines for installation care are given in Chapter 17.

15.2.4. Regular Maintenance and Calibration

Readout units, field terminals and any accessible embedded components should be maintained and calibrated on a regular schedule. Guidelines are given in Chapter 16.

15.2.5. Care During Data Collection, Processing, and Interpretation

Careful data collection, processing, and interpretation require a commitment by experienced and motivated people. Guidelines are given in Chapter 18.

In contrast to the goals for instrument ingredients, these five goals for people ingredients can usually be achieved. If we remember that the primary *people* need is **a high degree of motivation,** the goals will be within our grasp.

CHAPTER 16

CALIBRATION AND MAINTENANCE OF INSTRUMENTS

An instrument reading is useful only if the correct calibration is known. Zero shifts and scale span changes of transducers 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 and maintained properly.

16.1. INSTRUMENT CALIBRATION

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 (*acceptance tests*), and during service life. These are discussed in turn.

16.1.1. 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 5 indicates that the procurement specification should include required factory calibrations and quality assurance and provides guidelines on methods and documentation. The user may opt to **participate in** factory calibrations. This is strongly recommended whenever the procurement includes newly developed instruments, when calibration requirements differ from the manufacturer's standard procedures, or when users believe that their needs cannot be satisfied by the acceptance tests described in Section 16.1.2. The words *participate in* rather than *inspect* are used to convey the need for cooperation.

16.1.2. 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 *acceptance tests*. If an instrument does not work **perfectly** under laboratory conditions, it is unlikely to work at all when installed in the field (Cording et al., 1975).

As indicated in Chapter 5, the instruction manual should include a step-by-step acceptance test procedure. Whenever possible, acceptance tests should include a verification of calibration data pro-

vided 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 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, and the cable immediately adjacent to each cable-end connector should be flexed carefully to disclose any malfunction. The zero readings should agree with the readings supplied by the manufacturer. All electrical transducers intended for burial should be immersed in water for as long as possible to check the waterproofing. Instruments such as dial indicators, depth micrometers, and mechanical strain gages are generally reliable but should be checked against reference standards. Piezometers should be checked by immersion under known depths of water, and inclinometers may be function-checked by placing in sections of inclinometer casing kept fixed at some constant, known angle in the laboratory.

Any instrument that fails an 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.

Because standard transit insurance often requires notice of damage within a relatively short period after delivery, acceptance tests and function checks should be made before this time has expired. Sufficient lead time must be allowed to permit adequate testing and for any needed repair or replacement of faulty instruments.

16.1.3. Calibrations During Service Life

Calibrations or function checks of readout units are required during service life. Calibrations of any embedded components provided with in-place calibration check features are also required. These calibrations are usually performed by personnel responsible for data collection and should be under the direct control of the owner or instrument specialists selected by the owner.

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 acceptance test procedure. When this is insufficient, calibrations can often be made at local commercial calibration houses, using equipment traceable to an accepted standard agency. 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.

When readout instruments are attached within a permanent terminal enclosure, they also require regular calibration. Calibrations can be made by use of a standard device located in or carried to the terminal enclosure, or by detaching the instruments for calibration elsewhere. For example, a "master" gage can be carried to the terminal enclosure for twin-tube hydraulic piezometers and connected to the terminal system when calibrations of Bourdon tube pressure gages are required. All readout instruments attached within a permanent terminal enclosure should be arranged for easy detachment for calibration and/or replacement, and shutoff valves should be provided at all pressure gage connections.

When an in situ calibration or function check is possible, this should be included in the regular calibration schedule. For example, some fixed borehole extensometers can be checked for free-sliding of the wire or rod by moving the instrument head outward and measuring wire or rod elongation. Some instruments have two different types of transducer. Permeability tests can be made in open standpipe and twin-tube hydraulic piezometers to examine correct functioning. Sometimes it is possible to remove the load from load cells temporarily by making *lift-off* tests, allowing a check on the calibration.

Sections 8.4.3 and 8.4.17 provide guidelines on field checks of electrical instruments by using a circuit tester and a volt–ohm–milliammeter.

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. Certain instruments merit calibration spot-checks on each day that readings are taken. For example, an inclinometer should be checked on each day that readings are taken, in a test casing, field casing, or test stand as described in Section 12.8.6, and a portable depth micrometer should be checked within a portable reference standard. Other instruments require calibration spot-checks on a weekly or monthly basis, and a complete calibration is recommended at intervals of between 3 months and 1 year. A sticker on each instrument should indicate the last and next calibration dates.

When instruments are used for monitoring long-term performance, procedures for calibrations during service life should be included in the operating and maintenance manual.

16.2. INSTRUMENT MAINTENANCE

Regular maintenance required during service life is usually performed by personnel responsible for data collection and should be under the direct control of the owner or instrument specialists selected by the owner. Personnel responsible for data collection should always be on the lookout for damage, potential for damage, and deterioration or malfunction and should initiate repair or replacement without delay. Detailed maintenance requirements vary with each instrument and should be stated in the manufacturer's instruction manual (Chapter 5). The manual should include a troubleshooting guide, cleaning, drying, lubricating, and disassembly instructions, and recommended maintenance frequency. If batteries are required, service and charging instructions should be given in the manual.

As indicated in Chapter 5, appropriate spare parts should be ordered when instruments are procured to replace accessible malfunctioning components during service life. Provision should also be made for spare or standby readout units for use in case of malfunction. Options are procurement of a spare, arrangements with the manufacturer for rental of a spare if needed, and a standby equipment lease. Maintenance personnel should ensure that adequate spare parts and readout units are on hand or readily available. When instruments are used for monitoring long-term performance, maintenance procedures should be included in the operating and maintenance manual.

The following general guidelines indicate the factors to be considered when planning maintenance details.

16.2.1. Readout Units

Readout units should be kept clean and dry, following the manufacturer's instructions. Particular attention should be given to connections, including O-rings. If the readout unit includes gold-plated electrical connectors, these must be kept clean to prevent wear of the plating and corrosion of the underlying metal. If the readout unit is fitted with a humidity indicator, it should be dried whenever indicated. If no indicator is fitted, a special effort must be made to dry the unit when appropriate. Lubrication instructions should also be followed, as some readout units require periodic lubrication, whereas others must not be lubricated.

Protective caps and plugs are important, and their integrity should be maintained. If the manufacturer has not provided protective caps and plugs for terminals on readout units, users are advised to add these to their readout units. Caps and plugs should preferably be anchored to the readout unit to avoid loss and to remind reading personnel that they must be used.

Insufficient care is often given to battery charging and maintenance. Nicad and lead–acid batteries require different charging schedules, and either can be damaged by use of incorrect procedures. Guidelines on battery charging and maintenance, field checks with a volt–ohm–milliammeter, operating spares, and handling and transporting readout units are given in Section 8.4.17.

16.2.2. 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.

16.2.3. Embedded Components

Various embedded and normally inaccessible components require maintenance. For example, twin-tube hydraulic piezometer tubes usually require occasional flushing to remove gas bubbles, and tubing connected to pneumatic transducers may occasionally require flushing with a dry gas to remove moisture. 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 piezometers either suspended in standpipes or packed-off with pneumatic packers.

CHAPTER 17

INSTALLATION OF INSTRUMENTS

Installation of instruments requires a special effort. In his foreword to this book, Ralph Peck 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.

When planning for instrument installation, Murphy's law (*If anything can go wrong, it will*) should be foremost in mind. If any source of difficulty can be postulated, it should be taken care of at this stage.

Before installation of instruments can start, the monitoring program will have been planned and instruments will have been procured, following the guidelines given in Chapters 4 and 5. Planning includes selection of appropriate contractual arrangements for installing instruments, selection of instrument locations, preparation of detailed installation procedures, preparation of an installation schedule, and coordination of installation plans with other parties. These planning steps are described in detail in this chapter, followed by guidelines on field work and preparation of an installation report.

17.1. CONTRACTUAL ARRANGEMENTS FOR INSTALLING INSTRUMENTS

Five possible contractual arrangements for installing instruments are described in Chapter 6, required personnel qualifications are given, and a recommendation is made to avoid use of the low-bid method. Installation of instruments should preferably be under the direct control of the owner or of instrumentation specialists selected by the owner.

During the planning phase a designation will have been made of which tasks are within the capability of average construction contractors (*support work*) and which tasks require the skill of instrumentation specialists (*specialist work*). In the author's view, installation of instruments should be a team effort between support and specialist personnel, following the guidelines for cooperation given in Section 6.6.7. Drilling crews should not be relied on to install instruments in boreholes unless specialist personnel are intimately involved.

17.2. LOCATIONS OF INSTRUMENTS

Locations of instruments should be selected in accordance with the three-step approach given in Chapter 4, considering zones of particular concern, representative zones, and zones to serve as indices of comparative behavior. Locations should be shown on the contract plans, but the exact location of instruments should usually be determined in the

field, when geologic details and construction procedures are defined more closely than during the design phase.

17.3. DETAILED INSTALLATION PROCEDURES

Planning for installation of instruments should include preparation of detailed written installation procedures. If instruments are to be installed by the owner's personnel, 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, retaining key items for enforcement by the owner's representative, and the procedures will be used when reviewing the contractor's submittal of proposed installation methods.

The instrument manufacturer's instruction manual will often be helpful when preparing detailed installation procedures, but many manuals provide insufficient guidelines. When site-specific constraints must be met, as is normally the case, the manufacturer cannot be expected to provide complete and definitive procedures, and the user must plan detailed procedures to suit the specific site geotechnical conditions. However, these procedures must be flexible enough to account for unexpected conditions that arise during installation, such as unexpected ground conditions or changes in the construction contractor's procedure or schedule.

The various factors to be considered when preparing detailed installation procedures are described in Sections 17.4–17.10. Factors that are applicable to most instruments are included, and specific factors relevant to one instrument category are included in Part 3.

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. An example of a detailed installation procedure is included in Appendix G.

17.4. INSTALLATION AT THE GROUND SURFACE

Methods for installing instruments at the ground surface depend on the type of instrument, and various specific guidelines are given in Part 3. Some general guidelines are added in this section, particularly relating to protection of exposed components to minimize the chance of damage.

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. Flexible conduit is convenient, as leads can often be snaked through the conduit prior to installation, thereby minimizing interruption to normal contruction activities. Flexible spiral metallic conduit is often used but cannot be bent round a radius less than about 6 in. (150 mm). If watertightness is important, plastic sheathed flexible metallic conduit can be used (ends of conduit must also be sealed), and where small radius bends are required, rubber hose may be a better choice. In some cases rigid PVC or steel pipe or flexible corrugated polyethylene pipe may be preferable. When considering the options, users should bear in mind the smallest allowable bend for the tubes or cables and the need for waterproofing. If there is any chance of water entering a supposedly sealed conduit, either at the ends or through a defect, the sealing feature may in fact cause ponding of water, and an unsealed conduit may be a better choice. Cushioning must be provided between the leads and conduit at the exit points, for example, by using garden hose or by packing the space with burlap or soft rags.

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.

17.5. INSTALLATION IN BOREHOLES

Detailed procedures for installation of instruments in boreholes should ensure conformance, should satisfy requirements for borehole diameter, length, alignment, and wall roughness, and should include methods of drilling, installation of downhole components, and borehole backfilling. These issues are discussed in turn.

17.5.1. Conformance

When either changes in the length of the borehole or local shear displacements are anticipated during in-

strument 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. Alternatively, axially compressible corrugated plastic pipe can be installed around rigidly connected pipe, provided that the outer pipe is free to slide with respect to the inner pipe. When plastic tubes or electrical cables are installed in boreholes through soils that will undergo substantial vertical compression, tubes and cables can often be prespiraled prior to installation, by winding them around a pipe of suitable size. Spiral in a plastic tube can be made permanent by immersing the pipe and tube in hot water and allowing both to cool before removing the pipe (Handfelt et al., 1987). Of course, temperature limits for the plastic must not be exceeded, and tests should be made to determine the minimum required temperature and immersion time.

17.5.2. Borehole Requirements

Most instruments that are installed in boreholes require minimum and maximum allowable borehole diameters, and some have limitations on borehole alignment or wall roughness. Where the diameter is not critical, it should be ample, so that downhole components pass easily along the borehole during installation. Boreholes should generally be drilled a few feet longer than the instrument to ensure that debris falling or pushed to the end of the borehole will not obstruct the installation. Borehole requirements may include an enlargement or recess at the collar for the instrument head and additional small holes for mounting the head and protective arrangements.

17.5.3. 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. Detailed discussion and comparison of drilling methods are beyond the scope of this book, but methods in common use are described briefly. Dimensions of drill rods, flush-joint casing, diamond coring bits, hollow-stem augers, and U.S. pipe are given in Appendix F.

Wash Boring

The *wash boring* method (Figure 17.1) is used to advance driven casing through soil. Soil that enters the casing at the bottom during driving is removed by pumping water through a small-diameter wash pipe, with a washing or chopping bit attached to its lower end. Cuttings of soil are carried upward by the wash water within the annulus between the casing and wash pipe. Standard or heavy-duty threaded and coupled pipe is sometimes used as

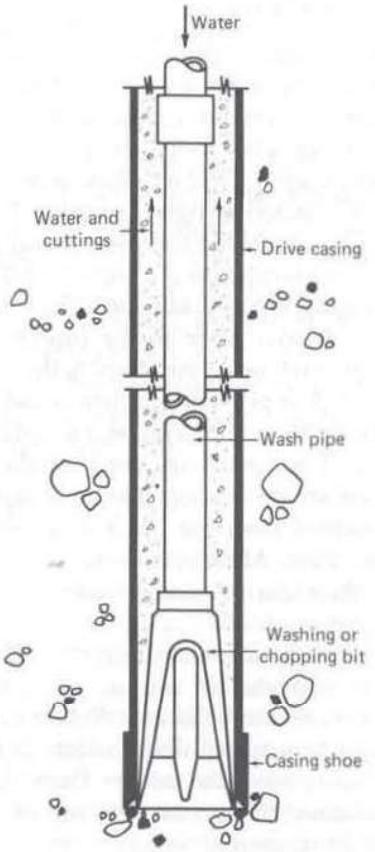


Figure 17.1. Wash boring method.

*Written with the assistance of Charles O. Riggs, Manager of Research and Marketing, Central Mine Equipment Company, St. Louis, MO.

drive casing, but the exterior couplings can impede casing removal, particularly in granular soils. It should be used only when removal will present no problems and, when it is used, individual pipe ends should be reamed to ensure a flush inside diameter at the couplings. *Flush-coupled* casing, of the type sometimes used to case overburden for core drilling, is generally not suitable for supporting boreholes that are to be used for installation of instruments, because the inside diameter at a coupling is significantly less than the inside diameter of the casing. *Flush-joint* casing, another type often used during core drilling, is flush inside and outside and is the most suitable choice.

The wash boring method is frequently used for installing instruments in soil when there is a need to be certain of a clean stable borehole, as is often the case when installing piezometers, inclinometer casings, probe extensometers, and fixed borehole extensometers. The method is slow and therefore expensive but, in the view of the author, it provides a good defense against Murphy's law.

Auger Drilling with Hollow-Stem Augers

Auger drilling can be performed using either continuous flight solid-stem or hollow-stem augers, but solid-stem augers are not usually applicable for advancing boreholes for installation of instruments. As shown in Figure 17.2, a hollow-stem auger column has a continuously open axial stem that allows the borehole to be drilled and cased simultaneously. Steel flights are welded to the outside of the stem to form the auger, which is advanced by rotation and application of axial force to the top of the auger column. It is often necessary to plug the aperture at the auger head to prevent problems caused by soil entering the hollow stem. The pilot assembly shown in Figure 17.2 is sometimes used for this purpose and also assists in drilling. The pilot assembly is removed before inserting an instrument through the hollow stem. Alternatively, a flat expendable *knock-off* plate can be used on the bottom of the auger column.

Drilling with hollow-stem augers is used for installation of piezometers and inclinometer casings in soil and occasionally for installation of other instruments for measuring deformation. It is usually necessary to remove the augers from the ground without rotation, to prevent twisting of downhole instrument components.

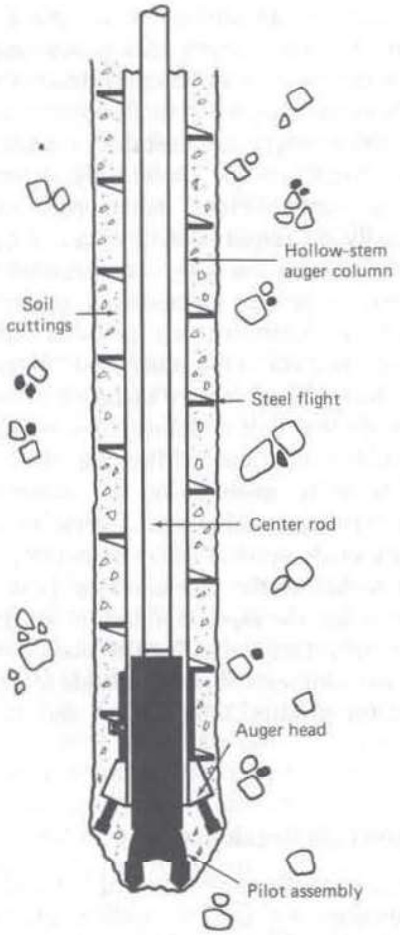


Figure 17.2. Auger drilling with hollow-stem augers.

Rotary Drilling

Rotary drilling (Figure 17.3), also called *hydraulic rotary drilling, straight rotary drilling,* or *mud rotary drilling,* can be used in soil or rock. No core is recovered while drilling but, when drilling in soil, samples can be recovered by intermittent sampling at the bottom of the borehole. A bit on the bottom of a column of drill rods is rotated and axially advanced. A drilling fluid is pumped downward through the drill rods, and cuttings of soil and rock are carried upward within the annulus between the drill rods and the wall of the borehole. A correctly "designed" drilling fluid will penetrate the wall of the borehole and form a thin "mud cake," thereby maintaining the level of fluid in the borehole to provide supporting pressure on the wall of the borehole. Bentonite (powdered sodium montmorillonite) is the most commonly used drilling fluid

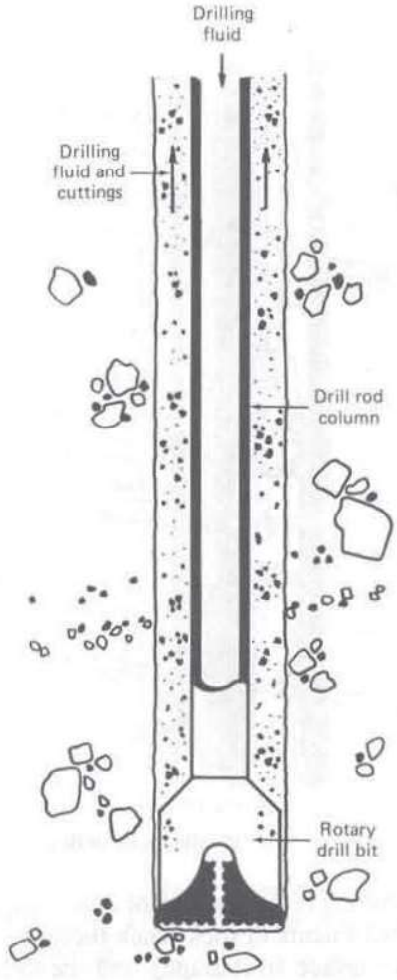


Figure 17.3. Rotary drilling.

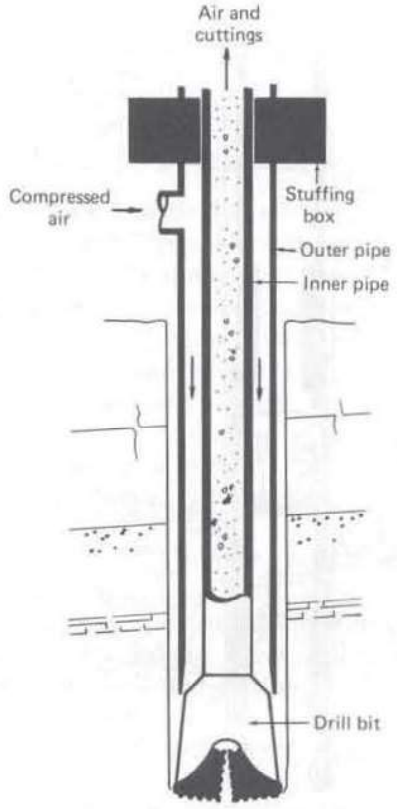


Figure 17.4. Drilling by double-tube reverse air circulation.

additive. The most commonly used bit is the *tricone* bit, fitted with three rotating toothed cones, but a *drag* bit with three or four blades is often used in soil.

Rotary drilling can be used for installation of instruments in soil and rock when the stability of the borehole is certain. If there are doubts about stability, casing or hollow-stem augers should be used instead of drilling fluid: the author has experiences of instruments becoming stuck in a fluid-filled borehole before they have reached the planned depth! Under some circumstances a flush-joint casing should be advanced with rotary procedures to ensure a stable borehole. Conventional bentonite drilling fluid should not be used in boreholes for piezometer installations, because the additives will permanently impede the functioning of the piezometer by severely decreasing the permeability of the surrounding soil. However, as described in Chapter 9, a drilling fluid containing a biodegradable organic polymer can be used.

A special type of rotary drilling, *double-tube reverse air circulation,* has some application for installation of instruments, and is shown in Figure 17.4. With this method an outer pipe is advanced as the cuttings are forced upward at high velocity within a small inner pipe.

Core Drilling

Core drilling can be used in soil or rock, and a core is recovered. A core barrel attached to a column of drill rods is rotated and advanced, and drilling fluid flushes and cools the bit on the end of the core barrel. The soil or rock at the advancing end of the core barrel is cut or abraded by the bit, cuttings are carried out of the borehole by the drilling fluid, and a cylinder of soil or rock (the *core*) enters and is protected by the core barrel. Drill casing is used when necessary to ensure that overburden soil or

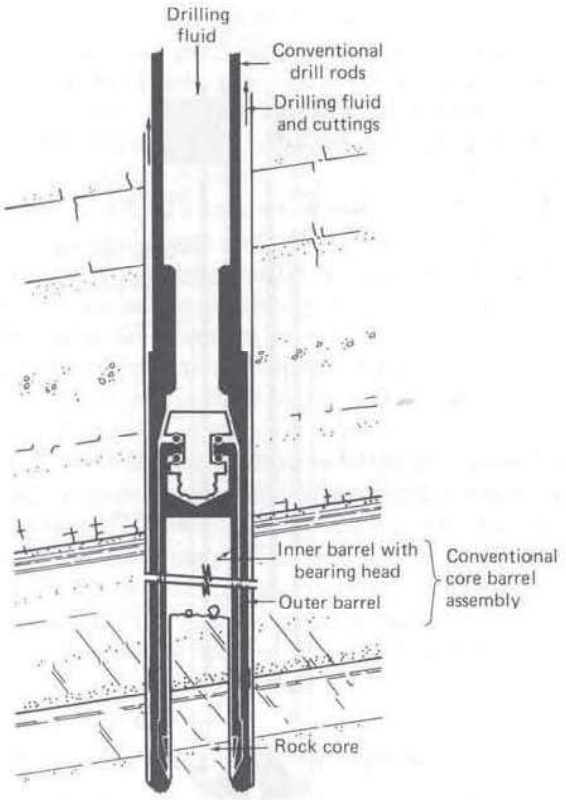


Figure 17.5. Conventional core drilling.

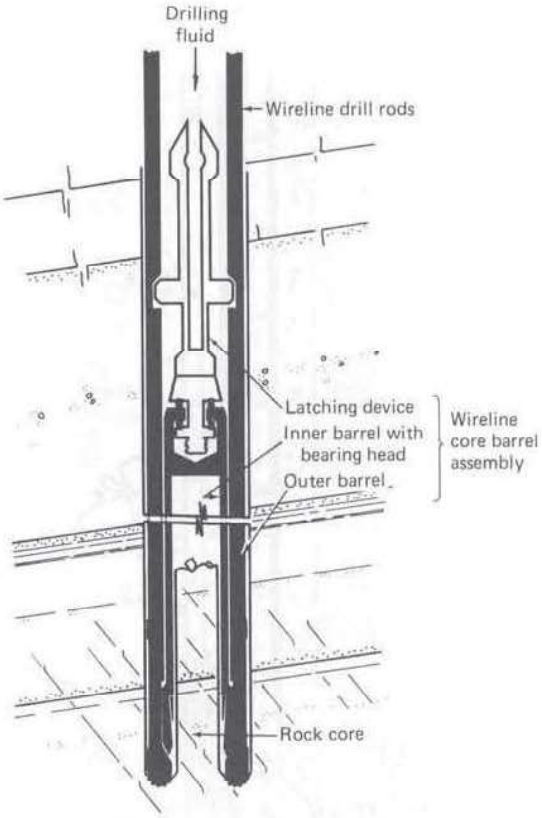


Figure 17.6. Wireline core drilling.

rock does not enter the borehole and interfere with coring, and it also provides a reliable conductor for the return of drilling fluid. Core drilling is performed either with *conventional* or *wireline* systems, as shown in Figures 17.5 and 17.6. When a conventional core barrel is used, the complete drill rod column must be withdrawn from the borehole to remove the increment of core from the core barrel. When a wireline core barrel is used, the inner barrel, containing the increment of core, is withdrawn from the borehole on a "wireline," which attaches to the latching device shown in Figure 17.6. The bit, outer barrel, and drill rods remain in the borehole. The hardest rocks, such as quartzite, are often cored most economically by using impregnated diamond bits, with small manufactured diamonds contained in the bit crown matrix. Softer rocks, such as limestone, are often cored using surface-set bits, with larger natural diamonds set at the surface of the crown matrix. Soft rock is often cored using carbide insert or polycrystalline (carbide and manufactured diamond) insert bits. The evaluation of rock cores is usually enhanced greatly if split inner tube core barrels are used.

Core drilling is the method of choice for installation of instruments in rock when there is a need to define subsurface stratigraphy and the locations of discontinuities: this is typically the case when installing instruments to measure deformation, stress change, and joint water pressure. Core drilling is not often used for installation of instruments in soil.

Rotary Percussion Drilling

Rotary percussion drilling (Figure 17.7) involves simultaneous rotation and percussion of a drill bit to fracture the material at the end of a borehole. No core is recovered. The most common drills are rotary-impact rock drills, called *drifters*, usually mounted on track carriers. Alternatively, down-the-hole rotary-impact hammers can be used. The borehole is flushed with air or water. Rotary percussion drilling is not a frequent choice for installation of instruments, because locations of discontinuities cannot be defined. Moreover, significant deviations in alignment often occur, so that the locations of downhole components are unknown. When these limitations are acceptable, the method can be used

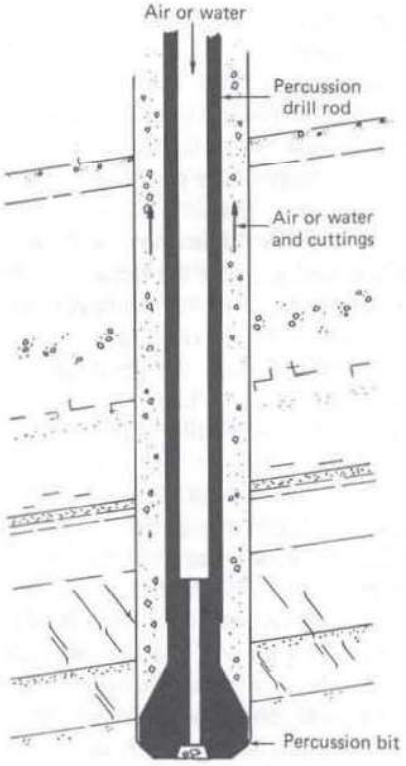


Figure 17.7. Rotary percussion drilling.

economically, particularly when drilling equipment is on the site for other purposes, such as drilling holes for blasting.

Controlling Borehole Alignment

When borehole alignment is critical, for example, when installing piezometers between closely spaced vertical drains after the drains have been installed or when installing fixed borehole extensometers or inclinometer casings above or alongside a future tunnel, care must be taken to set up the drill rig at the correct alignment and to use drilling procedures that minimize deviation from this alignment. When using the wash boring method to install drill casing in soft ground, an oversized casing shoe should be used and the casing advanced by spinning and washing, rather than by driving.

Directional drilling techniques can be used to maintain borehole alignment or to deviate deliberately from the initial alignment. They have been developed for use in production of oil and gas but are occasionally relevant for installation of instruments, for example, plumb lines and inverted pendulums. The technique involves inserting a deflection wedge at the bottom of the borehole. Many experienced and competent core drillers have never used directional drilling techniques.

Guidelines Applicable to All Drilling Methods

Whichever drilling method is selected, the detailed written installation procedures should include the drilling method and requirements for drilling equipment, sampling methods and frequency, maximum and minimum borehole diameters, casing or auger diameter, drilling fluid additives, drilling records, sample containers, and storage and submittal of samples.

During drilling, a conventional borehole log should be maintained, and any required samples should be taken.

After completion of drilling, the borehole should be thoroughly cleaned with air, water, or drilling fluid and the length and diameter checked. When water is used for drilling, many drillers will wash the borehole by inserting drill rods or a jet pipe to the bottom of the borehole and flushing water through the rods or pipe. However, better results are obtained by reversing the jetting pump and using the drill rods or jet pipe as the intake, with the lower end a few inches from the bottom of the borehole. The borehole is kept filled by pouring clean water in until all cloudy water is pumped out. This is referred to as the *reverse circulation method* and maximizes the velocity of outgoing water. When a borehole in rock is advanced by diamond coring methods, the hole should require minimal cleaning. Minimum diameter can be checked by passing a suitably sized "dummy" along the borehole and, where more complete data are required, a borehole caliper can be used. When borehole alignment, inclination, or straightness is critical, a borehole directional survey (Chapter 12) may be appropriate.

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. When using grout backfill in an unsupported borehole, the borehole should first be tested for grout-tightness by removing drill rods and conducting a permeability test, usually by running water into the borehole to maintain water level at the borehole collar. The author has used a maximum water inflow of 1 U.S. gallon per 10 feet (1.3 liters/meter) of borehole per 15 minutes as an ac-

ceptability criterion. If the inflow exceeds an acceptable value, the borehole must be sealed with bentonite or other sealing compound, or grouted and redrilled, and tested again. Such testing is not possible in boreholes containing drilling fluid or when drill casing is used throughout the borehole length, and a conservative estimate of grout-tightness must be made. Testing and sealing can of course be performed in any borehole length below the bottom of the drill casing.

17.5.4. Installation of Downhole Components

If significant time has elapsed since completion of drilling and checking for length, diameter, and grout-tightness, the checks should be repeated prior to inserting downhole components and the borehole again cleaned and sealed if necessary. The method of installing downhole components depends on the type of instrument and drilling method, but several general guidelines can be given.

When maximum tensile strength is required across socket couplings in plastic pipe or when plastic pipe is to be used for high-pressure applications, a primer should be used before the solvent cement is applied. The primer etches the surfaces and allows proper adhesion. Dimensions of steel and PVC pipe that are often used as downhole components are given in Appendix F.

Downward Boreholes

If the instrument is heavy, a ¼ in. (6 mm) manila rope or ⅛ in. (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. A hand-operated winch is useful for controlling this safety line when instruments are heavy or when boreholes are deep. The safety line will normally be cut after it has served its purpose and left in place. If, on the other hand, 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.

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 70 to 100 lb/ft^3 (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, 1 in. (25 mm) diameter plastic tubes or pipes are required, but a smaller diameter can be used for thin grout mixes, and a larger diameter may be required for very thick mixes or for compressed bentonite pellets (Chapter 9). 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 9.3, is usually the best choice: sizes are given in Appendix F. If tubes or pipes are to be left in place, they should be taped securely to other downhole components. If they are to be recovered, light taping at the bottom is necessary for tubes but usually unnecessary for pipes. If backfill is to be placed in stages, a single tremie tube or pipe can be pulled upward as backfilling progresses, or a separate length can be installed for each stage.

Many borehole instruments are shipped in short lengths, and installation time can often be reduced by preassembling components into longer lengths prior to installation. Certain instruments such as fixed borehole extensometers with spring-loaded anchors require insertion rods, and these must be inserted in the borehole as installation of downhole components proceeds. Other instruments such as inclinometer casings and multipoint fixed borehole extensometers require careful control of orientation as they are installed.

Near-Horizontal Boreholes

When installing in near-horizontal boreholes, insertion rods are often required. These can usually be rented from the instrument manufacturer, or lightweight pipe such as aluminum electrical conduit can be used. Fiberglass chimney sweep rods can also be considered for this purpose.

If a grout backfill has to be used, it is preferable to incline the borehole downward 5–10 degrees and to use a single grout tube with its outlet at the bottom of the borehole. If there is any doubt about the alignment of the borehole or if it must be as horizontal as possible, an alignment survey should be made prior to locating grout outlets and vents. For example, alignment surveys are essential when near-horizontal holes are drilled in rock by percussion methods and when a grout backfill will be used,

because borehole alignment is always uncertain. A borehole directional survey instrument or full-profile liquid level gage can be used. Downward boreholes that hold water can be surveyed by traversing a diaphragm piezometer along the hole, and at several depths measuring water head and lead length. If the borehole will not hold water, it may not hold grout, and it should be sealed as described above. Grout outlets must be provided at the lowest point in the borehole and at each trough and a vent at the highest point and at each crest.

Upward Boreholes

When installing in upward boreholes, insertion rods are also required. A hydraulic anchor designed for use with fixed borehole extensometers can often be attached to the upper end of the instrument and used to hold the system in place after insertion rods are withdrawn. If a grout backfill is to be used, stage grouting is necessary, first to seal around the instrument head at the collar of the borehole and later to fill the remainder of the borehole in one or more stages, and grout outlets and vents must be installed for each stage.

17.5.5. Selection of Backfill Material

Alternative backfill materials for boreholes include grout, granular fills such as sand and pea gravel, and bentonite pellets. Grout backfill is most frequently used, is suitable when the borehole must be sealed against water flow, and can be used for all borehole alignments. Grout backfill is more likely than granular backfill to fill the borehole completely but cannot be used if grout would bleed into the surrounding ground. Where permeable strata overlie impermeable strata, grout backfilling is possible in the impermeable zone, and granular backfill can be placed in the permeable zone after grout has set. Granular backfill cannot readily be installed in upward boreholes and should not be used where its high permeability would create nonconformance by allowing flow of water along the borehole. Bentonite pellet backfill is used for installation of piezometers in boreholes. Alternative backfill materials are discussed in Sections 17.5.6–17.5.8.

17.5.6. Backfilling Boreholes with Grout*

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. When probe extensometers rely 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 casings should satisfy criteria for maximum and minimum strength. For these reasons there is no universally suitable grout, and each installation must be considered individually.

While assembling material for this section of the book, by reviewing the contents of the technical literature and by discussing with engineers who have expertise in grouting, two overriding factors became evident. First, little is known on the subject of grouting around instruments in boreholes, presumably because this use of grout represents such a tiny proportion of overall grout use. The following are useful references, but none relates specifically to instrumentation: ACI (1981), Dames & Moore (1979), Deere (1982), Jefferis (1982), and Littlejohn (1982). Second, 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, these realizations 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.

Some typical mixes will be given, but it is emphasized that they should not be used as a "cookbook." 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. Where a more exact measurement of grout properties is required, laboratory tests of trial mixes will be needed, following the method described by ASTM (1980).

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

*Written with the assistance of Joseph P. Welsh, Vice President, GKN Hayward Baker, Inc., Odenton, MD.

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. Properties 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.

The following are some examples of grout mixes used during installation of instruments in boreholes.

1. Water/cement/bentonite, 1/0.15/0.06 by weight (Easton, 1984). Bentonite and water mixed first. Used for probe extensometers that rely on grout to ensure conformance in soft clay. The 28 day unconfined compressive strength tested as approximately 500 lb/ft^2 (25 kPa). Minnitti (1985) tested a similar mix, concluding with a 28 day unconfined compressive strength of approximately 300 lb/ft^2 (15 kPa). Minnitti also tested a water/cement/bentonite mix, 1/0.08/0.11 by weight, giving a 28 day unconfined compressive strength of approximately 400 lb/ft^2 (20 kPa).
2. Bentonite/cement ratio of 0.10/1 by weight, with sufficient water to allow pumping. Recommended by AASHTO (1978) for backfilling around inclinometer casings.
3. Hydrated lime/cement ratio of 1.6/1 by weight, with sufficient water to allow pumping. Recommended by AASHTO (1978) for backfilling around inclinometer casings. However, the high pH in hydrated lime can cause corrosion of aluminum and should therefore not be used with aluminum inclinometer casing.
4. Bentonite/cement ratio of 0.15/1 by weight, with sufficient water to allow pumping. Recommended by Corps of Engineers (1971) for sealing above open standpipe piezometers.
5. Sand/cement ratio of 5/1 by weight. Recommended by USBR (1974) for sealing above open standpipe piezometers.
6. Fetzer (1982) used two mixes to seal above open standpipe piezometers. A 5 ft (1.5 m) seal was first made with a water/bentonite, 1/0.3 mix by weight, followed by a water/cement/fine sand, 1/1.9/2.7 mix by weight, with expansion agent and retarder.

Whenever quoting proportions of grout mixes, it is essential to indicate whether proportions are by weight or by volume: there is significant confusion in the technical literature because this is not always done. Useful conversion factors for preparation of grout mixes are given in Appendix H.

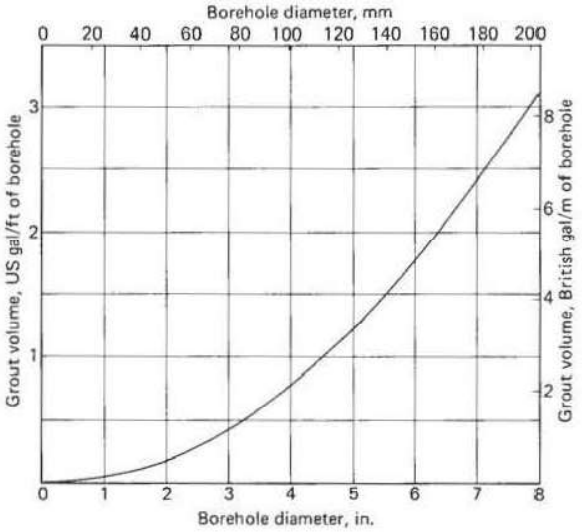


Figure 17.8. Required grout volumes for borehole installations (allowing for 20% waste).

Bentonite is available in a form specifically designed for use in salt water, but a much larger quantity of bentonite is required (Handfelt et al., 1987) and experience has not always been satisfactory. It may be more appropriate to mix conventional bentonite with fresh water when grouting boreholes in a salt-water environment or to use a chemical sealant (Driscoll, 1986; Senger and Perpich, 1983).

Sand is rarely used in grouts for backfilling boreholes, but additives such as expanders, fluidizers, and retarders* may be included, and on occasion chemical grouts may be appropriate, but in all cases the advice of grouting specialists should be obtained.

Various practical guidelines for backfilling boreholes with grout are given in this paragraph. Boreholes at all alignments can be backfilled with grout but, as discussed earlier in this chapter, multiple grout outlets and vents and stage grouting may be required. Prior to grouting, a check should be made to ensure that a sufficent quantity of each ingredient is readily available. Figure 17.8 indicates required grout volumes, including a 20% margin for waste, for various borehole diameters. Water should be circulated through the grout tube or pipe

*Some commercial sources are listed in Appendix D.

to verify that the circuit is open. When downhole components include a vertical pipe of adequate diameter, the borehole can be grouted from within the pipe by using one of the check valve arrangements shown in Figure 12.74 instead of an outside tremie pipe. These arrangements often allow use of a smaller-diameter borehole. In general, downhole vertical pipes and tubes should be filled with water or oil while grouting to avoid collapse owing to external pressure, and grout pressures should be controlled to avoid collapse. For very deep installations stage grouting may be required, in which case an outside tremie pipe must be used. When there is a possibility of grout entering instrument components, for example, the top of a vertical pipe when grouting outside the pipe, the possible grout entry point should be sealed. Grout should be mixed thoroughly by circulating through the grout pipe for sufficient time or by use of a colloidal mixer if available. A fine mesh such as window screen should be fitted over the suction intake during grouting to ensure that lines are not blocked by lumps resulting from incomplete mixing. When withdrawing any drill casing or augers during or after backfilling a borehole, rotation should not be allowed, otherwise downhole components may become spiraled.

When grouting under artesian conditions, one of the methods described in Chapter 9 for installing piezometers can be used. Alternatively, barium sulfate can be added to the grout to increase density, taking care to top up with grout while withdrawing casing.

17.5.7. Backfilling Boreholes with Sand or Pea Gravel

Use of sand or pea gravel backfill is limited to downward boreholes, and the borehole diameter should be large enough to discourage bridging, generally at least 2 in. (50 mm) larger than the outside diameter of the downhole components. 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 2 ft (600 mm) maximum layers. A flush-coupled rod or pipe, such as fixed piston sampler actuating rods with a special tamping foot, is useful for tamping, measuring depths, and breaking bridges. When it is important to center the components in the borehole, the sounding hammer described in Chapter 9 should be used. If possible there should be no protruding couplings in downhole components. Internal vibrators supplied by some manufacturers of inclinometer casing often have insufficient power for proper compaction of backfill.

Second, sand or pea gravel can be tremied to the bottom of the borehole. A 1 in. (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. For both methods, if drill casing or hollow-stem augers have been used, they should be withdrawn, without rotating, in small increments to avoid collapse of the borehole prior to backfilling. Experience with each installation will indicate the allowable increment, which may vary from 6 to 30 in. (150–750 mm). Withdrawal should usually precede backfilling, because if backfill is allowed to settle within the casing or augers it may "grab" downhole components and cause them to be lifted when casing or augers are withdrawn. Depth measurements should be made repeatedly while backfilling to verify correctness of the backfill level. It is helpful to mark the tamping rod or tremie pipe at a distance from its bottom equal to the length of casing or augers, so that the mark is alongside the top when the backfill is placed up to the bottom.

Figure 17.9 indicates the quantity of Ottawa sand required to fill various sizes of borehole.

17.5.8. Backfilling Boreholes with Bentonite Pellets

Backfilling boreholes with compressed bentonite pellets for installation of piezometers is described in Chapter 9. Figure 17.9 indicates the quantity of compressed bentonite pellets required to fill various sizes of borehole, assuming a unit weight of 74 lb/ft^3 (1.19 Mg/m^3). This unit weight is typical, but some commercial sources have unit weights up to 82 lb/ft^3 (1.31 Mg/m^3).

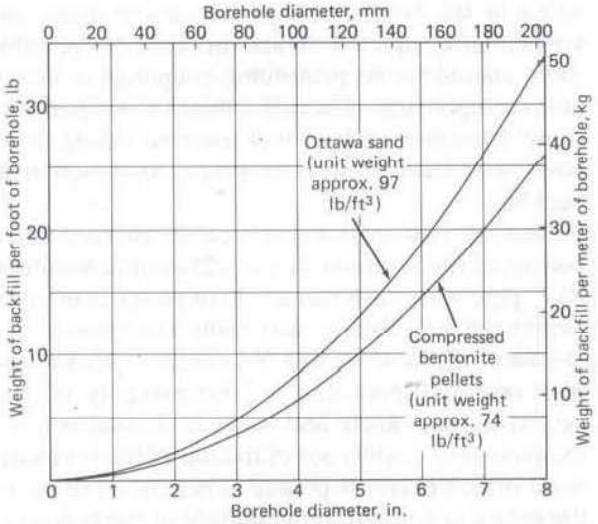


Figure 17.9. Required volumes of Ottawa sand and compressed bentonite pellets for borehole installations (no allowance for waste).

17.6. 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, inclinometer casings, and fixed embankment extensometers will normally require either axially compressible access pipe or telescoping couplings in rigid access pipe. Alternatively, a rigid access pipe can be surrounded by an axially compressible or telescoping pipe, provided that this outer pipe is free to slide with respect to the access pipe. As an illustration of damage that can be caused to pipes by deformation of fill, Figure 17.10 shows dramatic rupture of rigid plastic standpipes attached to piezometers, found in an exploratory shaft in an embankment dam. Plastic tubes and electrical cables also often require special measures to protect them from damage by localized ground deformation.

The following sections provide guidelines on installation of pipes, tubes, and cables. Factors relevant to a single instrument category and guidelines

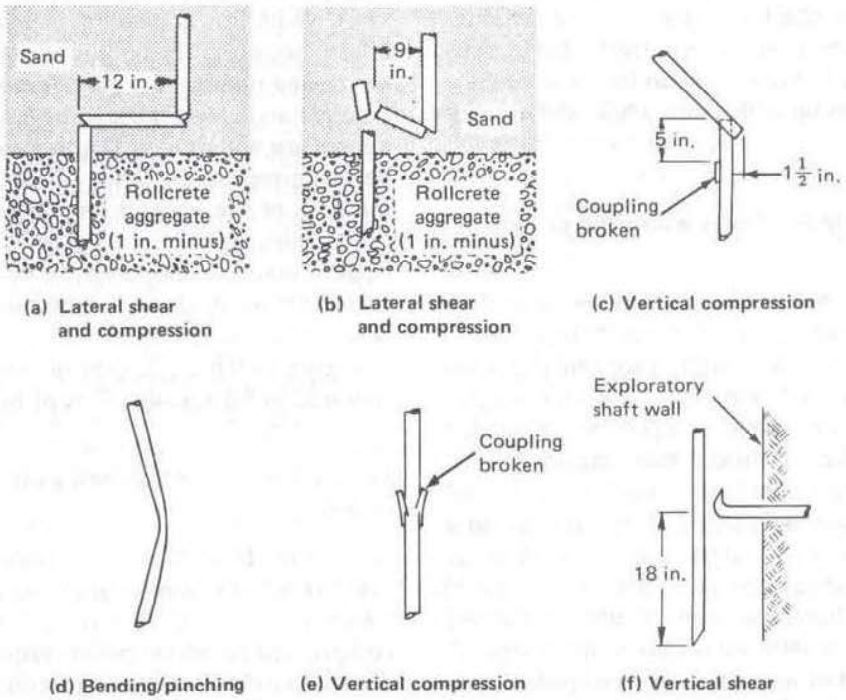


Figure 17.10. Rupture of piezometer standpipes found in an exploratory shaft in an embankment dam (after Mikkelsen and Wilson, 1983).

on installation of various individual instruments are included in Part 3. In general, the engineering properties of fill material around pipes, tubes, and cables should match, as closely as possible, the properties of the surrounding fill and should not provide a leakage zone for water. This need is of paramount importance in embankment dams. 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. As discussed in Section 17.5.4, when maximum tensile strength is required across socket couplings in plastic pipe or when plastic pipe is to be used for high-pressure applications, a primer should be used before the solvent cement is applied. Before tubing is installed, it should be tested at a pressure well in excess of the anticipated pressure in the field. Constrictions in tubing have been found on several occasions and, unless prior experience or testing indicates that constrictions are not present, an appropriately sized ball bearing should be blown through the tubing. Tubing should be stored away from direct sunlight, and ends should be capped during storage to prevent the entry of dust and insects. All pipes, tubes, and cables that are installed in groups should be marked individually at approximately 20 ft (6 m) intervals so that they can be identified if they are damaged and repairs are attempted.

17.6.1. Horizontal Pipes

Telescoping couplings must be sealed to prevent intrusion of fill material. O-rings form the best seal or, alternatively, soft rubber tubing can be installed to span between coupling and pipe, with a hose clamp at each end. Various filler materials can be packed in the annular space between pipe and coupling, including pliable sealing compounds manufactured for waterproofing and insulating electrical connections, or grease-impregnated tapes.

Pipes are usually placed in a trench 1–2 ft (300–600 mm) wide by 2–3 ft (600 mm to 1 m) deep, bedded on and covered by approximately 6 in. (150 mm) of suitably graded fill material. The allowable maximum particle size in the material depends on the required engineering properties of the backfill, the strength of the pipe, the depth of cover, and the type of construction equipment that will operate over the backfilled trench and will normally be between 0.2 and 0.5 in. (5–12 mm). Horizontal pipes installed in rockfill are normally surrounded with a layer of finer material such as sand or pea gravel, and intermediate layers of coarser material may be required to prevent migration of particles. A deeper trench may be necessary if heavy compaction equipment will operate over the backfilled trench. The remainder of the trench is then filled with excavated material in 4–8 in. (100–200 mm) lifts and compacted with hand-controlled power-operated equipment.

An alternative installation method, whereby pipes are installed on the surface of previously compacted fill rather than within a trench, is included in Section 17.6.3, describing installation of horizontal tubes and cables.

When horizontal pipes are installed to allow passage of a probe, such as a probe extensometer, inclinometer, or full-profile liquid level gage, a method of traversing the probe 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, three methods are possible. First, fiberglass chimney sweep rods can be used to push the probe up to about 200 ft (60 m), using a fitting on the probe for attachment of the rods. This method is suitable only if the pipe is reasonably straight and if there are no irregularities on the inside wall of the pipe, such as telescoping couplings. Second, 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. The third method uses a duct motor developed by post office engineers (Deadman and Slight, 1965; Penman and

Charles, 1973) for threading telephone wires along ducts. The duct motor is driven by compressed air and consists of a double-acting telescopic plunger with a valve mechanism causing it alternately to extend and contract as long as the air supply is maintained. The duct motor and probe are driven to the inaccessible end of the pipe and readings taken while withdrawing the probe.

17.6.2. Vertical Pipes

Telescoping couplings in vertical pipes must be sealed, as described above for horizontal pipes, to prevent intrusion of fill material. When settlement is anticipated, the couplings must be installed in an open position, and the sealing arrangements help to prevent closing during installation. When the instrument consists of a rigid access pipe surrounded by an axially compressible or telescoping pipe and when water leakage must be prevented, the annular space between the pipes should be filled with a slurry.

Two methods are available for extending a pipe upward as fill is placed: the *mounding* method and the *excavate and replace* method. In the mounding method, the pipe is installed to protrude above the fill level and protected with a hand-compacted mound of fill. Typical maximum height of the mound is 5 ft (1.5 m). In the excavate and replace method, the pipe is capped, filling and compaction continued over the pipe for about a 5 ft (1.5 m) depth, a local excavation made to locate the cap, a new pipe length added, and the excavation backfilled and compacted with hand-controlled power-operated equipment. The latter method usually results in a more straight and vertical extension than the former. Whichever method is used, great care is needed to avoid damaging the pipe, compaction around the pipe tends to be inferior, and interruption to normal filling operations is costly. Maximum particle size and engineering properties of fill adjacent to the pipe should follow the guidelines given above for horizontal pipes.

17.6.3. Horizontal Tubes and Cables

When planning installation of horizontal tubes and cables, special attention should be paid to routing, avoiding wherever possible traversing zones of large differential strain and providing slack at any such transitions. Tube and cable splices should be reserved for repair work and should not be included in the planned arrangements unless absolutely necessary. When splices are used, they should be made with great care as described in Chapter 8.

Horizontal tubes and cables are normally installed in trenches, bedded on and covered by suitably graded material. Trench size and backfilling arrangements are generally as described in Section 17.6.1 for horizontal pipes, but the allowable maximum particle size will usually be smaller than for pipes. Where armoring or conduit is not used, typical maximum size is 0.2 in. (5 mm). Particle size restrictions may also be needed in the first two lifts of backfill overlying the 6 in. (150 mm) covering layer. In rockfill, two or more layers of protection will usually be required, for example, sand and pea gravel, and gradations should follow filter design criteria. Individual tubes and cables should be separated by approximately 0.5 in. (12 mm) and, where several are installed, a wood or metal rake similar to a garden rake is helpful when controlling spacing. No tube or cable should be allowed to cross over another. Installation of tubing for twin-tube hydraulic piezometers, following the general procedures outlined previously, is described by Bartholomew et al. (1987) and USBR (1974).

Clements (1982) describes an alternative, which he refers to as the *surface installation method,* to installation of horizontal pipes, tubes, and cables in trenches. Clements describes in detail the disadvantages of trench installation, including high cost, lengthy time required, difficulty in excavating to a level bottom, difficulty in maintaining a steep-sided trench in granular fill, problems of maintaining the maximum particle size, and ponding of rainwater. The writer describes a "tramline" effect, whereby continuous lines of material are disturbed by backhoe teeth to a depth of up to 8 in. (200 mm) below the bottom of the trench. He states that there has been recent evidence to suggest that isolated instances of limited piping have occurred along instrumentation trenches within completed embankment dams and comments that although the tramline factor is not necessarily the cause of such happenings, it represents a weakness in the installation method that could result in piping effects.

The alternative recommended by Clements entails installation of pipes, tubes, and cables on the surface of previously compacted fill as shown in Figure 17.11. The first stage requires cutting a swath with a bulldozer and dumping a mound of loose parent material alongside the swath to provide protection from construction traffic. Figure 17.11

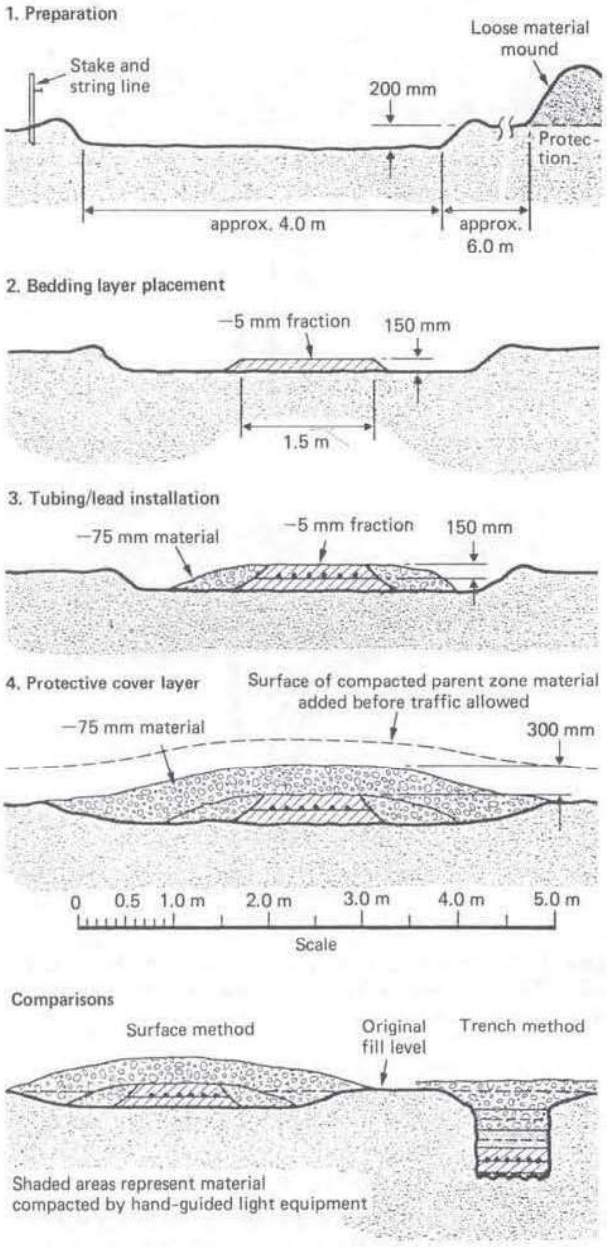


Figure 17.11. Stages in the *surface installation method* for tubes and cables (after Clements, 1982).

shows a swath 8 in. (200 mm) deep but, depending on the surface level control prior to cutting, the depth may vary. During the second stage, a bedding layer is placed, using a skid-mounted hopper approximately 8 ft (2.5 m) wide and 13 ft (4 m) long, with fixed top and side guide plates at the hopper discharge mouth to ensure a continuous and regular layer. The hopper is towed along the swath centerline by a bulldozer and filled using a front end loader. The layer is compacted using a self-propelled vibratory roller with a dynamic force of 5000 lb (22 kN), the first pass without vibration, followed by a specified number of vibrating passes. Instrument leads are then placed and covered using a skid-mounted guide and hopper. Before the initial protection layer is compacted, it is necessary to support the sides by placing parent zone material against it, and the ridge of protection material at the side of the swath provides a convenient source. Once placed, the material is hand-picked to remove rocks larger than 3 in. (75 mm) in diameter and to match the level of the uncompacted protection layer. The whole is then compacted with hand-guided vibratory plate compactors. This is the only stage at which heavy compaction equipment is not used. The final stage of the operation involves placing a 12 in. (300 mm) layer of up to 3 in. (75 mm) material over the swath area. Much of this work can be done by a bulldozer pushing loose material from the protective ridge at the side of the swath. Once spread, the material is hand-picked to remove oversize rocks. During this operation, care must be taken to prevent vehicle tracks from advancing over the instrument leads. Once the layer is leveled, compaction is accomplished by the self-propelled vibratory roller, the first pass being made without vibration. Normal construction and compaction techniques are restored for the next and subsequent lifts. It is also recommended that 12 in. (300 mm) of added cover is placed over the installation before loaded haulage equipment is allowed to cross it. The installation procedure is now complete and the area may be handed over to normal fill placement operations.

Clements indicates that the surface installation method is much quicker than the trench method, uses less personnel, is less disruptive to routine construction activities, results in better conformance between backfill and surrounding compacted fill, and does not allow ponding of rainwater. Clements also comments that, because the method is quicker, manual screening of protection material may create difficulties, and a small mechanical screening plant may be worthwhile.

Despite the advantages of the surface installation method, the author believes that it should be used only when there is good supervision of general fill placement: it is all too easy for mounds of fill to be pushed around during subsequent fill placement, and bulldozer drivers have a natural tendency to

level out the surface for the next layer. The attractiveness of the surface installation method clearly depends on specific site factors, and the primary application appears to be for installation of instrument leads during construction of embankment dams.

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. Conduit should not be used where passage of water along the instrument leads must be prevented, but even in these cases individual leads can be threaded within minimum-sized plastic tubing where they pass through zones that will experience significant strain, and the annular space filled with petroleum jelly. This arrangement, for example, can be used for instrument leads that pass from the core of an embankment dam to the downstream filter, the tubing extending 3–10 ft (1–3 m) either side of the interface.

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 17.12a. 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 17.12b.

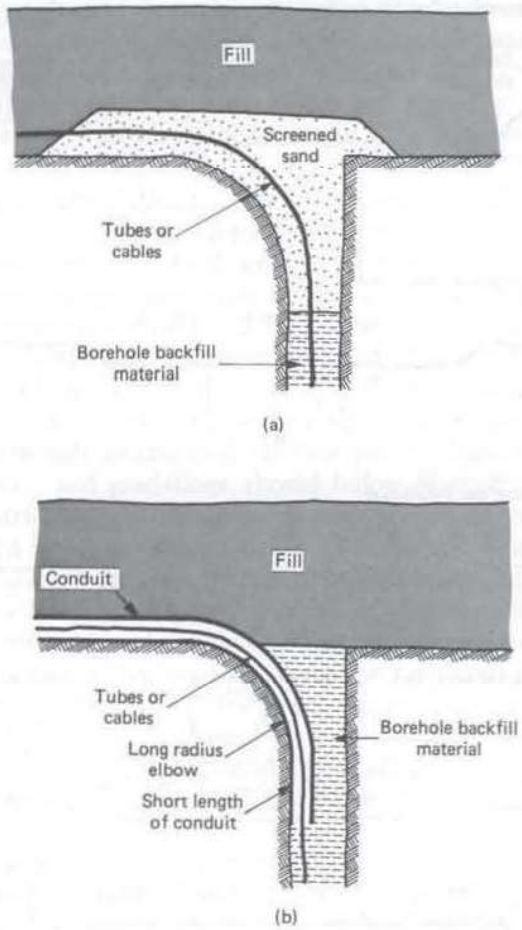


Figure 17.12. Typical arrangements for protecting tubes and cables where they emerge from a borehole: (a) without conduit and (b) with conduit.

17.6.4. Vertical Tubes and Cables

The guidelines on routing, splices, and identification tags given above for horizontal leads apply also to vertical leads. However, vertical runs of tubes and cables should be avoided wherever possible: they are a nuisance to construction, are liable to be damaged, and impede the quality of compaction. Vertical runs have sometimes been protected within a vertical pipe, leaving the pipe in place and without backfilling inside the pipe, but this practice runs the risk of damaging leads at the point of entry or exit and is usually unsatisfactory. When vertical runs cannot be avoided, they should be kept to a minimum. Protection options include surrounding the run with suitably graded material or installing within telescoping pipe fitted with large-radius 90 degree elbows at top and bottom.

17.7. INSTALLATION IN UNDERGROUND EXCAVATIONS

When instruments are installed within underground excavations, additional constraints often exist, and cooperation with the construction contractor is essential. The contractor's primary objectives will usually be to excavate soil or rock and to install support. Interruptions for installation of instruments can delay an entire construction cycle, and installation plans must be coordinated with the contractor well ahead of installation dates. Guidelines on coordination are given in Section 17.12.

Special attention must be given to the provision of any necessary platforms to support installation equipment and personnel and to the movement of installation equipment such as drill rigs. Whenever possible, any required boreholes should be drilled with equipment that is used for other construction activities. Special care must be taken to protect instruments from damage, following the guidelines given in Sections 17.4 and 17.8.

17.8. PROTECTION FROM DAMAGE

Various procedures for minimizing damage to surface and downhole components, to pipes, tubes, and cables installed in fill, and to instruments installed in underground excavations have been described in the previous sections. Some additional guidelines can be given to minimize the chance of 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. It is hoped that readers of this book will not endure the embarrassment of the author, who on one occasion buried terminals to instruments installed in a city suburb prior to construction of a tunnel and located them with respect to nearby buildings. The project was delayed, urban renewal took place, and when the project was reactivated extensive digging was required before the terminals were found!

All vertical pipes should be provided with a cap to prevent intrusion of debris. Where construction activities might damage the tops of vertical pipes or where vandals might block pipes by dropping stones, a removable plug should be installed at an appropriate depth. A suitable plug is shown in Figure 17.13. The plug is attached to an insertion tool, consisting of two concentric pipes, the inner pipe connecting to the left-hand thread on the plug, the outer pipe terminating in a socket wrench to fit the hexagonal nut. Each pipe has a tee handle on the upper end. The plug and tool are inserted within

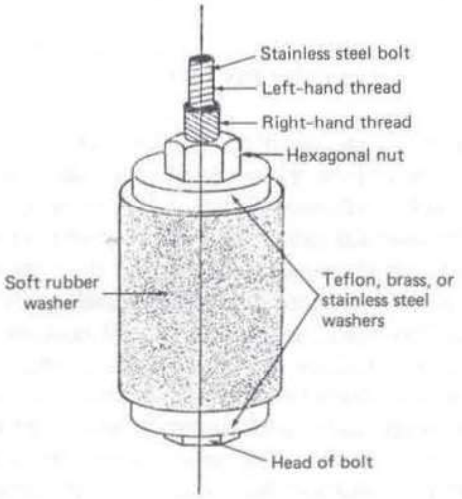


Figure 17.13. Removable plug for vertical pipe (courtesy of Syncrude Canada Ltd, Fort McMurray, Alberta, Canada).

the pipe, the plug held in place with the inner pipe and set by turning the outer pipe clockwise, the insertion tool removed, and the depth of the plug recorded. Plug removal follows the reverse procedure.

Protection from damage by the environment should consider all the factors listed in Table 4.4. Of particular relevance to terminals are corrosion, water, temperature extremes, ice, shock, dust, dirt, mud, chemical precipitates, humidity, and loss of accessibility. Piezometer tubes and electrical cables have occasionally been eaten by rodents, who apparently have a taste for plastic. When appropriate, protection from this hazard should be provided by use of metal conduit. All relevant environmental factors must be addressed on a case-by-case basis.

17.9. ACCEPTANCE TESTS

Acceptance tests should always be performed to ensure, to the extent practicable, that installations have been completed satisfactorily. For example, permeability tests can be made in twin-tube hydraulic piezometers, groove tracking tests and spiral survey measurements can be made in inclinometer casings, and rod sliding tests can be made in fixed borehole extensometers equipped with a disconnect between rod and anchor. 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.

17.10. INSTALLATION RECORDS

As discussed in Chapter 4, "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 the following list:

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

17.11. 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.

In cases where an instrument is required to monitor long-term performance but not required during construction, it may be possible and expedient to install the instrument in a borehole after construction is complete, thereby reducing interference to construction and the chance of damage to the instrument.

17.12. COORDINATION OF INSTALLATION PLANS

Detailed planning for installation should be coordinated among all parties involved, including the instrument supplier, owner's personnel, and the construction contractor. Arrangements must be made to ensure that instruments arrive on time.

When the owner's personnel are responsible for installation, a cooperative working relationship with the construction contractor is essential, and the owner's field personnel should make a special effort to establish a cooperative relationship. The best way of establishing such a relationship is for instrumentation personnel to initiate thorough communication with all levels of the contractor's personnel several weeks or months before the start of installation work. The instrumentation personnel should meet with the contractor's engineers and supervisors to explain what will be done, why it must be done, and what will be required of the contractor. They should discuss access for installation and prepare sketches of detailed installation arrangements, linked to the contractor's actual method of construction, to forewarn the contractor of the impact on normal construction work. They should provide lists of materials that will be required for support work and be willing to tailor their installation plans to create minimum interference to the contractor's work. In short, mutual respect can usually be established by thorough communication, dispelling the adversary relationship that sometimes arises between architect-engineers and construction contractors. Having established this respect, instrumentation personnel should maintain the contractor's respect by performing top-quality work, by being responsive to the effects of the program on the contractor, and by working with the contractor to minimize any adverse effects.

17.13. FIELD WORK

If the guidelines provided in this chapter have been followed, intensive planning will have been completed before field work is started. Decisions will have been made on contractual arrangements for installing instruments, locations of instruments, and installation schedules. Detailed installation procedures will have been planned, and they will have been coordinated with all parties involved.

Despite such intensive planning, surprises are the norm rather than the exception, and field personnel must make a special effort to complete each installation satisfactorily. Chapter 6 indicates that the basic requirements for field instrumentation personnel are reliability and patience, perseverence, a background in the fundamentals of geotechnical engineering, mechanical and electrical ability, attention to detail, and a high degree of motivation. If installation personnel possess these qualities, they should be well equipped to solve problems created by surprises.

Before starting installation work, field personnel should study and understand the detailed installation procedure. They should be aware that the instrument may not serve its purpose if a single essential but apparently minor requirement is overlooked during the installation. A complete check should be made to ensure that all listed materials and equipment are either on hand or readily available, and all components should be identified.

A few golden rules apply to field work. The first rules apply to care of hardware. All instruments and components should be stored and protected prior to installation. They should be handled carefully. Cables and tubes can readily be damaged by nicking, bending, or twisting, and they should always be removed from a spool by rotating the spool. Components should be kept as clean as possible, and ends of tubing and pipe, connectors, and threads should be covered with protective material or placed on clean above-ground surfaces. Adequate cleaning materials should be on hand. Second, when installing instruments in boreholes, listen to the advice of the driller. Experience with drilling can often contribute more to the success of a borehole installation than a university degree. Third, if problems arise while installing downhole components and if there is any possibility of collapse of the borehole or of premature setting of grout, act fast and discuss later: *if in doubt, pull it out*. Fourth, remember that Murphy's law was writ-

ten especially for installation of geotechnical instruments.

17.14. 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 performance specifications (photographs are often helpful).
- Details of calibration procedures.
- Details of installation procedures (photographs are often helpful).
- Initial readings.
- A copy of each installation record sheet.

CHAPTER 18

COLLECTION, PROCESSING, PRESENTATION, INTERPRETATION, AND REPORTING OF INSTRUMENTATION DATA

Chapter 4 indicates that data collection, processing, presentation, interpretation, and reporting procedures should be planned before instrumentation work commences in the field. This chapter provides guidelines on planning and execution.

A detailed draft of procedures should be prepared during the planning phase and finalized after instruments have been installed, by which time responsible personnel are more familiar with operation of instruments and specific site considerations. Chapter 5 includes a listing of data collection, processing, presentation, and interpretation information that should be included in a comprehensive instruction manual, but users should recognize that, although manufacturers can provide some basic information, they are usually not familiar with specific site conditions. Users must therefore prepare their own procedures.

18.1. 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 owner or instrumentation specialists selected by the owner. 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.

18.1.1. Personnel Qualifications and Responsibilities

Data are normally collected by technicians or junior engineers, under the supervision of a more experienced geotechnical engineer. Qualifications of data collection personnel should be similar to those of installation personnel: reliability, mechanical and electrical ability, computational skill, attention to detail, a background in the fundamentals of geotechnical engineering, and a high degree of motivation.

Data collection personnel should work as a team, the division of responsibilities depending on their individual capabilities and on project requirements. Technicians and junior engineers should be encouraged to take an interest in the instrumentation

program and be given as much responsibility as they can handle, thereby creating a sense of involvement and motivation and permitting the more experienced engineer to concentrate on aspects that require mature engineering capability.

Typical personnel responsibilities are given in Tables 18.1 and 18.2, so that data collection personnel can be aware of the full scope of their tasks. These responsibilities are listed as if data are to be collected manually. When automatic data acquisition systems are used, some of the tasks will be handled automatically.

18.1.2. Role of Automatic Data Acquisition Systems

Chapter 8 includes a description of automatic data acquisition and communication systems and a discussion of suitability for monitoring various transducers. This section is confined to a discussion of advantages and limitations of automatic systems.

Until a few years ago, almost all data were recorded by hand: the *long way*. The advent of automatic data acquisition systems 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. No automatic system can replace engineering judgment. These remarks should not be taken as a vote against the use of automatic data acquisition systems, but as a plea for an honest appraisal of their suitability before they are selected. Advantages and limitations of automatic data acquisition systems are given in Table 18.3.

The guidelines in the remainder of Section 18.1 are written as if data are to be collected manually because, when data are collected automatically, the goals are the same. When planning to use automatic data acquisition systems, users are encouraged to study these guidelines and to ensure that an essential need is not forgotten.

18.1.3. Written Procedure for Data Collection

The experienced geotechnical engineer responsible for data collection should finalize the draft procedure that has been written during the planning phase. The detailed written procedure will be based on information provided in the manufacturer's instruction manual and on specific site conditions and will include the items in the following list. It will be seen that this list has many similarities to the responsibilities listed in Tables 18.1 and 18.2, but it serves a different purpose.

- Needs and procedures for coordinating with other parties for access and other support work.
- Step-by-step procedure for equipment set-up and turn-on.
- Step-by-step procedure for taking readings, with appropriate illustrations, including a list of equipment and tools required during reading and cautions pertaining to personnel and equipment. (It is often advisable to read each instrument more than once and to use judgment in selecting the most appropriate value. Use of the same data collection personnel for all measurements throughout a particular project is highly desirable. Interchange of readout units should be avoided whenever possible, because readings are often dependent on the particular transducer and readout unit combination.)
- Special procedures for taking initial readings.
- Guidelines on reading frequency.
- Any required instructions for reading two or more instruments concurrently, so that data can be correlated on a common time basis.
- Guidelines on handling readout units, as given in Section 8.4.17.
- Field data record.
- Sample completed field data record.
- Guidelines to enable data collection personnel to answer the question, *Is the instrument functioning correctly?* (Procedures for ensuring reading correctness are discussed in Chapter 4 and include visual observations, duplicate instruments, a backup system, regular in-place calibration checks, consistency between two or more measured parameters, and data repeatability.)
- Procedure for comparing the latest readings with the previous readings, so that any significant changes can be identified immediately. (If data are recorded automatically, a comparison step should be programmed into the system. When manual recording and field data sheets are used, the previous set of readings should be taken to the field, either by copying the previous field data sheets or by transcribing key data to a column on the new data sheets. Originals of previous data sheets should not be taken into

Table 18.1. Typical Responsibilities of the Experienced Geotechnical Engineer

Phase of Work	Responsibility
Before and during collection of field data	Coordinating with and ensuring the cooperation of all parties Training and supervising technicians or junior engineers and assisting them whenever they have questions or concerns
Before collection of field data	Familiarity with the planning of the monitoring program, as described in Chapter 4 Familiarity with the project construction procedure Familiarity with the calibration and maintenance requirements described in Chapter 16 Familiarity with details of instrument installation, including the installation record sheets and installation report, as described in Chapter 17 Familiarity with the general requirements for data collection, as described in this chapter Preparing the final version of detailed data collection procedures Establishing and updating data collection schedules Conveying, to technicians or junior engineers, hazard warning levels and other critical measurements that should be brought to the immediate attention of the experienced engineer
During collection of field data	Reviewing data Advising the owner, the design consultant, or the construction contractor of any condition requiring their attention Communicating with personnel responsible for data processing, presentation, and interpretation Recording all factors that may influence measured data[a] Making visual observations of behavior, for correlation with instrumentation data[a]

[a]If technicians or junior engineers have sufficient ability and motivation, they can handle the last two responsibilities, under the close supervision of the experienced geotechnical engineer.

Table 18.2. Typical Responsibilities of Technicians and Junior Engineers

Phase of Work	Responsibility
Before collection of field data	Understanding why the instruments are necessary, how they work, and what parameters are being measured Understanding the project construction procedure
During collection of field data	Consciousness of safety considerations Reading instruments and recording data on field data records Comparing the latest readings with the previous readings Ability to answer the question, *Is the instrument functioning correctly?* Alerting the experienced geotechnical engineer when magnitudes of data change exceed predetermined critical magnitudes Calibrations during service life and maintenance of readout units, field terminals, and embedded components, as described in Chapter 16 Being on the lookout for damage, potential for damage, and deterioration or malfunction of instruments and initiating preventative action or repair or replacement without delay Verifying that sufficient spare parts, including spare readout units, are always on hand or readily available

the field. When data are recorded manually in a field book, previous data are in the book.)

- Procedure for alerting the experienced geotechnical engineer when magnitudes of data change exceed predetermined critical values and for advising the owner, the design consultant, or the construction contractor of any condition requiring their attention.
- Procedure for recording all factors that may influence measured data, including construction details and progress, geology and other subsurface conditions, and environmental factors

Table 18.3. Advantages and Limitations of Automatic Data Acquisition Systems

Advantages	Limitations
Reduced personnel costs for reading instruments and analyzing data	Replacement of a knowledgeable observer by an item of hardware (there is a real possibility that visual observations will not be made, that other factors influencing measured data will not be recorded, and that causal information will therefore not be available)
More frequent readings	Possibility of generating an excess of data, encouraging a "file and forget" attitude, and therefore failing to take timely action in response to data
Retrieval of data from remote or inaccessible locations	Possibility of blind acceptance of data, which may or may not be correct (garbage in, garbage out)
Instantaneous transmittal of data over long distances, using telemetry	High initial cost and often high maintenance cost
Increased reading sensitivity and accuracy	Currently, often requires some custom designed components that are unproven
Increased flexibility in selecting required data	Complexity, requiring an initial "debugging" period
Measurement of rapid fluctuations, pulsations, and vibrations	Need for regular field checks and maintenance by specialized personnel
Recording errors are fewer and immediately recognizable	Need for backup manual recording arrangements
Electronic data storage is in a format suitable for direct computer analysis and printout	Need for a reliable and continuous source of power
	Susceptibility to damage caused by weather conditions and construction activity

such as temperature, rainfall, snow, sun, and shade.

- Procedure for making visual observations of behavior for correlation with instrumentation data. (Special care should be taken to continue making visual observations during periods when instruments are read infrequently, so that any signs of adverse behavior are identified and a more intensive reading schedule resumed.)
- Procedure for inspecting for damage, potential for damage, and deterioration or malfunction of instruments and for initiating preventative action or repair or replacement.
- Procedure for calibrations during service life and for maintenance of readout units, field terminals, and embedded components.
- Procedure for communicating with personnel responsible for data processing, presentation, and interpretation.

18.1.4. Field Data Records

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.

CURIOUS GEORGE PROJECT
FIELD DATA SHEET FOR PNEUMATIC PIEZOMETERS

Date _____ Observer _____
Readout Unit No. _____ Air Temperature _____

Piez. Number	Time	Readings (lb/in.^{2})			Remarks
		Individual		Best	

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

_____

_____

_____

_____

_____

_____

Figure 18.1. Typical field data sheet for pneumatic piezometers.

Figure 18.1 shows a typical field data sheet for a pneumatic piezometer, assuming that a copy of previous data will be taken to the field. Space is provided for four individual readings, so that data correctness can be judged on the basis of repeatability and a judgmental *best* reading entered. The *best* reading can be an average or can be a median value. The median value is obtained by deleting highest and lowest values until only one central reading remains, this being the median. If two readings remain (as is the case for an even number of readings), the average of these two is taken as the median value. A space is also provided for air temperature, because pneumatic readout units can be sensitive to temperature, and on occasion a column for air temperature may be appropriate. If there is a

CURIOUS GEORGE PROJECT
FIELD DATA SHEET FOR PROBE EXTENSOMETER

Extensometer No. _____
Readout Unit No. _____
Survey Tape No. _____

Date _____
Start Time _____
Finish Time _____
Observer _____

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

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

_____

_____

_____

_____

_____

_____

Figure 18.2. Typical field data sheet for a probe extensometer, with deepest measuring point in bedrock.

predominant cause of pore water pressure change, a column may be appropriate for recording those causal data. For example, if the piezometers are installed in soft ground beneath an embankment, a column may be appropriate for the current elevation of the top of the fill. To avoid unnecessary writing, it may be convenient to enter instrument numbers on the original of the field data sheet.

Figure 18.2 shows a typical field data sheet for a probe extensometer, for which a set of data are required rather than a single value, and one full sheet is needed for a single set of readings of one instrument. In this example the deepest measuring point is always in bedrock. If there is a possibility of settlement at the deepest measuring point, a space will be required on the sheet for collar elevation, requiring a surveyed elevation of the point on the instrument terminal at which survey tape readings are

made. Again, it is assumed that a copy of previous data will be taken to the field. Space is provided for three individual readings, but the third set is necessary only if significant differences occur between the first two sets. The sheet should be designed for the largest number of measuring points at an individual instrument, in this case eight.

The U.S. Bureau of Reclamation (Bartholomew et al., 1987; Carpenter, 1984b) has pioneered use of hand-held field data entry devices to replace field data sheets. A bar-coded tag is attached to the field terminal for each instrument, containing the project name and instrument number. The tag is read using an optical reader pen attached to the data entry device. The device is programmed in a question and answer mode to prompt data collection personnel on what entry is expected, and previous readings are stored in the device so that new data can be compared immediately with previous data. When all field data are entered, the device is taken to a telephone or computer terminal where it transmits the data to a file at a central computer. Data processing personnel then access the file, add it to the project data base, and process the data by computer. Simpler devices are also available, without the bar-coded tag, such as are used by electric and gas meter readers.

18.1.5. 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 essential.

Many instruments, including instruments installed in boreholes and many types of strain gage, take a few days to stabilize after installation. Instruments should therefore be installed as early as possible, before they are affected by construction activities, and initial readings taken after stabilization.

The following guidelines are given as a goal when establishing initial readings. The guidelines can usually be followed with instruments requiring a single reading, such as strain gages and piezometers. However, for instruments requiring multiple readings, such as probe extensometers and inclinometers, this goal may be too demanding and should be tempered by considerations of a reasonable level of effort.

Typically, a minimum of two readings will be taken immediately after installation, as part of the acceptance test (Chapter 17). 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.

In summary, the goal for the initial reading process includes four steps: acceptance tests, daily readings until stable, formal initial readings, and daily readings to verify stability.

Readings are often dependent on the particular transducer and readout unit combination, and interchange of readout units should therefore be avoided whenever possible. When a spare readout unit has been procured, for use if the primary unit malfunctions, or when there is a possibility of interchanging units, initial readings should be made with each unit. When an interchange occurs and this has not been done, formal "initial readings" must be taken with the replacement unit, and for data continuity it must be assumed that either there has been no change since the previous reliable reading or data have changed in accordance with an established trend.

To avoid the possibility of disputed measurements of change, the contractor and owner's representative should jointly make initial readings and agree on appropriate values.

18.1.6. 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 instrumented 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 surcharge). It is often wise to increase the frequency of readings during heavy rainfalls. 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.

Cording et al. (1975) indicate that data collection needs generally fall into two categories: those that are construction dependent and those that are time dependent. A preliminary schedule should be developed on this basis during the planning phase so that requirements can be estimated for access and personnel. This preliminary schedule should assume a maximum reading frequency, and once construction has started and procedures are known, a more realistic schedule can be developed. The schedule should be flexible and should be reviewed frequently, in light of observed trends and predicted behavior.

As indicated in Chapter 6, if the construction contractor is responsible for data collection under a unit price contract, there should not be a predetermination of the reading schedule and frequency. Frequency should be at the discretion of the owner's representative, necessitating a *data set* or *man hour* pay item unit.

18.2. 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 owner or instrumentation specialists selected by the owner. Personnel requirements for these tasks are frequently underestimated, resulting in the accumulation of unprocessed data and failure to take appropriate action. The time required for data processing and presentation is usually similar to, and may even exceed, the time required to collect data.

18.2.1. Personnel Qualifications

Processing and presentation should be under the supervision of the experienced geotechnical engineer who has been responsible for data collection. The engineer will be assisted by other engineers or by technicians, but only those who have computational skill and a background in the fundamentals of geotechnical engineering. Processing and presentation require significant judgments and should not be delegated to inexperienced personnel.

18.2.2. Role of Automatic Data Processing and Presentation

The advent of automatic data processing and plotting systems has made an enormous increase in the efficiency of data processing and presentation. Despite their obvious advantages they have their limitations, of which users should be fully aware. No automatic system can replace engineering judgment.

Advantages of Automatic Data Processing and Presentation

DiBiagio (1979) lists the following advantages of automatic data processing and presentation:

- Computers provide for rapid and accurate processing of data, providing the programs are available and thoroughly tested.
- Once data are stored in a form that is readily accessible by a computer, it is a relatively simple task to reprocess batches of data if changes in processing or computational procedures are desired.
- Computers can be used to generate summary tables or results for direct use in reports. Thus, the costly and time consuming task of hand typing and proofreading tables is avoided.
- Curves or other forms of graphical representation of data can be generated automatically by the computer. Very little effort is required, for example, to change scales or replot data. This is often necessary in reporting data obtained from instrumentation projects, in particular when they extend over a long period of time.
- Highly skilled personnel are not required to administer routine computer data processing tasks.
- Computers or microprocessors can be used to scale signals or to perform computations on input

data in real time if immediate access to results is a requirement.

- On projects involving large amounts of data [such as offshore structures] computers may be used advantageously to perform a running statistical analysis of recorded data, to produce separate records of compact statistical data. Analysis of reduced data of this kind is useful as a first step in planning subsequent data processing and analysis procedures to be used on the bulk of the recorded data.

Disadvantages of Automatic Data Processing and Presentation

Disadvantages of automatic data processing and presentation include the following:

- Replacement of a knowledgeable engineer by an item of hardware. There is a real possibility that engineering judgment will be given second place, that correlations will not be made with visual observations and with factors that influence measured data.
- Computer programs must be thoroughly tested before they are accepted. Whenever possible, self-checks should be built into computer programs.

The author believes that when automatic data processing procedures are used they should be designed so that the product is never inferior to the product of thorough and judgmental manual data processing. With this in mind, the remaining subsections of Section 18.2 are written as if data are processed manually. This should not be taken as a vote against the use of automatic data processing, but merely as a way to define a quality standard to be used as a model when developing detailed manual or automatic data processing for a particular project.

Figures 18.3–18.5 show examples of data plots prepared by automatic systems during construction of the Mt. Baker Ridge Tunnel in Seattle.

Bartholomew et al. (1987) provide guidelines on automatic processing of data for embankment dam instrumentation.

18.2.3. Written Procedure for Data Processing and Presentation

The geotechnical engineer responsible for data processing and presentation should finalize the draft procedure that has been written during the planning phase. The detailed written procedure will be based on information provided in the manufacturer's instruction manual and on specific site conditions and will include the following:

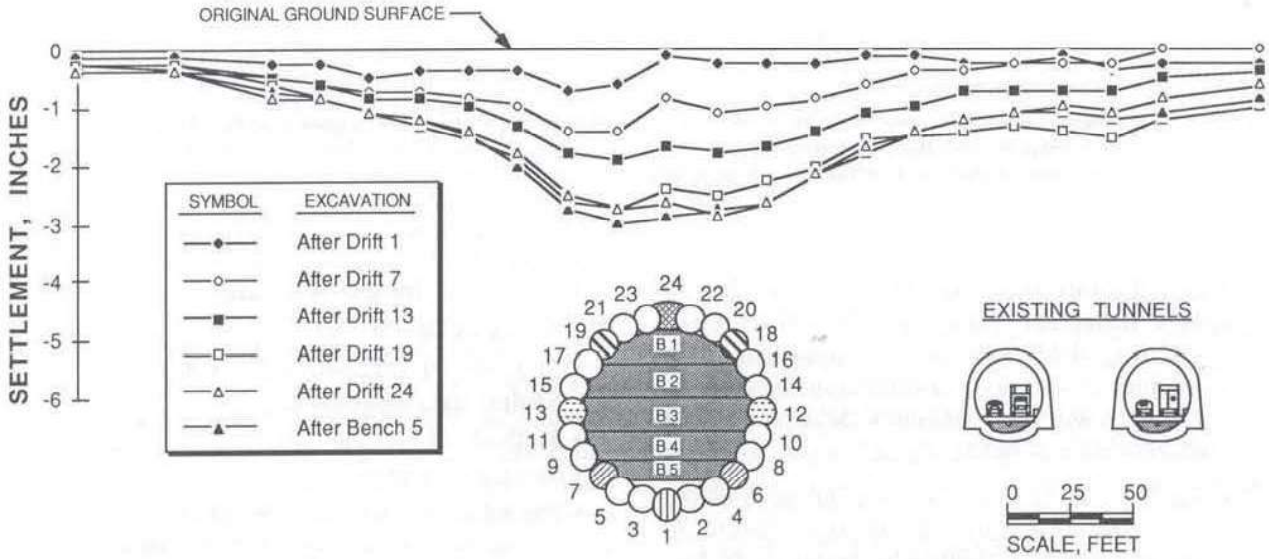


Figure 18.3. Typical plot of surface settlement data, prepared using Apple Macintosh 512k personal computer. Mt. Baker Ridge Tunnel, Seattle, WA (courtesy of Shannon & Wilson, Inc., Seattle, WA).

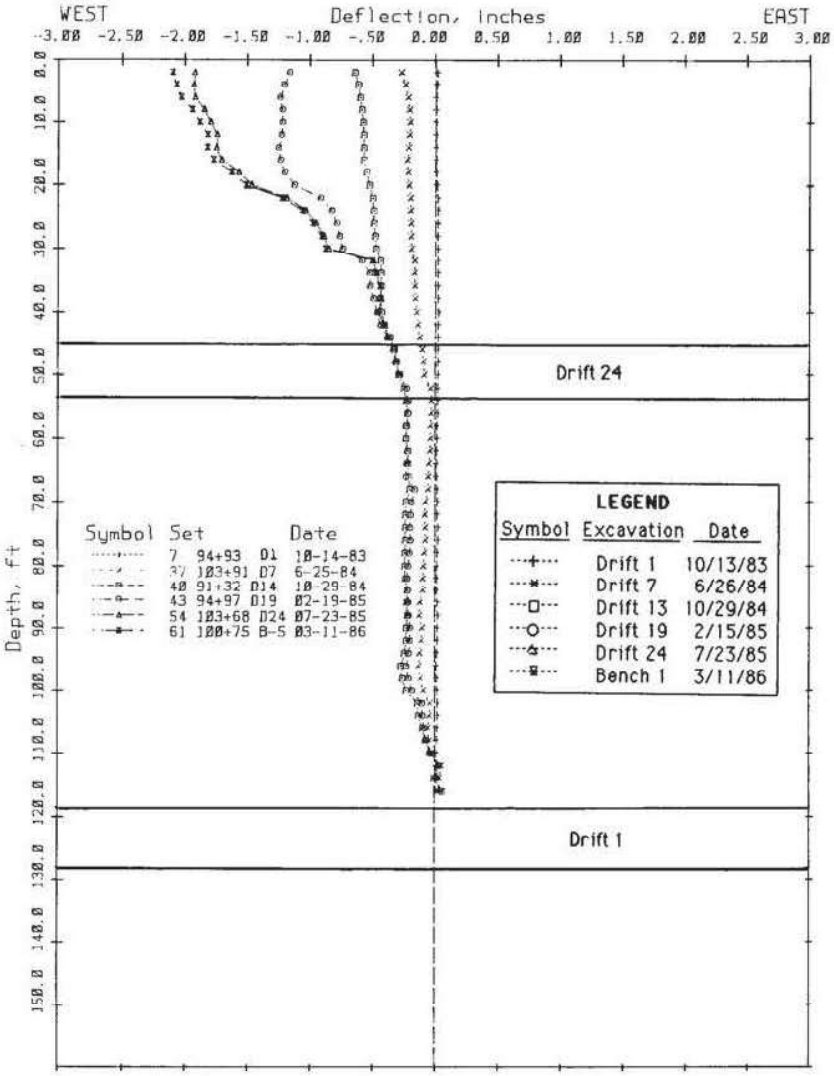


Figure 18.4. Typical plot of inclinometer data, prepared using Digital Equipment Corporation PDP 11/34 computer and Houston Instruments DMP 29 X-Y plotter. Mt. Baker Ridge Tunnel, Seattle, WA (courtesy of Shannon & Wilson, Inc., Seattle, WA).

- List of information that should be provided by data collection personnel, including initial readings, field data records, information on reading correctness, information on factors that may influence measured data, and visual observations of behavior.
- Factory calibrations, results of acceptance tests, and procedure for updating calibration data in accordance with calibrations made during service life.
- Use of automatic data processing procedures.
- Procedures for data screening.
- Data calculation sheet.
- Step-by-step calculation procedure.
- Sample data calculation.
- Methods of plotting data.
- Sample data plots.
- Procedure for advising the owner, the design consultant, or the construction contractor of any conditions requiring their attention.
- Procedure for communicating with personnel responsible for interpretation of data.

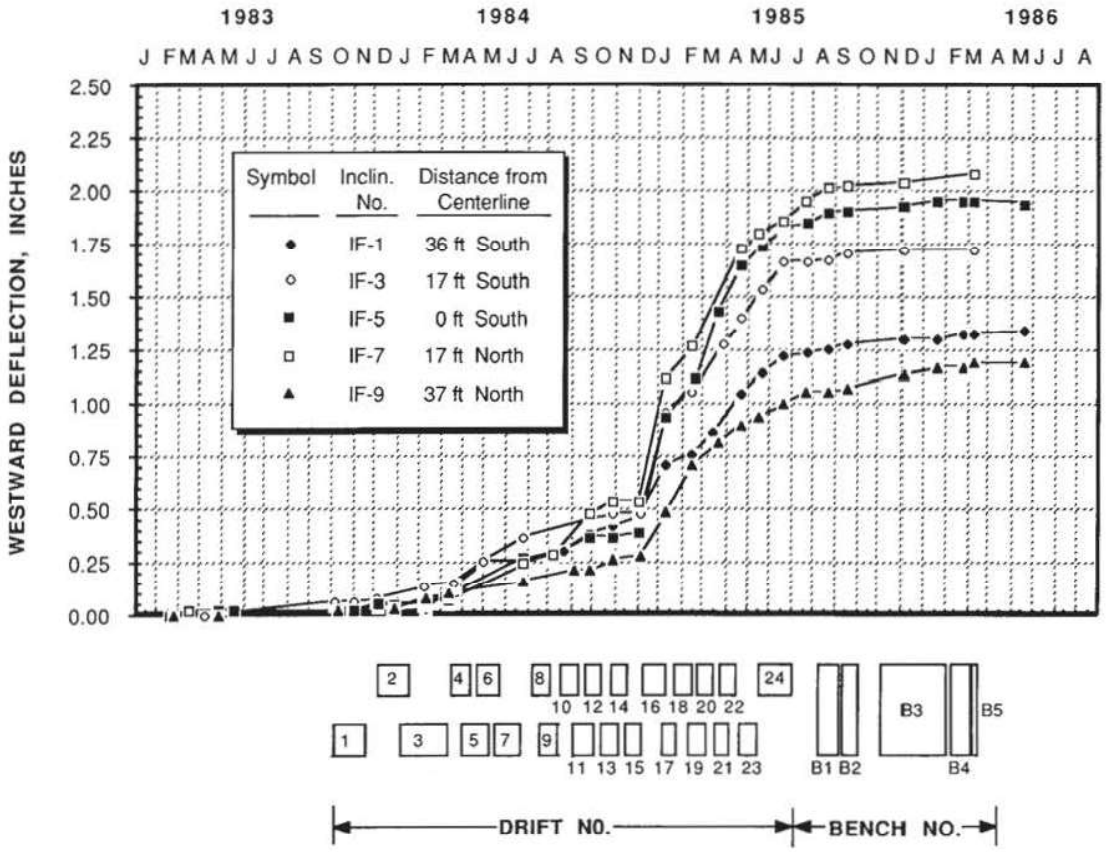


Figure 18.5. Typical plot of inclinometer data, prepared using Apple Macintosh 512k personal computer. Mt. Baker Ridge Tunnel, Seattle, WA (courtesy of Shannon & Wilson, Inc., Seattle, WA).

18.2.4. Screening of Data

Data are usually screened, in two steps, prior to data calculation and plotting.

The first data screening step is made in the field by data collection personnel when they examine reading correctness, by using one or more of the following procedures: visual observations, duplicate instruments, a backup system, regular in-place calibration checks, consistency between two or more measured parameters, and data repeatability. After this step, an assessment of data can be made in order to detect changes requiring immediate action.

The second data screening step is made in the office by data processing personnel and entails scrutinizing the field data, marking any obvious errors on the field data records, and reviewing the evaluations of data correctness made earlier in the field. When obvious errors are found, repeat readings will usually be required. On occasion, additional screening is possible by evaluating plots of raw data or computer-generated tabulations designed specifically for screening purposes, as discussed in Section 18.2.6.

18.2.5. Calculations

After data that appear false are discarded, raw readings are transcribed from field data records to calculation sheets for conversion to engineering units. Usually this task should be accomplished within 24 hours of data collection and should be followed immediately by updating routine plots of data versus time.

Calculation sheets should be made specially for each project and instrument type, to record project name, instrument type and number, date and time of readings, initials of persons making and checking

CURIOUS GEORGE PROJECT
CALCULATION SHEET FOR PNEUMATIC PIEZOMETER

Piezometer No. _____ Initial Piezometric Elev. _____ ft.

(1) Date & Time	(2) Best Reading (lb/in.^{2})	(3) Piezo-metric Head (ft) (2) × 2.31	(4) Elev. of Piez. (ft)	(5) Piezo-metric Elev. (ft) (3) + (4)	Calcs. by, Checked by		Remarks

Figure 18.6. Typical calculation sheet for a pneumatic piezometer.

calculations, readings transcribed from field data record, equations used for calculations (including any calibration or correction factors), and any remarks. Numbering the columns and using these numbers in defining each calculation step at the head of subsequent columns can be helpful.

Figure 18.6 shows a typical calculation sheet for a pneumatic piezometer. The initial piezometric elevation is included so that a comparison can be made with subsequent values. The first two columns are transcribed from the field data sheet (Figure 18.1). The third column is for a change of units from pressure to head and can include any instrument calibration factor. The fourth column is required so that head can be converted to elevation. The elevation must be either estimated or measured at a nearby settlement gage. The fifth column of data will be plotted against time. The remarks column will include key items transcribed from the field data sheet and may also include remarks on reading repeatability.

Figure 18.7 shows a calculation sheet for a probe extensometer, compatible with the field data sheet in Figure 18.2. In this case a full calculation sheet is required for each set of data. The survey tape constant is the distance between the sensing point in the probe and the zero on the survey tape. The elevation of the bottom measuring point is determined when initial readings are made and will not change, because in this example the deepest measuring point is in bedrock. Spaces (3), (5), and (6) are completed by transcription from the field data sheet. The collar is the point on the instrument terminal at which survey tape readings are made. The collar is settling, and data must be referenced to the fixed bottom measuring point by calculating as shown on the sheet. The calculated settlement for the bottom measuring point should of course be zero. Settlement will usually be plotted against time, and the plot for each measuring point will be labeled with the initial elevation of the measuring point. If there is no interest in elevation data and if the same survey tape is always used, the calculations can be simplified by omitting the survey tape constant and revising the calculation sheet.

CURIOUS GEORGE PROJECT
CALCULATION SHEET FOR PROBE EXTENSOMETER

Extensometer No. _____ Date _____ Survey Tape Const. _____ ft (1)
Elev. at Bottom Measuring Point _____ ft (2)
"Best" Tape Reading for Bottom Measuring Point _____ ft (3)
Collar Elev. _____ ft (4)[(1) + (2) + (3)] Calcs. by _____ Checked by _____

(5) Meas. Point No.	(6) Best Tape Reading (ft)	(7) Depth Below Collar (ft) (6)+(1)	(8) Meas. Point Elev. (ft) (4)−(7)	(9) Initial Meas. Point Elev. (ft)	(10) Settlement (ft) (9)−(8)	Remarks

Figure 18.7. Typical calculation sheet for a probe extensometer, with deepest measuring point in bedrock.

18.2.6. Plots of Data

After calculations have been made, plots of data should always be prepared. Some engineers and geologists struggle to interpret data that is presented on field data records or calculation sheets, but the human eye and brain are not able to do this efficiently. *Prepare a plot and all becomes clear!*

Various types of plot play a role in the data processing, presentation, and evaluation process. Some are described in the following subsections, and examples are given. The examples are intentionally simplistic, because their purpose is to show concepts rather than actual data.

Plots to Assist with Data Screening

As indicated in Section 18.2.4, data are screened both in the field and in the office. On occasion, data quality can be improved further by plotting raw readings or by generating computerized tabulations designed specifically for screening purposes and exercising engineering judgment. Two prerequisites are needed on these occasions: sufficient data for use as input to the screening process and the existence of a known truth about performance, for use as a basis when judging data. An example will illustrate the procedure.

A probe extensometer has been installed vertically in an alluvial deposit. The first stage of an embankment has been placed on the deposit, and probe extensometer readings are being used to evaluate the progress of consolidation settlements during a 1 month waiting period. The designers are analyzing vertical compression, within the deposit, in horizontal slices defined by known stratigraphic boundaries. Measuring points for the probe exten-

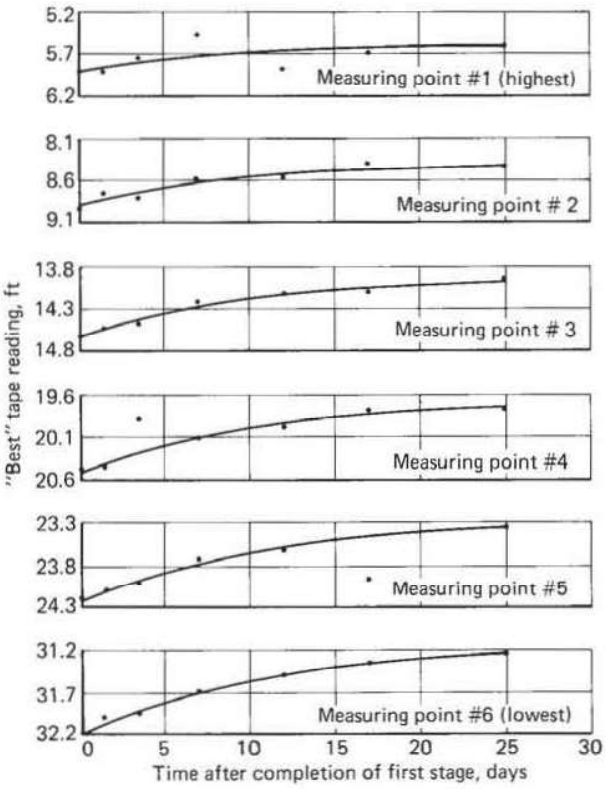


Figure 18.8. Plot of raw data for probe extensometer, to assist with screening.

someter have been installed at these boundaries. It would be possible to process data directly from raw field data by plotting vertical compression between each pair of measuring points versus time, but that procedure would not make use of the following known truth: *under constant applied loading conditions, a plot of settlement versus time should follow a logical pattern; the slope of the plot may change, but there should be no significant scatter on either side of a smooth plot.* This rule applies to all measuring points in the soil profile and allows an opportunity to improve data quality by correcting for random reading errors. Plots would first be made, for each measuring point, of tape reading versus time, and logical and smooth lines would then be drawn as best fits to the points, as shown in Figure 18.8. These smooth lines would then be used, in preference to the raw data, as a basis for analyzing vertical compression.

Routine Plots of Data Versus Time

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

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 full consolidation to occur. On other occasions, for example, when deformation is monitored around a tunnel 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 one assumed to be more correct. An example of such a plot, in which the load cell is assumed to be more correct, is given in Figure 18.9. When visual observations can be quantified, plots can be made to compare observations with measurements and used to assist with data interpretation.

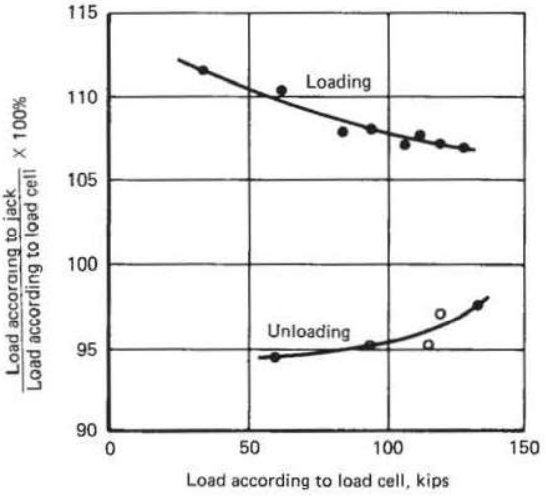


Figure 18.9. Comparison between load cell and hydraulic jack during proof test of tieback.

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, rainfall, snow, 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 18.10 shows plots of fill thickness and settlement for an embankment on soft ground, indicating the relationship between cause and effect.

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.

As an example of a summary plot, deformation can often be summarized by preparation of a contour plot on a plan of the deforming area, each contour indicating a different magnitude of deformation. Such plots are useful, for example, to show deformation of an embankment dam. Alternatively, vector plots can be prepared, showing arrows that indicate the magnitude and direction of deformation.

Cording et al. (1975) suggest that summary plots

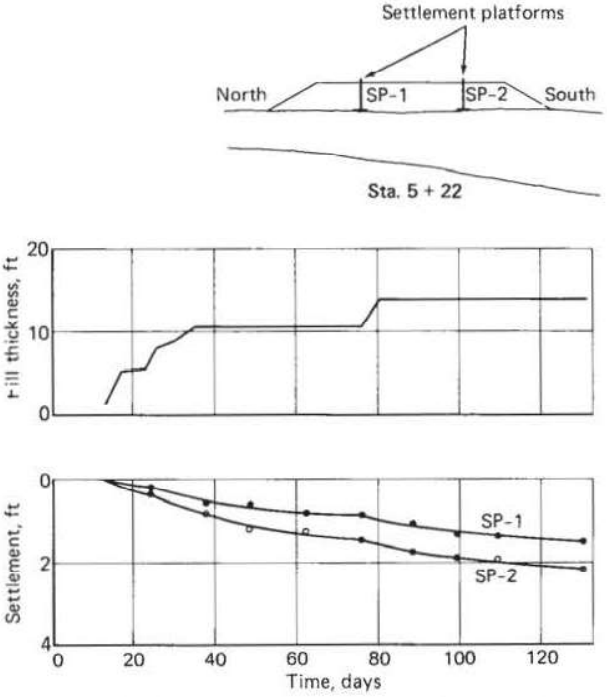


Figure 18.10. Plot to examine cause and effect relationships: settlement of soft ground below an embankment.

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.

Guidelines on Plotting

Lambe (1970) discusses portrayal of data:

> The proper portrayal of data is now, and always has been, essential to good soil [and rock] mechanics. Important trends can be completely hidden in poorly portrayed data. Further, valuable experimental or theoretical data incorrectly portrayed can be worthless, or even worse, can be misleading. The engineer needs to recognize that he must aim his plots towards the intended use. He might display his data one way, if they are to be used by a researcher looking for a specific phenomenon, but display them quite differently if he is trying to show a general trend to a non-technical person.
>
> Several preliminary stages are usually needed to produce an outstanding plot. To make an outstand-

ing plot, the engineer needs to know what is significant about the data, but to determine what is truly significant he needs a good plot! He thus must make a plot, study it for trends, mechanisms, etc., and after discovering what is the most important result to be gathered from the plot, finally prepare it to show very clearly the discovered result. The engineer is not likely to obtain outstanding plots if he does not fully understand his subject. He is usually doomed to failure if he depends on a laboratory or field technician aided by a draftsman to prepare his plots. The engineer is guaranteeing second-rate work if he so delegates this responsibility. Years ago, I made an unsuccessful attempt to establish rules for plotting data. This attempt failed, as one would expect, since the most important contribution to outstanding plots is the preparer's own understanding of his subject and his own creativity.

Good portrayal of data therefore requires a full understanding of the instrumentation program combined with imagination, ingenuity, sufficient time to digest the data, and some trial and error. Lambe (1970) and Hansmire (1978) provide guidelines on plotting, including the following:

- All work should be done in pencil. Use symbols to distinguish between different instruments or different times. Colored pencils should not be used, as the colors do not reproduce.
- Choose scales so that observations fill the space available, but do not use exaggerated scales that would magnify minor changes to make them appear alarmingly large.
- Draw plots slightly darker than the underlying grid so that when copied both are visible but the plots stand out. Make sure data points are visible.
- Plot elevations and depths on the vertical axis.
- Plots should be self-explanatory. Show project name, the type of instrument, the scale and units of measurement, and the time of measurement.
- Use sketches on the plot to show the locations of the instrument relative to the construction activity and the geology.
- Maintain consistency of scales in a given document so that plots can be compared.
- Prepare time plots showing the progress of construction as well as the measurements.
- Plot predicted behavior or the limits of safety on the same axes as the field readings.
- All plots should be initialed by the engineer responsible for their preparation, thereby ensuring a sense of responsibility.
- Whenever possible summary plots should be on a single sheet of report-sized paper.

Lambe (1970) presents Figure 18.11 to illustrate poor and good plots of the same data. The poor plot is on the left, the good plot on the right. Shortcomings of the poor plot include lines drawn too lightly, a poor selection of scales, use of a legend to denote the large and small footings rather than direct labeling, subsoil described in words rather than in a simple sketch, poor placement of numbers and titles on the axes, poor arrangement of title, and data points not shown.

18.3. 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.

Early data interpretation steps have already been described: screening of data in the field and office to examine reading correctness and to detect changes requiring 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.

18.3.1. Personnel Qualifications

Responsibility for data interpretation will have been determined during the planning phase and should be under the direct control of the owner, the design consultant, or instrumentation specialists selected by the owner. Interpretation is a task for experienced geotechnical engineers. They should be familiar with the planning of the monitoring program, the project construction procedure, and details of instrument installation, data collection, processing, and presentation. They should have computational and analytical skills and should be

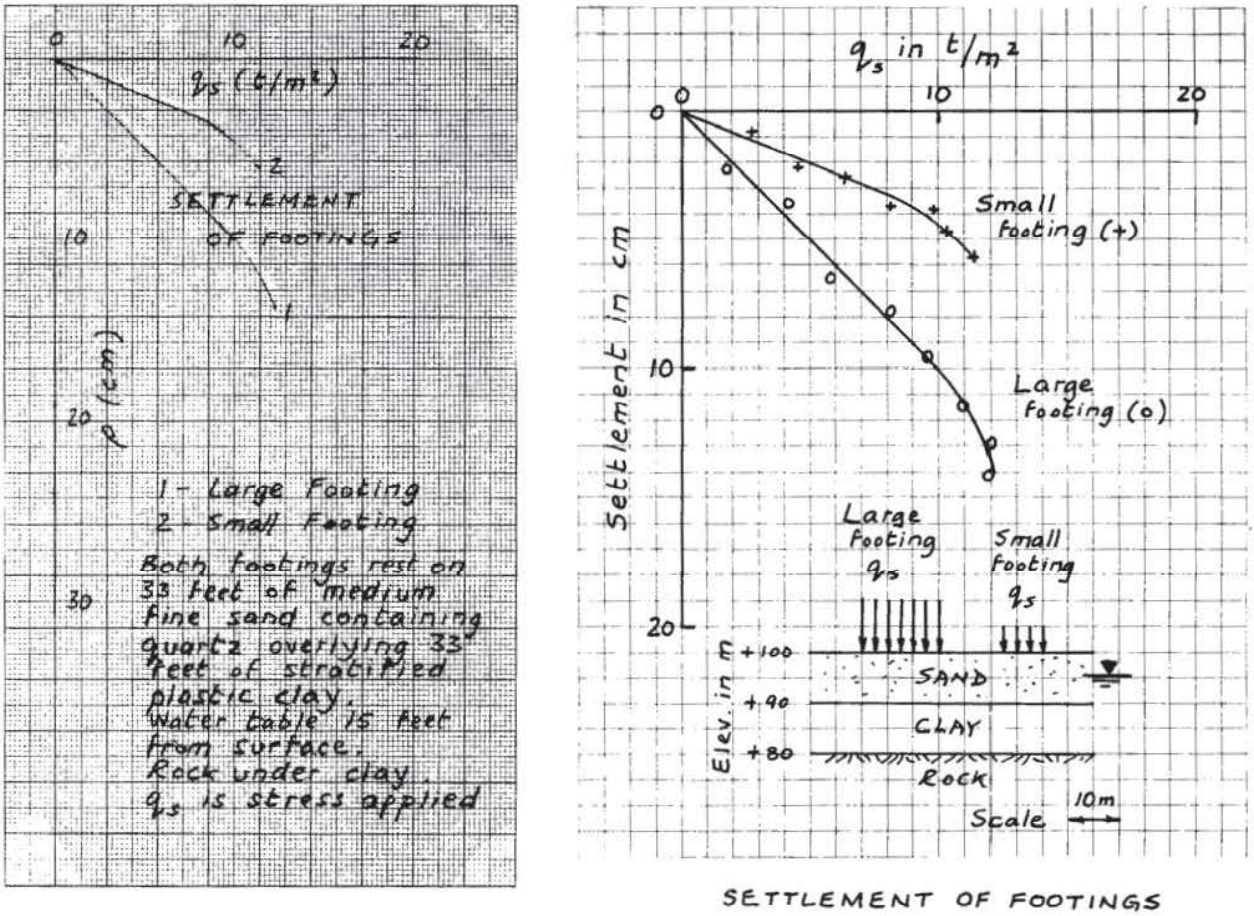


Figure 18.11. Poor and good portrayal of same data (after Lambe, 1970).

capable of exercising engineering judgment. In many cases, the same geotechnical engineer who has been responsible for data collection, processing, and presentation will play a lead role in data interpretation. If a lead role is inappropriate, this person must at least act in a coordinating and review capacity, because interpretation cannot be divorced from all preceding tasks.

18.3.2. General Guidelines on Interpretation of Data

Details of data interpretation are dependent on factors specific to each instrument and project, but a few general guideines can be given.

Preliminary Work

A draft procedure for data interpretation should have been written during the planning phase, and this should be studied so that interpretation is tuned to the intended purpose of the instrumentation program.

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.

Interpretation and Reinterpretation

Interpretation and reinterpretation is an ongoing process. Initial interpretations will be tentative, dependent on collection of further data. Interpretations may change as a clearer understanding of real behavior is developed. Assessment of the performance of an individual instrument often requires a study of data over a significant time period.

Selection of Plots

Various types of plot that are useful when interpreting data have been described in Section 18.2.6. They include routine plots of data versus time, plots to assist with predictions, plots for comparing observed and predicted behavior, plots for comparison of measurements and observations, and plots to examine cause and effect relationships. Selection of plots must be made on a case-by-case basis, with the purpose of the instrumentation program clearly in mind.

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 (Chapter 17) are often helpful when evaluating questionable data, because difficulties encountered during installation may cause abnormal data. Questionable data can also be evaluated by reviewing the studies of data correctness that have been made earlier, in the field during the data collection phase and in the office during the data processing phase.

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. 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.

18.4. 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, and a technical paper may be prepared.

18.4.1. 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

It may be appropriate also to include copies of calculation sheets, but the reports should be kept as simple as possible to avoid delay in preparation and submittal.

18.4.2. 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 the owner and design consultant so that any lessons learned may be incorporated into subsequent designs. If an installation report (Chapter 17) has been written, the final report can either reference that report or repeat appropriate content. The final report should contain at least the following information:

- Summary of the report.
- Introduction, including a brief description of the project and the reason for using geotechnical instrumentation.
- Any project design and construction information that is relevant to the monitoring program.
- Summary of the monitoring program planning phase.
- Description of instruments and readout units.*

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

- Plans and sections sufficient to show instrument numbers and locations.
- Appropriate surface and subsurface stratigraphic and geotechnical data.*
- Instrument calibration and maintenance procedures.*
- Instrument installation procedures.*
- Procedures for data collection, processing, presentation, and interpretation.*
- 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.

Whenever possible, the report should also include an assessment of the monitoring program. This assessment should cover instrument performance, calibration and installation techniques, adequacy of data collection, processing, presentation, and interpretation procedures, and recommendations for specifying future monitoring programs. Inclusion of such an assessment may entail some self-criticism but will provide valuable input to planning future monitoring programs. We learn from our mistakes, and we very much welcome learning from the mistakes of others!

When instruments are used for monitoring long-term performance, the final report should form part of the operating and maintenance (O&M) manual. The O&M manual should also provide guidelines to assist O&M personnel in judging satisfactory performance based on the measurement data. An example of guidelines for O&M personnel responsible for embankment dams is given by Dunnicliff (1981).

On completion of the final report, all data should be filed in an orderly manner in one location.

18.4.3. Technical Publication

Well-documented case histories of monitoring programs are invaluable for advancing the state of the art of geotechnical design and construction. The fact that many geotechnical journals contain more research-oriented papers than case history papers generally reflects the nature of papers submitted for publication rather than the intent of journal publishers to emphasize research. Well-documented case histories are welcomed by journal publishers and readers, and engineers who are in a position to prepare these papers are strongly encouraged to do so. The content will usually be similar to the content listed above for the final report.

This plea for well-documented case histories should not be taken as a plea for papers describing every monitoring project. Before embarking on writing a paper, the writer should have positive answers to two questions: *Do I have something new to say?* and *Will my work help to fill a knowledge gap?*

Part 5

Examples of Instrumentation Applications

The goal of Part 5 is to describe the role of instrumentation in addressing geotechnical questions that may arise during the design, construction, or performance of typical civil engineering projects.

The chapters are divided into four 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 third section provides an overview of what is typically *routine* monitoring and what is monitoring for *special applications*. The fourth section presents a tabular summary of selected case histories that illustrate effective use of geotechnical instrumentation. Case histories are summarized in chronological order, starting with the most recent.

The sequence of geotechnical questions in the second part of each chapter 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. Sketches are included to convey a possible layout of instruments for addressing some of the geotechnical questions. However, readers are cautioned that these layouts are intended merely to convey a general approach: as indicated in Chapter 4, the selection of instrument locations should reflect predicted behavior for each individual case and should be compatible with the method of analysis that will later be used when interpreting the data.

The format and content of this part of the book have been strongly influenced by DiBiagio and Myrvoll (1981), describing use of instrumentation for projects in or on soft clay.

These chapters are not intended to be exhaustive summaries, state-of-the-art papers, or "cookbooks." They are intended merely to open the minds of readers to the possible role of geotechnical instrumentation and to guide them toward implementation.

It is said in Chapter 4 that geotechnical instrumentation should not be used unless there is a valid reason that can be defended. 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. If a reader uses this book by (1) studying the chapters in Part 5, (2) noting the types of instruments, (3) noting the sketched layouts, (4) studying Part 3, *Monitoring Methods*, for details of the instruments, and (5) proceeding with a monitoring program, that reader is misusing the book. Readers are strongly encouraged to follow the more logical procedure outlined in Chapter 4.

CHAPTER 19

BRACED EXCAVATIONS*

Several types of wall are used for supporting the sides of braced excavations, including steel sheet piles, soldier piles and lagging, auger piles, interlocking wood sheeting, and slurry walls. Lateral support can be provided either by *internal* or *external bracing*. Internal bracing consists of structural members of steel or wood, placed either as horizontal *cross-lot bracing* or as inclined or horizontal *rakers*. The most commonly used external bracing consists of *tieback anchors*.

Figures 19.1–19.3 show the contrast between how projects will usually appear on engineering drawings, and what may actually happen in the field: the *field* figures are exaggerated to emphasize the contrast. Figure 19.1 depicts a typical excavation in clay, supported by interlocking sheet piles and cross-lot bracing, for the case where the sheet piles are not driven to bedrock or underlying firm stratum. Figure 19.2 shows a typical excavation supported by soldier piles, lagging, and tieback anchors. Figure 19.3 shows a cross section through a typical excavation for a slurry wall, prior to concreting.

19.1. GENERAL ROLE OF INSTRUMENTATION

The design of 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.

The choice between internal and external bracing has a major influence on the type and role of instrumentation for a braced excavation. 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.

*Written with the assistance of Mark X. Haley, Associate and Vice President, Steven R. Kraemer, Associate & Vice President, and Cetin Soydemir, Senior Engineer, Haley & Aldrich, Inc., Cambridge, MA.

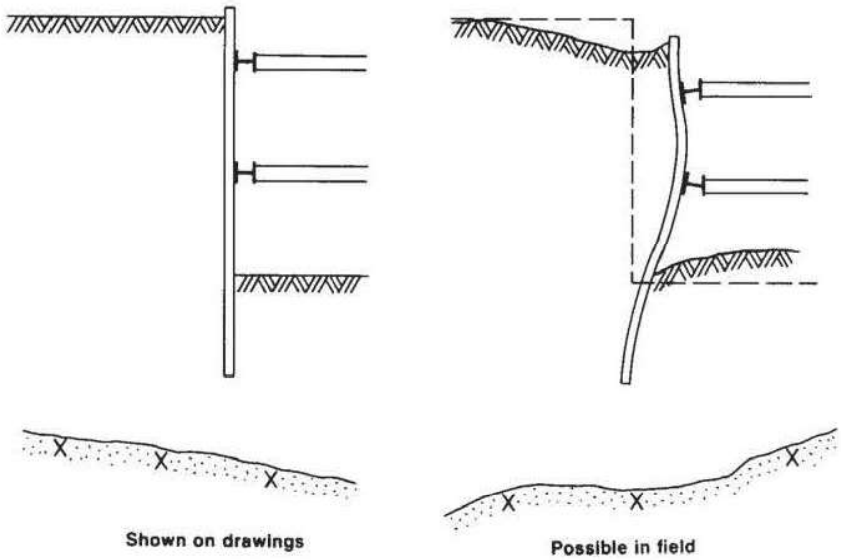


Figure 19.1. Open cut excavation supported by interlocking sheet piles and cross-lot bracing (after DiBiagio and Myrvoll, 1981).

19.2. PRINCIPAL GEOTECHNICAL QUESTIONS

The following principal questions are presented in the normal order of occurrence for a braced excavation. The order does not reflect a rating of importance.

19.2.1. What Are the Initial Site Conditions?

Initial site conditions are determined by use of conventional site investigation procedures, sometimes supplemented by in situ testing. 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 by representatives of the construction contractor's insurance company and some-

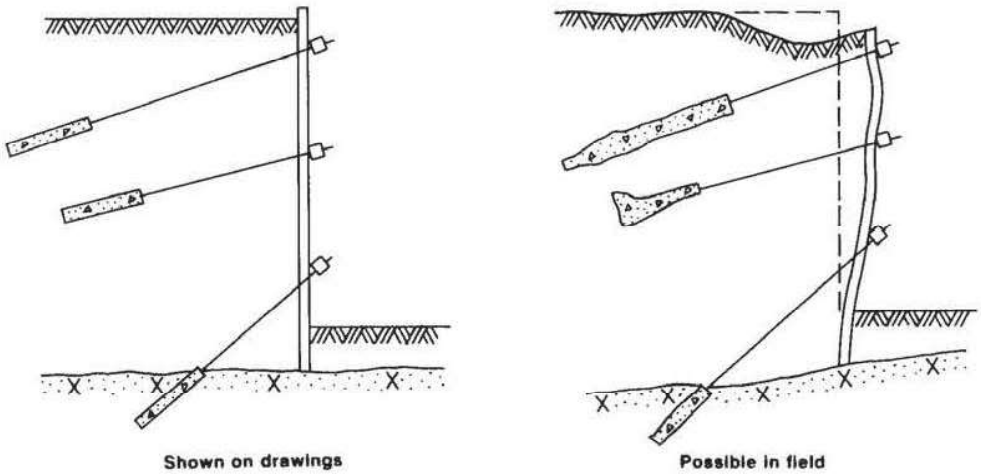


Figure 19.2. Open cut excavation supported by soldier piles, lagging, and tieback anchors (after DiBiagio and Myrvoll, 1981).

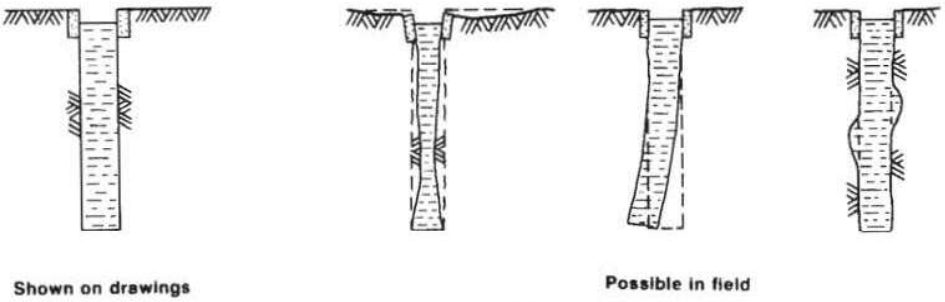


Figure 19.3. Excavated trench for slurry wall (after DiBiagio and Myrvoll, 1981).

times also by the owner. The purpose of the preconstruction survey is 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 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.

19.2.2. What Information Can Be Provided by Instrumenting a Full-Scale Test of the Complete Excavation and Support System?

The primary goals when designing support are to control ground movements and to establish a stable excavation. The designers may identify a combination of circumstances that lead them to opt for a full-scale test section, as a means of ensuring design adequacy and economy. The combination of the following four circumstances provides an example. First, the planned excavation is for a long subway or water tunnel. Second, subsurface conditions are judged to be relatively uniform along the length of the excavation. Third, there is little precedent for constructing a braced excavation in similar ground. Fourth, the designers believe that a full-scale test section can reduce the conservatism of the loading envelope and can therefore be economical.

Instrumentation and observation are essential components of a full-scale test section. The intensity of instrumentation is usually greater than for other applications, so that maximum information can be learned from the test. The types of instrumentation are discussed elsewhere in this chapter.

The test section should be long enough so that end effects are minimized. As a general rule, the length of the test section should be at least four times the width of the excavation, and it should be located at least three braces away from any nonuniformity in the cross section. However, this ideal must be tempered by cost considerations and by knowledge of site and project conditions.

19.2.3. 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 displacement at the anchor heads.

As indicated in Chapter 13, calibrated hydraulic jacks do not provide an accurate measurement of load, and a good-quality load cell should be used adjacent to the jack. Electrical resistance load cells are normally used, but mechanical, vibrating wire, and hydraulic cells are also suitable. The author favors use of the Proceq mechanical load cell (Figure 13.1) for this purpose: it is robust, has adequate accuracy, is read simply by observing a dial indicator, and does not require attachment of an electrical cable or remote readout unit.

Displacement is normally measured by attaching a long-range dial indicator to a survey tripod, with the dial indicator stem bearing on the anchor head. A universal joint at the connection between dial indicator and tripod eases the setting up task.

If the objective of test anchors includes determi-

nation of the pattern of load transfer in the grouted zone, strain gages and/or multiple telltales should be used, as described in Section 19.2.9.

19.2.4. 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 proved 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. Various aspects of a typical full-scale test are discussed in the following subsections.

Stability in Relation to Time

One objective of a full-scale test is to monitor lateral and vertical ground deformations in the vicinity of the trench in relation to time. The magnitude and rate of observed deformations will provide an indication of trench stability or instability.

Lateral deformations in the soil alongside the trench can be monitored by using inclinometer casings installed close to the walls of the trench. The alignment of boreholes for inclinometer casings should be controlled carefully, so that casings do not intercept the excavation. However, because neither the casing nor the trench will be exactly vertical, the top of an inclinometer casing should not be closer than about 18 in. (460 mm) to the wall of the trench, and a backup casing is advisable at about 4 ft (1.2 m) from the wall. Because inclinometers cannot provide data on lateral deformation of the wall of the trench itself, a comprehensive test includes monitoring of convergence across the trench, using one of the gages described in Section 12.11.2.

Vertical movements of the ground surface and settlement of the guide walls are normally monitored by surveying methods. Although not of primary importance, monitoring of subsurface vertical deformation may provide valuable input to the evaluation of the test and is usually justified. Probe extensometers will normally be used. Trends in measured pore water pressure in the surrounding soil can also serve as a guide to indicate how fast overall stability is deteriorating with time, and pore water pressures can be monitored with rapidly responding diaphragm piezometers. Figure 19.4 shows a possible layout of instrumentation for examining the stability of a slurry trench in relation to time. The piezometers remote from the test panel are required as reference piezometers, to monitor any variation in groundwater pressure that may result from other causes.

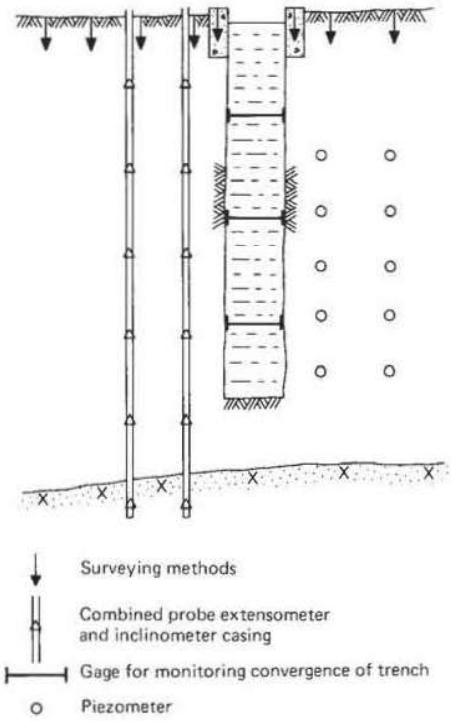


Figure 19.4. Possible layout of instrumentation for examining the stability of a slurry trench in relation to time (after DiBiagio and Myrvoll, 1981). Note: Use of surveying methods requires benchmark, remote from test panel. Piezometers also required, remote from test panel.

Stability in Relation to Panel Length

The limiting length to width ratio for which a single panel can be excavated safely can be established on the basis of full-scale test panels of varying length. Comprehensive measurements of the displacement field are needed to evaluate the restraint provided by the soil at the ends of a trench, using the instruments shown in Figure 19.4.

Stability in Relation to Slurry Level and Density

Trench stability is affected by the difference between the level of slurry and the groundwater level and by the density of slurry. The effects can be assessed by varying the slurry level and density

while monitoring ground deformation and groundwater level in the vicinity of the panel, again using the instruments shown in Figure 19.4.

Shape and Alignment of Slurry Trench

The shape and alignment of the trench may not be as intended. It is impossible to make a visual examination of an excavated trench; therefore, any final inspection must rely on instrumentation. The width of the trench at any point can be determined with remote reading caliper tools, and the same instrument can be used to search for local failures or overbreak, particularly in fissured clay where blocks of material may fall out during or subsequent to excavation. Errors in vertical alignment of slurry trenches may have serious consequences, and verticality of the test trench can be measured using an inclinometer, by lowering inclinometer casing temporarily into the trench and pressing the casing against one side with small double-acting pneumatic jacks. Other measurement methods are described in Section 19.2.6.

Deformation of Trench During Concreting

Filling a deep slurry trench excavation with fresh concrete may cause a significant increase in loading on the bottom and sides of the trench and may lead to local instability. If the trench is excavated in very soft clay, large local deformations may occur, and there may even be a chance of a bearing capacity type failure of the bottom or sides of the excavation. When a test panel is constructed, lateral and vertical deformations and pore water pressures should be monitored closely during concreting. Instrumentation is again as shown in Figure 19.4, with the addition of small settlement platforms set on the bottom of the trench. A sleeve should be provided around the riser pipe for each settlement platform.

19.2.5. Is the Bracing Being Installed Correctly?

Field measurements play a role in verifying that internal and external bracing is being installed correctly.

Internal Bracing

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, and a special effort must be made to ensure that the jack is calibrated and aligned correctly, as discussed in Chapter 13.

External Bracing

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 in Section 19.2.3 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.

When a tieback anchor is proof tested and locked-off, the load may be changed in adjacent anchors that have already been stressed and locked-off. For example, in the case of a slurry wall or a tied-back bulkhead, locking-off each anchor at a predetermined load does not necessarily produce the desired condition, and one or more rounds of adjustments may be required. Measurements are normally made by *lift-off* testing, using a load cell adjacent to the hydraulic jack.

19.2.6. 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. The author is aware of five methods 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.* Fifth, an inclinometer can be used, as described in Section 19.2.4, 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.

Sonic coring, as described in Chapter 24, can be used to test the integrity of concrete in a slurry trench.

19.2.7. 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.

Whenever there is concern for instability, surface deformation should be monitored, using surveying methods. Monitoring should include settlement of the ground, the top of the supporting wall, and nearby structures. When tiebacks are used with soldier piles and lagging, settlement measurements should normally be made on the tops of all soldier piles, to monitor any downward movement of a pile owing to the downward component of anchor load. A *rod settlement gage* (Section 12.7.7) alongside each important structure will provide accurate settlement data without need for a survey crew. Horizontal deformation measurements on the top of the supporting wall, using surveying methods and a horizontal control station, provide routine data for stability control when it is critical to minimize ground movements.

Crack gages across existing cracks in structures provide important information when there is concern that excavation may cause additional damage.

Subsurface settlement alongside important underground utilities should also be monitored, using subsurface settlement points.

If the monitoring program consists only of deformation measurements on the surface and alongside utilities, and the 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. When soldier piles and lagging are used, inclinometer casings can be attached to soldier piles, but the bottom should be deep enough to ensure base fixity, and this may require attaching steel pipes to the soldier piles and drilling through the pipes prior to installation of inclinometer casing. When a slurry wall is used, inclinometer casings can be installed within the trench prior to concreting, again ensuring that the bottom is deep enough for base fixity, possibly requiring drilling below the base of the trench. For all types of wall, inclinometer measurements are often supplemented by using convergence gages across the excavation. Fixed borehole extensometers are an alternative to inclinometers if external bracing is used. A simple plumb line can sometimes be useful for providing horizontal deformation data at low cost. Subsurface settlement measurements, using either probe extensometers or fixed borehole extensometers, can be used to provide additional causal data.

Figure 19.5 shows a possible layout of instrumentation for monitoring an internally braced excavation in clay when there is concern that a

*A commercial source is given in Appendix D.

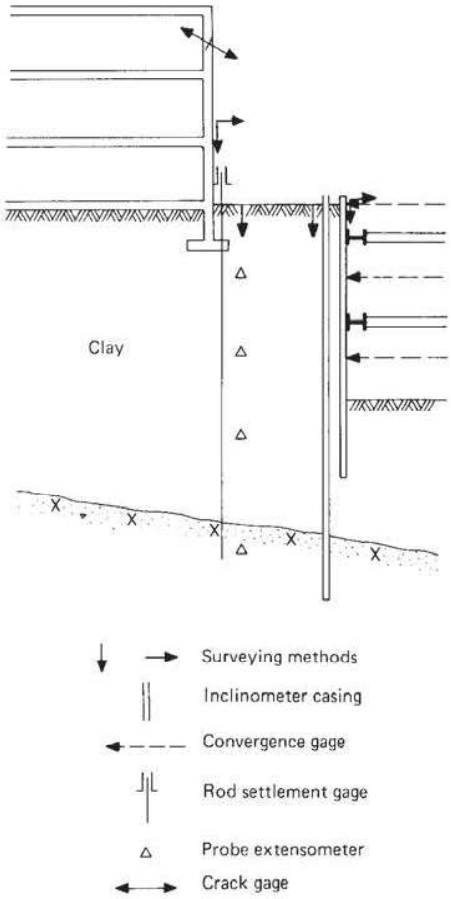


Figure 19.5. Possible layout of instrumentation to provide forewarning of adverse effect on a building alongside an internally braced excavation in clay. Note: Use of surveying methods requires benchmark and horizontal control station, remote from excavation. Piezometers also required if lowering of the groundwater table could cause problems.

building may be affected adversely by ground movements. This is only one example, and it must be stressed that many other configurations are possible, as indicated by the case histories described later in this chapter.

Adverse ground movements, of course, can also be caused by overload of individual braces, lowering of the groundwater table, or bottom heave, and these issues are addressed in subsequent sections.

Table 19.1 lists suitable instruments for monitoring whether the excavation is stable and whether nearby structures are being affected adversely by ground movements.

19.2.8. 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.

Table 19.1. Instruments Suitable for Monitoring Whether Excavation Is Stable and Whether Nearby Structures Are Being Affected Adversely by Ground Movements

Measurement	Suitable Instruments
Settlement of ground surface, structures, and top of supporting wall	Surveying methods Rod settlement gages
Horizontal deformation of ground surface, structures, and exposed part of supporting wall	Surveying methods Convergence gages Plumb lines
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 internal bracing	Surface-mounted vibrating wire strain gages Surface-mounted mechanical strain gages Calibrated hydraulic jack and load cell (lift-off test)
Load in external bracing	Load cells Calibrated hydraulic jack and load cell (lift-off test) Surface-mounted vibrating wire strain gages
Groundwater pressure	Piezometers
Bottom heave	Magnet/reed switch gages Mechanical heave gages Fixed borehole extensometers Inclinometers

Internal Bracing

For an internally braced excavation, strut loads should be monitored on as many struts as is practicable or economically justified, and reserve struts should be available for immediate installation in the event that measurements indicate that allowable limits have been reached. Surface-mounted vibrating wire strain gages are usually the preferred instruments, and mechanical 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.

When using strain gages for determination of strut loads, a check on strain gage performance can be made during preloading. O'Rourke and Cording (1975) report good correlation between hydraulic jack and strain gage measurements soon after calibration of the jack, but the correlation became less good over a period of months without recalibration. A load cell in series with each hydraulic jack provides a better check on strain gage performance. Chapter 13 includes a discussion of temperature effects on strut loads and on strain gage readings and indicates that an error can occur if the strain gage and the structural member are at different temperatures. This problem can be acute when sunlight causes differential heating of a steel strut, and the guidelines given in Chapter 13 should be followed.

External Bracing

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. Any of the load cells discussed in Section 19.2.3 are adequate, and *telltale load cells* provide an economical method, provided that sufficient annular space is available and that access to the anchor head is maintained. Surface-mounted strain gages, preferably of the weldable vibrating wire type, are an alternative to load cells if *threadbar* (threaded solid bar) tendons are used.

19.2.9. 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. The magnitude and distribution of load can be determined by making measurements in a test section of adequate length, as discussed in Section 19.2.2.

Internal Bracing

Determination of stress in internal bracing requires either lift-off testing or use of strain gages, as discussed in Section 19.2.8.

External Bracing

Determination of total load in external bracing requires either lift-off testing or use of load cells or strain gages, as discussed in Section 19.2.8.

Determination of the pattern of load transfer in the grouted zone requires measurement of strain, using strain gages and/or multiple telltales. Very great cooperation is essential between personnel responsible for instrumentation and personnel responsible for installation of the anchors. When an anchor instrumented for this purpose is installed, strain measurements will normally be made during the proof test to determine a relationship between measured strain and load and to determine the quality of strain data. An estimate of the combined modulus of the steel and grout is needed so that measured strains can be converted to stresses.

When detailed load transfer data are required for threadbar tendons, weldable vibrating wire strain gages are the first choice. Fuglevand et al. (1984) report that a manufacturer of threadbar tendons recommended against use of weldable strain gages, believing that the heat from welding could affect the metallurgical characteristics of the bar. However, gages were attached to test lengths of bar that were then pulled to failure, with no notable reduction in capacity. Weldable resistance strain gages are an alternative to vibrating wire gages but, as discussed in Chapter 13, they have many limitations. Damage to all types of strain gages and cables is not unusual while pulling drill casing and while grouting; there-

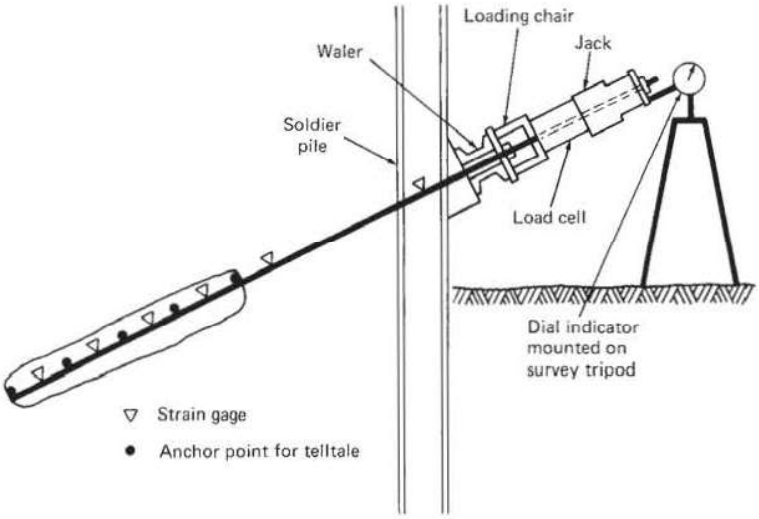


Figure 19.6. Setup during proof test, showing possible layout of instrumentation for a *threadbar* anchor, when detailed load transfer data are required.

fore, great care must be taken with protection arrangements, and significant redundancy is recommended. Whenever adequate annular space is available, multiple telltales are recommended as backup. Two configurations are possible: first, unstressed 0.25 in. (6 mm) diameter flush-coupled rods within oil-filled sleeves of PVC pipe; and second, the *slack-wire* type of fixed borehole extensometer system described in Chapter 12, with each wire housed within an oil-filled 0.25 in. (6 mm) outside diameter nylon tubing. The first alternative is simpler but requires more annular space than the slack-wire system.

Figure 19.6 shows a possible layout of instrumentation for a threadbar anchor when detailed load transfer data are required. The figure shows the setup during the proof test.

When detailed load transfer data are required for stranded cable tendons, attachment of strain gages is impracticable, and multiple telltales are the only available option. However, an additional difficulty arises with stranded cable tendons that does not arise with threadbar tendons. As the drill casing is pulled, it must normally be rotated, tending to cause a spiral in the tendons and binding of the telltale rods or wires within their sleeves, with significant reduction in accuracy of data. Provisions must be made for examining the extent of binding and, if possible, for gathering enough data so that frictional effects can be screened out. The slack-wire extensometer is a suitable choice because readings can be made at various wire tensions and adequate data obtained for screening. If unstressed rod telltales are used, they should be provided with a disconnect between the lower end of each rod and its anchor (Section 12.7.1), so that binding can be examined. If possible, both wire and rod telltales should be used on a stranded cable tendon, one as backup to the other. A telltale can be attached only to a single cable, and the assumption must therefore be made that the behavior of this single cable is representative of the behavior of all the cables that form the tendon: experience has shown that this appears to be a valid assumption.

Sheeting and Walers

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.

Slurry Walls

Stresses in slurry walls can be determined by use of strain gages, with a preference for the *sister bar* type of embedment gage with a vibrating wire transducer. Alternatively, strain measurements can be made and bending moments determined by measuring within sliding micrometer casings installed near

the front and back of the wall. 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 (DiBiagio and Roti, 1972).

19.2.10. 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. Whenever piezometers are installed in the vicinity of an excavation, reference piezometers should also be installed, remote from the excavation, to monitor any variation in groundwater pressure that may result from other causes.

19.2.11. 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.

Three types of heave gage are described in Chapter 12. First, there is a mechanical heave gage (Section 12.5.1) that is simple and inexpensive, but its accuracy is not great, its use requires a survey crew, and there is a risk of the borehole caving during excavation. Second, there is a magnet/reed switch gage (Section 12.5.7) that can be used to provide reliable and accurate data without interfering with excavation work. Third, there is a fixed borehole extensometer fitted with an electrical linear displacement transducer (Section 12.7.4) that is accurate but prone to damage and malfunction. For most applications the magnet/reed switch gage is the instrument of choice.

19.2.12. Is Long-Term Performance of the Bracing Satisfactory?

Internal bracing is either removed or incorporated into the final structure. The question is therefore limited to external bracing.

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 in Section 19.2.8 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.

Longevity of Load Cells

Longevity can be maximized by following the guidelines for reliability, given in Chapter 15.

The arrangement shown in Figure 19.7 can be

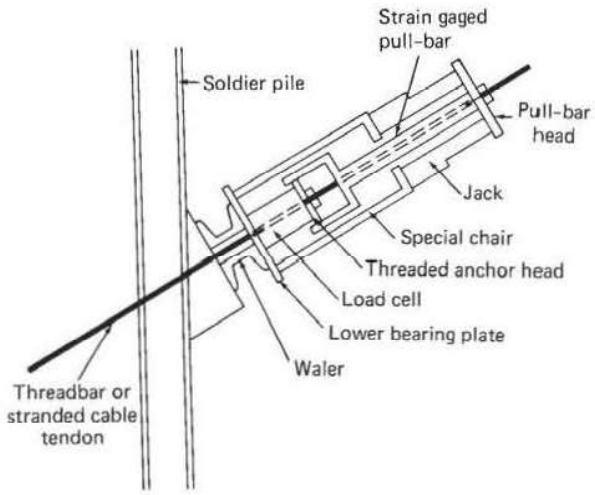


Figure 19.7. Schematic of arrangement for check-calibrating a load cell by *lift-off* testing.

adopted to check-calibrate a load cell by *lift-off* testing. The anchor head must have a threaded outside diameter or threaded central hole, for later connection to a *pull-bar*. The procedure involves placing a special chair over the load cell and anchor head, to bear on the lower bearing plate, threading a calibrated strain gaged pull-bar on to the anchor head, and inserting a center-hole hydraulic jack over the pull-bar to bear between the top of the chair and the pull-bar head. As the jack is actuated, load is transferred from the load cell to the pull-bar, a calibration of the load cell is made, and the zero reading of the load cell is determined after lift-off.

Nicholson Construction Company, Bridgeville, PA, has devised a method for removing a load cell without unloading the tieback, so that the cell can be checked and a new cell inserted if necessary. The arrangement is shown in Figure 19.8. A threaded anchor head is used, and the inside diameter of the load cell must be larger than the outside diameter of the anchor head/pull-bar arrangement. Prior to lock-off, split shims are inserted between the load cell and anchor head, with adequate total thickness for transferring load from anchor head to load cell. When preparing to remove the load cell for checking, a special chair is placed over the load cell and anchor head to bear on the lower bearing plate, with sufficient internal height to contain the load cell as described below. A pull-bar is threaded on to the anchor head, and a center-hole hydraulic jack is inserted over the pull-bar to bear between the top of the chair and the pull-bar head. The jack is actuated to lift the anchor head off the split shims, the split shims removed, the load cell lifted over the anchor head, and the split shims replaced together with additional split shims in the space previously

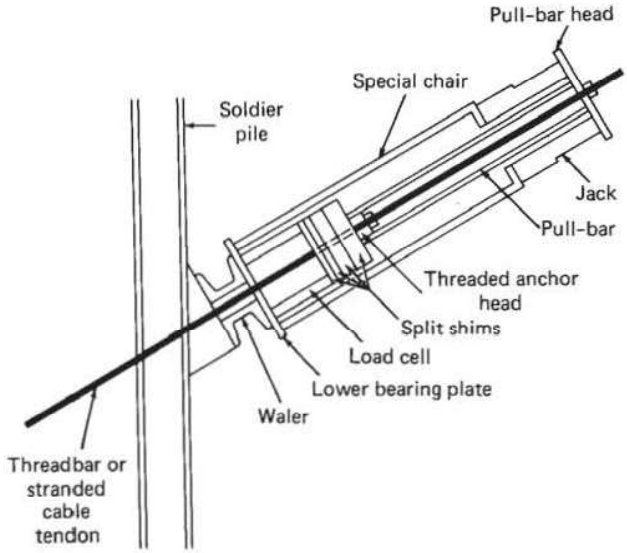


Figure 19.8. Schematic of arrangement for removing a load cell without unloading the tieback (after Nicholson Construction Company, Bridgeville, PA).

Table 19.2. Overview of Routine and Special Monitoring

Applications	Parameters
Routine monitoring: all types of ground support	Settlement of ground surface, structures, top of supporting wall, and utilities Horizontal deformation of ground surface, structures, and exposed part of supporting wall (when it is critical to minimize ground movements) Change in width of cracks in structures and utilities Convergence within excavation Groundwater pressure
Routine monitoring: external bracing	Load and deformation at anchor head during proof and performance testing of tieback anchors
Routine monitoring: internal bracing	Preload in bracing
Routine monitoring: slurry trench excavations	Level and properties of slurry Quantity of excavated material Verticality of trenches
Special applications: all types of ground support	Subsurface horizontal and vertical deformation around the excavation Bottom heave Stress in support system Deformation and support stress in full-scale test section of complete braced excavation and support system
Special applications: external bracing	Load and deformation at anchor head while loading test anchors during design phase or at the start of construction Long-term load and deformation in permanent tieback anchors
Special applications: slurry trench excavations	Deformation, groundwater pressure, level, and properties of slurry during full-scale test of slurry trench excavation Integrity of concrete

occupied by the load cell. The jack pressure is then released, and the pull-bar, jack, and special chair are removed. After checking, the load cell is replaced, using the reverse of the above procedure.

The second method of checking is more comprehensive than the straightforward lift-off test, because it allows for replacement of the load cell with a new cell in the event of serious malfunction. Both methods require use of special load cells designed for long-term use, and of course require that a sufficient length of tendon is left in place beyond the anchor head and that access is available for testing.

Interruption to Corrosion Protection

Instruments installed on a tieback anchor for monitoring long-term performance may interrupt the corrosion protection, and in general simple manual instruments create greater interruption than more complex remotely read instruments. Because simplicity leads to longevity, a dilemma is faced, and the designer must make a selection between two approaches.

First, simple instruments such as mechanically read telltale load cells can be used, or one of the above two methods for checking conventional load cells can be adopted, the interruption to corrosion protection accepted, and the monitored anchors regarded as *worst cases*. Second, a waterproof protective cover can be provided over the head assembly and filled with a rust-inhibiting protective grease, but this option requires use of inaccessible remotely read instruments, with reduced ability to check and replace them if necessary. Selection between the two approaches depends on likely consequences of interrupting the protection and on the availability of instruments that satisfy the criteria for long-term reliability, discussed in Chapter 15. For the particular case of monitoring load at the anchor head when threadbar tendons are installed, the best approach appears to be use of a pair of weldable vibrating wire strain gages on the bar (Figure 13.26), about 6 ft (2 m) from the nut. These gages can easily be incorporated into a double corrosion protection system, and are about the same size as a coupler in the tendon.

19.3. OVERVIEW OF ROUTINE AND SPECIAL APPLICATIONS

For those readers who seek general guidelines on what is typically *routine* monitoring and what is monitoring for *special applications,* Table 19.2 presents a broad generalization.

19.4. SELECTED CASE HISTORIES

Table 19.3 summarizes selected case histories of instrumented braced excavations.