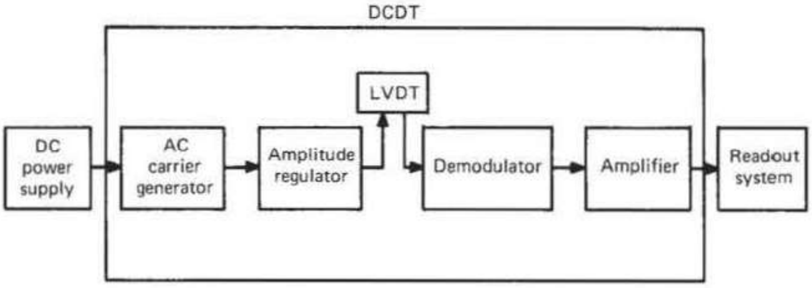



Figure 8.19. Schematic of direct current differential transformer (DCDT).

### 8.4.6. Direct Current Differential Transformer (DCDT)

DCDTs have similar applications to LVDTs and are usually preferred for geotechnical use. The need for special means of exciting the LVDT primary coil and modifying its secondary output voltage is provided for by miniaturizing the electrical circuitry and placing components within the transducer housing as shown in Figure 8.19. The device is now called a DC LVDT, or DCDT. The signal cable carries only DC voltages and unwanted cable effects are avoided.

DCDT manual data acquisition systems require only a stable DC power supply and a DC voltmeter. Most DCDTs have excellent resistance to humidity and corrosion effects and good long-term stability. They can also be protected within oil-filled housings.

### 8.4.7. Potentiometer

Linear potentiometers are an alternative to LVDTs and DCDTs for remote measurement of linear deformation. Rotary potentiometers are used for measurement of rotational deformation and where linear deformation can readily be converted to rotational deformation.

A potentiometer (*pot*) is a device with a movable slider, usually called a wiper, that makes electrical contact along a fixed resistance strip. As shown in Figure 8.20, a regulated DC voltage is applied to the two ends of the resistance strip and the voltage between *B* and *C* is measured as the output signal. The voltage varies between the voltage at *A* and the voltage at *B* as the wiper moves from *A* to *B*. When the device is used for measurement of linear deformation and the relationship between wiper position and output signal is proportional, the device is called a linear potentiometer. By reading voltages between *B* and *C*, and between *A* and *C*, a checking feature is created because the sum should equal the input voltage. An alternative method of readout is to measure the resistance between the wiper and one end of the resistance strip and to relate the measured resistance to the total resistance of the strip. In this way, the position of the wiper can be measured using an ohmmeter or digital multimeter. In a third readout method, the potentiometer forms two arms of a Wheatstone bridge circuit, and measurement of wiper position is made by using a portable indicator that contains a balancing potentiometer.

Potentiometers are not suitable for measuring rapidly varying motions. However, the readout is simple and can be arranged to give a high output voltage, which is not readily degraded by long lead wire effects or electrical noise. The resistance strip and wiper must be sealed to prevent moisture intrusion and, provided there are no leaks whatsoever, potentiometers can successfully be used for long-term measurements. In adverse environments, it may be worthwhile to seal potentiometers in oil-filled housings: oil-filled rotary potentiometers are available commercially, but the author is not aware of commercially available oil-filled linear potentiometers. However, because of the difficulty in en-




Figure 8.20. Schematic of linear potentiometer.

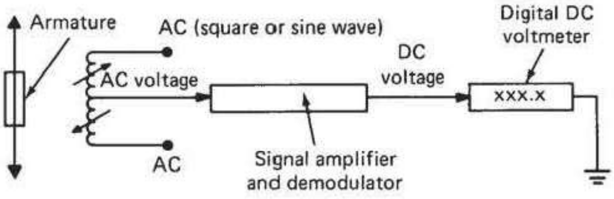



**Figure 8.21.** Schematic of variable reluctance transducer.

suring a long-term perfect seal on a linear potentiometer, LVDTs and DCDTs are now often preferred for long-term applications.

### 8.4.8. Variable Reluctance Transducer

Variable reluctance transducers (VRTs) are used in electrical crack gages and fixed embankment extensometers to measure linear deformation.

A VRT (Figure 8.21) consists of a center-tapped coil, positioned around a movable armature. The armature is magnetically permeable. An AC excitation voltage is applied to the ends of the coil, and an output voltage is sensed on the third wire. Movement of the armature away from the center, or null position, causes an imbalance in magnetic flux density between the two coil sections, which in turn causes the voltage in one section of the coil to increase, and the other to decrease. The signal amplifier and demodulator, shown in Figure 8.21, is normally positioned near the transducer. DC signals can be displayed on analog or digital meters or transferred to conventional data loggers and other voltage sensing devices. VRTs can also be read with portable strain indicators that are used with electrical resistance strain gages.

The coil is normally enclosed, both outside and inside, by stainless steel tubing sealed at the ends, and waterproofing of the lead wires is maintained to the signal amplifier and demodulator. Some leakage of moisture into the lead wires can be tolerated, and unless very low electrical resistance paths are created by free moisture between conductors, little degrading of the signal results. Water in the armature cavity is of no consequence. VRTs are therefore particularly suitable for installations below water, and they have been used very successfully in crack gages installed to monitor movement across perimeter joints on the upstream faces of concrete face rockfill dams. However, they should not be considered as the only transducers that are suitable for this purpose, since an LVDT or a DCDT could be sealed in a similar way to perform the same function.

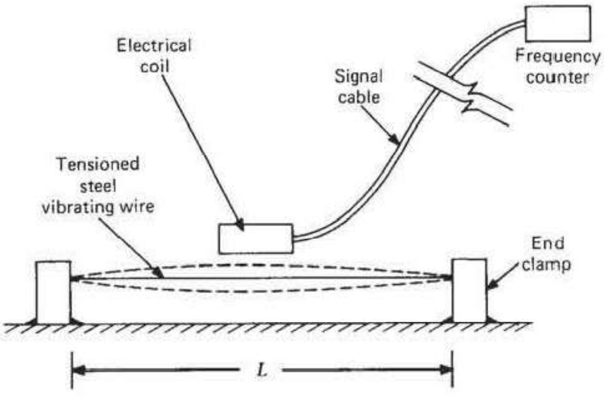



**Figure 8.22.** Schematic of surface-mounted vibrating wire strain gage.

### 8.4.9. Vibrating Wire Transducer

Vibrating wire transducers (Dreyer, 1977; Thomas, 1966) are used in pressure sensors for piezometers, earth pressure cells, and liquid level settlement gages, in numerous deformation gages, in load cells, and directly as surface and embedment strain gages.

#### *Operating Principle*

A length of steel wire is clamped at its ends and tensioned so that it is free to vibrate at its natural frequency. As with a piano string, the frequency of vibration varies with the wire tension, and thus with small relative movements between the two end clamps. The wire can therefore be used as a strain gage by plucking the wire, measuring natural frequency, and relating frequency change to strain. The wire is plucked magnetically by an electrical coil attached near the wire at its midpoint, and either this same coil or a second coil is used to measure the frequency of vibration. Figure 8.22 shows a vibrating wire transducer arranged for measuring surface strain.

#### *Reading Methods*

There are two methods of wire plucking and reading, the *pluck and read* method and the *continuous excitation* (or *autoresonant*) method. The latter allows measurement of low-frequency dynamic strains.

The pluck and read method entails application of one or more voltage pulses to the coil, thereby creating a magnetic attraction that causes the wire to vibrate. The coil then becomes a listening device because wire vibrations cause an alternating voltage to be induced in the plucking coil, of frequency identical to the natural frequency of the vibrating wire. The voltage signal is transmitted along the signal cable to a frequency counter, which is used to measure the time for a predetermined number of vibration cycles.

The continuous excitation or autoresonant method entails a similar procedure to initiate vibration, but a second coil is used to detect the frequency. The signal is fed back to the driving coil so that it applies a continuously pulsing voltage with frequency identical to the natural frequency of the vibrating wire. As the wire frequency changes, so does the driving frequency. The wire frequency can be determined, as above, by measuring the time for a predetermined number of vibration cycles. Alternatively, the pulsing voltage can be converted directly and continuously, by a frequency to voltage converter, into a voltage that is proportional to pulse frequency and can be displayed on a digital voltmeter or recorded on magnetic tape or a strip chart recorder. In this way, the transducer can be used for measuring dynamic strains, provided that the cycle frequency of the changing strain is less than about 100 hertz (Hz). It can also be used for control and alarm systems. However, when the continuous excitation method is used, readings can become unstable under the influence of radio frequency or electromagnetic interference, and the cable should be shielded as described in Section 8.4.17.

For both methods, frequency may be displayed directly on the frequency counter, requiring use of a calibration curve or table for calculating strain from frequency change, or the readout unit may include a linearizing circuit such that strain is displayed directly.

### *Relationship Between Frequency and Strain*

The equation for frequency of a vibrating wire in terms of wire stress (Hawkes and Bailey, 1973) is

$$f = \frac{1}{2L}\sqrt{\frac{\sigma g}{\rho}},$$

where $f$ = natural frequency (sec$^{-1}$),
$L$ = length of vibrating wire (in.),
$\sigma$ = stress in the wire (lb/in.$^2$),
$\rho$ = density of the wire material (lb/in.$^3$),
$g$ = acceleration due to gravity (in./sec$^2$).

In terms of wire strain,

$$f = \frac{1}{2L}\sqrt{\frac{Eg\varepsilon}{\rho}},$$

where $E$ = modulus of elasticity of the wire (lb/in.$^3$),
$\varepsilon$ = strain in the wire.

Thus,

$$\varepsilon = \frac{4L^2f^2\rho}{Eg} = Kf^2,$$

where

$$K = \frac{4L^2\rho}{Eg}.$$

Because the transducer is always installed with the wire under an initial tension, both the initial frequency $f_0$ and the new frequency $f$ enter into the calibration relationship:

$$\varepsilon = K(f^2 - f_0^2).$$

### *Primary Advantage of Frequency Signal*

The output signal contains the required information in the form of a frequency rather than magnitude of a resistance or voltage, and therefore undesirable effects involving signal cable resistance, contact resistance, leakage to ground, or length of signal cable are negligible. Very long cable lengths are acceptable. The stability of a frequency signal can be demonstrated dramatically by inserting bare conductors in water and observing no change in measured frequency: therein is a major reason for favoring a frequency rather than resistance or voltage signal when making measurements in a field environment.

### *Sources of Error*

Historically, the major disadvantages of vibrating wire transducers have been wire corrosion, creep of the vibrating wire under permanent tension, and slippage at the wire clamping points, all of which usually result in a reduction in vibrating frequency.

Corrosion can be minimized by selection of materials that are not subject to galvanic corrosion and by drying and hermetically sealing the cavity around the wire. Attempts to minimize corrosion in vibrating wire pressure transducers by circulating dry nitrogen through them continuously, while successful in the short term, have proved difficult to maintain, for example, because of changes in operating personnel.

If the vibrating wire creeps under permanent tension, or if slippage occurs at the wire clamping points, a frequency reduction occurs that is unrelated to strain, and a zero drift has occurred. Although various investigators indicate that zero drift is minor (e.g., Bordes and Debreuille, 1985; Browne and McCurrich, 1967; DiBiagio, 1986; Londe, 1982; Thomas, 1966), others report that zero drift can be significant (e.g., Bozozuk, 1984; Jaworski, 1973; Kleiner and Logani, 1982; O'Rourke and Cording, 1975; Szalay and Marino, 1981).

Tharp (1986) reports on measurements of zero readings made 17 months after the original calibrations, prior to installation of 16 vibrating wire piezometers manufactured by Irad Gage. Data are shown in Table 8.3. The accuracy quoted by the manufacturer is ±0.5% full scale. During subsequent discussions, a representative of the manufacturer (LeFrancois, 1986) indicated his personal experience that evaluations of data should not be made on the basis of readings from a single instrument:

> Advanced manufacturing techniques greatly minimize drift, but no manufacturer can ever guarantee that every unit built will not drift beyond the accuracy specified over an indefinite period of time. The remedy is, whenever the piezometer is retrievable, to perform periodic verification of the zero and calibration. Otherwise the accuracy is achieved by an average from a group of piezometers.

As can be seen from Table 8.3, 5 of the 16 piezometers show a change in zero reading greater than the quoted accuracy, but the average change (−0.27 lb/in.$^2$) is within the quoted accuracy (±0.5 lb/in.$^2$).

The author is aware of two unpublished cases in which vibrating wire transducers have apparently experienced significant zero drift. They both involve measurements with piezometers, supplied by two different manufacturers, in two embankment dams. During first filling of each reservoir, one vibrating wire piezometer indicated a high piezometric level that caused concern, and filling was stopped. The piezometer reading continued to rise and, when the indicated piezometric level rose above pool level, the measurements were discounted and filling continued. It may be noted that an excessively high piezometer reading is consistent with wire slippage or creep. The fact that several recent versions of vibrating wire piezometers are provided with an in-place check feature whereby zero drift can be checked at any time during the life of the instrument (Chapter 9) appears to support the contention that the potential for zero drift should not be ignored.

**Table 8.3. Changes in Zero Reading of Vibrating Wire Piezometers During 17 Month Period**

| Instrument | Change in Zero Reading | |
|---|---|---|
| Number | lb/sq.in.[a] | % full scale[b] |
| 10–603 | −2.58 | 2.58 |
| 10–606 | +0.65 | 0.65 |
| 10–607 | −0.16 | 0.16 |
| 10–655 | +0.09 | 0.09 |
| 10–663 | +0.23 | 0.23 |
| 10–666 | −0.33 | 0.33 |
| 10–667 | −1.12 | 1.12 |
| 10–668 | −1.15 | 1.15 |
| 10–669 | −0.36 | 0.36 |
| 10–670 | +0.53 | 0.53 |
| 10–671 | −0.19 | 0.19 |
| 10–685 | −0.06 | 0.06 |
| 10–686 | +0.09 | 0.09 |
| 10–689 | +0.08 | 0.08 |
| 10–697 | −0.49 | 0.49 |
| 10–699 | +0.49 | 0.49 |

[a] The negative sign indicates an increase in frequency of vibration.
[b] Range of piezometers: 100 lb/in.$^2$
*Source:* Tharp (1986).

Zero drift can be minimized by stress relieving the vibrating wire, clamps, and transducer body after they have been assembled, either by using high temperature or by load cycling, and most manufacturers insist that this is essential for long-term stability. On the other hand, some users report excellent results when this precaution was not taken. Wire attachment at the clamping points should not weaken the wire, and squeezed capillary tube clamps or swaged pins appear to be the preferred methods. Bordes and Debreuille (1985) recommend that wire tension should be within the range 9–13% of yield strength, and comment:

> Experience shows it is desirable for the instruments to undergo a period of aging after manufacture, to provide time for strain hardening of the components and attachment points and for the relaxation of internal stresses that are inevitably set up in the

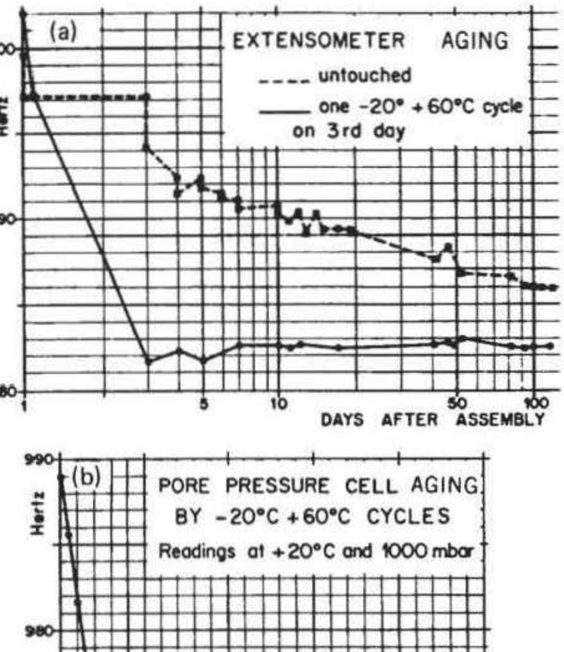



**Figure 8.23. Response of vibrating wire transducers to thermal cycling (after Bordes and Debreuille, 1985).**

> manufacturing process. The aging process can be accelerated by high temperature or a shaking table, although the two methods are not equally effective. [High] temperature accelerates the aging process.

Figure 8.23 illustrates the reactions of two Telemac vibrating wire transducers to thermal cycling. The upper figure shows readings of two embedment strain gages (the writers use the term *extensometer* for an embedment strain gage). The data indicate that readings stabilized immediately after a single thermal cycle (variations less than 1 hertz result from temperature and atmospheric pressure changes). The lower figure shows readings of a piezometer subjected to thermal cycles, indicating changes up to the 14th cycle. The writers make a distinction between the relatively simple strain gage arrangement, in which the only factors involved are changes in the wire itself and in its clamping arrangements, and the more complex transducer geometry of a wire attached to a diaphragm. In the second case, changes can occur at the point of attachment to the diaphragm and in the diaphragm itself, requiring more thermal cycles before stability is reached.

If a vibrating wire transducer is designed such that the electrical coil can be attached **after** the vibrating wire, clamps, and transducer body are assembled (e.g., the transducers manufactured by Geokon), the body can be stress relieved rapidly under high temperature, and no further aging is required.

In summary, on the issue of zero drift, four measures appear to be highly desirable: minimizing the potential for corrosion, use of appropriate wire attachment procedures at the clamping points, adherence to maximum tension limits, and appropriate aging by thermal and/or strain cycling. These measures are not adopted by all manufacturers. The author therefore does not support the viewpoint that **all** vibrating wire transducers are suitable for long-term applications. This opinion should not be taken as a vote in favor of alternative electrical transducers: when compared with other electrical transducers, a high-quality vibrating wire transducer is often the transducer of choice for long-term applications. However, the author favors the inherently more simple and reliable optical, mechanical, hydraulic, or pneumatic transducers for long-term measurements wherever they can provide the required data. There is a need for further documentation of long-term stability characteristics of vibrating wire transducers, both in the laboratory and in the field, and users are encouraged to examine the potential for zero drift and report their findings to the profession.

### *Applications when Transducers Are Subjected to Vibration*

If transducers will be subjected to vibration, the user should ensure that zero drift will not be caused. O'Rourke and Cording (1975) indicate that impact in the vicinity of surface-mounted strain gages during laboratory testing caused large zero drift, but Elson and Reddaway (1980) report insignificant zero drift when gages were embedded in driven piles. The writers conclude:

> Generally, the first few blows of the hammer (up to 100) produced a slight permanent extension in the strain gauges (typically 10 to 20 microstrain). Thereafter, driving the pile had little effect on the readings. It is not clear whether the permanent ex-

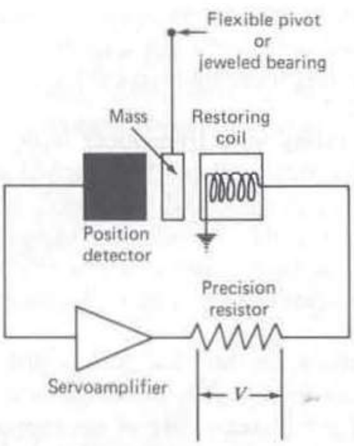



**Figure 8.24. Schematic of force balance accelerometer. Voltage *V* is proportional to the force required to hold the mass in the null position.**

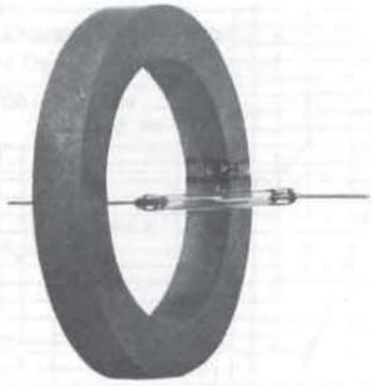


**Figure 8.25. Magnet/reed switch (after Burland et al., 1972). Reprinted by permission of Institution of Civil Engineers, London.**

tension recorded is a real or an apparent value, although it would be reasonable to suppose that the recorded extension reflected the development of microcracks in the pile.

Strain measurement on driven steel piles is best accomplished with low-mass weldable vibrating wire gages, with additional epoxy encapsulation for extra mechanical strength during driving. When transducers will be subjected to vibration, the user is advised to contact manufacturers whose transducers are being considered for names of others who have used their transducers in similar applications. If in doubt, tests should be conducted to check transducer performance under conditions similar to those anticipated in the field.

### 8.4.10. Force Balance Accelerometer

Force balance accelerometers are used as tilt sensors in tiltmeters, inclinometers, and in-place inclinometers.

The device consists of a mass suspended in the magnetic field of a position detector (Figure 8.24). When the mass is subjected to a gravity force along its sensitive axis, it tries to move, and the motion induces a current change in the position detector. This current change is fed back through a servoamplifier to a restoring coil, which imparts an electromagnetic force to the mass that is equal and opposite to the initiating gravity force. The mass is thus held in balance and does not move. The current through the restoring coil is measured by the voltage across a precision resistor. This voltage is directly proportional to the input force.

The simplest and least expensive manual data acquisition system is a portable digital voltmeter. Automatic data acquisition systems for use with inclinometers are described in Chapter 12.

Force balance accelerometers have a good track record when used in portable tiltmeters and inclinometers where the transducer can be reversed to eliminate errors caused by zero shift. However, they are not yet proved for long-term measurements requiring permanent embedment of the transducer, although recent design improvements have increased their longevity.

### 8.4.11. Magnet/Reed Switch

The magnet/reed switch system (Burland et al., 1972) is used in probe extensometers.

It is an on/off position detector, arranged to indicate when the reed switch is in a certain position with respect to a ring magnet, as shown in Figure 8.25. The switch contacts are normally open and one of the reeds must be magnetically susceptible. When the switch enters a sufficiently strong magnetic field, the reed contacts snap closed and remain closed as long as they stay in the magnetic field. The closed contacts actuate a buzzer or indicator light in a portable readout unit.

The repeatability of closure depends on the radial position of the reed switch within the ring magnet and also on the orientation of the reed about its own axis, because it is difficult to manufacture ring magnets that are polarized uniformly. If the reed switch remains within the middle third of a 1.25 in. (32 mm) inside diameter ring magnet, the repeatability will be within about ±0.01 in. (±0.25 mm). However, tests have shown (Burland et al., 1972) that with very precise guidance repeatability can be improved to about ±0.001 in. (±0.025 mm). Moreover, no detectable drift has been observed over a period of 12 months in the laboratory. The system is simple, reliable, precise, inexpensive, and well suited for long-term performance measurements. The only hazard known to the author is the possibility of permanent loss of magnetism if magnets are allowed in prolonged contact with each other prior to installation.

### 8.4.12. Induction Coil Transducers

Induction coil transducers are used in probe extensometers, fixed embankment extensometers, fixed borehole extensometers, and crack gages.

If an electrical coil is powered by an AC source, a magnetic field is created around the coil. This coil is referred to as the *primary coil*. If a second coil is within the influence of the magnetic field, a voltage is induced in this *secondary coil*. The principle is termed *inductive coupling*. The inductive coupling principle is used in three transducers for geotechnical applications. In all cases the two coils are separated both electrically and mechanically. The difference is in output and configuration.

#### *Soil Strain Gage*

In the first transducer, the *soil strain gage* (Section 12.6.5), the magnitude of voltage induced in the secondary coil is a function of spacing between the two coils. Thus, by maintaining constant input voltage and measuring induced voltage, coil separation can be determined. The readout unit contains a Wheatstone bridge circuit to null out the voltage from the secondary coil. Coil spacing is calculated from the amount of adjustment needed to balance the bridge.

#### *Transducer with Current-Displacement Induction Coil*

The second transducer is used in probe extensometers (Section 12.5.6). The secondary coil is a single steel wire ring, with no external electrical connection. When the primary coil is placed inside the ring, a voltage is induced in the ring, which in turn alters the current in the primary coil because its inductance changes. The current in the primary coil is a maximum when the primary coil is centered inside the ring; thus, by measuring primary coil current the transducer can be used as a proximity sensor. The readout unit contains an ammeter for current indication. In some versions of the probe extensometer, the secondary coil is a steel plate with a central hole instead of a single steel wire ring.

#### *Transducer with Frequency-Displacement Induction Coil*

The third transducer functions as a linear displacement gage with frequency output and is used in crack gages (Section 12.3.2), probe extensometers (Section 12.5.6), fixed embankment extensometers (Section 12.6.4), and fixed borehole extensometers (Section 12.7.4). The secondary coil is a nonferrous ring, with no external electrical connection. The primary coil includes a coil-capacitance resonant frequency circuit. As the primary coil moves relative to the secondary coil, the mutual inductance between the two changes. As the mutual inductance changes so does the resonant frequency of the circuit. Frequency can be measured and displayed automatically by digital electronics or by a manually operated frequency-sensitive bridge circuit, and displacement is determined from a frequency/displacement calibration.

Induction coil transducers have excellent long-term stability. For long-term applications any embedded steel components should be protected from corrosion.

### 8.4.13. Magnetostrictive Transducer

The magnetostrictive transducer (Hawkes, 1978) is used in fixed borehole extensometers, probe extensometers, and tube convergence gages.

The transducer is also referred to as a *sonic probe*. It consists of at least two permanent magnets attached to points between which displacement is to be measured. A nickel–iron alloy tube is placed alongside the magnets and a copper wire is threaded inside the tube, as shown in Figure 8.26. The nickel–iron alloy experiences a physical distortion when subjected to a change in magnetization, thus the term *magnetostrictive*. An instantaneous electrical pulse is applied to the copper wire, thereby inducing an instantaneous magnetic field along its length. The magnetic field interacts with the magnetic field at each of the permanent magnets and causes a strain pulse to be initiated in the nickel–iron tube. These strain pulses travel along the tube

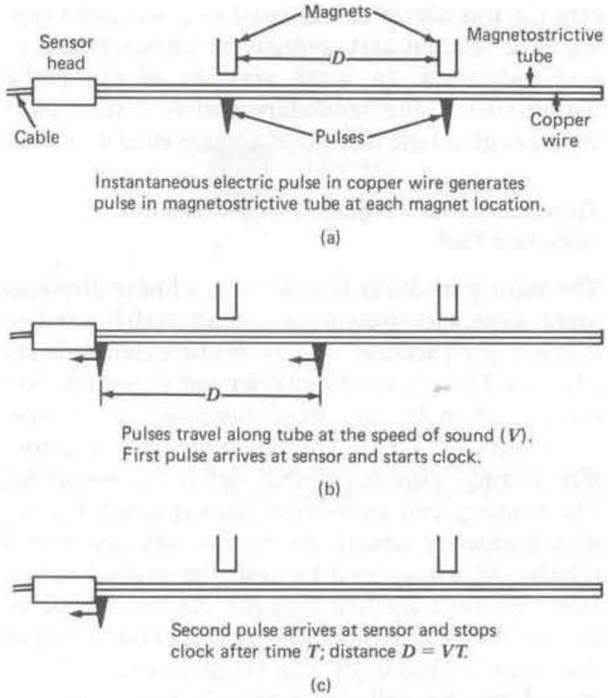



**Figure 8.26. Schematic of magnetostrictive transducer.**

at the speed of sound and are detected at the readout end of the tube. The first pulse, from the nearest magnet, arrives at the sensor head and starts a quartz crystal clock. The second pulse, from the next nearest magnet, stops the clock. Given the time between pulses and the velocity of pulse travel, the distance between adjacent magnets is determined readily.

Transducer accuracy is $\pm 0.001$ in. ($\pm 0.025$ mm) and the range can be as much as several feet. The transducer is noncontacting and has low hysteresis and high stability. Temperature effects are minimal, because the time measurement depends only on the pulse travel velocity and the distance between magnets. The nickel–iron alloy tube can expand or contract without affecting the time measurement. The time measurement is affected by change in the pulse travel velocity with temperature, but materials have been selected such that the error is negligible. The magnetostrictive transducer was not used in geotechnical applications until 1977; therefore, its longevity has not yet been proved. However, performance to date has been good and there appear to be no basic reasons for questioning longevity if access remains available to the sensor head for any necessary maintenance.

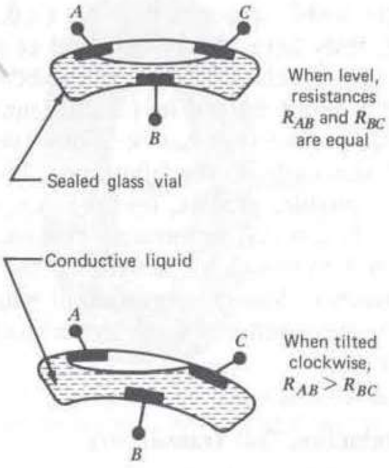



**Figure 8.27. Schematic of electrolytic level.**

### 8.4.14. Electrolytic Level

Electrolytic levels are used in tiltmeters and inclinometers.

An electrolytic level (Figure 8.27) consists of a sealed glass vial similar to the vial on a conventional builder's level, partly filled with a conductive liquid. Output resistance can be read with a portable strain indicator, in which case the two resistances form two arms of a Wheatstone bridge circuit, or the device can be read in the same way as a linear potentiometer. Criteria for the design of accurate electrolytic levels are discussed in Section 12.4.

### 8.4.15. Automatic Data Acquisition Systems for Electrical Transducers*

Advantages and limitations of automatic data acquisition systems, when contrasted with manual systems, are discussed in Chapter 18. This section is confined to a description of the equipment and a discussion of the applicability of automatic data acquisition systems for monitoring various transducers.

*Description of Equipment*

An automatic data acquisition system (ADAS) may consist of simple components or may be a complex computer system. All ADASs have certain common characteristics. First, the systems are programmed

*Written with the assistance of David A. Roberts, Senior Engineer, Shannon & Wilson, Inc., Seattle, WA.

to collect data automatically on a predetermined schedule, without human intervention. Second, the systems are designed to accommodate more than one transducer. Third, some type of signal conditioning is performed. Fourth, the data are either recorded or retransmitted to other equipment for recording.

The heart of an ADAS is either a data logger, data controller, or small computer. With the advancement of electronics, the differences between these three devices have blurred. The data logger is an instrument designed to record information from various transducers, usually at high speed. The data controller not only collects data but also uses a program to evaluate the data and to control various equipment such as an industrial process assembly line. The small mini, micro, or personal computer will generally be incorporated into the ADAS through the use of an interface device and will not have been designed for this particular application. Computers usually allow more sophisticated immediate data processing than the other two devices, but the most frequently used ADAS for geotechnical instrumentation applications is the data logger.

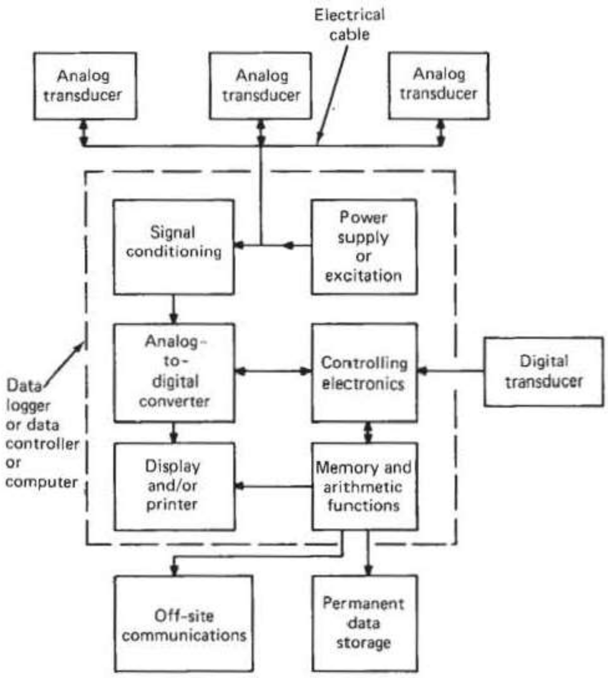



**Figure 8.28. Generalized block diagram of automatic data acquisition system (ADAS).**

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

Figure 8.28 shows the basic configuration of an ADAS. The power supply and signal conditioning convert the output of analog transducers into a signal that can be measured and converted to a number by an analog-to-digital converter. In most ADASs, the signals from the transducers are conditioned to produce a DC voltage or a binary decimal signal. The electronics controlling the ADAS performs various functions such as controlling measurement frequency, scaling and displaying the data, converting to engineering units, averaging data, checking for alarm limits, and controlling external equipment such as alarms and annunciators. The controlling electronics have some memory but data must then be stored or printed out for further analysis. The controlling electronics usually have the capability to store the data on a variety of media including paper, magnetic tape, and in some cases magnetic disk. Some ADASs also can be polled to collect data or to transmit data over the telephone to a separate processing center.

Recently, several manufacturers of geotechnical instrumentation have begun offering data loggers for field use, which are claimed to withstand extremes of temperature and humidity and yet are flexible enough to handle more than one or two types of transducer. The continuing improvement in low-power electronics permits these units to be programmed for unattended, battery operation from a month to up to a year. Figure 8.29 shows a data logger manufactured by Terrascience Systems Ltd., which is designed for installation within a protective enclosure at the collar of a borehole. A larger data logger is shown in Figure 8.30.

Slope Indicator Company has recently developed a line of "smart" vibrating wire transducers (Figure 8.31), the IDA™ (*intelligent data acquisition*) system, which includes additional electronics integral to the transducer package. The main pur-


Figure 8.29. In-borehole data logger. Terrascience Systems Model TERRA 8/D™ (courtesy of Terrascience Systems Ltd., Vancouver, BC, Canada).

poses of these "smart" transducers are to simplify on-site connection of large numbers of transducers, simplify data acquisition, and reduce cabling costs on large projects. This line of transducers is new and therefore no information is yet available on field reliability.

### *Applicability of ADASs for Monitoring Various Transducers*

The primary considerations for transducer selection should be reliability and required accuracy. If an ADAS is required, these primary considerations

Figure 8.30. Computer-based data logger for 1000 instruments, Waste Isolation Pilot Plant, Carlsbad, NM (courtesy of Soil & Rock Instrumentation Division, Goldberg-Zoino & Associates, Inc., Newton, MA).

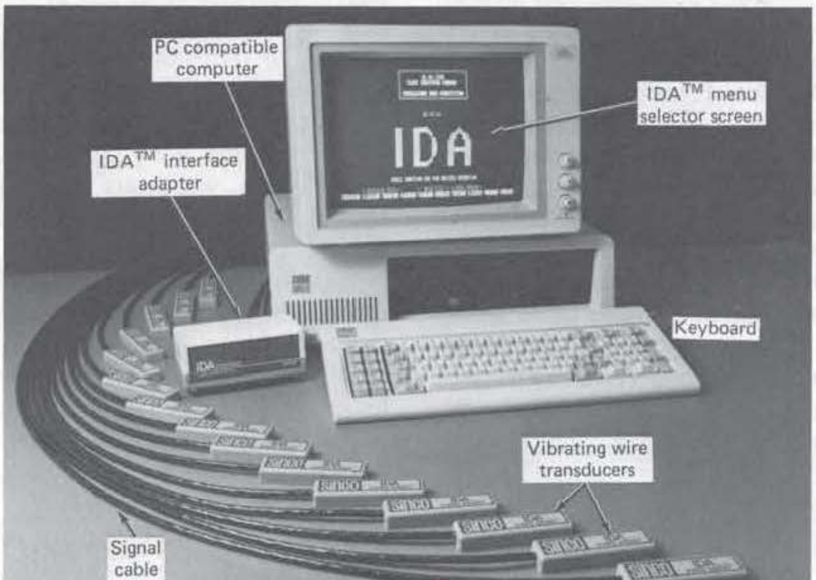



Figure 8.31. Slope Indicator Company IDA™ system (courtesy of Slope Indicator Company, Seattle, WA).

should override considerations of easy interfacing, and significant effort may be needed to design, build, and test interfacing between the preferred transducer and existing ADAS equipment.

A single model of data logger, data controller, or computer is not likely to be compatible with all transducers that are normally used in geotechnical instrumentation. If the transducers are supplied by a single manufacturer, and the same manufacturer sells a compatible dedicated data logger, this is likely to be the first choice. The following discussion is generally limited to flexible data loggers, data controllers, and computers. Applicability of ADASs for monitoring various transducers can be grouped into four categories, discussed in turn below.

Use of an ADAS is straightforward with some transducers. Included in this category are transducers that produce a full-scale DC output voltage of 1 volt or higher because these require minimum interfacing and signal conditioning. Examples are LVDTs, DCDTs, potentiometers, force balance accelerometers, electrolytic levels, high-output capacitance pressure transducers, and high-output electrical resistance strain gage networks. High-output electrical resistance strain gage networks are a relatively new development in which amplifying and signal conditioning electronics are built into the transducer, producing a full-scale DC output voltage of approximately 5 volts. Thermocouples, thermistors, and resistance temperature devices (RTDs), described in Chapter 14 and used for temperature measurement, require only minor interfacing. However, as discussed in Chapter 14, a different method is used for each of the three transducers.

The second category of transducers requires more complex signal conditioning. Examples include all five of the low-output electrical resistance strain gages described in Section 8.4.1, the type of signal conditioning depending on the bridge network. Some ADASs can provide this conditioning in the form of plug-in modules. Alternatively, standalone electronic interface equipment can be purchased to produce a high-level output voltage suitable for most commercially available ADASs. Other transducers in this second category include vibrating wire, magnetostrictive, and induction coil transducers of both the soil strain gage type and the type with frequency output, all of which require either special excitation or signal conditioning when used with flexible data loggers, data controllers, or computers. Vibrating wire transducers require an ADAS that is able to convert frequency to a digital signal. Soil strain gages require an adequate excitation voltage and an ADAS that is sufficiently sensitive to resolve small voltage changes resulting from changes in coil separation. Magnetostrictive transducers rely on the measurement of pulse travel time; therefore, an interface is required for connection to an ADAS, and at the present time there is no flexible data logger, data controller, or computer that could be connected directly to these transducers. For all transducers in this second category, a dedicated data logger is the best choice, preferably supplied by the manufacturer of the transducer.

The third category consists of electrical transducers that require manual operation, normally by traversing a sensing probe through a pipe, and thus there is little reason to use an ADAS. Examples are magnet/reed switch and induction coil transducers used with probe extensometers.

Fourth, several non electrical transducers can be read with ADASs, using arrangements described elsewhere in this book. These include pneumatic transducers (Section 8.3), plumb lines and double-fluid full-profile settlement gages (Chapter 12), and twin-tube hydraulic piezometers (Appendix E).

Publications describing geotechnical applications for automatic data acquisition systems include Bailey (1980), Brough and Patrick (1982), Carpenter (1984b), Carpentier and Verdonck (1986), Chedsey and Dorey (1983), DiBiagio (1979), DiBiagio et al. (1981), Green and Roberts (1983a, 1983b), ICOLD (1982), Lytle (1982), Murray (1986), Steenfelt (1983), USCOLD (1988) and Weeks and Starzewski (1986).

### 8.4.16. Power Supplies and Communication Systems for Electrical Transducers*

Electrical transducers generally require an electric power supply, either to a portable readout unit, an ADAS, or in some cases directly to the transducer. Options are mains power, batteries, portable generators, and solar, wind, and water power. Electrical instruments also require a communication linkage between the transducer and data acquisition system and in some cases between the data acquisition system and a remote facility. Options are hard-wiring, telephone lines, and radio transmission.

*Written with the assistance of David A. Roberts, Senior Engineer, Shannon & Wilson, Inc., Seattle, WA.

*Overview of Electrical Power Supplies*

The primary sources of electrical power are mains power and battery power. Mains power in the United States is 110 volts or 220 volts AC, 60 hertz single phase, and is usually available only near inhabited areas. Some other countries have 240 volts AC, 50 hertz single phase as a standard. Battery power is used in most portable instruments, and batteries may be disposable or rechargeable. Alternative power supplies, used when batteries cannot provide sufficient power economically and when the site is too remote for mains power, include diesel or gasoline generators, thermoelectric generators, and solar, wind, and water power. A good overview of some electrical power supply systems is given by Ball (1987).

*Mains Power*

Mains power is preferred for automatic data acquisition systems. Many components of an ADAS require 110 or 220 volts AC, and control of environmental conditions such as temperature and humidity is simplified when mains power is used. The quality and reliability of mains power varies considerably, and conditioning may be necessary where quality is poor. Low voltage, commonly called *brownout*, can cause erratic behavior and can damage electronic equipment. Overvoltage caused by lightning strikes can destroy unprotected equipment, and high-voltage transients can cause errors in data collected by ADASs. Undervoltage, overvoltage, and transients can be cured by a combination of lightning protection equipment (Section 8.4.17), circuit breakers, and power line conditioners. The problem of power outages can be addressed by use of uninterruptable power supplies: these are essentially switching devices that use rechargeable batteries to supply power during outages of mains power.

*Disposable Batteries*

Disposable batteries are used in portable readout units that have low power consumption and also in areas where rechargeable batteries would be difficult to service. Common types are carbon–zinc, mercury, and alkaline cells. Alternatively, air-depolarized cells can be used in remote areas where other power supplies are inadequate and where high capacity is required. These cells provide only 1.2 volts each but can be connected in series to provide the required voltage. They are available with capacities of up to 2000 ampere-hours and will operate at temperatures as low as −40°C. Their main disadvantages are high price, large bulk, and the caustic property of the electrolyte.

*Rechargeable Batteries*

Rechargeable batteries are of two general types, *nicad* and *lead–acid*. Nicad (nickel–cadmium) batteries are normally used where power requirements are small, physical space is limited, and repeated deep discharge cycles are anticipated. Lead-acid batteries are available with a wide variety of voltages and power capacities, but they are vulnerable to damage if fully discharged for a long period. Automobile batteries are lead–acid type. Low-capacity sealed lead–acid batteries that use a gelled electrolyte are now used widely in portable readout units. Shoup and Dutro (1985) describe various types of disposable and rechargeable batteries.

*Diesel and Gasoline Generators*

In remote areas where high-capacity power is required, diesel or gasoline electric generators are commonly used. Diesel units are generally preferred because maintenance is minimized and stored fuel is safer, but use of either type entails maintenance and refueling costs. Most diesel generators in the United States are designed to produce 110 or 220 volts AC, but convertors can be added to reduce the power output to a convenient DC voltage such as 12 volts.

*Solar Power*

Solar power is used with increasing frequency, but several variables must be considered before selecting this approach. Northern areas such as Canada and Alaska receive much smaller amounts of solar radiation than other parts of North America and have a wide variation in summer/winter radiation. When solar power is required in northern areas, either the system must be sized on winter needs, entailing significant overcapacity in summer, or additional batteries are required for winter use. In more southerly latitudes, similar needs can arise if a south-facing location is not available or if the solar panels are affected by shade. For high-capacity solar power, large arrays of panels must be used, and specialized batteries are required to smooth the variations in power supply.

### *Thermoelectric Generators*

In remote locations that are unsuitable for solar power and where power requirements are moderate, thermoelectric generators are sometimes used in preference to diesel or gasoline generators or batteries. These generators are powered by natural gas, propane, or butane, with either a flame burner or catalytic burner. Capacities range from 10 to 90 watts, and various output voltages can be selected. These units are relatively small, weigh between 30 and 200 lb (14–90 kg), have very low maintenance needs, but of course require periodic refueling.

### *Wind and Water Power*

Wind- and water-powered generators can also be used for power supply in remote areas. However, both methods are relatively expensive, require frequent maintenance, and are rarely used in geotechnical applications.

### *Ranges of Power Output for Various Electrical Power Supplies*

Table 8.4 presents a range of power output available from various electrical power supplies. The values cited in this table are approximate, and the ranges are those that are economically feasible for instrumentation purposes.

**Table 8.4. Typical Range of Power Output for Various Electrical Power Supplies**

| Power Supply | Power Output: Minimum (watts) | Power Output: Maximum (watts) |
|---|---|---|
| Diesel generator | 1000 | 50,000 |
| Solar panel[a] | 4 | 70 |
| Thermoelectric generator | 7 | 90 |
| Rechargeable battery[b] | 0.4 | 4 |
| Nonrechargeable battery[c] | $4 \times 10^{-4}$ | 3 |

[a] For a location in central United States. Normally more than one panel is required to obtain maximum power output.
[b] One month recharge cycle.
[c] Yearly replacement.
*Note:* All calculations based on a 12 volt DC system.

### *Communication Systems*

Electrical transducers are generally *hard-wired* to the data acquisition system, using electrical cable. Cable may be buried in the ground or concrete, or attached to the surface of a structure, or suspended in air. Guidelines for selection of cable are given in Section 8.4.17.

When an automatic data acquisition system is used, data can be collected from the system by visiting the site. Alternatively, a communication system can link the on-site system to an off-site master collection station as shown in Figure 8.32, so that data can be collected and processed without visiting the site. Communication between the on-site junction boxes and collection station will often be via hard-wiring, or occasionally radio transmission may be used when rugged terrain, roads, or other obstacles prevent the use of hard-wiring. Radio transmission between remote on-site stations and the on-site collection station is usually performed with low-power line-of-sight transmitters.

Communication between on-site and off-site stations will usually be via telephone lines or radio. When available, telephone lines provide the most convenient and least expensive link. Telephone lines are rented from local telephone companies and can be either dedicated or standard voice grade. Dedicated lines can handle higher rates of transmission but are more expensive and may not be available. Standard dial-up voice grade lines are the more common method and require a modem at each end of the line. Transmission rates for dial-up lines vary from 10 to 120 characters per second and depend both on the sophistication of the transmitting and receiving equipment used and on the quality of the telephone lines over which the signal is sent. Data transmission over telephone lines can be either analog or digital but the digital method, using an ASCII protocol, is generally the preferred method. Data transmission from the on-site station to an off-site station can be initiated by any of four methods: manually from the on-site station, timed automatically from the on-site station, polled manually from the off-site station, or polled automatically from the off-site station.

Where telephone lines are not available, and the site is too remote for installation of a private line to the nearest telephone line, radio transmission can be used to link the on-site and off-site stations, but radio transmission over more than a few miles requires use of a relay system. Relay systems can be simple single channel relays, radio networks, or satellites. It is likely that, in the future, satellite

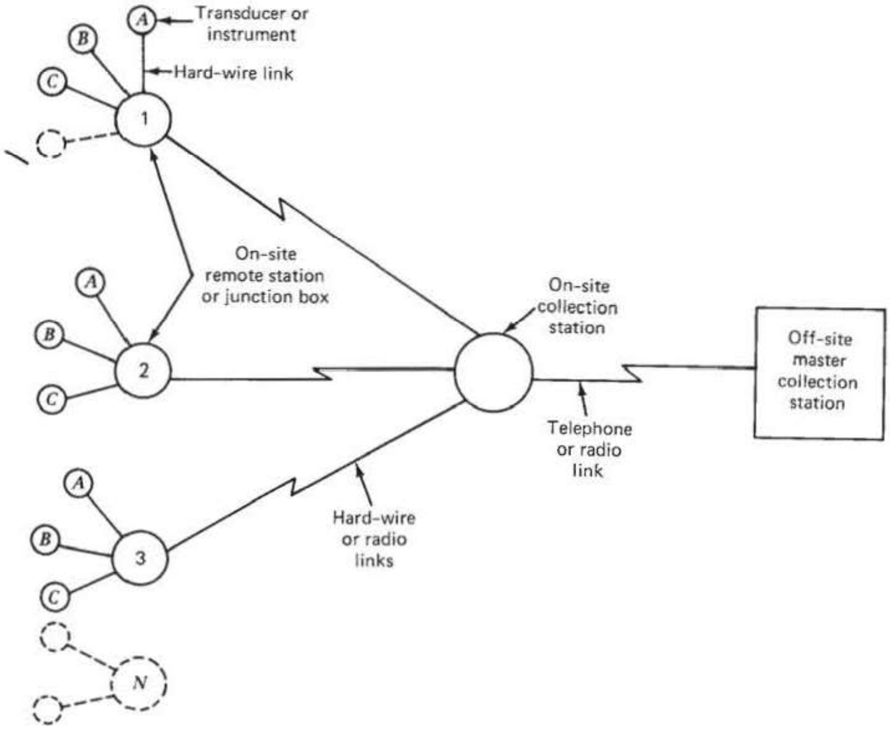



Figure 8.32. Schematic of transducers and communication system for remote monitoring.

links will play an increasing role in communication of data. As for telephone lines, radio transmission of data can be either analog or digital, but digital transmission usually has less errors. Radio frequencies, power, and protocols for transmitting data vary greatly among different systems and are usually regulated by government agencies. Baur (1987) presents a good overview of the role of radio transmission systems.

An innovative system is described by Teal (1986) for communicating data from inaccessible transducers to an on-site data collection station. The system uses a technique akin to that used in geophysical resistivity surveys. An alternating electrical potential field is established between a pair of buried electrodes and a digitally encoded signal is transmitted. This signal is received at the monitoring station by a second pair of electrodes, thereby avoiding hard-wiring. Arrangements are being made to install a prototype system in England.

Whenever data are transmitted without use of hard-wiring, there is a possibility of transmission error. The data should be transmitted more than once and any errors identified by comparing transmissions.

### 8.4.17. General Guidelines on Use of Electrical Transducers and Data Acquisition Systems*

This section includes only brief general guidelines. Most of the issues discussed require knowledge beyond that of the typical user of instrumentation. Unless users have sound experience with field electronics, they are encouraged to seek the detailed guidance of others with appropriate experience.

#### *Selection of Cable*

Manufacturers of geotechnical instruments normally standardize the type of cable for each instrument: to do otherwise would create difficulties in supply to the end user. However, the standard type may not be suitable for all applications, and the user should participate in selection of cable. The following items must all be considered:

*Written with the assistance of Howard B. Dutro, Vice President, Slope Indicator Company, Seattle, WA, and Charles T. McNeillie, Senior Instrumentation Technician, Soil & Rock Instrumentation Division, Goldberg-Zoino & Associates, Inc., Newton, MA.

- Length of run
- Frequency and magnitude of signal
- Environment (e.g., underground, aerial, indoor, marine sunlight)
- Temperature of environment
- Altitude
- Longevity requirements
- Susceptibility to damage (e.g., need for conduit or armoring, likelihood of axial or shear strains such as when embedded in fill)
- Proximity to sources of electrical noise

*Electrical noise* is a term used to cover random measurement variations caused by external factors, and excessive noise in a system may mask small real changes. It is therefore important that electrical signals should be as free as possible from noise. Primary causes of noise are radio frequency interference (RFI) and electromagnetic interference (EMI) from such sources as power lines, electrical generators and motors, commercial TV, radio or radar stations, defective fluorescent signs, electronic navigation systems, nearby thermostats and other switch closures, welding, and dirty terminals of power line transformers. Problems created by some of these sources can be corrected, for example by replacing defective fluorescent signs and cleaning terminals of power line transformers. The best protection against noise is the use of shielding.

Individually shielded twisted pairs, bundled inside an overall shield, are recommended for all applications. For some applications this may indeed be "overkill," but it is inexpensive insurance against the many unknowns associated with a field environment. As a rule of thumb, the shield should be grounded with respect to the measuring circuit at **one end** only, as close as possible to the chassis ground for a portable instrument and as close as possible to the earth ground on a mains instrument.

The primary sources of guidance on selection of cable are the manufacturers of cable, and several manufacturers who have applications engineers are listed in Appendix D. The end user should make the final decision, after discussions with manufacturers of the instrument and the cable. From a delivery and cost standpoint, it is always cost effective to use a standard stocked cable.

### *Cable Connectors*

The most suitable types of connector for field instrumentation are the *Mil. Spec.* (*MS*) bayonet lock type. MS connectors are available with from two to several hundred connecting pins, which are normally gold plated to provide long-term stable connections. The crimp versions are preferable to the soldered versions because, although they are more expensive, they provide the most reliable connection and are less dependent on variations in quality of work when connecting cables to connectors. The back shells of the connectors are environmentally sealed, and when properly mated by turning 90°, the male/female connection is also sealed. However, the connectors are **not** waterproof.

It is advisable to discourage water from wicking along the cable from a cut in the outer insulation and entering the connector. This is best done by sealing the back shell around the individual conductors where they fan out. The two-part epoxy sealers used in cable splice kits (see next subsection) are suitable, and alternatively room temperature curing silicone rubber (e.g. *RTV, room temperature vulcanize*) can be used. *Noncorrosive* RTV should be used*—the type available from electronic supply houses, not the type available from hardware stores. RTV should be applied and allowed to cure in layers not exceeding 0.25 in. (6 mm) thick.

It must be emphasized that the connection method described above will not create **waterproof** connections. Sealing materials will generally not bond to cable insulation materials such as Teflon™ and polyethylene. In extreme cases, such as underwater applications, it is necessary to use special marine connectors, but these are very expensive.

Connectors should be kept scrupulously clean and dry. A non-contact cleaner—a high pressure inert gas contained in a spray can—is best for cleaning.† Whenever connectors are not mated, they should be sealed by dust caps and each attached to the connector by a chain so that it will not be lost.

### *Cable Splices*

Cable splices should be reserved for repair work and should not be planned into a system unless there is no alternative. Commercial splice kits should be used, available from the manufacturers listed in Appendix D. The kits generally consist of crimp splices and a two-part epoxy sealer with a forming mold. It is essential to match the crimp splice size with the cable conductor size, and to use a rachet-type crimping tool that can be calibrated to

*For example, Dow Corning Type 3140 or 3141.
†For example, Chemtronics 70 p.s.i. cleaner.

the proper crimp size. Great attention to detail must be paid when making a splice, and the instructions provided with the kits assume significant knowledge. The guidance of a good electronics technician, such as a person who repairs TVs, should be sought.

"Homemade" splice kits, consisting of crimp splices, mastic, RTV, and electrical tape should not be used. They generally develop leakage to moisture over the long term and deteriorate more rapidly than the commercial kits. Unless made indoors by skilled personnel, soldered splices will generally be inferior to a high-quality crimp splice.

### *Lightning Protection*

Lightning can damage electrical instruments in three ways: power surges caused by a direct strike, induced transients caused by a nearby strike, and electromagnetic pulses induced by the magnetic field of a strike. *Transient protection devices* should be incorporated into the system whenever there is a possibility of serious damage by lightning. Commercial sources are listed in Appendix D. Both the power input and the transducer should be protected. Good sources of information on lightning protection include Baker (1978, 1980), Burkitt (1980), DiBiagio and Myrvoll (1985), and General Electric (1976).

### *Protection from Water*

If it can find a way to the inside of an electrical system, water will find a way. Methods of preventing water from entering cables, connectors, and splices have been described earlier in this section. *Water-blocked* cable is available to prevent migration of water along the conductors in case of damage to a cable.

Readout units should be kept dry, and a humidity indicator can be incorporated to indicate when there is a need to dry out the unit. The indicator contains a chemical that changes color when the humidity exceeds a predetermined level.

### *Field Checks with Volt–Ohm–Milliammeter*

Section 8.4.3 describes the use of a circuit tester to check the integrity of electrical resistance strain gages and cables. The integrity of cables connected to other types of instrument can be checked with a volt–ohm–milliammeter (VOM). The VOM should be the type used for testing radios and TVs, not the type used for power line measurements. Specific procedures appropriate for checking each instrument should be obtained from the manufacturer of the instrument.

### *Charging and Maintenance of Batteries*

Rechargeable batteries are of two general types, *nickel-cadmium* (*nicad*) and *lead-acid*. Recharging procedures for the two types are quite different. Nicad batteries must be discharged completely at maximum intervals of 3 months, whereas lead–acid batteries must never be discharged completely and should be fully charged at maximum intervals of 3 months when not being used. Shoup and Dutro (1985) give detailed guidelines on charging and maintenance of both types.

### *Operating Spares*

Components of an instrumentation system can malfunction as a result of mechanical damage, water damage, deterioration, defects, or wear and tear. Chapter 5 includes a recommendation for procurement of appropriate spare parts at the time of initial procurement, together with suggestions for obtaining spare or standby readout units. The stock of operating spares should be reviewed on a regular basis during service life.

### *Handling and Transporting Transducers and Data Acquisition Systems*

Instruments should be handled with great care. As an illustration of this need, when transporting instruments and personnel in a pickup truck, the instruments should be on the seat and the personnel in the back, not the other way around. A classic and recurrent blunder is to carry an instrument by its signal cable: this should never be permitted.

### *Regular Calibration and Maintenance of Readout Units*

General guidelines for calibration and maintenance of readout units are given in Chapter 16.

### *Use of Electrical Transducers and Data Acquisition Systems in Cold Conditions*

Guidelines for use of electrical instruments in cold conditions are given by Atkins (1981).

# CHAPTER 9

# MEASUREMENT OF GROUNDWATER PRESSURE

## 9.1. INSTRUMENT CATEGORIES AND APPLICATIONS

Definitions of the terms *groundwater level, pore water pressure,* and *joint water pressure* are given in Chapter 2. In this book the term *piezometer* is used to indicate a device that is sealed within the ground so that it responds only to groundwater pressure around itself and not to groundwater pressures at other elevations. Piezometers are used to monitor pore water pressure and joint water pressure. The term *pore pressure cell* is sometimes used as a synonym for *piezometer*. An *observation well* is a device that has no subsurface seals, and it creates a vertical connection between strata.

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

Applications for observation wells are very limited. In current practice they are frequently installed in boreholes during the site investigation phase of a project, ostensibly to define initial groundwater pressures and seasonal fluctuations. However, because observation wells create a vertical connection between strata, their **only** application is in continuously permeable ground in which groundwater pressure increases uniformly with depth. This condition can rarely be assumed. The author believes that many practitioners do not appreciate the need to make pore water pressure measurements at many different depths, rather than measuring at one or two points and assuming a straight line pressure–depth relation. Detailed pore water pressure data provided by recently developed multipoint piezometers with movable probes (Section 9.9) have shown that pressure–depth profiles can be quite irregular and very different from that which one would infer from measurements at one or two points. Perhaps the major reason for the frequent and continued use of observation wells is that they can be installed by drillers without the participation of geotechnical personnel: this is certainly not the case for installation of piezometers. Observation wells are therefore inexpensive, but in the view of the author are often misleading and should be used only when the groundwater regime is well known. If the groundwater regime is well known, measurements may be unnecessary: therefore, a practitioner must have a strong argument in favor of an observation well before selecting this option.

Additional applications for measurement of groundwater pressure are given in Part 5.

Piezometers can be grouped into those that have a diaphragm between the transducer and the pore or joint water and those that do not. Instruments in the first group are piezometers with pneumatic, vibrating wire, and electrical resistance strain gage transducers. Instruments in the second group are open standpipe and twin-tube hydraulic piezometers.

In general, piezometers used for measuring pore water pressure in soil are no different from piezometers used for measuring joint water pressure in a rock mass: the difference is in the installation arrangements. These differences are discussed in Section 9.18.

Various instruments for measuring groundwater pressure are described and compared below. Other descriptions and comparisons are given by Bozozuk (1960), Cording et al. (1975), Corps of Engineers (1971), Hanna (1985), USBR (1974), and Wilson and Mikkelsen (1978).

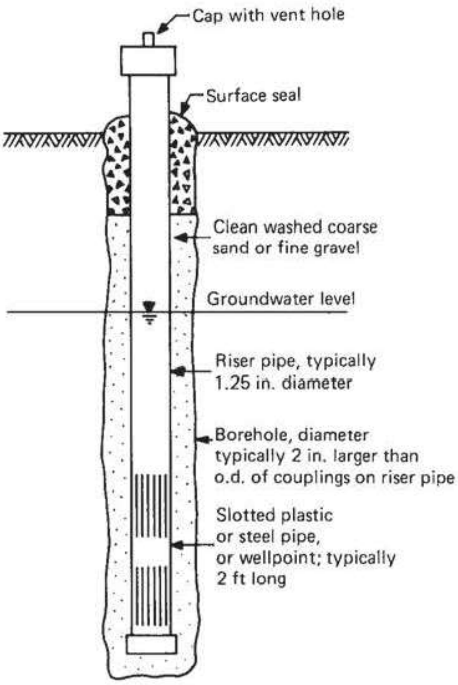



**Figure 9.1. Schematic of observation well.**

## 9.2. OBSERVATION WELLS

As shown in Figure 9.1, an observation well consists of a perforated section of pipe attached to a riser pipe, installed in a sand-filled borehole. The surface seal, with cement mortar or other material, is needed to prevent surface runoff from entering the borehole, and a vent is required in the pipe cap so that water is free to flow through the wellpoint. The elevation of the water surface in the observation well is determined by sounding with one of the probes described in Section 9.3.2 for measurements within open standpipe piezometers.

As discussed in Section 9.1, observation wells create an undesirable vertical connection between strata and should rarely be used. The water level within the observation well is likely to correspond to the head in the most permeable zone and will usually be misleading. At sites where a contaminant exists in one aquifer, installation of an observation well also leads to contamination of other aquifers.

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

## 9.3. OPEN STANDPIPE PIEZOMETERS

### 9.3.1. Description

An open standpipe piezometer requires sealing off a porous filter element so that the instrument responds only to groundwater pressure around the filter element and not to groundwater pressures at other elevations. Piezometers can be installed in fill, sealed in boreholes (Figure 9.2), or pushed or driven into place.

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

The open standpipe piezometer is also referred to as a *Casagrande piezometer,* after publication of measurement methods for monitoring pore water

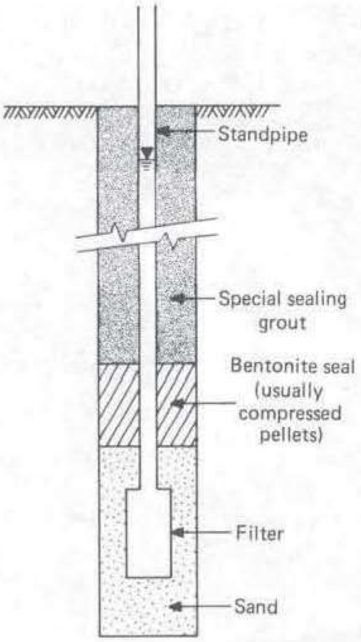



**Figure 9.2. Schematic of open standpipe piezometer installed in a borehole.**

pressure during construction of Logan Airport in Boston (Casagrande, 1949, 1958). The Casagrande version consisted of a cylindrical porous ceramic tube, connected with a rubber bushing to 0.375 in. (10 mm) inside diameter saran plastic tubing. Today, high-density porous hydrophilic polyethylene (Figure 9.3) usually replaces the brittle ceramic, and PVC or ABS plastic pipe or polyethylene tubing replaces the saran tubing, which becomes brittle with age and exposure to sunlight. Standpipe inside diameter typically ranges from 0.2 to 3 in. (5–76 mm).

Flush-coupled Schedule 80 PVC or ABS pipe is a good choice for standpipes, with either cemented or threaded couplings that allow easy passage of the reading device and the sounding hammer (Section 9.17.8). When cemented couplings are used, one end of each length of pipe is machined as a male, the other as a female, and the coupling connected using solvent cement. A primer should be used to etch the surfaces and ensure proper adhesion of the cement. When threaded couplings are used, they should be of the self-sealing type shown in Figure 9.3. These special threads are undersized at the female end and create a watertight connection without any sealer, merely by tightening with vice grips or small pipe wrenches. Conventional tapered pipe threads are

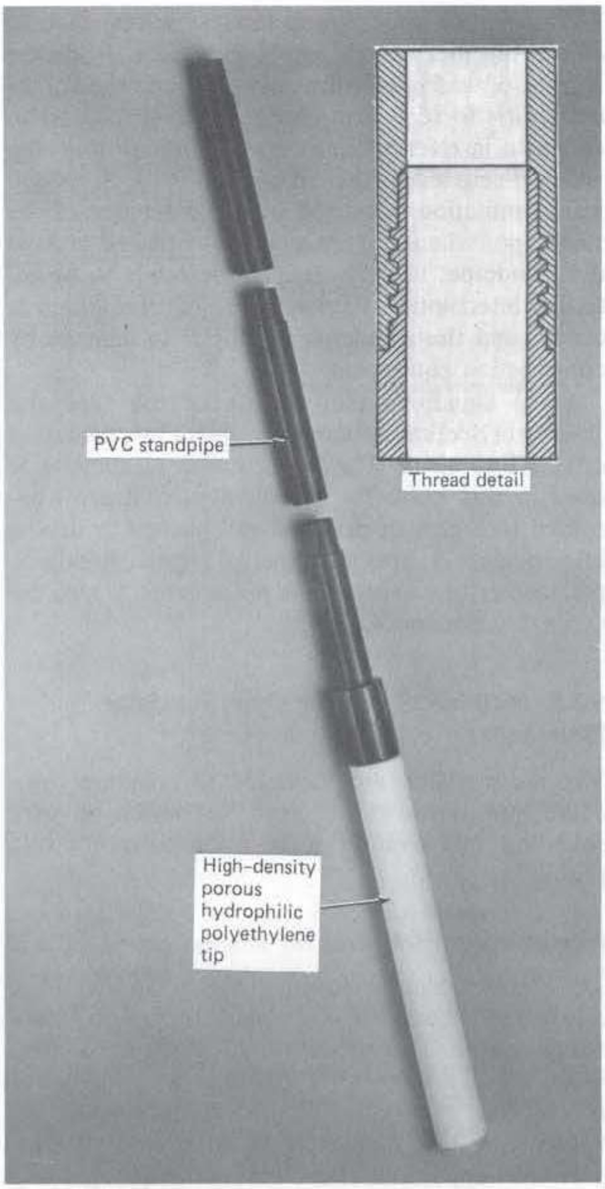



**Figure 9.3. Open standpipe piezometer with porous polyethylene filter and self-sealing threaded PVC standpipe (courtesy of Piezometer Research & Development, Bridgeport, CT).**

not suitable for flush couplings, and conventional square threads will often not sustain significant internal water pressure.

Open standpipe piezometers are generally considered to be more reliable than other types, and the reliability of unproven piezometers is usually evaluated on the basis of how well the results agree with those of adjacent open standpipe piezometers. Advantages and limitations are summarized in Section

9.12. A major limitation is their slow response to changes in piezometric head, because a significant volume of water must flow out of or into the soil or rock mass to register a change in head. This slow response is referred to as *hydrodynamic time lag* and is discussed further in Section 9.10. A second major limitation is caused by the existence of the standpipe: when embankment fill is placed around the standpipe, nearby compaction tends to be inferior, interruption to normal filling operations is costly, and the standpipe is subject to damage by construction equipment.

Open standpipe piezometers of the type described in Section 9.8 can be installed by pushing or driving into place. The term *push-in* piezometer is used in this book for piezometers that are connected to a pipe or drill rod and pushed or driven into place. A special type of open standpipe piezometer, the *heavy liquid piezometer,* is also described in Section 9.8.

### 9.3.2. Methods of Reading Open Standpipe Piezometers

Various methods are available for reading open standpipe piezometers, most of which involve sounding the elevation of the water surface with a probe.

#### *Electrical Dipmeter*

The most commonly used probe is an *electrical dipmeter* (Figure 9.4), consisting of a two-conductor cable with a cylindrical stainless steel weight at its lower end. The weight is divided electrically into two parts, with a plastic bushing between, and one conductor is connected to each part. The upper end of the cable is connected to a battery and either an indicator light, buzzer, or ammeter. When the probe is lowered within the standpipe and encounters the water surface, the electrical circuit is completed through the water and the surface indicator is actuated. For small-diameter standpipes where the water level is no deeper than about 15 ft (5 m), a coaxial cable with bared ends can be used.

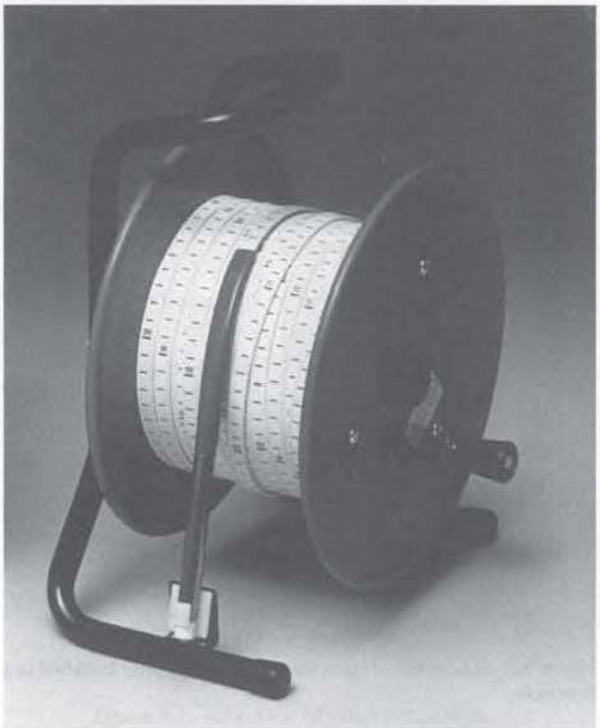


Figure 9.4. Electrical dipmeter (courtesy of Geotechnical Instruments (U.K.) Ltd., Leamington Spa, England).

#### *Capillary Reader*

An alternative device is manufactured by Piezometer Research and Development and shown in Figure 9.5. The term *capillary reader* originates from an earlier device that relied on the capillary action of liquid in a small-diameter tube. The air valve is held open to allow the two surfaces of colored liquid to stabilize at the same level, and the nylon tubing is pushed down the standpipe until the lower end is submerged below the water surface. Submergence is indicated by a lowering of liquid level in the graduated sight tube. The nylon tubing is then slowly withdrawn and the level of colored liquid rises until the surface of the water in the standpipe is reached, and at this point the graduation on the tubing is read. A 0.125 in. (3 mm) outside diameter tubing is normally used, such that it can be inserted in standpipes with inside diameter as small as 0.2 in. (5 mm): an electrical dipmeter cannot normally be used in such small-diameter standpipes. When used in a large-diameter standpipe a small weight is usually added to the lower end.

#### *Audio Reader*

Sandroni (1980) describes an audio reader, shown in Figure 9.6, that can be assembled from readily available components. The arrows on the figure show the path of a noise created, for example, by a transistor radio. The graduated measuring tube is

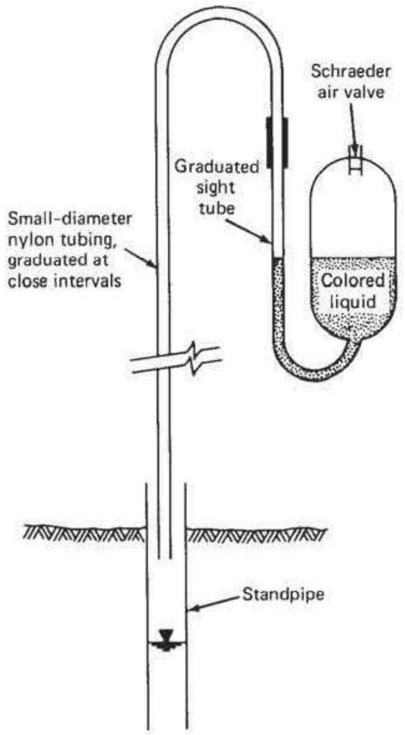



**Figure 9.5. Schematic of capillary reader.**

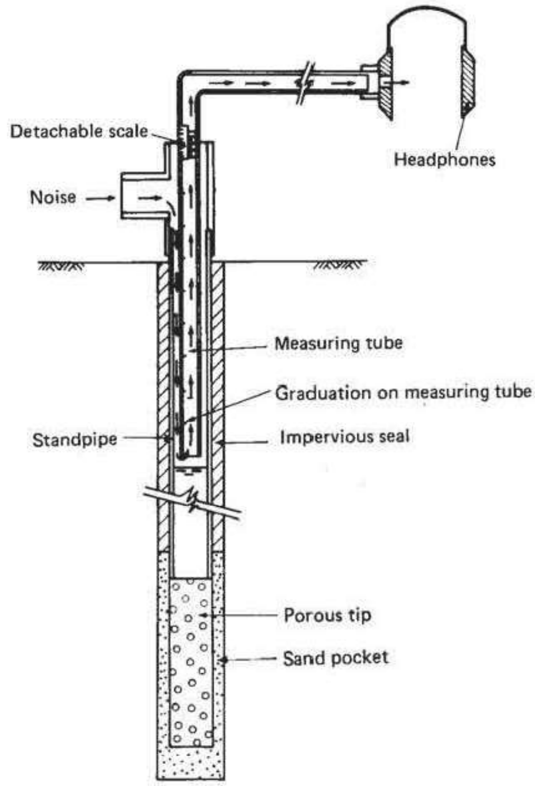



**Figure 9.6. Audio reader (after Sandroni, 1980). Reprinted by permission of Institution of Civil Engineers, London.**

inserted within the standpipe and, when the lower end of the tube touches water, the noise transmitted to the headphones ceases. The detachable scale is used to subdivide graduations on the measuring tube.

### *Survey Tape and Weight*

If the standpipe diameter is large enough, a survey tape with a weight on its lower end can often be used by listening for the weight to contact the water surface. A small bell, with the striker removed, is sometimes used to increase the noise on contact.

### *Pressure Transducer*

A pneumatic, vibrating wire, or electrical resistance strain gage pressure transducer can be inserted into the standpipe below the lowest possible piezometric level, thereby allowing readings to be made at a remote location. The transducer can be left hanging in place and recovered for periodic recalibration.

Alternatively, an open standpipe piezometer can be converted to a diaphragm piezometer by inserting a pressure transducer within the standpipe below the lowest possible piezometric level and sealing just above the transducer. A packer inflated with air can be used for the seal and allows recovery of the transducer for recalibration (e.g., Tao et al., 1980). This adaptation shortens the response time and reduces the possibility of clogging by repeated inflow and outflow of water through the porous tip.

### *Purge Bubble System*

The *purge bubble* principle can be used to read an open standpipe piezometer, as shown in Figure 9.7, and this system also allows readings to be made at a remote location. A length of plastic tubing is inserted within the standpipe to a point below the lowest piezometric level, and the elevation of the lower end is recorded. A small controlled flow of air depresses the level of water within the plastic tubing until it falls to the lower end of the tubing, at which time air bubbles rise to the surface of water in the standpipe. The measured air pressure is then equal to the water pressure at the lower end of the plastic tubing, and the piezometric elevation is cal-

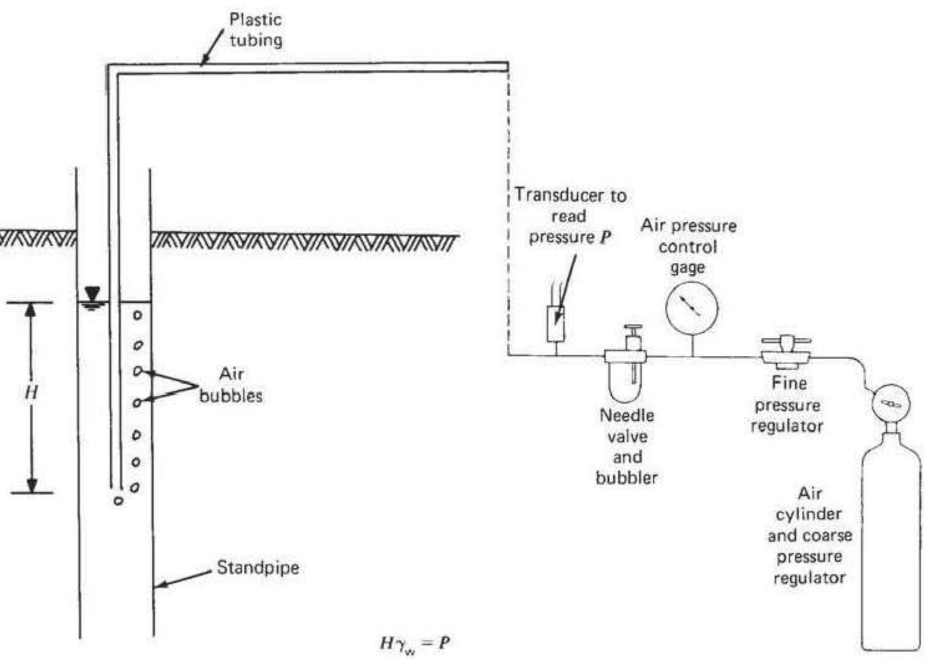



**Figure 9.7. Purge bubble principle (after Penman, 1982).**

culated as shown in the figure. Most readout units for pneumatic transducers can readily be adapted for this purpose.

The system shown in Figure 9.8 can be used for continuous recording by maintaining a constant small flow of air and recording pressure (e.g., Brand et al., 1983; Pope et al., 1982).

When the purge bubble system is used, the standpipe need not be vertical, provided that the elevation of the lower end of the plastic tubing is known. However, the system should not be used if the inside diameter of the standpipe is less than 0.3 in. (8 mm) because, as discussed in Section 9.3.3, the air bubbles may not rise to the top of the standpipe, resulting in a false reading.

### *Halcrow Bucket*

Brand et al. (1983) describe the *Halcrow bucket* for recording peak water levels in standpipes. A chain of plastic cylinders, each with an intake hole, is fixed along a weighted nylon string at selected depth intervals above the normal base water level in the standpipe. When the string of buckets is withdrawn, the highest transient water level is indicated by the upper limit of water-filled buckets. This system is cheap and simple, and it can be used to provide important design information when expensive automatic monitoring systems cannot be justified.

### *Float and Recorder*

Open standpipe piezometers are sometimes read with an automatic recorder by inserting a float and counterweight arrangement in the standpipe. However, this arrangement requires a minimum standpipe diameter of about 3 in. (76 mm) and therefore has a large response time.

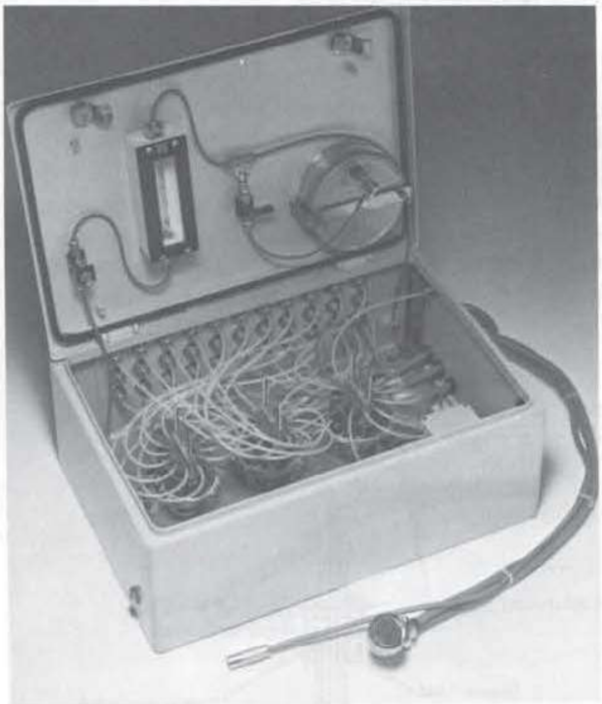
### *Bourdon Tube Pressure Gage*

If the piezometric level rises above the top of the standpipe, a Bourdon tube pressure gage can be attached.

## 9.3.3. Open Standpipe Piezometers in Unsaturated or Gaseous Soils

When an open standpipe piezometer is installed in unsaturated or gaseous soils, gas may enter the standpipe. If the inside diameter of the standpipe is less than about 0.3 in. (8 mm), gas bubbles may not rise to the top of the standpipe, and the water level in the standpipe will be elevated to give a false read-


(a)



(b)

**Figure 9.8. Continuous recording purge bubble system: (a) data logger and (b) scanner system used in conjunction with logger (courtesy of Geotechnical Instruments (U.K.) Ltd., Leamington Spa, England).**

ing. In such soils the inside diameter of the standpipe must therefore be greater than 0.3 in. (8 mm).

### 9.3.4. Open Standpipe Piezometers in Consolidating Soils

When an open standpipe piezometer is installed in consolidating soils, the standpipe may buckle or shear, creating a leak or obstructing passage of the readout probe. Examples are shown in Figure 17.10. When this potential exists, either a telescoping pipe should be installed around the standpipe or an alternative type of piezometer should be used. When it is necessary to prevent leakage of water along the annulus between the pipes, the annulus should be filled with a slurry.

### 9.3.5. Freezing of Water in Open Standpipe Piezometers

If the piezometric level rises above the frost line, the water in the standpipe will freeze and the piezometer will become inoperable. However, the upper portion of water in the standpipe can be replaced with an antifreeze mixture with specific gravity less than unity. A mixture containing 4 parts methanol, 2 parts glycerol, and 3 parts water by volume has been used (Soderman, 1961). The mixture has a specific gravity of approximately 0.99 at 26°C (79°F) and a freezing point of −51°C (−60°F). However, methanol can damage some plastics, and it is safer to use a thin oil such as hydraulic oil, and to make a correction to the piezometric level because of the lower specific gravity. In this case an electrical dipmeter will not function, and one of the alternative reading methods must be used. In freezing situations, an alternative approach is use of the *heavy liquid piezometer*, described in Section 9.8.

### 9.3.6. Checking Integrity of Seal

A falling or constant head permeability test can be made through the piezometer in an attempt to check the integrity of the seal. The test should be made only after allowing dissipation of any pore water pressure generated during installation, as indicated by a stable water level in the standpipe. However, Vaughan (1969) points out that these tests are only of limited value because a seal has to be extremely bad before it will be detected by this method. Despite this limitation, the test appears to be worthwhile as an acceptance test.

Equations for calculating permeability from falling and constant head tests are given by Hanna (1985) and Hvorslev (1951).

## 9.4. TWIN-TUBE HYDRAULIC PIEZOMETERS

The twin-tube hydraulic piezometer (Figure 9.9), sometimes referred to as a *closed hydraulic piezometer*, was developed for installation in the

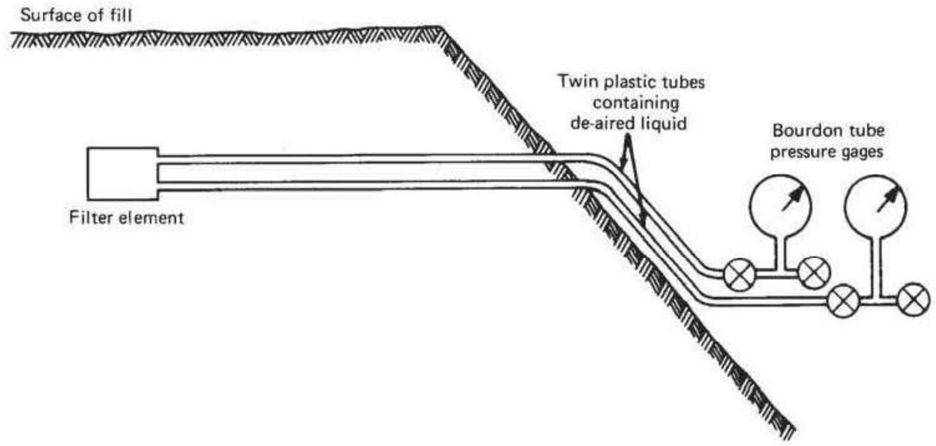



**Figure 9.9. Schematic of twin-tube hydraulic piezometer installed in fill.**

foundations and fill during construction of embankment dams. It consists of a porous filter element connected to two plastic tubes, with a Bourdon tube pressure gage on the end of each tube. U-tube manometers or electrical pressure transducers can be used instead of Bourdon tube pressure gages. The piezometric elevation is determined by adding the average pressure gage reading to the elevation of the pressure gages. If both plastic tubes are completely filled with liquid, both pressure gages will indicate the same pressure. However, if gas has entered the system through the filter, tubing, or fittings, the gas must be removed by flushing: this is the reason for requiring two tubes.

Figure 9.10 shows a method of converting an open standpipe piezometer to a twin-tube hydraulic piezometer. The standpipe is cut to a level such that the special head is protected by installing it slightly below the ground surface. The instrument is initially read by using the purge bubble principle. When the piezometric level rises to a level above the top of the standpipe, a valve at the upper end of the central tube can be closed permanently, and the instrument is read as a twin-tube hydraulic piezometer.

The application for twin-tube hydraulic piezometers is almost exclusively limited to long-term monitoring of pore water pressures in embankment dams, and therefore longevity is a primary need. Twin-tube hydraulic piezometer systems have been developed in the United States by the U.S. Bureau of Reclamation (Bartholomew et al., 1987; USBR, 1974) and in England by Imperial College, London (Bishop et al., 1960; Penman, 1960). The USBR piezometer tip is shown in Figure 9.11 and the Imperial College (Bishop) tip in Figures 9.12 and 9.13. Each system has been used widely in embankment dams throughout the world with very variable success. The system developed in England appears to have a better success record than the system developed in the United States, and successful long-term

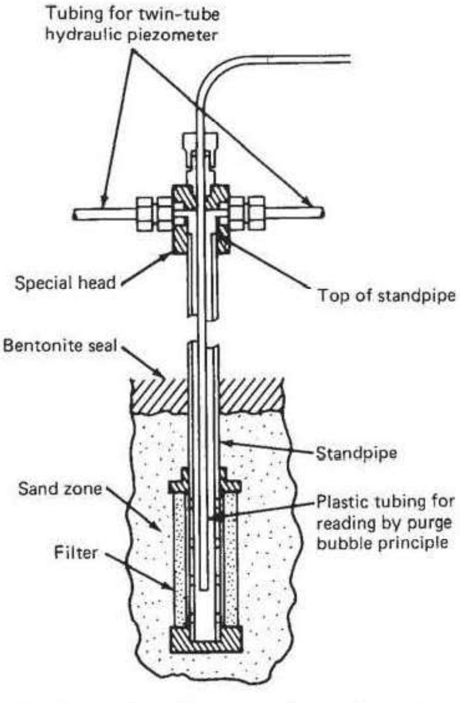



**Figure 9.10. Conversion of open standpipe piezometer to twin-tube hydraulic piezometer (after Penman, 1982).**

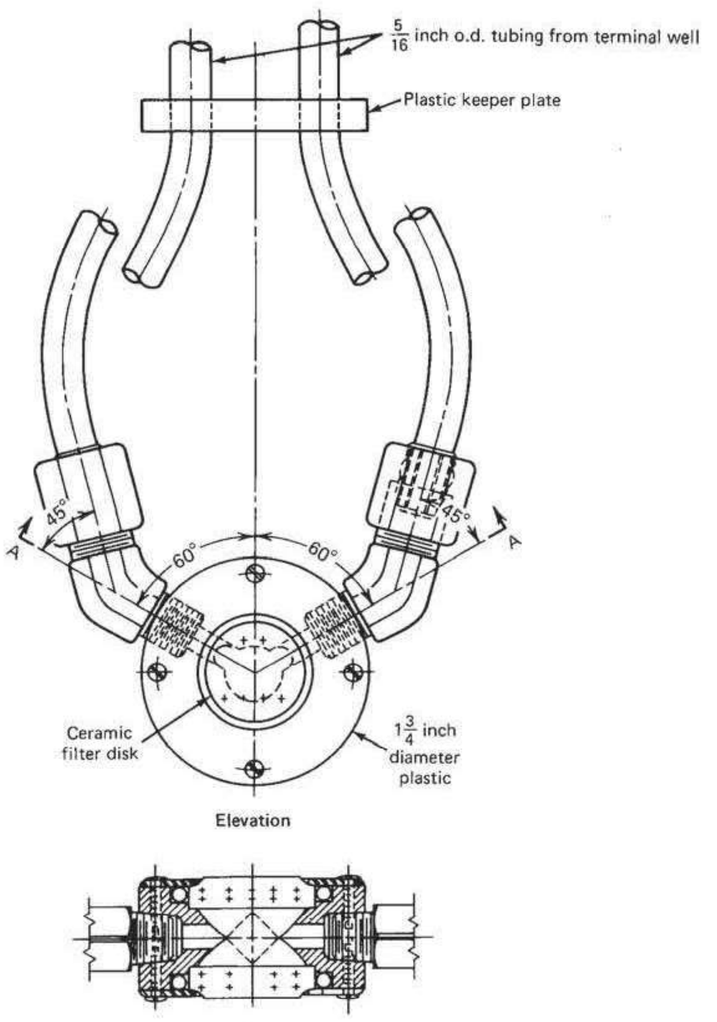



**Figure 9.11. USBR embankment-type twin-tube hydraulic piezometer (after USBR, 1974). Courtesy of Bureau of Reclamation.**

use of twin-tube hydraulic piezometers requires close adherence to many proven details. Some engineers believe that the twin-tube hydraulic piezometer should be superseded by diaphragm piezometers, but the author has a different view, and the arguments for and against both points of view are given in Chapter 21. Because the many proven details are not available in compact form in the literature, and because the author advocates the use of twin-tube hydraulic piezometers for long-term monitoring of pore water pressure in embankment dams, these details are given in Appendix E. Details relate to selection of components, method of installation, initial filling with liquid, and maintenance.

When compared with diaphragm piezometers, the major advantages of twin-tube hydraulic piezometers are the absence of inaccessible moving parts or electrical components and the capability of flushing the piezometer cavity. They are therefore very attractive for long-term monitoring. The system developed in England is reliable and has a long successful performance record. When installed in fill, the integrity of twin-tube hydraulic piezometers can be checked after installation by performing falling or constant head permeability tests. Automatic data acquisition systems are available, with a scanner allowing an electrical pressure transducer to be coupled hydraulically to each piezometer in turn.

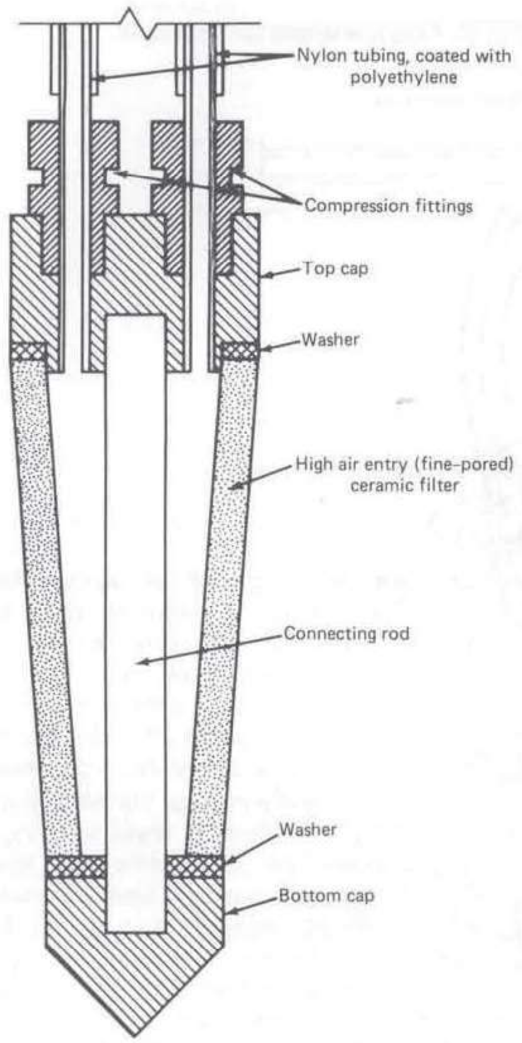



Figure 9.12. Bishop twin-tube hydraulic piezometer (after Penman, 1960).

Disadvantages include the need for a terminal enclosure to contain the readout and flushing arrangements; in addition, the enclosure must be protected from freezing either by heating or by constructing it below the frost line. Also, routing of tubing must be planned so that it does not rise significantly above the minimum piezometric elevation, otherwise the liquid will be required to sustain a subatmospheric pressure, and the liquid is likely to become discontinuous. However, if all components are selected to minimize gas entry, as described in Appendix E, and if high-quality de-aired water is used, the tubing can be installed up to 20 ft (6 m) above the minimum piezometric elevation.

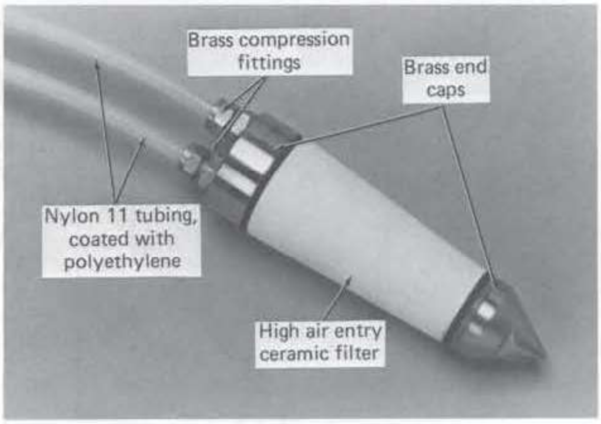



Figure 9.13. Bishop piezometer tip with attached tubing (courtesy of Soil Instruments Ltd., Uckfield, England).

Periodic flushing with de-aired water may be needed to remove any gas bubbles that have accumulated, and some engineers contend that this maintenance burden is unacceptable. However, the author believes that the maintenance burden has been exaggerated by improper selection of components, and flushing is seldom necessary in a properly designed and installed system when positive pore water pressures are being monitored.

## 9.5. PNEUMATIC PIEZOMETERS

The various types of *normally closed* and *normally open* pneumatic transducers and associated data acquisition systems are described and compared in Chapter 8. When these transducers are used for monitoring pore water pressure, a filter is added to separate the flexible diaphragm from the material in which the piezometer is installed (Warlam and Thomas, 1965). Figure 9.14 shows a schematic arrangement of a pneumatic piezometer, and a photograph of the version manufactured by Thor International, Inc. is shown in Figure 9.15. Similar piezometers, in which oil is used instead of gas, are available; however, they do not appear to offer any advantages over pneumatic piezometers.

Pneumatic piezometers can be packaged within push-in housings but are subject to the limitations discussed in Section 9.16. A special version of pneumatic piezometer, designed for pushing into place below the bottom of a borehole, is described in Section 9.8.

Some pneumatic piezometers can be used for

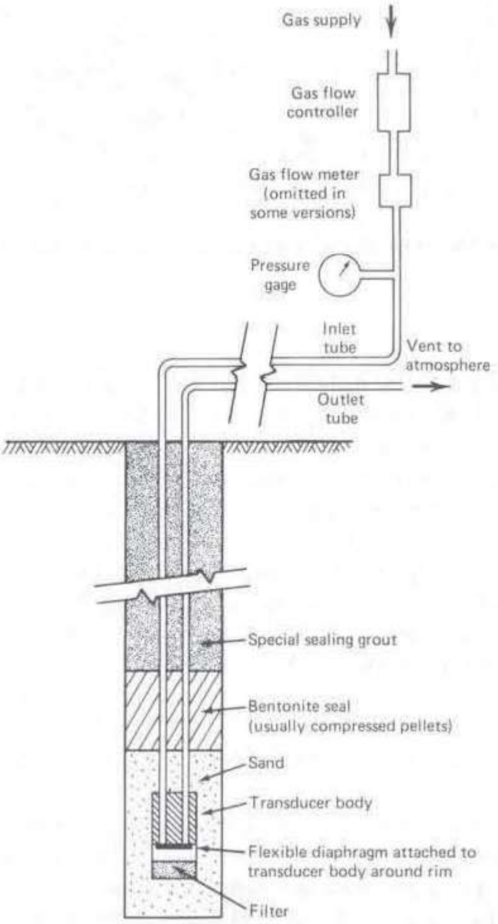



**Figure 9.14. Schematic of pneumatic piezometer installed in a borehole. (*Normally closed* transducer, with two tubes, read as gas is flowing.)**

monitoring negative (subatmospheric) pore water pressures by applying a vacuum to the outlet tube.

The *normally open* transducer is available from Terra Technology Corporation with one or two additional plastic tubes connected to the cavity between the filter and the flexible diaphragm. The intent of adding one tube is apparently to reduce the error caused by diaphragm displacement (Chapter 8) and also to allow unclogging a blocked filter by applying water pressure to this tube. If the piezometric level is below the end of the additional tube, and the end of the tube is capped, an air space is likely to exist above the water in this tube such that the error caused by diaphragm displacement is reduced, but the rapid response feature is impaired. If the piezometric level is above the end, and the end is capped, the error caused by diaphragm displacement remains. If the end is not capped, water will rise in the tube as the check valve closes during reading, possibly creating a false reading because of the increased head. Therefore, addition of this third tube appears to be a disadvantage in all cases. The intent of adding a fourth tube and connecting it to the cavity is apparently to permit flushing any accumulated gas from the cavity, thus minimizing response time and ensuring that pore water pressure rather than pore gas pressure is measured in unsaturated soils. However, the limitations described above for the single additional tube are applicable, and all the limitations of twin-tube hydraulic piezometers have been added. The author believes that a better solution to problems caused by large diaphragm displacement is use of an appropriately designed *normally closed* pneumatic piezometer.

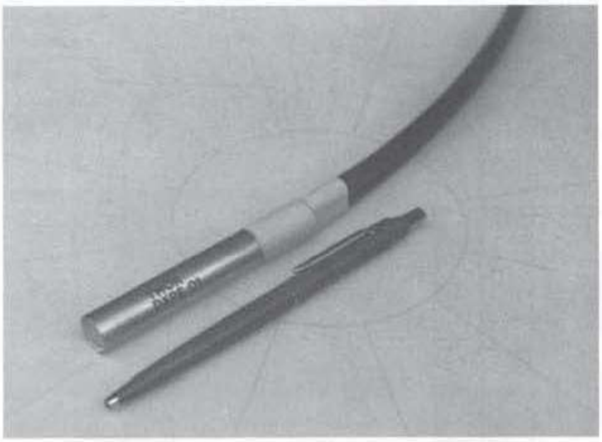


**Figure 9.15. Pneumatic piezometer (courtesy of Thor International, Inc., Seattle, WA).**

When selecting a pneumatic piezometer, a user must pay attention to the details discussed in Section 8.3. The author does not favor use of *normally open* pneumatic piezometers.

## 9.6. VIBRATING WIRE PIEZOMETERS

The vibrating wire piezometer has a metallic diaphragm separating the pore water from the measuring system. As shown in Figure 9.16, a tensioned wire is attached to the midpoint of the diaphragm such that deflection of the diaphragm causes changes in wire tension, and measurements are made by following the procedure described in Chapter 8. Figure 9.17 shows the version manufactured by Telemac.

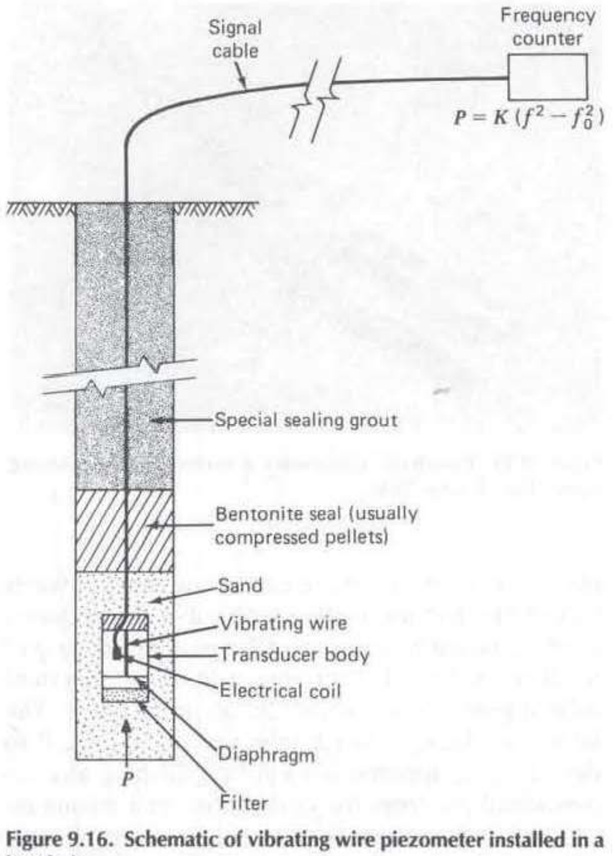



**Figure 9.16. Schematic of vibrating wire piezometer installed in a borehole.**

Chapter 8 describes advantages and limitations of vibrating wire transducers, identifies the major historical disadvantages as the potential for errors caused by zero drift and by corrosion of the vibrating wire, and indicates methods for minimizing errors. Most vibrating wire strain gage piezometers have a dried and hermetically sealed cavity around the vibrating wire, thereby minimizing the corrosion problem.

In recognition of the concern for zero drift, several versions of vibrating wire piezometer are provided with an in-place check feature whereby the zero reading and in some cases the calibration can be checked at any time during the life of the piezometer. These versions are described in Section 9.8.

There has been recent evidence in Norway that if standard vibrating wire piezometers are installed in compacted fill, they may show a reading change caused by total stresses acting on the piezometer body. The problem can be overcome at the manufacturing stage by constructing the piezometer within a thick-walled cylinder (DiBiagio and Myrvoll, 1985) or within an outer protective shell, and this appears to be a worthwhile precaution for all vibrating wire piezometers that are to be installed within embankments. More details of this rubust instrument are given in Section 9.8.

**Figure 9.17. Vibrating wire piezometer (courtesy of Telemac, Asnières, France).**

Vibrating wire piezometers can be packaged within push-in housings, as shown on Figure 9.18. However, they are subject to the limitations discussed in Section 9.16.

## 9.7. ELECTRICAL RESISTANCE PIEZOMETERS

Operating principles and commercial versions of unbonded and bonded electrical resistance strain gage piezometers are shown in Figures 9.19–9.22.

The unbonded version uses a transducer invented by Roy Carlson in 1928 and patented by him in 1936.

Bonded versions generally include semiconductor resistance strain gages. Although highly stable bonded resistance strain gage transducers are available, their cost is high, at present they cannot be used in piezometers that are competitively priced, and commercially available piezometers have transducers for which long-term stability is uncertain. Bozozuk (1984) reports that numerous cases of long-term drift have been observed in the field. However, ongoing improvements in semiconductor

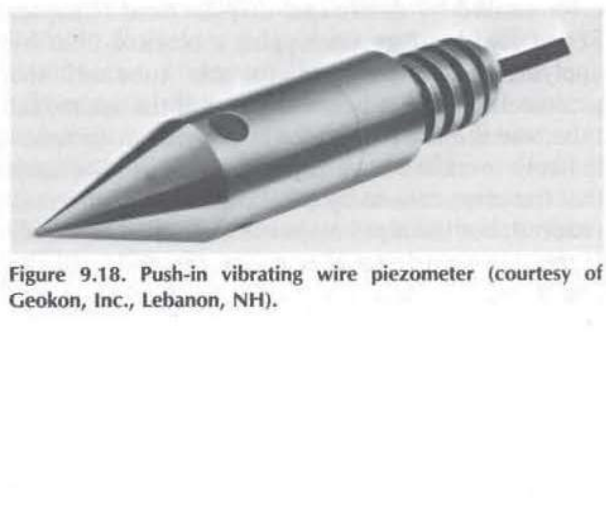


**Figure 9.18. Push-in vibrating wire piezometer (courtesy of Geokon, Inc., Lebanon, NH).**

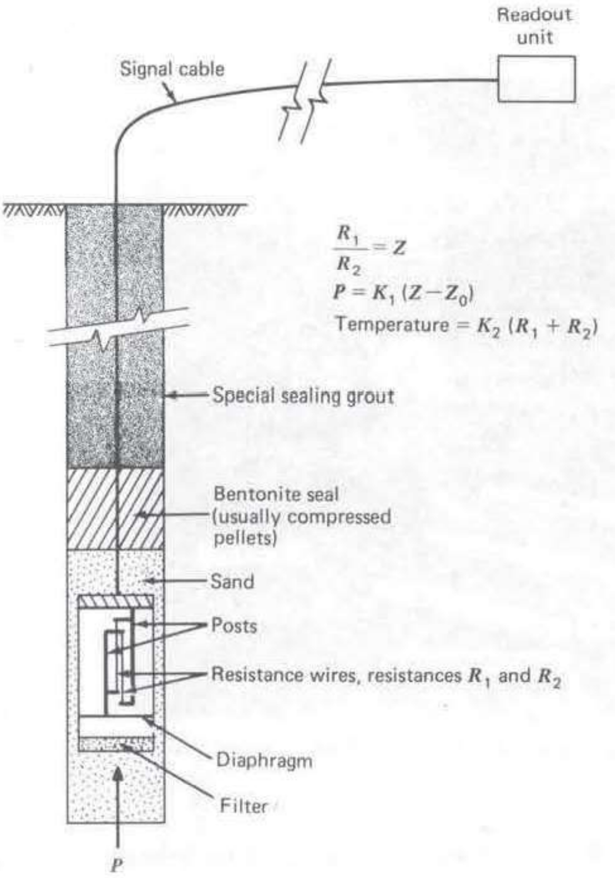



**Figure 9.19. Schematic of Carlson unbonded electrical resistance strain gage piezometer installed in a borehole.**

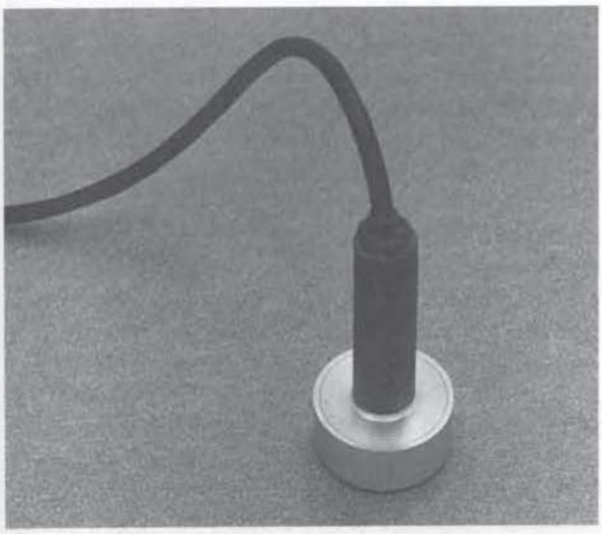


**Figure 9.20. Unbonded electrical resistance strain gage piezometer (*pore pressure cell*) (courtesy of Carlson Instruments, Campbell, CA).**

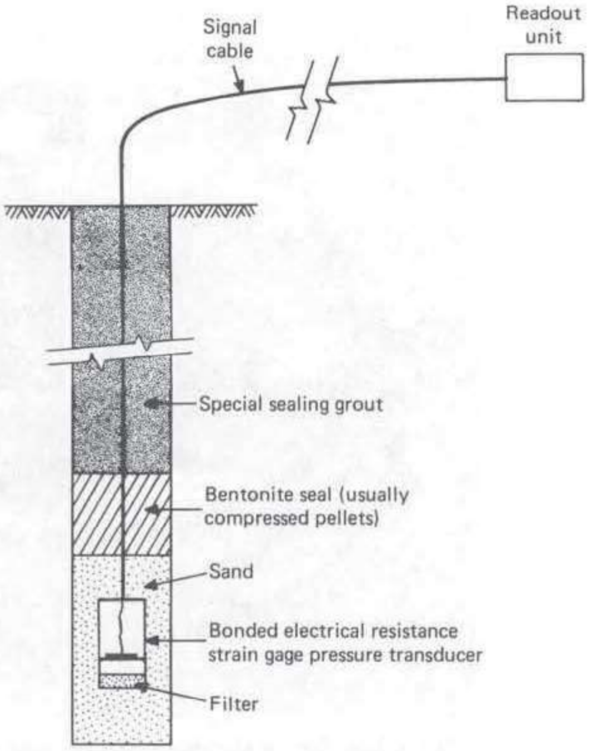



**Figure 9.21. Schematic of bonded electrical resistance strain gage piezometer installed in a borehole.**

strain gage technology may lead to development of more reliable and competitive piezometers of this type in the near future.

Bonded electrical resistance strain gage piezometers can be packaged within push-in housings (e.g., Torstensson, 1975; Wissa et al., 1975) and are used primarily for in situ determination of soil properties, where small size is critical and periodic recalibration is possible. They can, however, be left in place for monitoring pore water pressure, in which case they are subject to the limitations discussed in Section 9.16. They can also be used as *profiling piezometers* for measuring pore water pressure at many points, by pushing in increments and waiting for the reading to stabilize after each push.

A version of the bonded electrical resistance strain gage piezometer referred to as the *BAT piezometer* is arranged so that the transducer can be detached from the porous tip. The piezometer is described in Section 9.8.

When electrical resistance strain gage piezometers are installed in compacted fill they should, as described above for vibrating wire piezometers, be

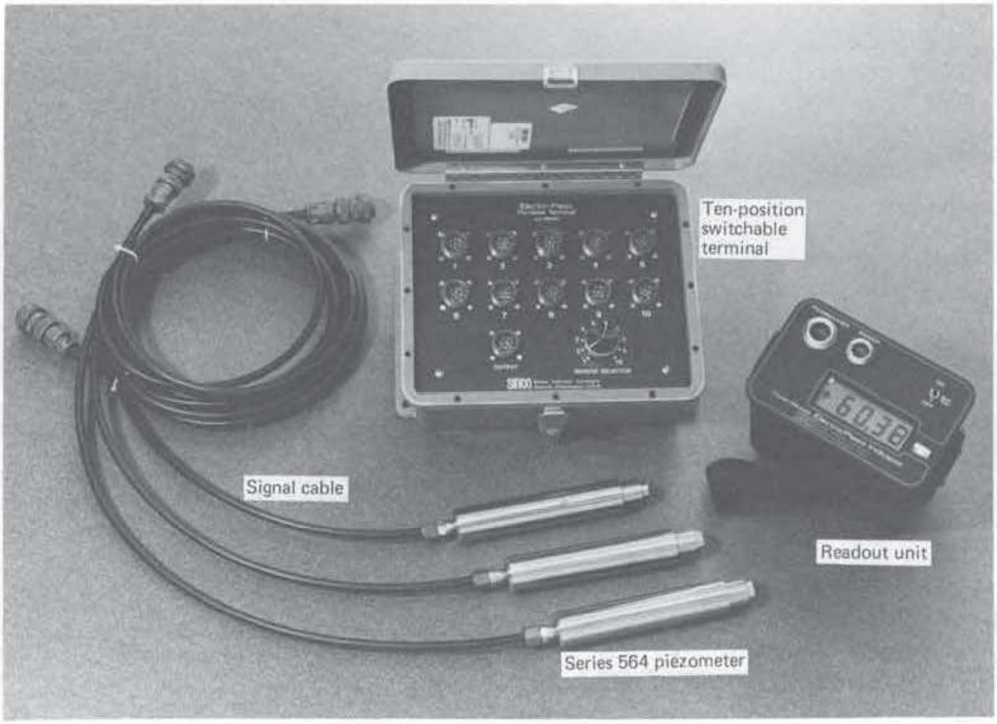



**Figure 9.22. Bonded electrical resistance strain gage piezometers (courtesy of Slope Indicator Company, Seattle, WA).**

housed within an adequately strong body so that reading changes are not caused by total stresses acting on the outside of the body.

## 9.8. MISCELLANEOUS SINGLE-POINT PIEZOMETERS

Numerous single-point piezometers are available, based in principle on one of the five types described above. The following sections describe examples that have a practical application.

### 9.8.1. Push-in Open Standpipe Piezometer: Wellpoint Type

A standard wellpoint (Section 9.2) is connected to galvanized pipe with threaded couplings. Nominal pipe diameter is typically 1.0 or 1.25 in. (25 or 32 mm). The author believes that use of this standard system will not result in an adequate seal above the piezometer and that it must be amended by adding a length of larger-diameter pipe above the wellpoint, as described in Section 9.16. Section 9.16 also describes other limitations of push-in open standpipe piezometers.

### 9.8.2. Push-in Open Standpipe Piezometer: Geonor Type

The Geonor *Model M-206 field piezometer* (Figure 9.23) was developed for use in soft sensitive clays in Norway and is pushed into place by attachment to EX-size drill rods.

EW-size drill rods will also fit on the piezometer, but the connection between an EX male and EW female thread will not be watertight, and excess pore water pressure is likely to bleed through this connection and dissipate along the inside of the drill rod. If EW rods are used, a special effort must be made to seal the connection. Sintered bronze filters are standard, and epoxy filters are used in corrosive environments. The outside diameters of EX and EW drill rods (Appendix F) are slightly larger than the outside diameter of the piezometer (1.30 in., 33.0 mm). The rods remain in place, and the piezometer and rods are recovered when measurements are no longer required, but economy can be achieved by using 1 in. (25 mm) steel pipe with threaded couplings above a sufficient length of EX rod for adequate sealing.

The piezometer has been used successfully as a *profiling piezometer* for measuring pore water pres-

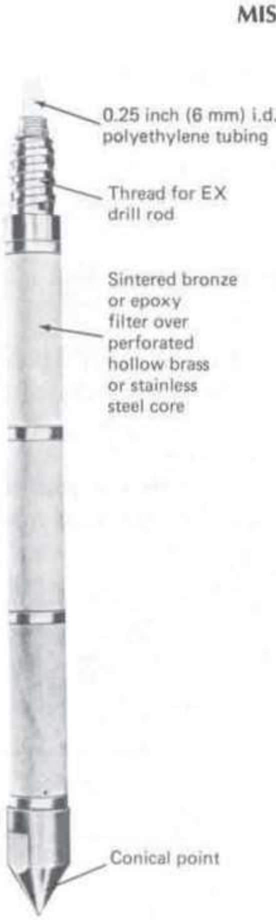



**Figure 9.23.** Geonor Model M-206 field piezometer (courtesy of Geonor A/S, Oslo, Norway).

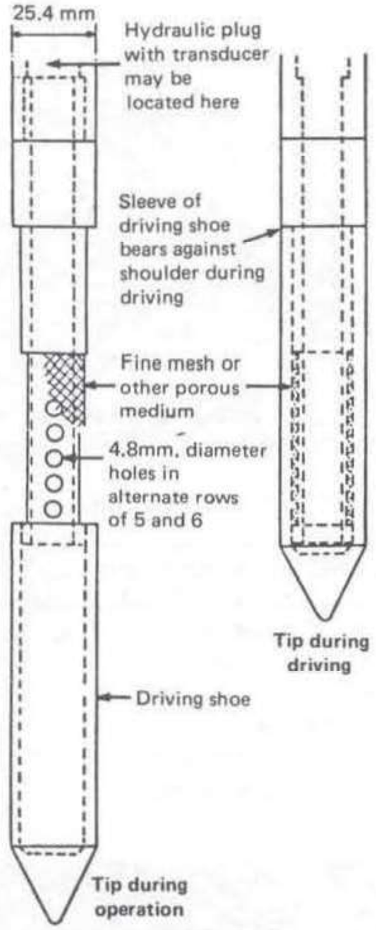



**Figure 9.24.** Cambridge drive-in piezometer (after Parry, 1971). Reprinted by permission of the Institution of Civil Engineers, London.

sure at many points in a very soft marine clay by pushing in 1.5 ft (0.5 m) increments and waiting for the water level in the standpipe to stabilize after each push, normally requiring an overnight wait (Handfelt et al., 1987). In addition to providing data at many points, this procedure can be used to check permanently installed piezometers. The version of piezometer with sintered bronze filters should be used for this purpose, because the version with epoxy filters has a smaller-diameter core that tends to break while pushing or withdrawing the piezometer.

### 9.8.3. Push-in Open Standpipe Piezometer: Cambridge Type

The *Cambridge drive-in piezometer* (Parry, 1971) is shown in Figures 9.24 and 9.25 and was developed for pushing or driving through soil containing stones and hard seams without damaging, smearing, or clogging the porous element. All parts of the tip except the gauze mesh are mild steel. The piezometer is attached with a flush joint to a length of steel pipe and driven into the ground with the sliding driving shoe covering the porous section as shown on the right side of Figure 9.24. When in place, a mandrel is inserted within the standpipe and the driving shoe tapped down approximately 900 mm (35 in.) relative to the standpipe, thereby exposing the porous element to the soil.

The piezometer is available from Geotechnical Instruments (U.K.) Ltd. and Soil Instruments Ltd.

### 9.8.4. Heavy Liquid Piezometer

The *heavy liquid piezometer* (Figure 9.26) is a special type of open standpipe piezometer, developed by Piezometer Research and Development. The

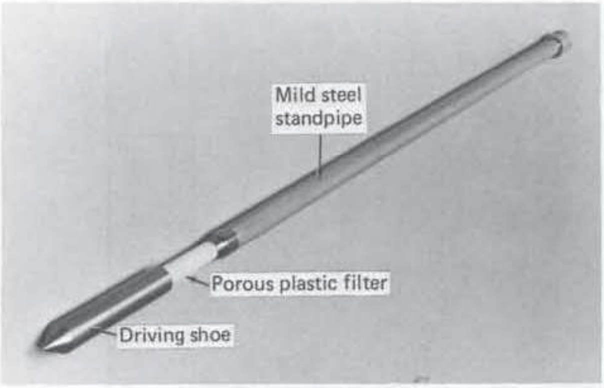



**Figure 9.25. Cambrdge drive-in piezometer (courtesy of Soil Instruments Ltd., Uckfield, England).**

standpipe and part of the piezometer tip are filled with acetylene tetrabromide, a liquid with a specific gravity of 2.96, and in the tip there is a stable interface between acetylene tetrabromide and water. The surface of liquid in the standpipe is therefore lower than the piezometric level, and the known specific gravity is used to convert measured liquid level to piezometric level.

When compared with a conventional open standpipe piezometer, the heavy liquid piezometer reduces response time and overcomes freezing problems. When the piezometric level rises above the top of the standpipe it also overcomes the need to attach a Bourdon tube pressure gage and therefore reduces the possibility of damage by vandals.

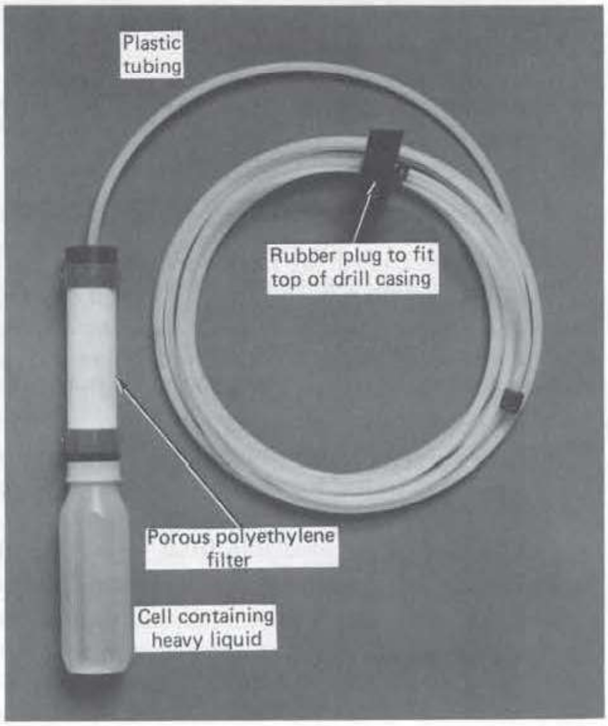



**Figure 9.26. Heavy liquid piezometer (courtesy of Piezometer Research & Development, Bridgeport, CT).**

### 9.8.5. Diaphragm Piezometers with In-Place Check Features

Several versions of vibrating wire piezometers are provided with an in-place check feature whereby the zero reading can be checked at any time during the life of the piezometer.

DiBiagio (1974) describes a feature, shown in Figure 9.27, for checking the zero reading and the

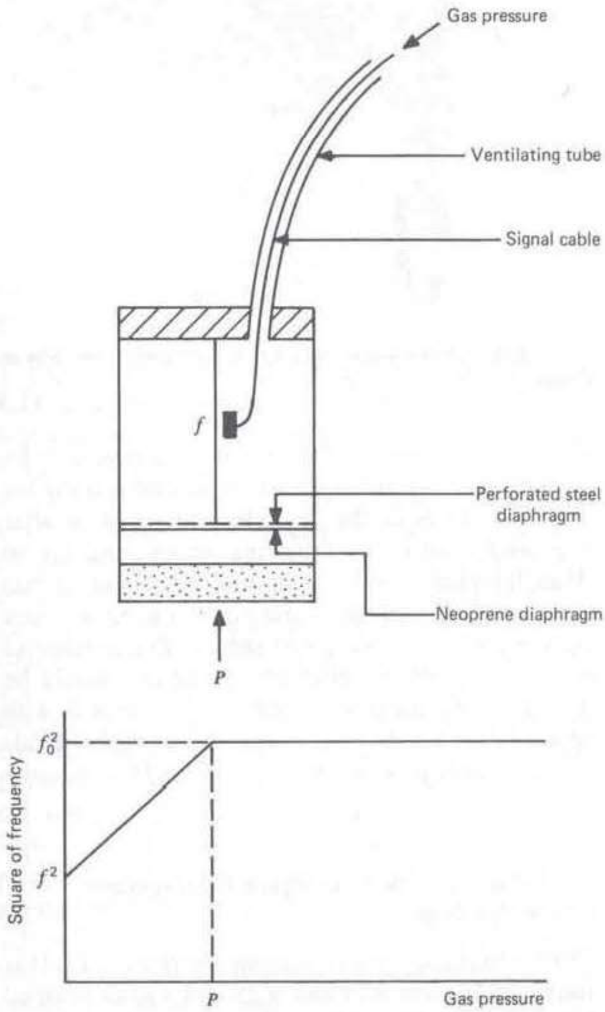



**Figure 9.27. Schematic of Norwegian Geotechnical Institute vibrating wire piezometer with feature for checking calibration in place.**

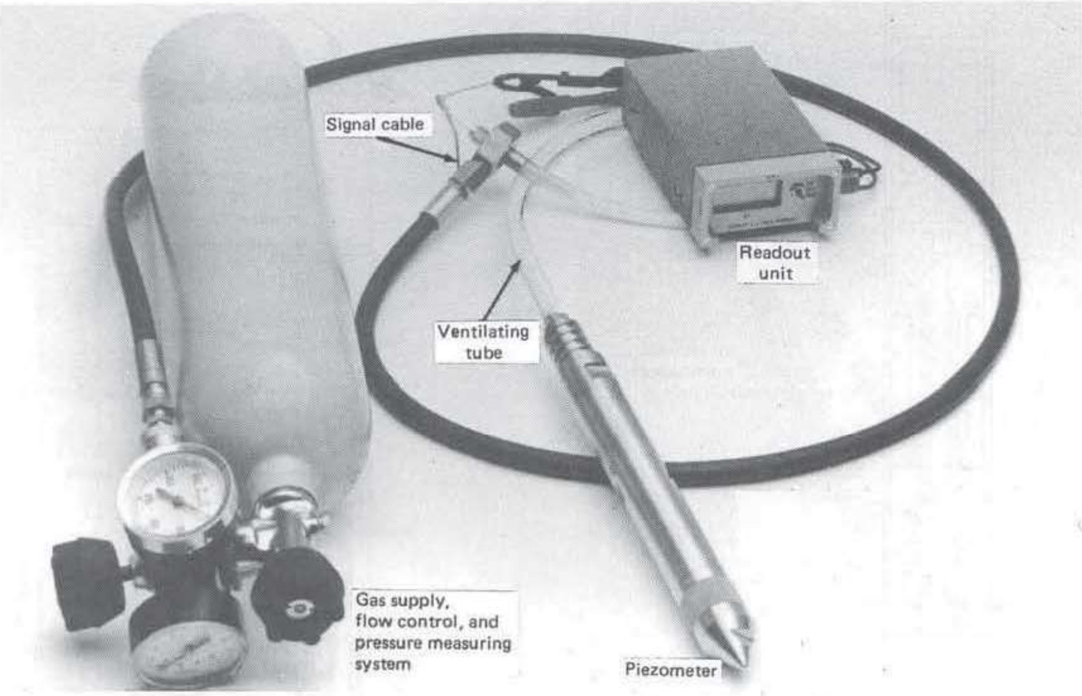



**Figure 9.28. Vibrating wire piezometer with feature for checking calibration in place (courtesy of Geonor A/S, Oslo, Norway).**

complete calibration in place. The instrument is available from Geonor (Figure 9.28). The body of the gage is ventilated through a tube, and the back of the neoprene diaphragm is normally at atmospheric pressure. Under normal working conditions the two diaphragms are in contact, and the instrument behaves as a conventional vented vibrating wire piezometer. To check the zero reading and calibration, gas pressure is applied to the back of the diaphragm through the ventilating tube. Readings of gage wire frequency are taken at various pressure increments until eventually, when the gas pressure is slightly greater than the pore water pressure, the neoprene diaphragm moves away from the metal diaphragm, leaving it with equal gas pressure on both sides, and the frequency of vibration of the gage wire reaches an asymptotic value. Any further increase in the applied gas pressure does not cause further change in the vibrating frequency of the gage wire, and this limiting frequency value is the zero frequency for the instrument. The calibration of the instrument is indicated by the slope of the pressure versus frequency plot.

Various manufacturers of vibrating wire piezometers now offer this feature as an option, and the feature could also be used for checking electrical resistance piezometers. Although an ingenious arrangement, it is incompatible with a hermetically sealed cavity within the piezometer tip and increases the potential for corrosion because any damage to the ventilating tube may provide a direct path for moisture to enter the cavity. Deterioration of the neoprene diaphragm may also result in malfunction. The instrument is therefore best suited to obtaining verified data of high accuracy over a short term, rather than as an in-place check over a long term.

The Swedish manufacturer Geotech AB has developed a method for checking the zero reading in place without the above limitation. As shown in Figure 9.29, bellows and a stem are added between the diaphragm and the filter. The zero reading can be checked (but not the slope of the calibration) by applying a voltage to the electromagnet to move the stem downward, thereby separating the stem from the diaphragm and returning the vibrating wire to its zero condition. The downward movement is approximately 0.05 mm (0.002 in.).

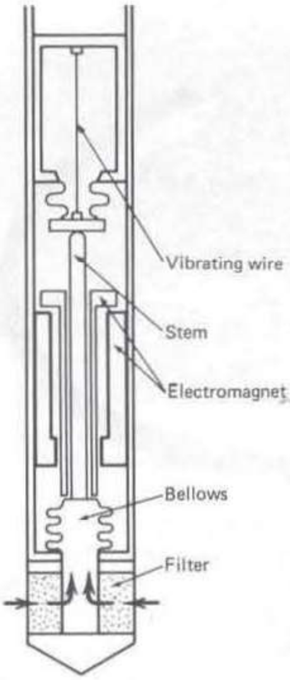



Figure 9.29. Schematic of Geotech AB vibrating wire piezometer Model PZ 4000, with feature for checking zero reading in place.

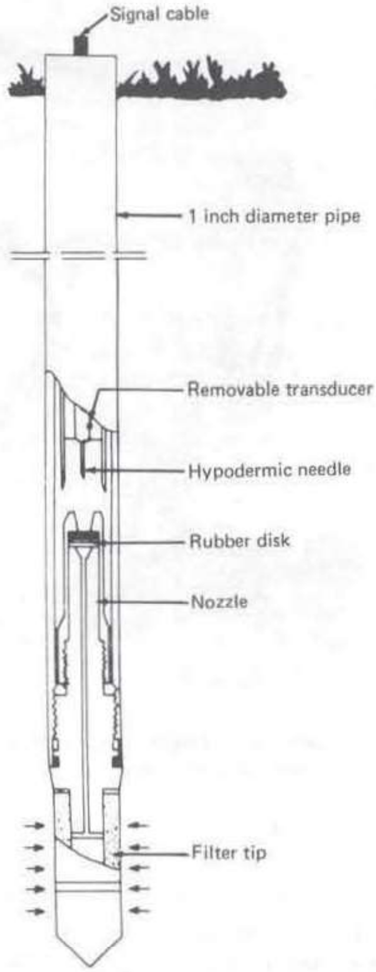



Figure 9.30. BAT piezometer (after Torstensson, 1984a). Reprinted by permission of *Ground Water Monitoring Review*, 

### 9.8.6. BAT piezometer

The *BAT piezometer* (Torstensson, 1984a) allows a bonded resistance strain gage transducer to be detached at any time from the porous tip, permitting servicing and recalibration of the transducer. The instrument is available from BAT Envitech, and a single transducer can be used for many piezometers. The porous tip has a short nozzle at its top, sealed with a rubber disk. The tip is saturated, attached to 1 in. (25 mm) galvanized pipe, and installed by the push-in method. Whenever a measurement of pore water pressure is required, a transducer unit is lowered down the standpipe to mate with the porous tip. A hypodermic needle at the base of the transducer unit pierces the rubber disk to create a hydraulic connection, and the disk is self-sealing when the transducer unit and needle are withdrawn. The arrangement is shown in Figure 9.30. Torstensson (1984a) reports that the rubber disk can be penetrated by the hypodermic needle several hundred times without loss of its self-sealing function. Separate units can be lowered down the standpipe for groundwater sampling and for in situ permeability testing.

### 9.8.7. Piezometer Designed to Be Installed by Pushing in Place Below the Bottom of a Borehole

A special version of pneumatic piezometer, manufactured by Slope Indicator Company, is designed for pushing into place below the bottom of a borehole (Handfelt et al., 1987). Figure 9.31 shows a version suitable for use in very soft clay (standard penetration test blowcount of less than about 2 blows per foot). Figure 9.32 shows a photograph of this version and also a more robust version suitable for pushing into stiffer clay (this has been installed in clay with a standard penetration test blowcount of 15 blows per foot, but is likely to be suitable for installation in stiffer clays). A similar packaging

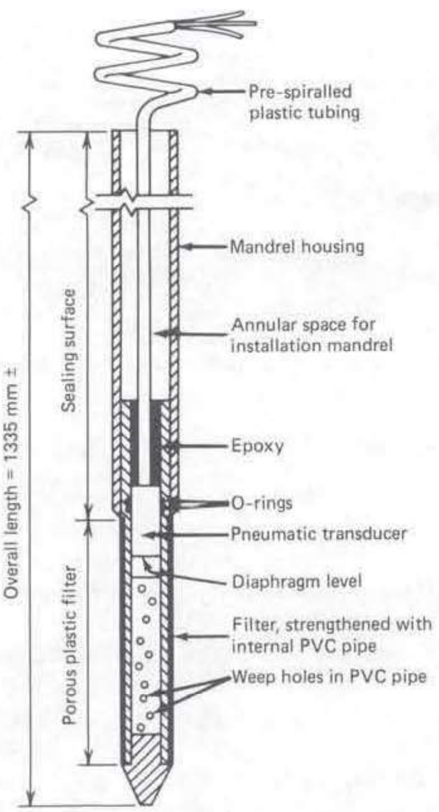



**Figure 9.31. Pneumatic piezometer for installation by pushing in place below bottom of a borehole in very soft clay (courtesy of RMP Encon Ltd. and Dames & Moore).**

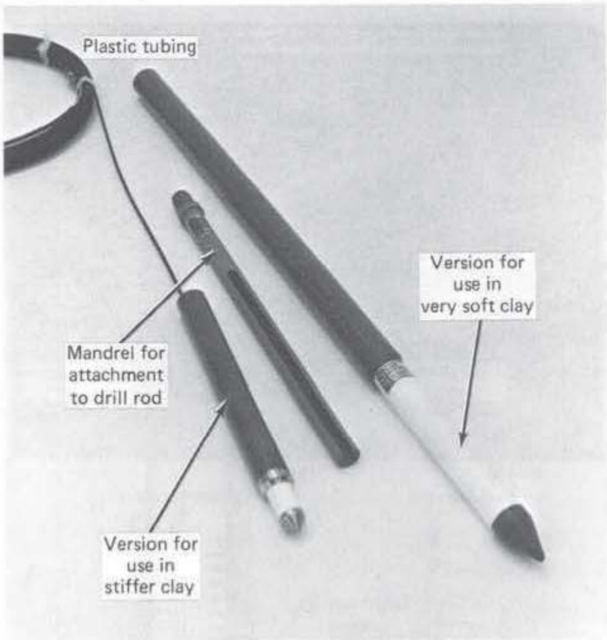



**Figure 9.32. Pneumatic piezometers for installation by pushing in place below bottom of borehole (courtesy of Slope Indicator Company, Seattle, WA).**

could be arranged for other types of diaphragm piezometer. The method of installation is described in Section 9.17.1.

### 9.8.8. Vibrating Wire Piezometers for Embankment Dams

DiBiagio and Myrvoll (1985) describe a vibrating wire piezometer specifically designed for installation in embankment dams. The piezometer is manufactured by Geonor and is shown in Figures 9.33 and 9.34. It has a thick-walled stainless steel housing to prevent the vibrating wire transducer from responding to total stresses acting on the housing, a very robust and water-blocked signal cable, heavy-duty seals, and long high air entry filters. The piezometer, and also a heavy-duty vibrating wire piezometer manufactured by Geokon, are good examples of instruments designed for long-term survivability.

### 9.8.9. Piezometers with Duplicate Transducers

Piezometers are available with one pneumatic and one vibrating wire transducer housed in the same body, so that one transducer acts as a backup to the other. An example is shown in Figure 9.35. Alternatively, two separate piezometers can be installed, but the combination arrangement simplifies installation of the tubing/signal cable. Vibrating wire piezometers and also pneumatic piezometers are

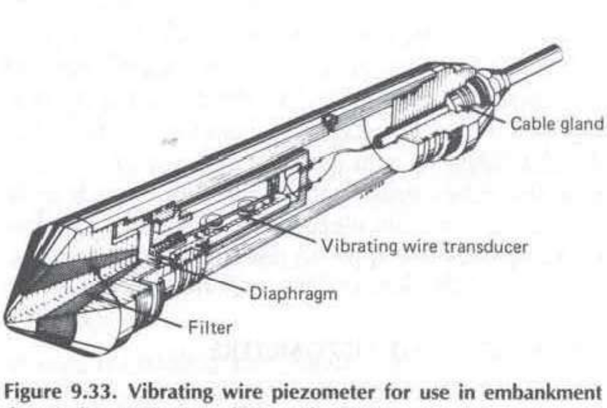
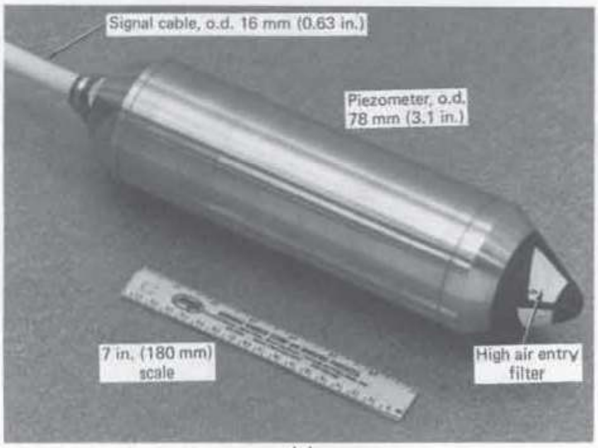
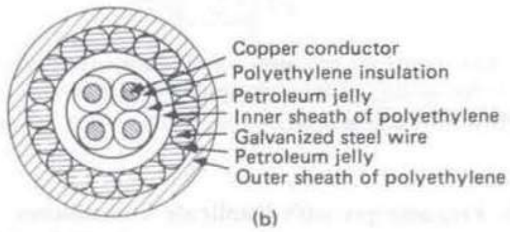



**Figure 9.33. Vibrating wire piezometer for use in embankment dams (after DiBiagio and Myrvoll, 1985).**




(a)




(b)

**Figure 9.34. Vibrating wire piezometer for use in embankment dams: (*a*) Model S-411 piezometer and (*b*) cross section of P-430 signal cable (courtesy of Geonor A/S, Oslo, Norway).**

available with two hydraulic tubes connected to the cavity between the filter and diaphragm, thereby adding a twin-tube hydraulic piezometer as backup, and this arrangement also allows for flushing any gas out of the cavity via the hydraulic tubes.

Whenever considering a piezometer with duplicate transducers, the combination will generally be subject to the combined limitations of both but will not necessarily be subject to the combined advantages of both. For example, addition of twin hydraulic tubes to a pneumatic piezometer requires that the tubes must not be installed significantly above the minimum piezometric elevation, a limitation that does not apply to pneumatic piezometers.

## 9.9. MULTIPOINT PIEZOMETERS

Multipoint piezometers can be created by four methods, described in turn.

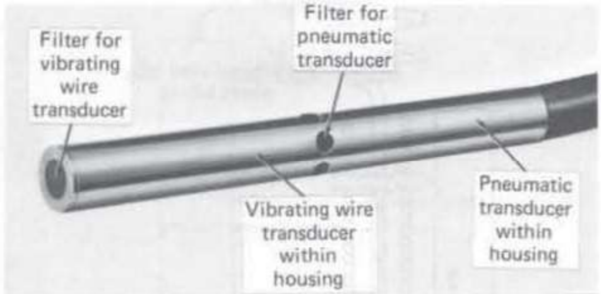



**Figure 9.35. Piezometer with one pneumatic and one vibrating wire transducer (courtesy of Geokon, Inc., Lebanon, NH).**

### 9.9.1. Multipoint Piezometers with Packers

Several piezometers can be assembled with a packer above and below each piezometer to create a seal.

Inflatable packers (e.g., Figure 9.36) are most frequently used and are filled with air, water, or epoxy. Commercial sources of inflatable packers, with multiple central holes for passage of leads from lower piezometers and packers, are given in Appendix D.

An innovative system has recently been developed by Solinst Canada Ltd., based on a design by the University of Waterloo (Cherry and Johnson, 1982). The arrangement, shown in Figure 9.37, uses Dowell chemical sealant (Section 9.17.8), which swells when in contact with water that is poured down the central PVC pipe. The figure shows standpipe piezometers, but alternatively pneumatic or vibrating wire piezometers can be used. The instrument can also be adapted for groundwater sampling. The system is supplied in modular form, with threaded connections in the PVC pipe between modules. An O-ring seal is included in each connection, as shown in Figure 9.37, to ensure that connections are watertight.

### 9.9.2. Multipoint Piezometers Surrounded with Grout

Several piezometers can be packaged within a length of pipe to create a multipoint piezometer, which is then inserted within a borehole in soil and entirely surrounded with grout.

Vaughan (1969) describes a multipoint open standpipe piezometer that is installed in this way. Vaughan reports that the response times of the piezometers in grout are adequate for most engineering purposes for all permeabilities of the soil.

Figure 9.36. Multipoint pneumatic piezometers with inflatable packers (courtesy of Thor International, Inc., Seattle, WA).

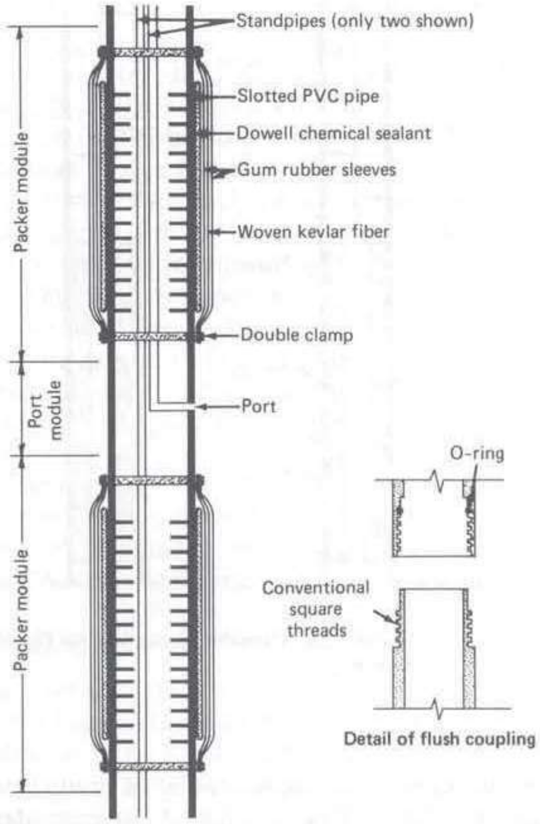



Figure 9.37. Schematic of Waterloo Multilevel System (courtesy of Solinst Canada Ltd., Burlington, Ontario, Canada).

When the permeability of the grout is greater than that of the soil the response time of the piezometer in grout is only slightly greater than that of a piezometer with a sand filter. When the soil has a permeability more than two orders of magnitude greater than the permeability of the grout, the response time is controlled by the grout annulus only. However, bearing in mind permeability and conformance criteria, the author believes that the applicability of this method is limited to use in a uniform clay of known properties, such as the core of an embankment dam. It is also limited by the difficulties of manufacturing and installing a grout so that its in-place compressibility and permeability are reasonably uniform.

### 9.9.3. Multipoint Push-in Piezometers

A multipoint open standpipe piezometer can be assembled from flush-coupled pipe with several filter zones and plastic tubes and pushed into place. Massarsch et al. (1975) report on good performance of a multipoint vibrating wire piezometer, with transducers 1 m (3 ft) apart and installed by pushing into clay, and indicate that leakage did not occur along the flush-coupled pipe. Carpentier and Verdonck (1986) describe a multipoint piezometer with electrical resistance strain gage transducers, used for measuring the influence of tides and waves on pore water pressures below the seabed.

Push-in piezometers are subject to the limitations discussed in Section 9.16.

### 9.9.4. Multipoint Piezometers with Movable Probes

A pipe is installed and sealed within a borehole and a movable probe periodically inserted within the pipe. The development of multipoint piezometers with movable probes has created a significant

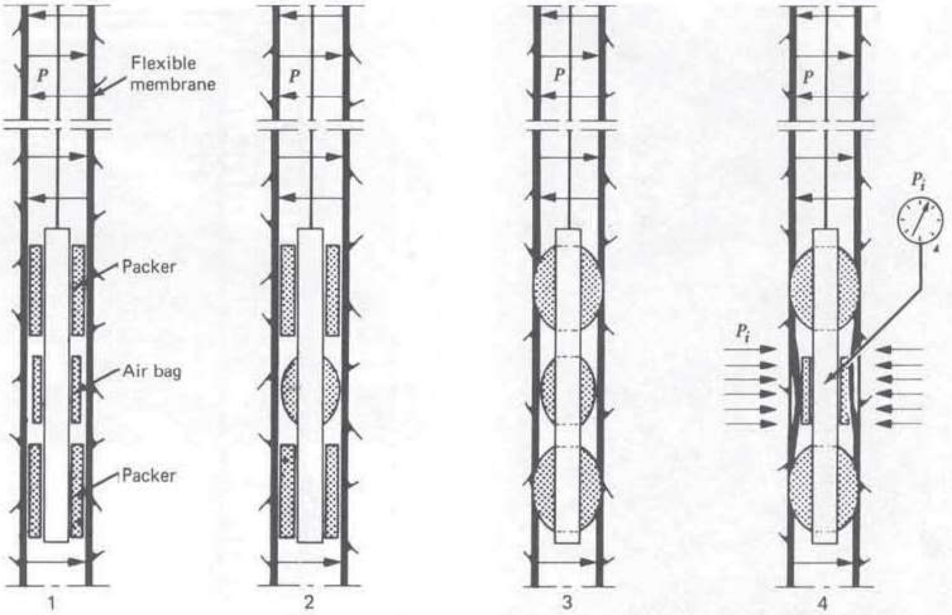



**Figure 9.38. Operating procedure for Piezofor multipoint piezometer (courtesy of Telemac, Asnières, France).**

breakthrough in the measurement of groundwater pressure. They allow a detailed pressure–depth profile to be defined, with an almost unlimited number of measurement points. Although they have a high initial cost, their use may result in substantial cost savings by ensuring that a project is designed with a reliable knowledge of the pressure–depth profile.

Three configurations are commercially available.

### *Piezofor System*

The *Piezofor* system was developed in France for measurement of joint water pressure in rock (Hoek and Londe, 1974; Londe, 1982). A slotted pipe is grouted into a borehole with a brittle slurry that cracks as it shrinks. A soft waterproof tubular membrane is inserted into the pipe and pressurized to a higher value than the expected groundwater pressure.

Joint water pressure measurements are taken by following the steps shown in Figure 9.38. The Piezofor probe is lowered to the desired point, and water is forced out of the test section between the packers by inflating the air bag. The packers are inflated to isolate the test section and the air bag is deflated. The membrane is now forced inward by the outside pressure from the groundwater, causing the inside pressure to rise asymptotically until the two balance. The inside pressure is monitored with an electrical pressure transducer. The procedure can be repeated along the entire length of the borehole to provide the full piezometric profile, without affecting the natural seepage pattern. The rate at which equilibrium occurs between inside and outside pressures allows an estimate to be made of the permeability of the ground at that level. A pressure lock can be fitted at the top of the pipe if required, so that the probe can be inserted within the membrane without depressurizing the borehole.

The system has been used successfully in boreholes 330 ft (100 m) deep, and a system for use in 5000 ft (1500 m) boreholes is currently being tested.

### *Westbay Multiple Piezometer*

More recently, a movable probe device has been developed by Westbay Instruments Ltd. in Canada (Patton, 1979; Rehtlane and Patton, 1982) and is referred to as the *MP (multiple piezometer) System*. Installations can be made in soil and rock.

Figure 9.39 shows the general arrangement and the probe. The system consists of pipe, couplings and packers permanently installed in a borehole, a portable pressure measurement probe, and installa-

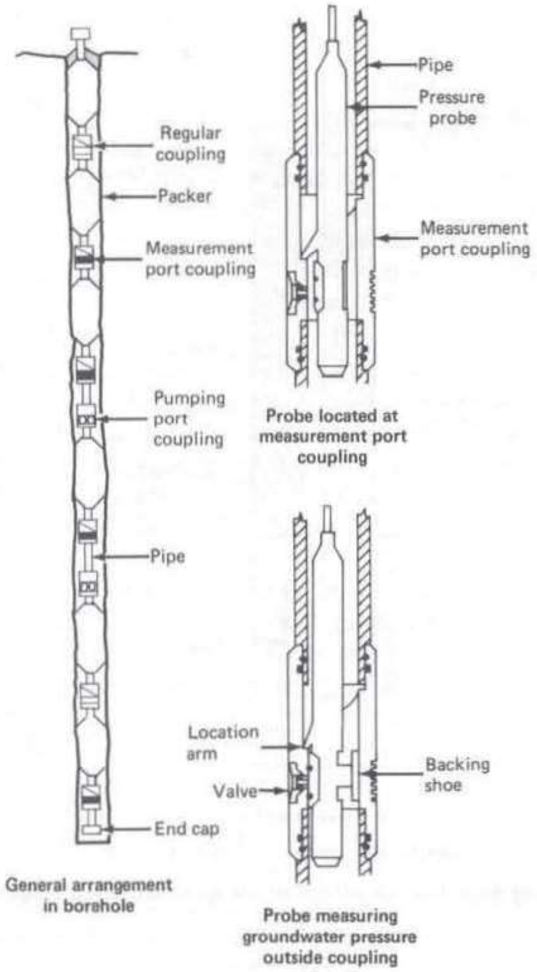



**Figure 9.39. Multiple piezometer (MP) system (courtesy of Westbay Instruments Ltd., North Vancouver, BC, Canada).**

tion tools. *Measurement port couplings* are installed in the pipe wherever groundwater pressure measurements are desired. Each of these couplings has a hole through its wall, with a filter on the outside and spring-loaded check valve on the inside. The remainder of the pipe is connected with sealed couplings, and a packer is installed around the pipe above and below each measurement port coupling. The assembly is lowered into the borehole, and the packers inflated one at a time with water, using a probe temporarily inserted within the pipe. To take readings, a pressure measurement probe is lowered within the pipe to locate a measurement port, jacked against the opposite wall of the pipe to open a check valve, and a measurement made. The procedure is repeated at each measurement port.

Pipe and couplings are manufactured in PVC or stainless steel. Pneumatic pressure probes are available for depths up to 250 ft (75 m) and standard electrical probes for depths up to 1000 ft (300 m). Recently a system has been used in boreholes 3000 ft (900 m) deep with 10–15 measurement ports in each and is designed for operation to a depth of 5000 ft (1500 m). Separate pneumatic and electrical probes are also available for taking pressurized or unpressurized samples of groundwater through the measurement ports. Additional valved couplings, referred to as *pumping port couplings,* can be included in the pipe for permeability testing or decontamination pumping. Grooved inclinometer casing can be used instead of smooth-walled pipe to create a combined piezometer and inclinometer system (Westbay Instruments Ltd., CPI system).

### *Piezodex System*

The *Piezodex* system has recently been developed by the Federal Institute of Technology, Zürich, and Solexperts AG (Kovari and Köppel, 1987). Pressure transmitters are mounted in the wall of the access pipe and isolated with pneumatic packers. Each pressure transmitter consists of a piston with very low volumetric displacement. When the movable probe makes contact with a piston, the piston moves outward slightly (0.1–0.2 mm, 0.004–0.008 in.), and groundwater pressure is transmitted to a force transducer mounted in the probe. The depth capability is currently limited to 50 m (165 ft), but a system with twice this depth capability is under development.

## 9.10. HYDRODYNAMIC TIME LAG*

When a piezometer is installed and groundwater pressure changes, the time required for water to flow into or out of the piezometer to effect equalization is called the *hydrodynamic time lag*. It is dependent primarily on the type and dimensions of the piezometer and the permeability of the ground. Open standpipe piezometers have a much greater hydrodynamic time lag than diaphragm piezometers

*Sections 9.10–9.14 have been written with the assistance of Arthur D. M. Penman, Consulting Geotechnical Engineer, Harpenden, England.

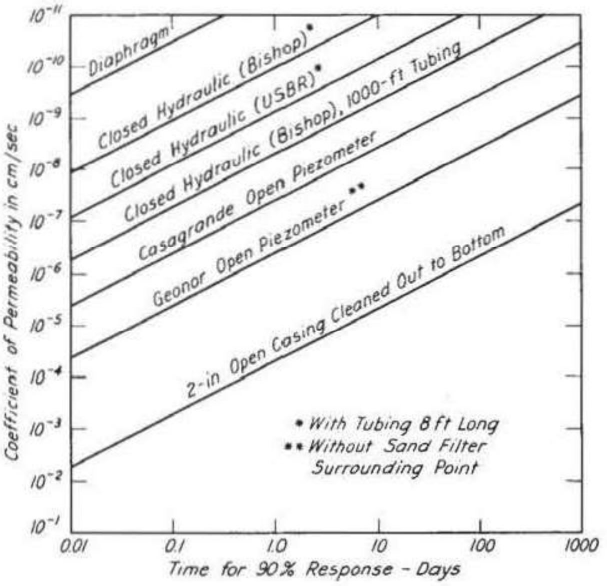



**Figure 9.40. Approximate response times for various types of piezometer (after Terzaghi and Peck, 1967).**

because a much greater movement of pore or joint water is involved. The term *slow response time* is used to describe a long hydrodynamic time lag.

Methods of estimating time lag are presented by Hvorslev (1951) and Terzaghi and Peck (1967). Brooker and Lindberg (1965) evaluate time lag effects for twin-tube hydraulic piezometers. Brand and Premchitt (1982), Penman (1960), Premchitt and Brand (1981), and Vaughan (1974a) report on the effects for various types of piezometers.

The order of magnitude of the time required for 90% response of several types of piezometer installed in homogeneous soils can be obtained from Figure 9.40, in which the *Geonor open piezometer* is the push-in instrument shown in Figure 9.23. The 90% response is considered adequate for many practical purposes, and of course the time for 100% response is infinite. The response time of open standpipe piezometers can be estimated from equations given by Penman (1960). For example,

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

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

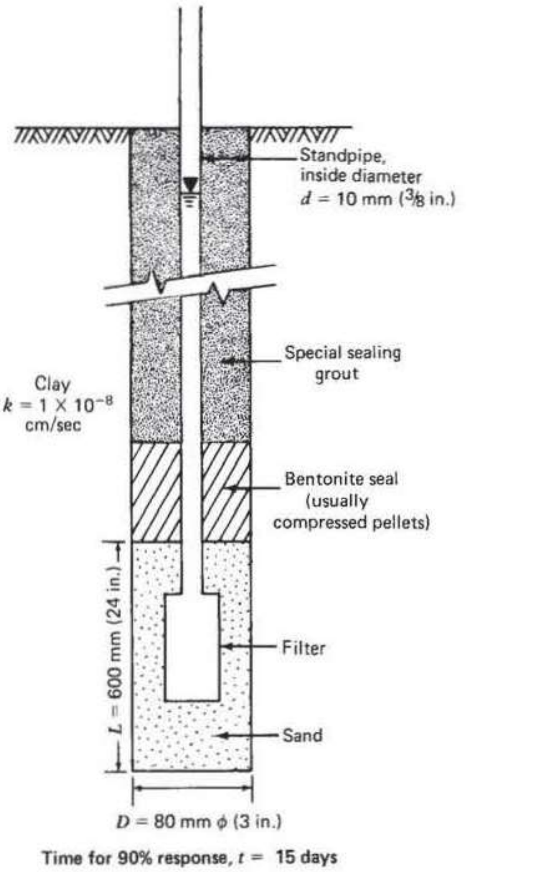



**Figure 9.41. Example of time lag for open standpipe piezometer.**

Figure 9.41 gives an example for an open standpipe piezometer surrounded by a sand zone. It should be noted that time lag can be minimized by using a minimum-diameter standpipe and a maximum-sized sand zone. However, as discussed in Section 9.3, a minimum-diameter standpipe can nullify the self-de-airing feature of the piezometer and will often create problems with insertion of the readout probe. For borehole installations, the diameter of the sand zone is limited by the economics of drilling large-diameter boreholes, and the length is often limited by the need to make measurements within a short length of the borehole.

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

## 9.11. TYPES OF FILTER

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

Figure 9.42 shows a fine-pored filter placed between two parts of a container, one part containing gas, the other part containing water. The gas pressure is higher than the water pressure, but the fluids are in equilibrium because the pressure differential is balanced by surface tension forces at the gas/water interface. The smaller the radius of curvature of the menisci at the interface, the larger can be the pressure difference between water and gas. The minimum radius of curvature of the menisci is dictated by the pore diameter in the filter; thus, the finer the filter, the greater can be the pressure differential. The *air entry value* or *bubbling pressure* of the filter is defined as the pressure differential at which blow-through of gas occurs. Thus, a filter with a high air entry value (or high bubbling pressure) is a fine filter that will allow a high pressure differential before blow-through occurs. Because such a filter also has a low permeability, the terms can be confusing. Although the fluids in Figure 9.42 are in equilibrium, the longevity of the fluid separation is uncertain because gas may enter the water by diffusion.

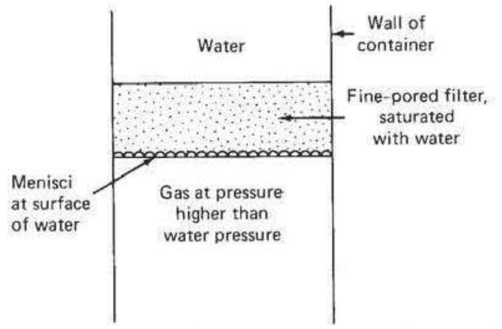



Figure 9.42. Separation of gas from water by a fine-pored filter.

Low air entry filters are coarse filters that readily allow passage of both gas and water. Typical low air entry filters have a pore diameter of 0.001–0.003 in. (0.02–0.08 mm, 20–80 microns). Typical air entry values range from 0.4 to 4.0 lb/in.$^2$ (3–30 kPa). A filter with a pore diameter of 0.002 in. (50 microns) has a permeability to water of about $3 \times 10^{-2}$ cm/sec.

High air entry filters are fine filters that can be used when piezometers are installed in unsaturated soil with the intent of measuring pore water pressure as opposed to pore gas pressure, in an attempt to keep gas out of the measuring system. Filters used for this purpose typically have a pore diameter of $4 \times 10^{-5}$ in. (0.001 mm, 1 micron), an air entry value of at least 15 lb/in.$^2$ (100 kPa), and a permeability to water of about $3 \times 10^{-6}$ cm/sec.

## 9.12. RECOMMENDED INSTRUMENTS FOR MEASURING GROUNDWATER PRESSURE IN SATURATED SOIL AND ROCK

### 9.12.1. General

General guidelines on the selection of instruments are given in Chapter 4, and a recommendation is made to maximize reliability by using the simplest instrument that will achieve the purpose. Advantages and limitations of instruments for measuring groundwater level are summarized in Table 9.1.

**Table 9.1. Instruments for Measuring Groundwater Pressure**

| Instrument Type | Advantages | Limitations[a] |
|---|---|---|
| Observation well (Figure 9.1) | Can be installed by drillers without participation of geotechnical personnel | Provides undesirable vertical connection between strata and is therefore often misleading; should rarely be used |
| Open standpipe piezometer (Figure 9.2) | Reliable<br>Long successful performance record<br>Self-de-airing if inside diameter of standpipe is adequate<br>Integrity of seal can be checked after installation<br>Can be converted to diaphragm piezometer<br>Can be used for sampling groundwater<br>Can be used to measure permeability | Long time lag<br>Subject to damage by construction equipment and by vertical compression of soil around standpipe<br>Extension of standpipe through embankment fill interrupts construction and causes inferior compaction<br>Porous filter can plug owing to repeated water inflow and outflow<br>Push-in versions subject to several potential errors: see Section 9.16 |
| Twin-tube hydraulic piezometer (Figure 9.9) | Inaccessible components have no moving parts<br>Reliable<br>Long successful performance record<br>When installed in fill, integrity can be checked after installation<br>Piezometer cavity can be flushed<br>Can be used to measure permeability | Application generally limited to long-term monitoring of pore water pressure in embankment dams<br>Elaborate terminal arrangements needed<br>Tubing must not be significantly above minimum piezometric elevation<br>Periodic flushing may be required<br>Attention to many details is necessary: see Appendix E |
| Pneumatic piezometer (Figure 9.14) | Short time lag<br>Calibrated part of system accessible<br>Minimum interference to construction: level of tubes and readout independent of level of tip<br>No freezing problems | Attention must be paid to many details when making selection: see Section 8.3<br>Push-in version subject to several potential errors: see Section 9.16 |
| Vibrating wire piezometer (Figure 9.16) | Easy to read<br>Short time lag<br>Minimum interference to construction: level of lead wires and readout independent of level of tip<br>Lead wire effects minimal<br>Can be used to read negative pore water pressures<br>No freezing problems | Special manufacturing techniques required to minimize zero drift<br>Need for lightning protection should be evaluated<br>Push-in version subject to several potential errors: see Section 9.16 |
| Unbonded electrical resistance piezometer (Figure 9.19) | Easy to read<br>Short time lag<br>Minimum interference to construction: level of lead wires and readout independent of level of tip<br>Can be used to read negative pore water pressures<br>No freezing problems<br>Provides temperature measurement<br>Some types suitable for dynamic measurements | Low electrical output<br>Lead wire effects<br>Errors caused by moisture and electrical connections are possible<br>Need for lightning protection should be evaluated |

**Table 9.1. (*Continued*)**

| Instrument Type | Advantages | Limitations[a] |
|---|---|---|
| Bonded electrical resistance piezometer (Figure 9.21) | Easy to read<br>Short time lag<br>Minimum interference to construction: level of lead wires and readout independent of level of tip<br>Suitable for dynamic measurements<br>Can be used to read negative pore water pressures<br>No freezing problems | Low electrical output<br>Lead wire effects<br>Errors caused by moisture, temperature, and electrical connections are possible.<br>Long-term stability uncertain<br>Need for lightning protection should be evaluated<br>Push-in version subject to several potential errors: see Section 9.16 |
| Multipoint piezometer, with packers (e.g., Figures 9.36 and 9.37) | Provides detailed pressure–depth measurements<br>Can be installed in horizontal or upward boreholes<br>Other advantages depend on type of piezometer: see above in table | Limited number of measurement points<br>Other limitations depend on type of piezometer: see above in table |
| Multipoint piezometer, surrounded with grout | Provides detailed pressure–depth measurements<br>Simple installation procedure<br>Other advantages depend on type of piezometer: see above in table | Limited number of measurement points<br>Applicable only in uniform clay of known properties<br>Difficult to ensure in-place grout of known properties<br>Other limitations depend on type of piezometer: see above in table |
| Multipoint push-in piezometer | Provides detailed pressure–depth measurements<br>Simple installation procedure<br>Other advantages depend on type of piezometer: see above in table | Limited number of measurement points<br>Subject to several potential errors: see Section 9.16<br>Other limitations depend on type of piezometer: see above in table |
| Multipoint piezometer, with movable probe (e.g., Figures 9.38 and 9.39) | Provides detailed pressure–depth measurements<br>Unlimited number of measurement points<br>Allows determination of permeability<br>Calibrated part of system accessible<br>Great depth capability<br>Westbay Instruments system can be used for sampling groundwater and can be combined with inclinometer casing | Complex installation procedure<br>Periodic manual readings only |

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

Reliability and durability are often of greater importance than sensitivity and high accuracy. The fact that the actual head may be in error by 1 ft (300 mm) as a result of time lag may not matter in some cases provided the piezometer is functioning properly. If a malfunction occurs, it is of little importance that the apparent head can be recorded to 0.01 in. (0.25 mm). It is generally believed that if the instrument is installed correctly, that it is functioning, and that no time lag remains, the accuracy of all piezometers can be within 6 in. (150 mm) of water head. High-accuracy requirements of course necessitate selection of high-accuracy components, such as *test quality* pressure gages in pneumatic readout units.

### 9.12.2. Use of Observation Wells

As indicated in Section 9.1, applications for observation wells are very limited. They create a vertical connection between strata, and their **only** application is in continuously permeable ground in which groundwater pressure increases uniformly with depth. This condition can rarely be assumed. In the view of the author, observation wells are often misleading and should be used only when the groundwater regime is well known. If the groundwater regime is well known, measurements may be unnecessary: therefore, a practitioner must have a strong argument in favor of an observation well before selecting this option.

### 9.12.3. Use of Open Standpipe Piezometers

For measurements of pore or joint water pressure, an open standpipe piezometer is the first choice and should be used provided that the time lag and other limitations listed in Table 9.1 are acceptable. When these limitations are unacceptable, a choice must be made among the remaining piezometer types.

### 9.12.4. Short-Term Applications

Short-term applications are defined as applications that require reliable data for a period of a few years, for example, during the typical construction period. When open standpipe piezometers are unsuitable, the choice is generally between pneumatic and vibrating wire piezometers. The choice will depend on the factors in Table 9.1, on the user's own confidence in one or the other type, and on a comparison of cost of the total monitoring program.

When the economics of alternative piezometers are being evaluated, the **total** cost should be determined, considering costs of instrument procurement, calibration, installation, maintenance, monitoring, and data processing. The cost of the instrument itself is rarely the controlling factor and should never dominate the choice. Until recently, vibrating wire piezometers were preferable to pneumatic piezometers when automatic acquisition of data was required, but now both types can be read with automatic data acquisition systems, and signals can be transmitted using the various communication systems described in Chapter 8.

Twin-tube hydraulic piezometers are rarely chosen for short-term applications: their primary role is for long-term monitoring of pore water pressure in embankment dams. Resistance piezometers are the only type capable of monitoring pore water pressures that are varying with high dynamic frequency, such as during earthquakes or while piles are driven nearby. However, the author does not favor their use except under circumstances where frequent checks on zero reading and calibration can be made. They provide a convenient method for monitoring and recording pressures during short-term tests, such as pumping tests to determine permeability in situ, and in such cases frequent calibration presents no difficulties.

### 9.12.5. Long-Term Applications

For long-term applications, selection criteria are similar, but twin-tube hydraulic piezometers become an attractive option. General recommendations for long-term applications are given in Chapter 15. Recommendations specific to long-term monitoring of pore water pressure in embankment dams are given in Chapter 21.

### 9.12.6. Detailed Monitoring of Pressure–Depth Profile

When detailed monitoring of the pressure–depth profile is required, multipoint piezometers will normally be the instruments of choice. Comparative information is given in Table 9.1.

### 9.12.7. Filter Requirements

When selected for installation in saturated soil or rock, all piezometers should have low air entry filters. Open standpipe and twin-tube hydraulic piezometers that are sealed into boreholes below the water table require low air entry filters so that the seal can be checked after installation by performing permeability tests. The permeability of the filter should be at least ten times greater than the permeability of the ground. There is no benefit in using a high air entry filter on a diaphragm piezometer installed in saturated soil or rock and, if such a filter is used and not fully saturated, readings will be incorrect.

## 9.13. RECOMMENDED INSTRUMENTS FOR MEASURING PORE WATER PRESSURE IN UNSATURATED SOIL

If the pores in a soil contain both water and gas, the pressure in the gas will be greater than the pressure in the water (Chapter 2). In fine-grained soils the pressure difference can be substantial, and special techniques may be required to ensure measurement of pore water pressure rather than pore gas pressure. Applications for pore water pressure measurements in unsaturated soil are generally limited to two situations: first, measurements in compacted fills for embankment dams, and second, measurements in organic soil deposits, in which gas is generated as organic material decomposes. These applications are discussed in turn.

### 9.13.1 Measurements in Compacted Fills for Embankment Dams

When measurements of pore water pressure are required in compacted fills for embankment dams, piezometer selection criteria are similar to those given in Section 9.12. However, open standpipe piezometers are generally unsuitable for installation during construction, because they interfere with fill placement.

Twin-tube hydraulic piezometers and diaphragm piezometers should have high air entry filters to ensure that pore water pressure rather than pore gas pressure is measured.

Twin-tube hydraulic piezometers should have filters sized to minimize response time, and the size used in the Bishop piezometer (Figures 9.12 and 9.13 and Appendix E) is recommended. Penman (1960) has compared the measured response time of the filter used in the Bishop piezometer with that of a smaller filter in the form of a 2 in. (50 mm) diameter disk and reports on the significant advantage of the larger filter. Clearly, it is undesirable for a filter unit to present too small an area to the soil, but a diaphragm piezometer, because it requires a much smaller volume of water to operate its transducer, can tolerate a smaller contact area than a twin-tube hydraulic piezometer.

When required for installation in compacted fills for embankment dams, diaphragm piezometers should either have hollow cylindrical filters (e.g., the Telemac instrument shown in Figure 9.17) or filters emerging at the tapered nose of the piezometer (e.g., the Geonor instrument shown in Figure 9.34), so that the surface of the filter can be installed in intimate contact with the fill. Sherard (1980) reports on measurements with vibrating wire piezometers installed in fine clayey soil at Porto Colombia Dam in Brazil, where all piezometers read zero during a time when the true pressures were clearly subatmospheric. The piezometers, manufactured by Maihak, had disk-shaped filters on their ends. It is believed that, with this arrangement for the filter, it is difficult to avoid entrapping air between the filter and soil; therefore, subatmospheric pressures will not be monitored.

Despite use of saturated high air entry filters, the longevity of filter saturation is uncertain because gas may enter the filters by diffusion. The compacted fill in an embankment dam may remain unsaturated for a prolonged period after the reservoir is filled, and in fact the fill may never become permanently saturated by reservoir water. Increase of water pressure causes air to go into solution, and the air is then removed only when there is enough flow through the fill to bring in a supply of less saturated water. The pressure and time required to obtain saturation depend on the soil type, degree of compaction, and degree of initial saturation. Pore gas pressure may therefore remain significantly higher than pore water pressure for a substantial length of time, perhaps permanently. The vibrating wire piezometer shown in Figures 9.33 and 9.34 has a high air entry filter that is much longer than in other versions, so that filter saturation is maintained for a maximum time. However, even with this long filter, permanent saturation is not ensured.

Twin-tube hydraulic piezometers allow for flushing of the filter and cavity with de-aired liquid, thereby ensuring that pore water pressure continues to be measured, and are preferable to diaphragm piezometers when long-term measurements are required. A more detailed justification for this recommendation is given in Chapter 21.

As discussed in Section 9.15, no sand pocket should be placed around piezometers when they are installed in compacted fills for embankment dams.

### 9.13.2. Measurements in Organic Soil Deposits

When pore water pressure measurements are required in organic soil deposits, difficulties caused by the presence of gas are usually overcome by installing open standpipe piezometers fitted with low air entry filters and with standpipes of adequate diameter to ensure self-de-airing. The use of twin-tube hydraulic piezometers may be limited by a piezometric elevation that will be too far below the elevation of the tubing.

If the response time or other limitation of an open standpipe or twin-tube hydraulic piezometer is unacceptable, a diaphragm piezometer must be used. A short-term method entails use of a diaphragm piezometer with high air entry filter. The

**Table 9.2. Filter and Saturation Requirements for Piezometers**

| | | Saturated Soil or Rock | | Unsaturated Soil | |
|---|---|---|---|---|---|
| Type of Piezometer | Method of Installation | Type of Filter[a] | Need for Saturation[b] | Type of Filter[a] | Need for Saturation[b] |
| Open standpipe | Push-in | Low | Yes | Low[d] | Yes |
| | Sealed within borehole[c] | Low | Yes[e] | Low[d] | Yes[e] |
| | Placed in fill | — | — | Low[d] | Yes[e] |
| Twin-tube hydraulic | Push-in | Low | Yes | High | Yes |
| | Sealed within borehole[c] | Low | Yes | High | Yes |
| | Placed in fill | — | — | High | Yes |
| Diaphragm | Push-in | Low | Yes[f] | High | Yes[f] |
| | Sealed within borehole[c] | Low | Yes[f] | High | Yes[f] |
| | Placed in fill | — | — | High | Yes[f] |

[a] "Low" indicates low air entry. "High" indicates high air entry.
[b] "Yes" indicates that complete saturation is required, including any cavity between filter and diaphragm.
[c] Surrounded by zone of saturated sand.
[d] Minimum inside diameter of standpipe is 0.3 in. (8 mm).
[e] Saturation not required if permeability tests will not be made through the piezometer.
[f] See discussion in text on saturation of filters attached to diaphragm piezometers.

filter and the cavity between filter and diaphragm must be completely saturated before installation but, as described previously, the longevity of the fluid separation is uncertain. Piezometers with long high air entry filters have been made for this purpose, to prolong the time for gas to infiltrate. For longer-term applications, two hydraulic tubes can be connected to the cavity between the filter and the diaphragm (Section 9.8.9), to allow flushing any gas out of the cavity, using the procedure described in Appendix E.

## 9.14. SATURATION OF FILTERS

### 9.14.1. Reasons for Saturating Filters

Saturation of filters serves four major purposes. First, if the pores in the filter are filled with water, the chance of clogging is reduced. Second, if gas is removed from the filter the response time is minimized. Third, saturation allows permeability tests to be made through open standpipe and twin-tube hydraulic piezometers. Fourth, filter saturation is necessary if the filter is to separate water from gas in unsaturated soils.

### 9.14.2. Recommendations for Type of Filter and Need for Saturation

Recommendations for filter type and the need for saturation are summarized in Table 9.2.

Table 9.2 indicates that filters attached to diaphragm piezometers should always be saturated, but there is one exception to this rule. Saturation is not necessary for a diaphragm piezometer that is sealed within a borehole in saturated soil or rock if there is no interest either in minimizing the time after installation during which readings are perturbed by the installation process or in minimizing subsequent response time. Saturation of a diaphragm piezometer should include the cavity between the filter and the diaphragm and of course requires selection of a piezometer that **can** be saturated. Some diaphragm piezometers are available with permanently attached high air entry filters and are supplied by the manufacturers in a dry condition. Such piezometers will cause the measured pressure to be higher than the pore water pressure because of trapped air, and they should not be used.

### 9.14.3. Methods of Saturating Low Air Entry Filters

Low air entry filters can be saturated by forcing water through the pores.

A low air entry filter on an open standpipe piezometer can be saturated by immersing the filter in water and applying a suction to the standpipe, for example, with a bilge pump: when a piezometer is installed in a borehole, this can usually be done within the borehole. A low air entry filter on a twin-tube hydraulic piezometer can be saturated in the same way or the filter can be saturated before attachment to the tubing by plugging one tube and connecting a pressurized supply of water to the other tube. If a low air entry filter on a diaphragm piezometer can be removed from the piezometer, it can be saturated in a similar way. If the filter cannot be removed and saturation is required for minimizing response time, a vacuum should be applied to the dry filter (having determined from the manufacturer that the instrument will not be damaged by the vacuum) and the filter then allowed to flood with water as the vacuum is released. The cycle should be repeated two or three times until no gas bubbles appear from the filter as the vacuum is applied.

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

### 9.14.4. Methods of Saturating High Air Entry Filters

High air entry filters require a more intensive saturation procedure than low air entry filters. Some manufacturers recommend total immersion in water for 24 hours, but this is ineffective because air will not escape.

A high air entry filter for a twin-tube hydraulic piezometer can be saturated, as shown in Figure 9.43, by attaching short lengths of tubing and placing the dry filter in a container of de-aired water for 24 hours. The water will pass through the filter and drive out the air. Even though some air will enter the de-aired water through the water surface, the amount appears to be small during the 24 hour period provided that the container is not moved or the water stirred. Suction can be applied to the tubes to speed up the process, but this is usually unnecessary.

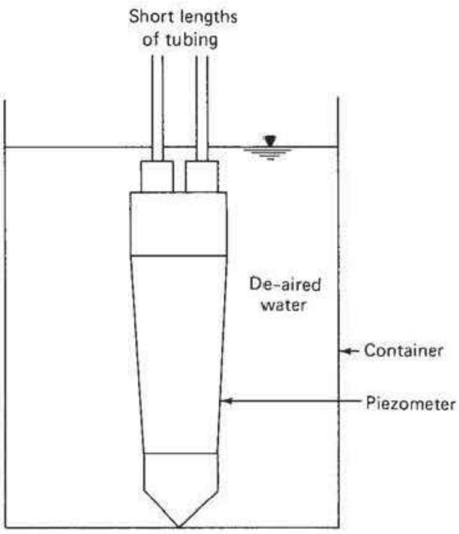



**Figure 9.43. Method of saturating twin-tube hydraulic piezometer.**

A high air entry filter for a diaphragm piezometer can be saturated by detaching it from the piezometer, placing the dry filter in a container, and applying a vacuum. The filter should then be allowed to flood with water, preferably warm de-aired water, allowing it to rise in the container so that it wets the lower side of the filter and rises slowly through it, thereby driving out the air. If the vacuum is high enough, the water will boil. The vacuum should then be released gradually and the filter kept submerged in the water for at least 1 hour. Attempts to saturate a high air entry filter by complete immersion in water and applying suction will often not be successful. Whatever the applied suction, air within the pores of the filter may be retained by the surrounding water and simply expand and contract as the pressure is lowered and raised.

There is no general rule about whether high air entry filters should be saturated at the manufacturer's facility or at the site. Because saturation requires significantly more care and effort than for low air entry filters, there is an argument favoring saturation at the manufacturer's facility. Bordes (1986) reports that, because of numerous problems in the field, Telemac saturates its filters in its workshop and, in addition, that freezing of submerged presaturated filters in a sealed container at −40°F (−40°C) has resulted in no damage. However, if appropriate facilities are available at the site, field saturation may be the more conservative approach.

## 9.15. INSTALLATION OF PIEZOMETERS IN FILL

As indicated in Section 9.6, there has been recent evidence that diaphragm piezometers that are installed in fill can show a reading change caused by total stresses acting on the piezometer body. Diaphragm piezometers that are to be installed in fill should therefore have thick-walled housings or an outer protective shell.

### 9.15.1. Compacted Clay Fill

A trench should be excavated to contain the piezometer tubes or cables, as described in Chapter 17, and piezometers are normally pushed into the wall of the trench. If the clay contains any material larger than 0.2 in. (5 mm), a pocket should be hand-excavated for each piezometer, alongside the trench, and backfilled with screened clay. The pocket is typically 18 × 18 × 30 in. (460 × 460 × 760 mm) deep. The backfill material should have a water content and density similar to the adjacent compacted clay and should be compacted with hand-operated equipment in 4 in. (100 mm) layers.

Because piezometers with high air entry filters will be used, it is essential that a good contact be obtained between the filter and the embankment material. Even a small thickness of loose, poorly compacted soil adjacent to the filter can prevent the piezometer from functioning correctly. Piezometers with conical filters, such as Bishop twin-tube hydraulic tips, are normally installed by pushing a mandrel approximately 12 in. (300 mm) into the clay soil and pressing the piezometer into the hole. The mandrel is exactly the same size as the piezometer and thus the piezometer makes close contact with the soil. As discussed in Section 9.13, diaphragm piezometers should have either hollow cylindrical filters as shown in Figure 9.17 or filters emerging at the tapered nose of the piezometer as shown in Figure 9.34. They should be installed by forming a cylindrical hole in the side of the trench, with diameter slightly smaller than the piezometer body, and forcing the piezometer into the hole. Smearing the filter with a thin coat of saturated soil paste, mixed to a consistency near the liquid limit, is probably desirable to aid in ensuring continuity between the water in the pores of the filter and the pore water in the embankment material (Sherard, 1981). The length of the hole above the piezometer is backfilled with tamped layers of embankment material, after removal of any larger stones.

No sand pocket should be allowed around the piezometer: sand would create a reservoir of air.

Guidelines for installation of piezometer tubing and cables are given in Chapter 17.

### 9.15.2. Granular Fill

Piezometers in granular fill are normally installed by excavating pockets alongside the trench, as described above for compacted clay, and placing the piezometer within backfill in the pocket. In rockfill or coarse granular material, it is necessary to place a zone of coarse clean sand around the piezometer to prevent damage. In rockfill, it may also be necessary to place a graded filter, from coarse gravel to sand, to prevent passage of material through the rockfill.

## 9.16. INSTALLATION OF PIEZOMETERS BY THE PUSH-IN METHOD

Three types of push-in open standpipe piezometer have been described in Section 9.8: wellpoint type, Geonor type, and Cambridge type. Pneumatic, vibrating wire, and bonded electrical resistance strain gage piezometers are also available as push-in versions. Multipoint push-in piezometers have been described in Section 9.9.3.

Installation of push-in piezometers is much less expensive than installation and sealing within a borehole, and therefore they tend to be used without recognition of their limitations. In addition to the limitations given in Sections 9.3–9.9 and summarized in Table 9.1, push-in piezometers are limited by the potential for an inadequate seal, for smearing or clogging, and for false readings caused by gas generation. These limitations are discussed in turn and installation guidelines are given.

### 9.16.1. Sealing

Sealing of a push-in piezometer requires an adequately intimate contact between the soil and pipe immediately above the piezometer.

The open standpipe configuration with wellpoint and steel pipe has a standard coupling immediately above the wellpoint, with a pipe of smaller outside diameter above the coupling. During installation, this coupling scores a hole of larger diameter than the outside of the steel pipe, and adequate sealing assumes that the soil will squeeze in to make intimate contact with the pipe. However, despite recommendations by AASHTO (1976) in favor of this procedure, the author believes that this may be an unwarranted assumption and that the configuration should not be used unless a length of pipe, with minimum diameter equal to the coupling diameter, is attached to the top of the wellpoint. This length of pipe will then be in contact with the soil for sealing purposes.

The drill rods attached to the Geonor field piezometer (Figure 9.23) have an outside diameter slightly larger than the outside diameter of the piezometer and have no protruding couplings; thus, they are in intimate contact with the soil. A minimum of 10 ft (3 m) of EX-size drill rod is generally used when the piezometer is installed in soft sensitive clays. Parry (1971) obtained good data by installing the Cambridge drive-in piezometer (Figures 9.24 and 9.25) in porous deposits underlying clay, using a 1 m (3 ft) sealing length of pipe. Torstensson (1984b) reports that, when using the BAT piezometer, a sealing length of 20 in. (500 mm) has been shown to be adequate for installations in clays, silts, and fine sands. The author has used a push-in open standpipe piezometer for falling head permeability measurements in the core of an embankment dam, with a 6 ft (2 m) sealing length of pipe. The core material was decomposed granite, a clay containing residual grains of angular coarse sand, and leakage occurred along the soil/pipe contact, presumably along channels created by the sand grains.

It is therefore evident that push-in piezometers are not appropriate for all soil conditions. No positive guidelines can be given on the length of the sealing pipe because the length depends on soil conditions, but it appears unwise to economize on the length: the longer the better. When a pipe surrounds the piezometer leads or standpipe, for use during pushing, connections in this pipe should also be sealed so that hydraulic short circuits are prevented. For example, when installing the Geonor field piezometer, all drill rod threads should be sealed. When an open standpipe piezometer is installed by the push-in method, the adequacy of the seal should be checked by allowing dissipation of any pore water pressure generated during installation and then performing a falling or constant head permeability test as described in Section 9.3.6.

### 9.16.2. Smearing and Clogging

Various approaches are taken to prevent excessive smearing or clogging of push-in piezometers during installation.

The Geonor field piezometer has a maximum pore size of 0.03 mm (0.001 in.) and is installed by first filling the standpipe totally with water and plugging the top, so that soil cannot enter the pores as the piezometer is pushed into place. Alternatively, water is pumped down the standpipe (a conventional garden spray tank, fitted with a pressure gage and flowmeter, is suitable) during all except the last 10 ft (3 m) of the push and a valve closed at the top of the standpipe for the remainder of the push. This alternative procedure requires that the pressure supply system is disconnected when each new length of pipe or rod is added during installation but ensures complete filling with water and minimum possibility of clogging. Pore water pressures are generated at the tip as the piezometer is pushed into clay and these must be allowed to dissipate before the standpipe plug is removed, to discourage clogging. The waiting time of course depends on the soil type: in the Norwegian soft sensitive marine clays for which the piezometer was developed, 24 hours is a typical waiting time.

The Cambridge drive-in piezometer is protected from smearing and clogging by use of the sliding driving shoe and by selection of an appropriate mesh size.

When a diaphragm piezometer is packaged within a push-in housing, the filter and cavity between the filter and diaphragm should be thoroughly saturated with water prior to installation, to prevent smearing and clogging.

### 9.16.3. Gas Generation

A further limitation of push-in piezometers results from the potential for gas generation if dissimilar metals are in contact.

DiBiagio (1977) reports that when piezometers were recovered after each use and reused at other sites, they sometimes provided good data and sometimes highly erratic data. The piezometers had vibrating wire transducers housed within stainless steel bodies and were attached to mild steel EX-size drill rod and installed by the push-in method. The drill rods remained in place, so that the mild and stainless steels were in contact. At sites where good measurements were obtained, the soil was quick clay, which has a low concentration of salt in the pores. At sites where erratic measurements were obtained, the clay had a high salt content in the pore water, such that the two dissimilar metals and the adjacent soil formed a galvanic cell (battery circuit), which can influence the measured pore water pressure in two ways. First, the corrosion process may include liberation of hydrogen gas at the stainless steel cathode (the piezometer). Second, the electric current generated by the corrosion process may have an electro-osmotic effect on the pore water. Both these effects tend to increase the pore water pressure around the stainless steel piezometer and also could account for fluctuations in measured pressures. A simple nylon bushing, inserted between the piezometer and the extension rods, was all that was needed to separate them electrically and eliminate the problem: this has been confirmed by numerous subsequent piezometer installations.

There is no reason to believe that this potential error is limited to vibrating wire piezometers. Presumably, the same problem could occur with any type of piezometer that is installed by the push-in method, if the push pipe is left in place.

### 9.16.4. Some Detailed Guidelines for Installation of Piezometers by the Push-in Method

Several details must be addressed when using push-in piezometers.

As a piezometer is pushed through soil, temporary changes in pore water pressure will generally occur around the piezometer. If a diaphragm piezometer is used, and these pore water pressures exceed the measuring range of the instrument, the piezometer may permanently be damaged. Piezometer readings should therefore be made continuously as the instrument is pushed and the rate of pushing controlled as appropriate. The feature shown in Figure 9.27, for checking the calibration of a diaphragm piezometer in-place, allows the application of a controlled backpressure during pushing: this type of piezometer can therefore often be installed more rapidly than a piezometer with a sealed cavity behind the diaphragm.

When installed in clays that are later subject to consolidation, the push pipe or rod will be subjected to downdrag forces, the piezometer tip will be pushed downward with respect to clay around it, and in extreme cases this may cause the piezometer tip to move from one stratum to another. It may be necessary to monitor the changing elevation by surveying methods on the top of the pipe and to interpret data accordingly. If the Cambridge drive-in piezometer is installed in consolidating clays, the downdrag forces will tend to close up the sliding arrangement at the tip, and the gap between the shoulder and driving shoe should be checked periodically by probing inside the standpipe and the driving shoe tapped further down if necessary.

When a piezometer is installed in very soft clay by the push-in method, the drill rod or pipe used for pushing may descend under its own weight after the piezometer is in place, and a steel plate should be clamped around the pipe to rest on the ground surface. This of course aggravates the problem just described, because the piezometer tip will then move downward by the full amount of vertical compression between tip and steel plate.

## 9.17. INSTALLATION OF PIEZOMETERS IN BOREHOLES IN SOIL

When it is not possible or satisfactory to install a piezometer below the ground surface by pushing or driving from the surface, a borehole method is required. The author is aware of seven methods that are in use for installing single piezometers in boreholes in soil, each of which is included in Table 9.3 and outlined in Sections 9.17.1–9.17.7.

The first method entails pushing from the bottom of a borehole, while the remaining six entail placement of the piezometer within a sand pocket and sealing above with bentonite pellets and/or grout. Recommendations and various practical details are given after the methods have been outlined.

### 9.17.1. Method 1. Push-in Below Bottom of Borehole

Installations in very soft to stiff clay can be made by using a variation of the push-in method, employing the instruments described in Section 9.8.7. A borehole is advanced to a short distance above the piezometer elevation and the piezometer pushed into place below the bottom of the borehole.

A borehole is drilled to the planned elevation of the top of the mandrel housing (Figure 9.31), the piezometer saturated, the tubing prespiraled as described in Chapter 17, and a mandrel inserted into the housing. The mandrel consists of a length of

**Table 9.3. Methods for Installing Single Piezometers in Boreholes in Soil**

| Method | Method of Borehole Support | Casing With-drawn | Ability to Take Soil Sample | Sand Around Piezometer | Sealing Method | Advantages | Limitations |
|---|---|---|---|---|---|---|---|
| 1. Push-in below bottom of borehole | Casing or bio-degradable drilling mud | Yes | Yes | No | Grout and contact between soil and mandrel housing | Rapid installation method | Possible only in relatively soft soils<br>Requires special piezome-ter<br>Precautions necessary to prevent excessive smearing or clogging |
| 2. Bentonite pellet seal, casing with-drawn | Casing | Yes | Yes | Yes | Pellets and grout | High-quality seal | Requires significant time for installation<br>Great care needed to clean inside of casing<br>Spiraled tubing or cable cannot be used |
| 3. Bentonite pellet seal, casing left in place (AASHTO method) | Casing | No | Yes | Yes | Pellets and casing/ soil contact | None | Casing not recovered<br>Relies on seal between soil and outside of casing<br>Great care needed to clean inside of casing<br>Risk of damage to piezom-eter if casing is dragged down<br>Problems often caused by sticky bentonite pellets |
| 4. Bentonite pellet seal, casing left in place (Piezometer R&D method) | Casing | No | No | Yes | Pellets and casing/ soil contact | Use of bottom plug keeps casing wall clean and mini-mizes bridging problems<br>Rapid installation method | Casing not recovered<br>Relies on seal between soil and outside of casing |
| 5. Grout seal, casing with-drawn | Casing | Yes | Yes | Yes | Pellets and grout | Rapid installation method | Requires skill to design grout mix<br>Great care needed to clean inside of casing<br>Layer of bentonite pellets recommended over sand to avoid contamination by grout<br>Grouting pressure must be minimized |
| 6. Bentonite pellet or grout seal, using hollow–stem augers | Augers | N/A | Yes | Yes | Pellets and grout | Convenient method if augers are more readily available than casing | Grout seal has same lim-itations as Method 5<br>Great care needed to clean inside of augers<br>Requires large volume of backfill material<br>Augers must not be with-drawn by rotation |
| 7. Bentonite pellet or grout seal, using bio-degradable drill-ing mud | Biodegradable drilling mud | N/A | Yes | Yes | Pellets and grout | Convenient method if local practice favors use of biodegradable drilling mud for site investigation borings | Requires skill and experi-ence to avoid borehole collapse and ensure en-zyme breakdown<br>Grout seal has same limita-tions as Method 5<br>Difficult to ensure that ben-tonite seal is in place if poured into mud-filled borehole<br>Increases response time of diaphragm piezometers |

appropriately sized drill rod, with a slot milled out of one side to allow passage of the tubing. A nylon line is attached to the top of the housing with a hose clamp and used to hold the piezometer on to the mandrel as more drill rods are added and the piezometer is lowered to the bottom of the borehole. On contacting the bottom of the borehole, the piezometer is pushed into place using the mandrel, the borehole filled with a cement/bentonite grout pumped down the drill rods as they are withdrawn, and the borehole topped up with grout. The borehole may be either cased temporarily, withdrawing casing without rotation, or supported by biodegradable drilling mud (Section 9.17.7).

### 9.17.2. Method 2. Bentonite Pellet Seal, Casing Withdrawn

The steps for this method are as follows:

1. Drive the casing, without a drive shoe, to 1 ft (300 mm) below the elevation of the bottom of the piezometer, taking a soil sample at the piezometer elevation, and wash until the water runs clear. The washing is extremely important because, if clay adheres to the inner wall of the casing, backfill materials will stick to the casing and form a plug. The *reverse circulation method,* described in Section 9.17.8, should be used.
2. Pull the casing 6 in. (150 mm) and pour in enough sand to fill the borehole below the bottom of the casing. Check the borehole depth.
3. Repeat step 2. Lower the piezometer to the top of the sand.
4. Pull the casing 6 in. (150 mm) and pour sand to fill the borehole below the bottom of the casing. Repeat until the sand and casing are 1 ft (300 mm) above the top of the piezometer, checking the depth after each pour.
5. Pull the casing 6 in. (150 mm) and pour compressed dry bentonite pellets to fill the borehole below the bottom of the casing. Repeat until a 4 ft (1.2 m) layer of pellets is in place, checking the depth after each pour.
6. Fill the casing with cement/bentonite grout, withdraw all casing, without rotation, and top up the borehole with grout.

### 9.17.3. Method 3. AASHTO Method: Bentonite Pellet Seal, Casing Left in Place

The method is described by AASHTO (1984) and is based on the procedure described by Casagrande (1949, 1958). For the reasons given later in this section and in Section 9.17.8, this method is not recommended for use but is included here for completeness. The steps for this method are as follows:

1. Drive 2 in. (50 mm) diameter casing, without a drive shoe and without a coupling in the lowest 10 ft (3 m), to 1 ft (300 mm) below the elevation of the bottom of the piezometer, taking a soil sample at the piezometer elevation. Do not wash in advance of the casing as the last 10 ft (3 m) are driven. After driving is complete, wash until water runs clear, using the *reverse circulation method.*
2. Pull the casing 1 ft (300 mm) while pouring in enough sand to fill the borehole below the bottom of the casing. Check the borehole depth. Lower the piezometer to the top of the sand.
3. If less than 3 ft (900 mm) of settlement is anticipated, pull the casing 1 ft (300 mm) while pouring in enough sand to fill to a level 30 in. (760 mm) above the bottom of the casing. Check the borehole depth. If more than 3 ft (900 mm) of settlement is anticipated, the casing is not pulled during this step.
4. Pour in a 1 in. (25 mm) layer of 0.5 in. (13 mm) rounded pebbles and tamp.
5. Pour in a 3 in. (76 mm) layer of bentonite pellets, followed by a 1 in. (25 mm) layer of pebbles, and tamp.
6. Repeat step 5 four times, to form a five-layer seal.
7. Pour in a 2 ft (600 mm) layer of sand, cover with pebbles, and tamp.
8. Repeat steps 5 and 6 to form a second five-layer seal.
9. Fill at least 10 ft (3 m) of the casing with sand.

The method suffers from several disadvantages. The cost of each installation includes the cost of casing left in place. The method relies on the adequacy of the seal between the soil and a 10 ft (3 m) length of casing. Although this is no doubt justified for installations in soft clay as described by Casagrande (1949), there may be doubts about adequacy in other soils. If the bottom of the casing is above the piezometer tip and the natural ground above the tip is subjected to vertical compression, the casing will be dragged down with respect to the tip, possibly resulting in damage to the installation. Blocked standpipes have been reported on several projects where significant vertical compression has occurred, indicating kinked standpipes. The provisions of step 3 are supposed to overcome this problem, but the author believes that the casing should not be pulled during step 3 even when as little as 6 in. (150 mm) of settlement is anticipated.

The method was developed before the advent of compressed dry bentonite pellets, and bentonite was prepared by mixing powdered bentonite with water to the consistency of putty, and rolling into small balls. Tamping was necessary to ensure a good seal, and pebbles were needed over each layer of bentonite to prevent bentonite from sticking to the tamping hammer, but use of pebbles risks damage to the piezometer leads while tamping. As discussed later, compressed dry bentonite pellets do not need tamping and do not need a layer of pebbles.

### 9.17.4. Method 4. Piezometer R&D Method: Bentonite Pellet Seal, Casing Left in Place

A variation on Method 3 is described by Piezometer R&D (1968, 1983). The major variations are as follows:

- Use of a temporary bottom plug in the casing, thus ensuring that the inside of the casing is clean. Sand can therefore be poured inside the casing before it is pulled, without risk of forming a "sand-grab." All casing ends are reamed to discourage bridging.
- Use of plastic centering fins around the piezometer to allow the piezometer to be installed half in and half out of the lower end of the casing, so that when casing is dragged downward as vertical compression occurs the piezometer and/or standpipe will not be damaged.
- Use of 2.5 in. (63 mm) instead of 2 in. (50 mm) casing.
- Use of compressed dry bentonite pellets, without pebbles.

The primary steps for this method are as follows:

1. Ensure that all casing ends are reamed and that the bottom section is 10 ft (3 m) long without threads on the bottom end. Insert a special steel and rubber plug in the bottom end. Drive casing to 2 ft (600 mm) below the bottom of the piezometer, adding water as each new section is connected.
2. Lower a tamping hammer to rest on the plug, and tap lightly. A solid "feel" indicates that no soil has bypassed the plug. Verify the casing depth. Pull the casing 3 in. (76 mm), and ensure that the plug leaves the casing by observing no upward movement of the tamping hammer cable. (If the plug follows the casing, it is likely that artesian conditions or excess pore water pressures exist, and the procedure described in Section 9.17.8 should be followed.) Remove the tamping hammer.
3. Pour in sufficient sand to fill 2 ft (600 mm) of the borehole.
4. With the casing water-filled, lower the piezometer, fitted with plastic centering fins, to rest on the sand.
5. Pour in sufficient sand to fill around the piezometer and 2 ft (600 mm) of borehole above the piezometer.
6. Pull the casing 2 ft (600 mm) plus half the length of the piezometer. The sand and piezometer will remain at their original elevation.
7. Pour in a 3 in. (76 mm) layer of bentonite pellets and tamp.
8. Repeat step 7 four times, to form a five-layer seal.
9. Pour in a 2 ft (600 mm) layer of sand.
10. Repeat steps 7 and 8 to form a second five-layer seal.
11. Fill at least 2 ft (600 mm) of the casing with sand. The remainder of the casing is not filled.
12. Install a rubber plug in the top of the casing, with the piezometer lead passing through a central hole and clamped above the plug.

Use of this method overcomes most of the problems associated with bridging of backfill, because the inside of the casing is flush and clean. However, no soil sample can be taken, and use of the bottom plug creates a closed-ended driving method and concern for unwarranted soil disturbance. However, Piezometer R&D (1983) report on many tests to evaluate the effect of the plugs, by installing pairs of piezometers 5 ft (1.5 m) apart, one by the bottom plug method and one by the more conventional method of driving casing and washing out thoroughly. Falling head permeability tests and piezometer readings as fill was placed over the piezometers indicated no difference in behavior.

### 9.17.5. Method 5. Grout Seal, Casing Withdrawn

The method is described by USBR (1974) and is essentially the same as Method 2, except that grout replaces the bentonite pellets.

The Corps of Engineers (1971), Fetzer (1982), and USBR (1974) suggest various grout mixes: details are given in Chapter 17. Lambe (1959) describes use of a chemical grout and, although the grout used by Lambe is no longer acceptable for environmental reasons, other chemical grouts are a viable option provided that the advice of grouting specialists is obtained and that the chemicals have no adverse effects on piezometer materials. Lambe recommends a thin layer of bentonite pellets above the sand to prevent the grout from invading the sand around the piezometer tip, and in general a thin grout should be avoided for this reason. A general and significant concern with any grout seal is the possibility of grout mixing with the sand around the piezometer as it flows under pressure from the end of the tremie pipe, and sand has on occasion been washed upward and out of the borehole. Grout pressure should therefore be minimized, and a grout pipe with a bottom plug and several side ports is preferable to an open-ended grout pipe.

Vaughan (1969) indicates that in relatively permeable material it is not difficult to select a grout with a lower permeability than the surrounding material, but in an impermeable material this may not be possible. For example, the permeability of a pumpable cement/bentonite grout may be on the order of $5 \times 10^{-8}$ cm/sec, whereas the in situ permeability of an overconsolidated clay may be as low as $2 \times 10^{-9}$ cm/sec. However, Vaughan describes a method for calculating the reading error when the grout has higher permeability than the ground and indicates that the error is minor if the two differ by one order of magnitude.

Whenever one is designing a grout mix for sealing piezometers in boreholes, the recommendations given in Chapter 17 should be followed.

### 9.17.6. Method 6. Bentonite Pellet or Grout Seal, Using Hollow-Stem Augers

The method is essentially the same as either Method 2 or 5, depending on the selection of sealing material, except that hollow-stem augers are used instead of drill casing.

### 9.17.7. Method 7. Bentonite Pellet or Grout Seal, Using Biodegradable Drilling Mud

Biodegradable mud* is composed of an organic polymer (no bentonite) that self-destructs through enzyme breakdown with time, leaving only the water used to mix the mud in the borehole. This type of drilling mud provides the necessary hole stabilization and cutting removal during drilling and then, upon reverting to water, leaves the borehole free of the organic polymer that was originally added to the water, providing a suitable environment for groundwater pressure measurement.

The primary steps for this method are as follows:

1. Mix the biodegradable mud with water until it is free from lumps.
2. Advance a borehole to 1 ft (300 mm) below the elevation of the piezometer.
3. Flush the borehole with "clean" mud, to remove any clay that has mixed with the mud while advancing the borehole. This clean mud should be as thin as possible, so that placement of backfill material is not obstructed, but not so thin that the borehole wall collapses. Clearly, this step requires judgment and experience.
4. Proceed with installing the sand, piezometer, and either a bentonite pellet or grout seal, generally as described above for Method 2 or 5.
5. If standpipe piezometers are used, "develop" the completed installation by clearing any filter cake of silt or clay from the borehole wall alongside the piezometer. Outward flow from the piezometer is usually not successful. The preferred method involves using compressed air to blow water out of the standpipe, allowing formation water to flow inward and break down the filter cake. If diaphragm piezometers are used, the filter cake cannot

*For example, Revert. See Appendix D.

be removed and hydrodynamic time lag is likely to be increased.

Where potable water supplies can be affected, a mud formulation that meets environmental protection standards must be used.

Biodegradable mud that is supplied in liquid form is easier to mix but is expensive and needs an additive to trigger breakdown. Quality control tends to be difficult, and this material is not recommended.

### 9.17.8. Recommendations for Installing Single Piezometers in Boreholes in Soil.

The author believes that, with the exception of Method 3, all of the above seven methods are viable. Each method may be the method of choice, the selection depending on the specifics of each application. Users are therefore advised to study the descriptions of the seven methods, Table 9.3, the many details included in Section 9.17.8, to answer the questions listed in Table 9.4, and to plan a method most suited to their particular case. The planning should include preparation of written step-by-step procedures, including a detailed listing of required materials and tools, following the guidelines given in Chapter 17.

Reservations about Method 3 stem from the possibility of damage to the piezometer when casing is dragged down, use of small-diameter casing and hand-rolled sticky pellets that aggravate the bridging problem, use of pebbles that may damage the piezometer leads, and reliance on the seal between the casing and soil. The author believes that Method 3 should not be used.

It appears that, if the soil is sufficiently clayey so that a seal can be assured between the soil and the outside of the casing and if the longer response time is acceptable, the push-in method using the Geonor field piezometer or Cambridge drive-in piezometer will perform as well as Method 4 and will be more economical. If adequacy of the seal is in doubt, none of these methods should be used.

Various practical details for installing piezometers in boreholes in soil are given below.

#### *Role of Drillers and Specialist Personnel*

Drilling crews should not be relied on to install piezometers in boreholes unless specialist personnel are intimately involved: the task should be a team effort.

**Table 9.4. Some Questions to be Answered when Selecting a Method for Installing Single Piezometers in Boreholes in Soil**

1. What is the soil type?
2. Can a piezometer be pushed from the bottom of the borehole, and is sufficient lead time available for procuring special piezometers?
3. Are there artesian conditions?
4. Are there excess pore water pressures?
5. What are the local drilling customs?
6. What drilling methods are available?
7. What skill/care/experience is available among personnel who will be responsible for installations?
8. Is adequate skill available for designing an appropriate grout mix?
9. How much vertical compression will occur in the soil above the piezometer?
10. Are soil samples required?
11. How can the borehole be supported?
12. Can the soil be relied on to seal against the outside of the casing?
13. How much time is available for installation?
14. How much time is available between completion of installation and the need to establish zero readings?
15. What are the requirements for response time?
16. What borehole diameter will be used?
17. How will the casing or augers be prepared?
18. How will the casing or augers be cleaned?
19. How will the casing or augers be backfilled?
20. How will the casing or augers be pulled?
21. If required, what will be the design of the sounding hammer?
22. Do bentonite pellets swell adequately, with or without a retarder?
23. What is the required waiting time for swell?
24. Is there a need to retard the swell and, if yes, what method will be used?
25. Do pellets behave satisfactorily when tested in flowing water?
26. Is one of the alternative bentonite seals preferable to compressed dry bentonite pellets?
27. Is there a need to form a seal below the piezometer?
28. If artesian conditions or excess pore water pressures exist, how will they affect the installation?
29. Which method of installation is most appropriate?

#### *Borehole Diameter*

Borehole diameter will be controlled primarily by the size of the piezometer or leads, the need for an annular space adequate for backfilling purposes, sampling needs, and the availability of drilling equipment. Boreholes drilled with hollow-stem augers may be as large as 10.2 in. (260 mm) in diameter, but casing as small as 2 in. (50 mm) has been used, for example, by Casagrande (1949), although this minimizes the space available for backfilling and is not recommended. A common borehole diameter in the United States is 3 in. (76 mm), and NW casing is often used.

### *Guidelines for Preparation and Cleaning of Casing and Augers*

When casing is used with Method 2, 3, 4, or 5, all casing ends should have a flush inside diameter so that backfill material will not bridge. If standard pipe is used as casing, the inside wall will normally turn inward at the ends to form a sharp protrusion, and ends must be reamed.

When Method 2, 3, 5 or 6 is used, the casing or augers must be cleaned very thoroughly before sand is poured. If this is not done, sand and bentonite pellets are likely to stick to the casing or augers and prevent a good installation. The most effective method is to reverse the jetting pump and to use the jet pipe as the intake, with its 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 in this way the velocity of the outgoing water is maximized.

### *Guidelines for Backfilling and Pulling Casing and Augers*

General guidelines are given in Chapter 17.

When backfill is placed around piezometers as casing or augers are pulled, they should be pulled in short increments, to avoid collapse of the borehole prior to backfilling. They should be pulled without rotation, to avoid spiraling piezometer leads. The installation methods outlined above refer to increments of as little as 6 in. (150 mm), and this is a good starting point for each installation. However, experience with each installation will indicate whether larger increments are acceptable or whether smaller increments are necessary.

For all methods except Method 4, pulling of casing should precede backfilling with solid material, because if backfill is allowed to settle within the casing it may "grab" the piezometer leads, causing the piezometer to be lifted when casing is pulled. The same applies to the use of hollow-stem augers. An exception to this rule, at a site where borehole caving was of great concern, is described by Fetzer (1982). A standpipe piezometer with 3 in. (76 mm) diameter PVC flush jointed standpipe was installed through 6 in. (150 mm) casing. Sand was poured to a level as high as 3 ft (1 m) above the bottom of the casing before the casing was pulled, but the piezometer could be held in place by weighting the top. This would not be possible with a less robust standpipe or with another type of piezometer.

Volumes of solid backfill should be controlled by using a small container with a volume equal to a known depth of borehole, remembering to calculate required volumes using the **outside** diameter of casing or augers. Quantities of Ottawa sand and compressed bentonite pellets, required to fill various sizes of borehole, are given in Figure 17.9.

Sand should be saturated with water before pouring and should be poured slowly to avoid bridging in the borehole. Any washed and screened sand between U.S. standard sieve size #20 and #40 (approximately 0.034 and 0.017 in.; 0.9 and 0.4 mm) is suitable and is usually available from suppliers of sand blasting materials. Screening and washing concrete sand is usually impracticable. Ottawa sand is often used, but it should be noted that this term does not define the gradation, and gradation limits must be included in specifications. No sand finer than #40 sieve size should be included because this will take too long to settle and will mix with the sealing material. These gradation requirements are made in consideration of filter criteria and must be maintained for sand around open standpipe piezometers. However, the upper size limit can be relaxed for diaphragm piezometers, because no significant flow of water occurs. The lower size limit should be maintained to ensure that sand falls to the bottom of the borehole. Sand falls through water in the borehole at a rate of about 3 seconds per foot (10 seconds per meter).

Bentonite pellets should be poured very slowly, and the best way to avoid bridging is to drop them individually at a rate of about 3 per second and to allow enough time for their descent before measuring borehole depth. They fall at a rate of about 1 second per foot (3 seconds per meter). If given sufficient space to do so, compressed dry bentonite pellets will swell to between 10 and 15 times their original volume, and they should **not** be tamped. If they are tamped, the dry pellets may be forced into the soil beyond the walls of the borehole, where they are not required. Moreover, as discussed later in this section, if tamping causes the pellets to assume a denser packing, expansion of the pellets against the soil may create artificially high piezometer readings for a substantial time after installation. The rule is: *make sure that the pellets are where they should be, and leave them alone.* The adequacy of a seal created by this method can be demonstrated by placing one layer of pellets at the bottom of a glass filled with water, and observing how they swell and form a complete seal around the glass. No pebbles are needed over a layer of compressed bentonite pellets. It is absolutely essential that pellets are allowed to expand before overlying material is placed, otherwise overlying material may descend through the pellets and prevent a seal.

When pouring sand or bentonite pellets in the borehole, the piezometer leads should be kept taut to discourage bridging, and depth measurements should be made repeatedly to verify correctness of the backfill level.

When a grout seal is used (Method 5, 6, or 7), care must be taken to avoid contaminating the sand with grout. It appears worthwhile to place a 6 in. (150 mm) layer of bentonite pellets above the sand and to allow them to expand before grouting, for a time determined while testing pellets in water as described later in this section. If a stiff grout has been selected, with a slump of about 4 in. (100 mm) or less, the bentonite pellets are not needed. In either case, a grout pipe with a bottom plug and several side ports should be used, and care should be taken to pump grout slowly and to raise the grout pipe slowly.

### *Sounding Hammer*

A *tamping hammer* is often used to tamp the bentonite seal and to center the piezometer leads or standpipe. In fact the tamping hammer serves two more purposes: to measure depths and to assist in dislodging any bridges of backfill material that may form in the borehole. As indicated above, compressed bentonite pellets should **not** be tamped. However, the device is required for the other three purposes: centering, depth measurement, and dislodging bridges. By persisting with the term *tamping hammer,* users are encouraged to use the device for tamping, and the author proposes the new term *sounding hammer.*

The need for centering the leads or standpipe within a bentonite pellet seal has been demonstrated during laboratory tests by Guarino (1985). When a single lead ran continuously along the inside wall of a pipe, filling the pipe with bentonite pellets and flooding with water did not result in a good seal.

A sounding hammer is required whenever placing sand and whenever using a bentonite pellet seal and is advisable for centering purposes when using a grout seal around all piezometer leads except exterior coupled standpipes.

When used around a single piezometer lead or a multiple-lead with a single jacket, the sounding hammer should be a steel cylinder with outside diameter 0.3–0.5 in. (8–13 mm) less than the inside diameter of the casing or augers, inside diameter approximately 0.2 in. (5 mm) larger than the outside diameter of the piezometer lead, a length of not less than 2 ft (600 mm), and a weight of approximately 15 lb (7 kg). The diameter limits are necessary so that backfill is forced to the bottom of the borehole, the length limit so that the hammer does not become angled and stuck in the borehole, and the weight limit for handling purposes. For small-diameter boreholes, the hammer can be made from solid steel, but usually it is necessary to weld one pipe inside another, with donut-shaped top and bottom plates, to keep the weight adequately low. If the required outside diameter cannot be achieved by using standard steel pipe, the next smaller pipe size can be used and the top and bottom plates made to protrude beyond the outside diameter by the required amount. All corners on the hammer should be thoroughly rounded. A bridle should be attached to the top of the hammer and a cable attached to the bridle with a secure smooth connection that will not damage the piezometer lead. A 0.125 in. (3 mm) stainless steel airplane cable, covered with a plastic sheath, is ideal for the bridle and cable. The cable should be graduated, using a hot stamping machine, with zero at the bottom of the hammer. Figure 9.44 shows a typical sounding hammer.

The arrangement shown in Figure 9.44 requires that the sounding hammer is inserted over the entire length of piezometer lead: a laborious task for long leads. This can be overcome by milling a slot along one side of the cylinder so that the lead can be inserted sideways. Removable top and bottom plates, each with a slot, are attached to the cylinder with their slots rotated 180 degrees with respect to the slot in the cylinder, so that they can also be inserted sideways but can retain the lead within the cylinder.

When a piezometer has two separate leads, for example, a pneumatic or twin-tube hydraulic piezometer for which a common tubing jacket has not

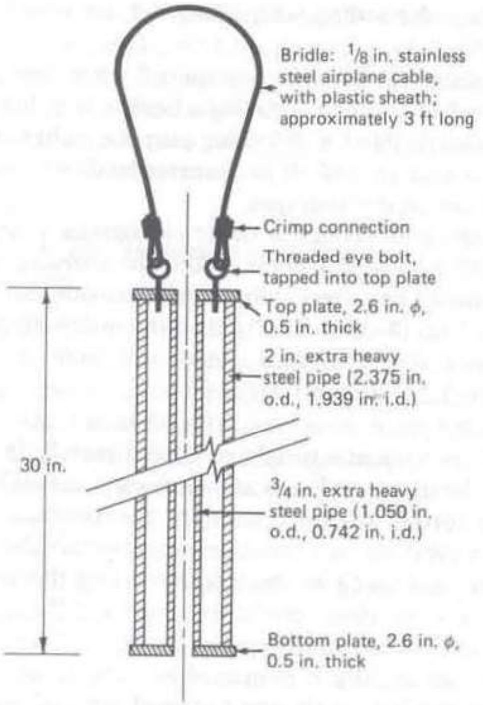



**Figure 9.44. Typical sounding hammer for use inside NW flush-joint casing (casing i.d. 3.0 in.). Note: (1) plates and pipe welded together; (2) all corners well rounded; (3) weight approximately 15 lb (7 kg); (4) graduated cable attached to bridle with loop and crimp connection.**

been used, the tubes must be separated by the sounding hammer. If the arrangement for a single lead is used, the two leads will be forced into close contact and a seal may not form along the contact. Separation is achieved by use of a bottom plate with two holes or by welding two separate small-diameter pipes within a larger pipe.

There is a temptation to use a sounding rod rather than a sounding hammer, because a rod is much easier to use. However, this practice is likely to push the standpipe or leads to one side of the borehole and may result in an inferior seal (Guarino, 1985). Bozozuk (1960) describes a sounding hammer consisting of an enlarged foot attached to pipe, and both foot and pipe are inserted over the piezometer lead. While a suitable method for shallow holes, the need for repeated coupling and uncoupling of pipes makes this a more laborious procedure.

### *Bentonite Pellets*

Compressed dry bentonite pellets have been shown to form an adequate seal (Filho, 1976) provided that

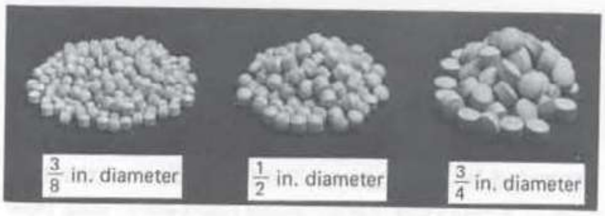



**Figure 9.45. Bentonite pellets (courtesy of Piezometer Research & Development, Bridgeport, CT).**

they can be inserted in the borehole at the required location. Typical pellets are shown in Figure 9.45.

The swelling properties of bentonite pellets depend both on the properties of the constituents of the pellets and on the chemistry of the water in which they are immersed. Users should always verify that the pellets swell to at least five times their original volume when placed in a container of water for 24 hours. The water should be from the source that will be used in the field. During this test a judgment should be made on the length of waiting time required before overlying material can be placed without risk of the material descending past the bentonite. A typical waiting time is 30 minutes.

Most people who have installed piezometers in boreholes with bentonite pellets have experienced the problem of pellets becoming stuck part way down the borehole, and the problem is aggravated if pellets are poured too fast. Fetzer (1982) reports that, when installing an open standpipe piezometer with a 3 in. (76 mm) flush coupled standpipe within a 6 in. (150 mm) casing, pellets became lodged in the annular space. Some manufacturers add retarding agents to their pellets.* Users have tried a variety of methods for retarding the onset of swelling and stickiness. Brief soaking in diesel fuel, hydraulic oil, alcohol, or varnish has sometimes been successful but on other occasions has caused pellets to break apart or to become sticky very rapidly. Other methods include coating with hair spray, or soaking in a shellac solution and allowing to dry. Biodegradable drilling mud is also used: the powder is mixed with water, using no more than 0.1 lb per U.S. gallon (12 g/liter), which creates a very viscous liquid. The mixture is allowed to hydrate for at least 2 hours before briefly soaking the pellets and placing them in the borehole. Other users have tried freezing pellets before use. When there is a need to re-

*For example, Piezometer Research & Development "Non-stick" pellets.

tard the onset of swelling and stickiness, no unique procedure can be recommended for all pellets and all water, and users must therefore make their own tests.

When making such tests, pellets should be from the same shipment as those to be used in the field, because pellets do not always have uniform properties. The water should be from the field source. Although swelling can be demonstrated by placing pellets in a container of water, this is an inappropriate test for investigating the onset of swelling and stickiness when pellets are falling through water, because the action of flowing water has a marked effect on behavior. Piezometer Research & Development base their reported pellet behavior on tests in flowing water, and the author recommends this approach. A short length of transparent pipe, with diameter similar to the diameter of the borehole, is placed vertically and plugged at its lower end.* A water supply is connected to an opening in the plug, the pipe filled with water, pellets inserted at the top, and the water flow controlled so that pellets remain at about the midheight of the pipe. This arrangement is now a good model of field conditions, pellet behavior can be observed, and the effect of retarders can be evaluated. The time required to achieve a particular observed condition can be recorded and converted to an equivalent borehole depth by using the measured flow of water spilling from the top of the pipe.

If a retarder is employed, users should verify that pellets swell to at least five times their original volume when placed in a container of water for 24 hours.

Commercial sources of bentonite pellets are given in Appendix D.

### *Alternative Bentonite Seals*

The above methods have referred to use of compressed dry bentonite pellets, installed by pouring into the borehole, but several other methods have been used, one of which may be the method of choice when planning an installation.

Casagrande (1949) describes use of bentonite balls of 0.5 in. (13 mm) diameter, formed by mixing powdered bentonite to a putty-like consistency, rolled in talcum powder to minimize sticking, and stored in glass jars to prevent drying. This procedure has in general been superseded by the advent

*The test apparatus is available from Piezometer Research & Development. See Appendix D.

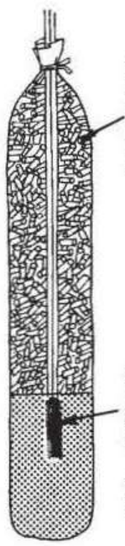



**Figure 9.46. Piezometer with integral sand pocket and bentonite pellet seal (courtesy of Thor International, Inc., Seattle, WA).**

of compressed dry bentonite pellets but may be used if the manufactured pellets are not available. However, as described in Section 9.17.3, tamping is needed, as also is a layer of pebbles to prevent sticking to the sounding hammer. Despite great care during installation, pellets tend to stick to the casing, and a heavy residue of bentonite tends to remain in the casing, obstructing the passage of pellets and reducing the quality of the seal. This method should be avoided wherever possible.

When piezometers are installed offshore or in other environments where groundwater is saline, conventional compressed bentonite pellets will not swell adequately unless special precautions are taken. Attempts to solve the problem with pellets made by compressing salt water bentonite appear to have been unsuccessful, and better success has been achieved by creating a local freshwater environment in which the conventional pellets can swell and remain swollen. This has been done successfully by using Method 4 and filling the casing with fresh water. Alternative approaches are to use a push-in method, Method 1, Method 5, 6, or 7 with a suitable grout, or a chemical sealant (Driscoll, 1986; Senger and Perpich, 1983).

Pellets can be prepackaged within a cylindrical bag as shown in Figure 9.46. The sand-filled part of the bag will normally be of canvas and the upper part of a coarser mesh. The arrangement must be sized to match the diameter of the borehole and the required lengths of sand and seal zone and substantially decreases the time and effort required for seal placement. As with all bentonite seals, sufficient waiting time must be allowed for swelling before placing overlying materials.

Pellets can be tremied to the bottom of the borehole by adding them to flowing water, using the method described in Chapter 17 for backfilling boreholes with sand or pea gravel, but with a larger-diameter pipe. A 1.5 in. (38 mm) Schedule 80 PVC pipe with flush threaded couplings is inserted within the borehole as the piezometer is lowered, 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 circulated down the pipe until it spills out at the top of the borehole. Pellets are 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 same tremie pipe arrangement can be used for placing sand.

Bentonite gravel (Appendix D) is often used in eastern Canada in preference to bentonite pellets. The gravel ranges in size from 0.3 to 0.8 in. (8–20 mm) and is significantly less expensive than compressed pellets.

Peters and Long (1981) describe the development of bentonite rings, installed with the aid of a modified tamping hammer around open standpipe piezometers in 6 in. (150 mm) diameter boreholes. Each bentonite ring was made by compacting bentonite at 75% water content in a 3.5 in. (90 mm) diameter by 6 in. (150 mm) long cylindrical mold and forcing a 0.8 in. (20 mm) rod through the center. Each ring was lowered around the standpipe with the special tamping hammer shown in Figure 9.47. To compact a ring, the hammer was lowered to the bentonite ring and the sliding steel weight raised and dropped. A series of four to six compacted rings constituted a seal. This method appears to be an attractive alternative to bentonite pellets but necessitates an effort by the user to prepare the rings and special hammer.

"Volclay sausages" are available from American Colloid Company (Appendix D), consisting of granular bentonite packaged within soluble plastic tubes 2 in. (50 mm) in diameter by 22 in. (560 mm) long. They are designed for curing loss of circulation in wireline drilling operations by using a downhole grouting technique. The author is not aware of their use for sealing piezometers, but smaller-diameter tubes appear to have a potential for this purpose.

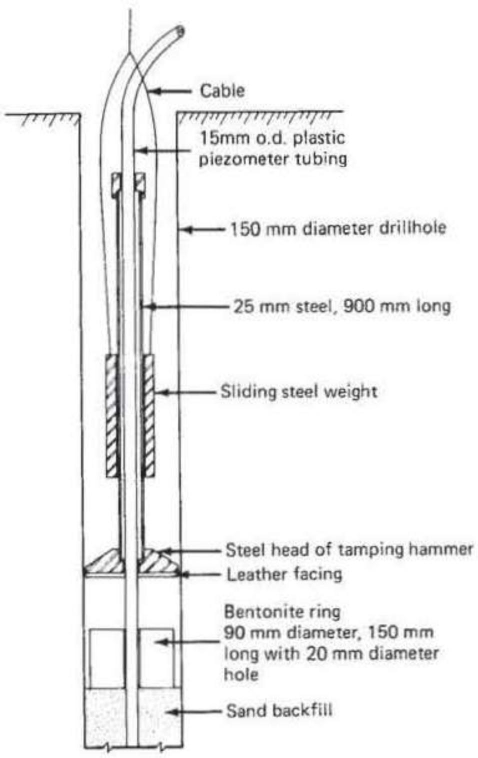



**Figure 9.47. Tamping hammer and bentonite ring seal (after Peters and Long, 1981).**

### *Chemical Sealant*

A chemical sealant has been developed by the Dowell Division Laboratory of Dow Chemical Company, Tulsa, OK, for sealing groundwater monitoring wells where the groundwater is highly mineralized (Senger and Perpich, 1983). The sealant contains a polymer that swells in water and can be preformed by pouring into a mold of appropriate size and allowing it to set overnight to a consistency of soft rubber. Swelling time can be adjusted by changing the formulation of the sealant.

Dowell chemical sealant is used in the Waterloo Multilevel System (Figure 9.37) and is being considered as a borehole seal during studies relating to disposal of high-level nuclear waste.

This chemical sealant appears to have applications for sealing conventional piezometers in boreholes, although the author is not aware of practical experience to date. Because of the very substantial effort involved in ensuring that compressed dry bentonite pellets are placed in the borehole at the required location, the alternative of an easy-to-place chemical sealant is extremely attractive. Users are encouraged to conduct comparative tests between seals made with compressed dry bentonite pellets and with chemical sealant and to report their findings to the profession.

### *Change of Pore Water Pressure Caused by Swelling of Bentonite*

The high-expansive property of bentonite pellets clearly has the potential for causing changes of pore water pressure. Depending on the soil conditions, confining conditions, and the bulk density of the pellets in place, there is a potential for an increase in pore water pressure as the pellets expand against the soil, but on the other hand there is a potential for a decrease in pore water pressure as the pellets hydrate and draw water from the pores.

Casagrande (1949) indicates that the purpose of the sand below, within, and above the bentonite seals in Method 3 is "to minimize the effect of swelling pressures of the overlying bentonite." The sand also increases frictional forces between backfill and casing, thereby discouraging a "blow-out" of the seal.

Binnie & Partners (1979) warn that "compressed bentonite pellets, which swell to many times their original size . . . can artificially reduce the pore pressures for long periods and should, therefore, only be used with caution." Guarino (1985) has made two sets of tests to examine this hypothesis. In the first set, a sand zone containing a piezometer was placed in the bottom of a closed-ended pipe, followed by a layer of pellets and water. Pressure was applied to the water and the piezometer monitored. In every case the piezometer reading decreased as hydration of the bentonite progressed. In the second set of tests, a piezometer was installed in a 12 in. (300 mm) diameter clay-filled pipe, in an attempt to model field conditions. A sand zone was provided around the piezometer and pellets above. The magnitude of pressure changes and the elapsed time before equilibrium pressure was established depended on the method of installation of the pellets. Use of a tamping hammer to pack the pellets in place caused excessively high readings of pore water pressure, in some cases for more than 1 month after installation. When pellets were poured into the "borehole" but not subsequently tamped, the piezometer readings decreased as hydration progressed, but equilibrium pressure was always established within 1 day. It is apparent from these tests that the rule is: *make sure that the pellets are where they should be, and leave them alone.*

In general, whenever bentonite pellets are used, seals should be installed well ahead of the need for measurements of pore water pressure so that any changes caused by the swelling of bentonite have had time to dissipate.

### *Sealing Below Piezometer*

When a piezometer is installed at the bottom of a borehole, no seal is needed below the piezometer. However, if the borehole has been advanced below the elevation where measurements are required, a seal must be placed up to the bottom of the sand zone. Sealing options and procedures are the same as for the seal above the piezometer, but care must be taken to prevent sealing material from contacting the wall of the borehole where the sand zone will be placed, usually by protecting the zone with casing.

### *Installation Under Artesian Conditions*

When a piezometer is to be installed under artesian conditions or where excess pore water pressures exist, arrangements must be made to place the seal while water is not flowing up the borehole and to overcome the problem of soil rising up inside the casing or augers. The most straightforward method is to extend the casing or augers to a level above the piezometric level and to counterbalance the flow with a static head of water, but this can only be done for a limited excess head. The author is aware of four methods for coping with the problem for larger excess heads.

First, in appropriate soil conditions a push-in piezometer can be installed.

Second, Method 4 can be used. Under artesian conditions the casing plug tends to follow the casing when the casing is first pulled. An extension is added to the casing, filled with water to minimize any flow of water around the casing plug, the lower sand, piezometer, and upper sand placed within the casing, the casing pulled to its final position, and the extension casing removed. The remainder of the procedure is as described in Section 9.17.4.

Third, Kinner and Dugan (1982, 1985) describe installation of piezometers offshore in sands and gravels in which the piezometric elevation ranged from 25 to 35 ft (8–11 m) above mean sea level. The arrangement is shown in Figure 9.48. The boreholes were advanced with heavy-weight conventional drilling mud. The wellpoint, riser pipe, and packer

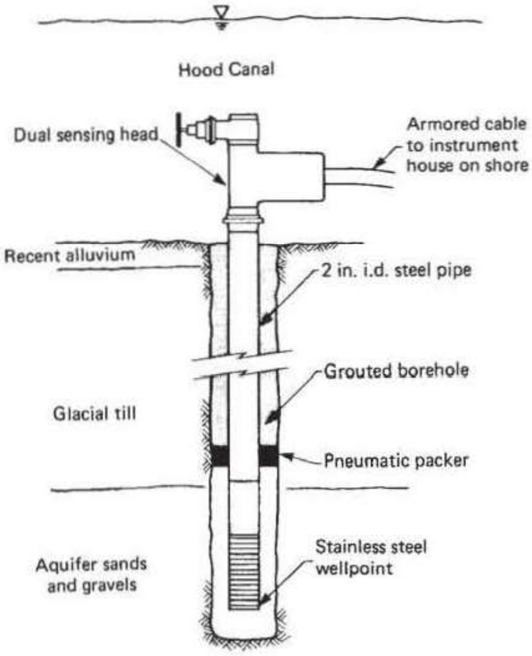



**Figure 9.48. Piezometer installed in soil under artesian conditions (after Kinner and Dugan, 1982).**

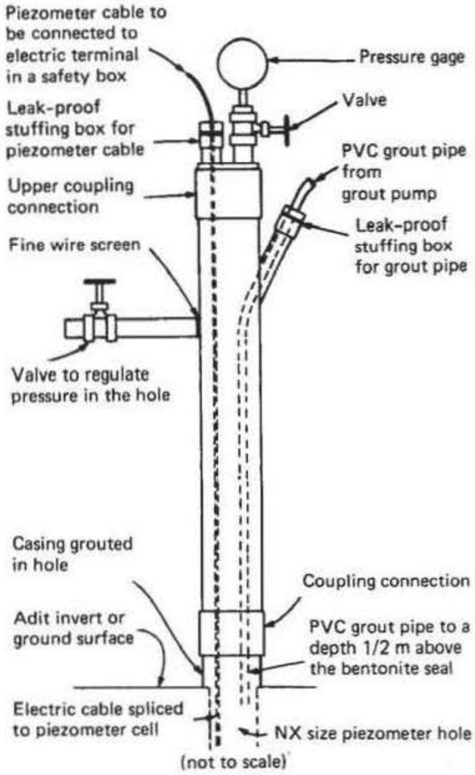



**Figure 9.49. Special equipment for installation of a piezometer under artesian conditions (after Logani, 1983).**

were inserted into the mud-filled borehole, the packer inflated, and the borehole grouted. After grout setup, the wellpoint and riser pipe were purged with water until artesian flow was generated, thereby cleansing the system of drilling mud. While the sensing head was being attached, the valve at the top of the assembly was opened to allow water flow and closed after making the connection. The dual sensing head contained both a pneumatic transducer and a single hydraulic tube and was packaged in this way to allow replacement of the head by a diver if damage or malfunction occurred. For a more conventional on-shore installation, a diaphragm piezometer could be set within the standpipe with an inflatable packer.

Fourth, Logani (1983) describes a method for installing piezometers under artesian conditions in rock, which could perhaps be adapted for use in soil provided that the borehole could be supported adequately during installation. The equipment shown in Figure 9.49, but without the grout pipe and piezometer cable, was attached to the borehole collar and the artesian pressure recorded. The pressure regulating valve was opened to allow free water flow and to release artesian pressure, and coarse sand was poured into the borehole by removing the upper coupling. The upper coupling was replaced and the pressure regulating valve closed, allowing the sand to settle under no-flow conditions. After again opening the valve and removing the upper coupling, the piezometer, placed within a saturated sand-filled pervious bag, was lowered into the borehole, followed by more sand, using the same procedure to allow the sand to settle. A 2 m (7 ft) layer of bentonite pellets was placed in increments in the same way and the grout pipe lowered within the borehole. The borehole was filled with a cement/bentonite grout under pressure to counter the artesian head. At this site the artesian pressure was as much as 40 ft (12 m) above the ground surface. Logani (1985) and Reyes (1985) both describe methods for installing piezometers under higher artesian pressures, by using a two-valve pressure loading chamber for introducing backfill material into the borehole under very low or no-flow conditions.

### *Some "Don'ts"*

Never advance the borehole using conventional drilling mud (unless using the method shown in Figure 9.48 for an installation under artesian conditions), as this will seal the soil and inhibit piezometer response.

When casing is withdrawn after placing a bentonite seal, never fill the remainder of the borehole with sand or drilling cuttings. Always use a cement/bentonite grout. The practice of filling the remainder of the borehole with sand presumably has its origin in the procedure described by Casagrande (1949), in which the sand is bounded by casing. If the casing is withdrawn and sand is placed, conformance has been violated by creation of a pervious column that allows drainage of pore water pressure and may feed surface runoff into the zone around the piezometer.

When installing diaphragm piezometers, do not allow any damage to the outer sheath of the tubing or cable. Pneumatic piezometer tubing often consists of individual tubes with a common sheath, and in many cases water can travel between the tubes and the sheath. Electrical piezometers may have cable that allows "wicking" inside the sheath. If the sheath is damaged just above the piezometer, a pathway may be created for drainage of pore water pressure and/or for water ingress to the transducer.

Whenever planning to use a bentonite pellet seal, do not allow any subsurface couplings in plastic tubes, connections in electrical cables, or exterior couplings in steel or plastic pipes. Apart from creating weaknesses, these protrusions interrupt the passage of the pellets and sounding hammer.

### 9.17.9. Installation of More than One Piezometer in a Borehole in Soil

#### *Adaptations of Methods for Installing Single Piezometers*

It is generally agreed that if more than one piezometer is installed in a borehole in soil by adapting one of the seven methods described previously, seals may be inferior. Two piezometers can be installed successfully if great care is taken and if each piezometer has only a single lead, but more than two are not recommended. For this purpose multiple tubes encased within a common jacket are considered as single leads.

When two piezometers are installed in a borehole, clearly Methods 1, 3, and 4 are unsuitable. The lower piezometer is installed by using one of the remaining four methods, and a seal must then be formed up to a level just below the upper piezometer. Bentonite pellets can be used if the distance is not great, but more normally the intervening space is grouted, and it is generally necessary to allow the grout to set overnight. The upper piezometer is then installed in the same way, with special attention to the seal around and between the two leads. When a bentonite pellet seal is used, the sounding hammer must ensure separation between the two leads, as described in Section 9.17.8. When a grout seal is used, spacers should be placed between the leads to ensure that grout seals the intervening space: a short length of small-diameter PVC pipe can be taped every few feet to the lead for the upper piezometer. Because these spacers interfere with placement of a bentonite pellet layer between sand and grout, a low slump grout should be used to avoid contamination of the sand.

When more than one open standpipe piezometer is installed in a single borehole, acceptance testing should include a verification of seal integrity by adding water to each standpipe in turn and verifying that the others do not respond.

#### *Installation of Multipoint Piezometers*

Section 9.9 describes several methods for installing more than two piezometers in a borehole in soil, using multipoint piezometers. Methods include use of inflatable or chemical packers, piezometers surrounded with grout, push-in piezometers, and movable probes. Advantages and limitations of the methods are given in Table 9.1.

## 9.18. INSTALLATION OF PIEZOMETERS IN BOREHOLES IN ROCK

As indicated in Chapter 2, measurements of joint water pressure in rock must be made within zones of open jointing. The core should be examined carefully, and additional in-hole studies such as water pressure tests using packers, sonic velocity profiles, borehole camera photographs, and impression packers may be required. Piezometers must be installed within a section of the borehole of sufficient length to intersect several discontinuities, and the borehole should be flushed thoroughly with water to clean out the fractures prior to placing backfill.

### 9.18.1. Installation of Single Piezometer in a Borehole in Rock

If support is required for the borehole, a single piezometer can be installed by using Method 2, 5, or

7, as described previously for boreholes in soil, but with longer sand zones. Simplified versions of Methods 2 and 5 are suitable for installing a single piezometer when support is not required.

### 9.18.2. Installation of More than One Piezometer in a Borehole in Rock

*Adaptations of Methods for Installing Single Piezometer*

As described in Section 9.17.9 for boreholes in soil, two piezometers can be installed successfully in a borehole in rock by using Method 2, 5, or 7, with longer sand zones, if each piezometer has only a single lead and if great care is taken.

More than two piezometers have been installed by using alternate layers of granular backfill and grout. Although not acceptable in soil because seals beween three or more leads are likely to be inferior, in rock the interval between piezometers will tend to be long, allowing use of long grout seals and overcoming concerns for leakage along the seals. Two methods are possible.

First, if the borehole requires support, biodegradable drilling mud and an adaptation of Method 7 can be used, with a grout seal. An example of this approach is described by Deardorff et al. (1980), who installed three pneumatic piezometers per hole in 6.5 in. (165 mm) diameter boreholes up to 900 ft (275 m) deep. A single 1.5 in. (38 mm) diameter steel pipe was used both as a grout pipe and for circulation of drilling mud. Gravel was poured into the borehole to create the pervious zone around each piezometer, and the grout was stiff enough to provide a firm base to support successive gravel zones and grout without waiting for individual seals to harden. The writers indicate that grout penetration into the gravel was negligible provided that the gravel was given adequate time to settle through the drilling mud, and provided that care was taken to pump grout slowly and to raise the pipe slowly after completion of a grout stage. Deardorff et al. (1980) also describe installation of pneumatic piezometers in boreholes drilled downward at an angle of 40 degrees to the horizontal and upward at an angle of 45 degrees. Packers were used as plugs while grouting the upward holes.

Second, if the borehole does not require support, an adaptation of Method 5 can be used. An example of this approach is described by Lang (1983), who installed four open standpipe piezometers per hole in 1200 ft (365 m) deep boreholes drilled with HQ wireline equipment. A 0.5 in. (13 mm) diameter PVC pipe was used to deliver alternate layers of sand and grout down the borehole.

Both of the above procedures are painstaking and laborious, and use of a multipoint piezometer is likely to be more economical.

*Installation of Multipoint Piezometers*

Section 9.9 describes several methods for installing more than two piezometers in a borehole in rock, using multipoint piezometers. Methods include use of inflatable or chemical packers, piezometers surrounded with grout, and movable probes. Advantages and limitations of the methods are given in Table 9.1. In almost every case where more than two piezometers are required in a single borehole in rock, a multipoint piezometer system with inflatable or chemical packers, using either several single piezometers or a movable probe, is the system of choice.

### 9.18.3. Installation Under Artesian Conditions

When a single piezometer is to be installed in a borehole in rock under artesian conditions, it may be possible to set an inflatable packer in an aquaclude above the piezometer, thereby stopping the flow and allowing the borehole above the packer to be grouted. Alternatively, either the third or fourth method described in Section 9.17.8 (Kinner and Dugan, 1982, 1985; Logani, 1983, 1985; Reyes, 1985) can be used. Multipoint piezometers with inflatable packers can also be used in this application.

# CHAPTER 10

# MEASUREMENT OF TOTAL STRESS IN SOIL

## 10.1. INSTRUMENT CATEGORIES AND APPLICATIONS

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

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

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

Most earth pressure cells are designed to measure static or slowly varying stresses only. When cells are required for seismic or large-scale dynamic loading studies, they must be designed to have a sufficiently rapid response time.

## 10.2. EMBEDMENT EARTH PRESSURE CELLS

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

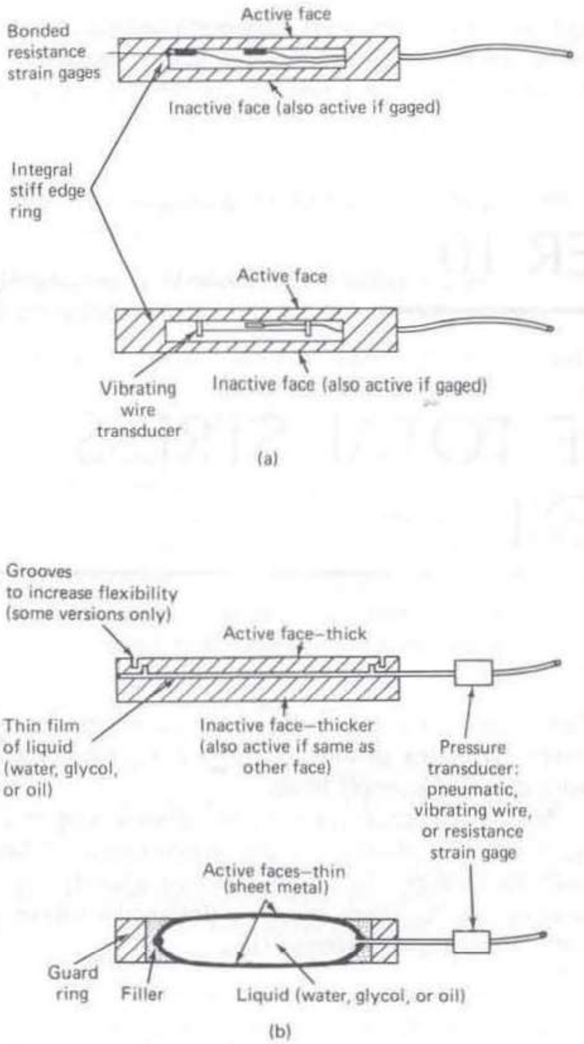



Figure 10.1. Basic types of earth pressure cell: (a) diaphragm cells and (b) hydraulic cells.

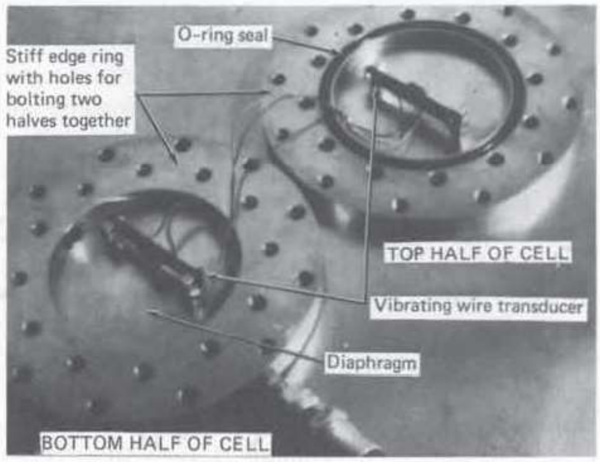



Figure 10.2. Diaphragm earth pressure cell with vibrating wire transducers (courtesy of Soil Instruments Ltd., Uckfield, England).

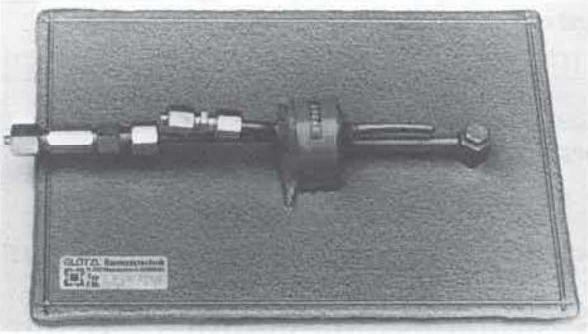
### 10.2.1. Types of Embedment Earth Pressure Cells

There are two basic types of embedment earth pressure cells: *diaphragm cells* and *hydraulic cells*. Examples of the two types are shown in Figure 10.1.

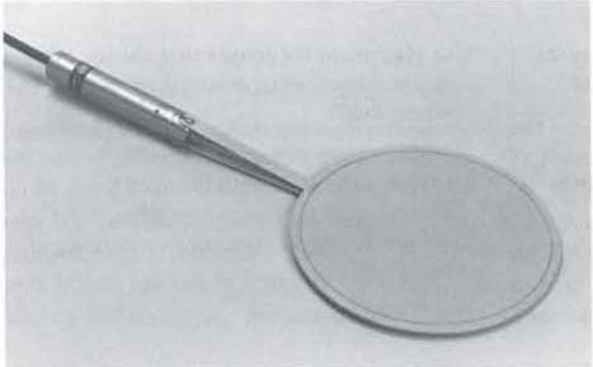
In the diaphragm type, a stiff circular membrane, fully supported by an integral stiff edge ring, is deflected by the external soil pressure. The deflection is sensed by an electrical resistance strain gage transducer bonded directly on the interior face of the cell or by a vibrating wire transducer, as shown in Figure 10.2. The vibrating wire transducer is usually mounted on two posts located at the points of contraflexure of the diaphragm, causing maximum rotation of the posts, magnifying the strain, and creating adequate sensitivity. A diaphragm cell may have one or two independent active faces. In the latter case, the two independent measurements provide an important check on the quality of the installation, in particular whether both faces are in similar contact with the surrounding soil. Thomas and Ward (1969) describe the design, construction, and performance of a diaphragm cell with two active faces and vibrating wire transducers. Induction coil transducers of the soil strain gage type (Chapter 8) have also been used in diaphragm cells.

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

Two versions of hydraulic cell are shown in Figure 10.1. In the first version, also shown in Figure 10.3, the active face is relatively thick (typically 0.1–0.25 in., 2.5–6 mm). Grooves are sometimes machined around the edge to increase edge flexibility, so that the active face tends to work as a piston. In addition, the layer of liquid is thin (0.02–0.08 in., 0.5–2 mm) so that the stiffness of the cell is high and more closely matches that of the surrounding soil, and the installed cell experiences minimal effects caused by thermal expansion and contraction of the liquid. In the second version of the hydraulic cell,



(a)



(b)

Figure 10.3. Hydraulic earth pressure cells with grooved thick active faces. (a) Cell with pneumatic transducer, (courtesy of Glötzl GmbH, Karlsruhe, West Germany and Geo Group, Inc., Wheaton, MD). (b) Cell with vibrating wire transducer (courtesy of Soil Instruments Ltd., Uckfield, England).

both faces are fabricated from relatively thin sheet metal, usually with a rolled edge. The layer of liquid is much thicker (0.1–0.4 in., 2.5–10 mm), so that the cell is less stiff. It is therefore subject to conformance errors in stiffer soils and is more susceptible to temperature effects. A guard ring may be provided to protect the thin sheet metal cell from radial edge loads. The active faces are usually domed outward, which makes installation against a prepared surface more difficult than a flat-faced cell.

Pressure transducer options for both versions of hydraulic cell are the same as for diaphragm piezometers: pneumatic, vibrating wire, and bonded or unbonded resistance strain gage type, with the advantages and limitations given in Chapter 8 and 9.

Comprehensive evaluations of various types of commercially available earth pressure cells, along with criteria for cell design and manufacture, are given by Brown (1977), Corbett et al. (1971), Hvorslev (1976), O'Rourke (1978), Reese et al. (1968), Selig (1964), State of California (1968, 1971), and Weiler and Kulhawy (1978, 1982). A report by the International Society for Rock Mechanics (ISRM, 1981c) includes a comprehensive description of the hydraulic type of cell with a pneumatic transducer, in addition to guidelines on installation, reading, calculation, and reporting procedures.

### 10.2.2. Limitations Imposed by Soil Environment

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

1. An earth pressure cell that will not appreciably alter the state of stress within the soil mass because of its presence (conformance).
2. A large enough sensing area to average out local nonuniformities.
3. Minimum cell sensitivity to nonuniform bedding.
4. A method of installation that will not seriously change the state of stress.

The last requirement generally limits these measurements to fills and other artificial soil conditions. However, some success has been achieved in measuring horizontal stress in soft soils by pushing specially designed earth pressure cells downward into natural ground (e.g., Massarsch, 1975). Other instruments used for this purpose include the stepped blade (Handy et al., 1982), pressuremeters (Baguelin et al., 1972, 1977; Wroth and Hughes, 1973), and the flat plate dilatometer (Marchetti, 1980; Schmertmann, 1982). Earth pressure measurements have also been made in natural ground by hydraulic fracturing through hydraulic piezometers (e.g., Penman, 1975). Wroth (1975) discusses and compares the various methods. Attempts to measure stress in soil by advancing a large-diameter borehole, inserting earth pressure cells, and backfilling around the cells are generally subject to gross conformance errors.

### 10.2.3. Factors Affecting Measurements

Hvorslev (1976), Selig (1964), and Weiler and Kulhawy (1978, 1982) report on studies of factors affecting measurements with embedment earth pressure cells. Table 10.1 is based on a table presented by Weiler and Kulhawy (1982), with substantial re-

**Table 10.1. Major Factors Affecting Measurements with Embedment Earth Pressure Cells**

| Factor | Description of Error | Correction Method[a] |
|---|---|---|
| Aspect ratio (ratio of cell thickness to diameter) | Cell thickness alters stress field around cell | Use relatively thin cells ($T/D < 1/10$) |
| Soil/cell stiffness ratio (ratio of soil stiffness to cell stiffness) | May cause cell to under- or overregister<br>Error will change if soil stiffness changes | Design cell for high stiffness and use correction factor |
| Size of cell | Very small cells subject to scale effects and placement errors<br>Very large cells difficult to install and subject to nonuniform bedding | Use intermediate size of cell: typically 9–12 in. (230–300 mm) diameter[b] |
| Stress–strain behavior of soil | Measurements influenced by confining conditions | Calibrate cell under near-usage conditions[b] |
| Placement effects | Physical placement and backfilling causes alteration of material properties and stress field around cell | Use placement technique that causes minimum alteration of material properties and stress field[b] |
| Eccentric, nonuniform, and point loads | Soil grain size too large for cell size used<br>Nonuniform bedding causes nonuniform loading | Increase active diameter of cell[b]<br>Use hydraulic cells with grooved thick active faces in preference to other types[b]<br>Take great care to maximize uniformity of bedding[b] |
| Proximity of structures and other embedded instruments | Interaction of stress fields near instruments and structure causes errors | Use adequate spacing |
| Orientation of cell | Changing orientation while placing fill over cell causes reading change | Use placement methods that minimize orientation changes<br>Attach tiltmeters to cell |
| Concentrations of normal stress at edges of cell | Causes cell to under- or overregister, depending on stiffness of cell relative to soil | For diaphragm cell, use inactive stiff edge ring to reduce sensitive area ($d/D \approx 0.6$)[b]<br>For hydraulic cell, use grooved thick active face and thin layer of liquid |
| Deflection of active face | Excessive deflection of active face changes stress distribution around cell by arching | Design cell for low deflection: for diaphragm cell, diaphragm diameter/diaphragm deflection at center > 2000–5000; for hydraulic cell, use thin layer of liquid[b] |
| Placement stresses | Overstressing during soil compaction may permanently damage cell | Check cell and transducer design for yield strength (hydraulic cells with pneumatic transducers have high overload capacity)[b] |
| Corrosion and moisture | May cause failure of cell by attacking cell materials | Use appropriate materials and high-quality waterproofing[b] |
| Temperature | Temperature change causes change of cell reading | Design cell for minimum sensitivity to temperature; if significant temperature change is likely, measure temperature and apply correction factor determined during calibration[b] |
| Dynamic stress measurements | Response time, natural frequency, and inertia of cell cause errors | Use appropriate type of cell and transducer, together with dynamic calibration[b] |

[a] $D$ = cell diameter; $T$ = cell thickness; $d$ = diaphragm diameter.
[b] Applies also to contact earth pressure cells. See Section 10.3.2.

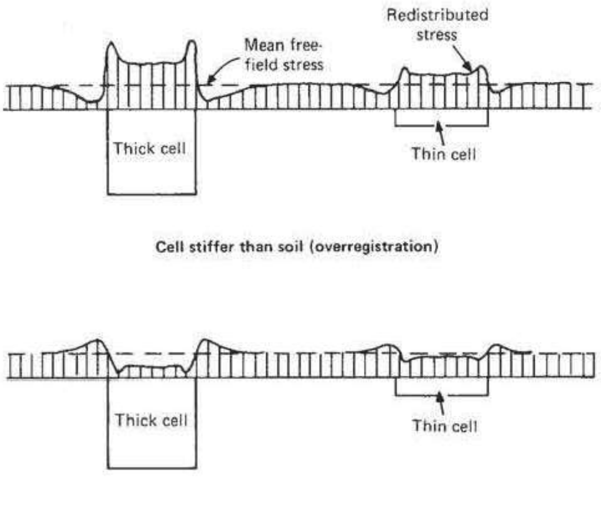



**Figure 10.4.** Effect of embedment earth pressure cell aspect ratio and soil/cell stiffness ratio (after Selig, 1964).

visions by the author, and summarizes the major factors and correction methods. Various factors that affect measurements are discussed in the following subsections.

### *Aspect Ratio and Soil/Cell Stiffness Ratio*

The effects of aspect ratio (ratio of cell thickness to diameter) and soil/cell stiffness ratio (ratio of soil stiffness to cell stiffness) are illustrated in Figure 10.4. The error resulting from these causes can be minimized by designing for high stiffness and an aspect ratio of less than 1:10. It can be seen from Figure 10.4 that if the transducer is mounted above the cell, the error will tend to be larger than if it is mounted at the end of a steel tube emerging through the rim of the cell as shown in Figure 10.3b. Mounting the transducer above the cell provides greater protection for the transducer, but makes compaction of soil around the cell more difficult.

### *Size of Cell*

The typical diameter of cells available for field use is 9–12 in. (230–300 mm). Small cells of 2–3 in. (50–75 mm) diameter are available, but they are not recommended for general field use because scale effects and placement problems are likely to cause greater errors than for larger cells. Very large hydraulic cells (e.g., Alberro and Borbón, 1985; Sparrow, 1967), up to 3 or 4 ft (1 or 1.2 m) in diameter, have been used in limited numbers but, although they may provide measurements of the average stress over a large area and thus be more accurate than cells of conventional size, they are very expensive and awkward to handle and install, and it is very difficult to create a uniform bedding over such a large area.

Figure 10.5 shows stresses measured at Chicoasén Dam in Mexico by installing a group of embedment hydraulic earth pressure cells of different sizes and shapes. Sizes and shapes are identified on

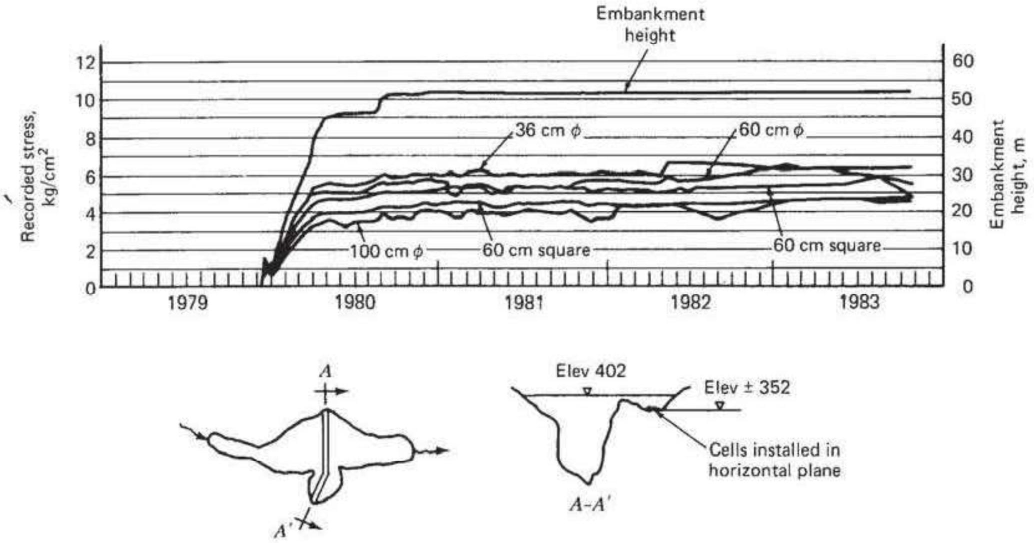



**Figure 10.5.** Measurements with embedment earth pressure cells installed in Chicoasén Dam (after Alberro and Borbón, 1985).

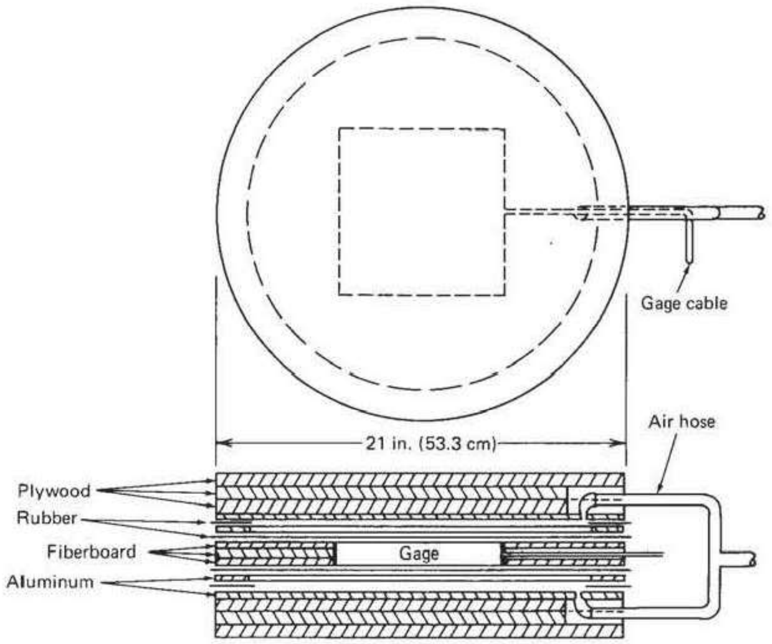



**Figure 10.6. Assembled fluid calibration chamber for earth pressure cells (after Selig, 1980). Reprinted with permission from ASTM *Geotechnical Testing Journal.* Copyright ASTM, 1916 Race Street, Philadelphia, PA 19103.**

the plots. The cells have thin active faces and guard rings, and the layer of liquid is 0.375 in. (9.5 mm) thick. The data show a scatter of about ±25% with respect to the mean value and generally indicate that the measured stress decreased with increasing size of cell.

### *Laboratory Calibrations*

Each cell should be calibrated under fluid pressure to be sure that it is functioning correctly and not leaking, and most manufacturers of commercial earth pressure cells provide a calibration chart based on all-round fluid pressure loading, using air or water. Selig (1980) describes a simple and inexpensive calibration chamber for use with a laboratory load reaction frame or compression testing machine for performing all-round fluid pressure calibrations. The chamber is constructed of easily fabricated layers of metal, plywood, and rubber and assembled as shown in Figure 10.6.

Unless installations are to be made in soft clay, fluid pressure calibrations are insufficient. If measurement accuracy must be maximized, each cell should be calibrated in a large calibration chamber, using the soil in which it will be embedded. Cell design and soil placement details have a very substantial influence on measurements, and during laboratory calibrations it is most important that installation procedures represent the intended field methods as closely as possible. Calibration within large chambers is an expensive and difficult undertaking, and chambers that have been used are described by Alberro and Borbón (1985), Bozozuk (1970, 1972b), Hadala (1967), and Selig (1980). The chamber should be at least three times, and preferably five times, the diameter of the cell.

The chamber shown in Figure 10.7 was used by Hadala at the U.S. Army Corps of Engineers Waterways Experiment Station at Vicksburg, Mississippi, to calibrate 2 in. (50 mm) diameter cells in both sand and clay. The chamber would also be suitable for calibrating the 9–12 in. (230–300 mm) diameter cells that are more typically used in the field. Hadala reports on the effect of the placement method used. The cells satisfied the Table 10.1 recommendations for stiffness and diaphragm deflection, and the aspect ratio was 1:9, yet Hadala concludes the following:

- The placement method used has a definite influence on the mean over-registration ratio and the scatter of the data about the mean. In some cases, over-registration ratio changed as much as 40 per-

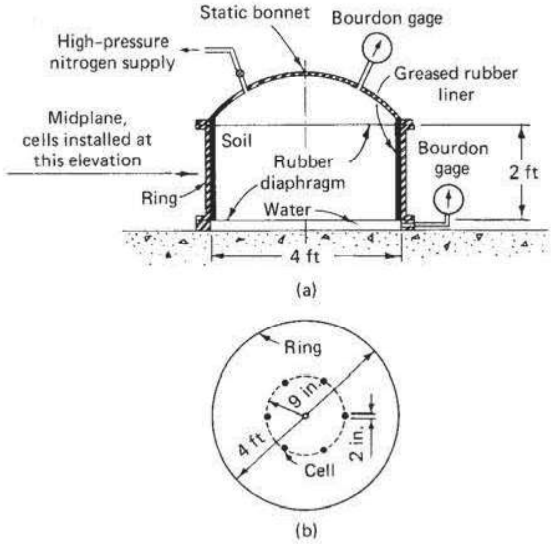



**Figure 10.7. (a) Test chamber for calibration of embedment earth pressure cells and (b) array of cells at midplane (after Hadala, 1967).**

cent due to placement method changes. In the case of scatter, it was noted that the simpler procedures resulted in less data scatter.

- Both over- and under-registration of this very stiff gage are possible, depending on the soil in which it is embedded and the way it is placed.
- The tests in clay generally exhibited lower over-registration ratios than those in sand.
- Of the sand test placement methods, the set-on-surface method [cells simply set on the surface, followed by normal construction procedures to complete the fill] was determined to be the best, while of the clay placement methods examined, the cut/no-cover method [cells placed in a shallow excavation of the same diameter and depth as the cell] was selected as the best.
- The data scatter noted in this study suggests that the use of a single soil stress gage to measure the magnitude of stress in a soil mass with any reasonable degree of confidence is a fruitless effort. When attempting to measure earth pressure magnitudes in virgin-loaded, compacted specimens similar to those used in this study, the average of at least three soil stress gage measurements should be used if 20 percent accuracy is required nine times out of ten.

It is likely that the small size of Hadala's cells aggravated placement effects, and it is believed that the effects are not so severe for more conventional field-sized cells.

Selig (1980) describes a chamber similar to Hadala's and provides practical guidelines on its use for calibrating field-sized cells. Bozozuk's chamber consisted of a heavily reinforced plywood box constructed in two sections, one fitting on top of the other. The cell was installed in the same plane as the junction between the two sections. As each loading increment was applied, the two parts of the box were jacked vertically apart to create a thin, continuous crack separating the two parts of the box. This crack forced all of the applied vertical load to be transmitted past the cell in grain-to-grain contact, since none of the load was lost in wall friction.

The Comision Federal de Electricidad at their Experimental Laboratories in Mexico City have constructed a large laboratory facility to test the response of embedment earth pressure cells to applied loads (Alberro and Borbón, 1985). The diameter of the test chamber is 3 m (10 ft) and the height 3.18 m (10.4 ft). Preliminary calibrations have been made to date in sand, and additional tests are planned to examine the effects of placement procedure, geometry and dimensions of the cells, and cell location and orientation within the chamber.

### *Field Placement Effects*

The above comments on placement effects refer to laboratory calibrations, and field placement effects add an additional source of error that may well be of even greater magnitude. As discussed in Section 10.2.4, the accepted field installation procedure involves compacting fill with heavy equipment, installing the cells in an excavated trench, and backfilling around and over them by hand tamping or light machine. The probability is high that cells are therefore surrounded by a zone of soil with greater compressibility than the remainder of the fill, that imposed stresses are therefore redistributed by arching, and that substantial underregistration occurs.

Binnie et al. (1967) report that when diaphragm cells with two active faces and vibrating wire transducers were installed in washed gravel fill at Mangla Dam, no cell recorded more than half the calculated added vertical stress. Opposite faces generally recorded different pressures, even for initial readings. The initial difference persisted as fill was placed, and in some cases increased so that one face showed a pressure less than half that of the other face. The writers comment that the initial differ-

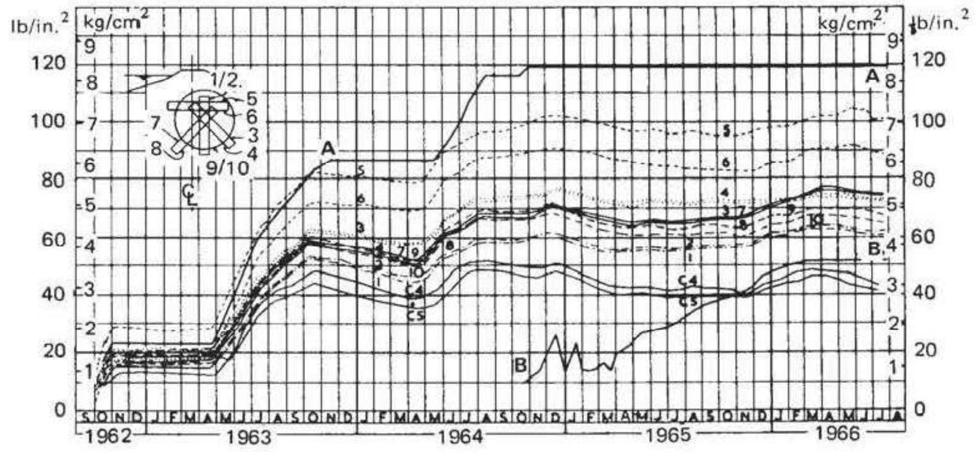



**Figure 10.8. Measurements with embedment earth pressure cells in the clay core at Balderhead dam: A—overburden pressure; B—reservoir pressure (after Kennard et al., 1967).**

ences were probably caused by nonplanar trimming of fill surfaces or uneven backfilling, whereas a part of the later differences may be due to stress variations in the fill caused by the cells themselves.

Kennard et al. (1967) and Thomas and Ward (1969) report on use of diaphragm cells with two active faces and vibrating wire transducers in the clay core of Balderhead Dam. Cells placed vertically showed good agreement between the two faces, as might be expected. During the first two filling seasons the average reading from a cell placed horizontally coincided approximately with the calculated overburden pressure, but thereafter the readings from the upper face increased at a greater rate than those from the lower face. On completion of the dam the average reading was about 80% of the overburden pressure. Data are shown in Figure 10.8.

Wilson (1984) comments that when earth pressure cells are installed in a horizontal plane in compacted fills for embankment dams, the cells typically register only 50–70% of the calculated added vertical stress as embankment construction continues. Wilson also comments that, because it is often difficult to shape the bottom of the excavation to the exact shape of the cell, bedding may be uneven and the application of vertical stress may deform the cell and cause additional error.

It appears to the author that two general observations can be made from a review of reported field placement effects. The first relates to the method of installation, the second to selection of type of cell.

First, although the conventional installation procedure usually prevents damage to the cells, its limitations are clear, and further research on the behavior of embedment earth pressure cells is needed to establish an improved procedure for installation within compacted fills. There is a need to develop a controlled method of field compaction around the cells that prestresses the soil to match the prestress in the remainder of the fill that is compacted by heavy equipment, without damaging the cells. It is hoped that improved installation techniques will result from the tests now in progress in Mexico (Alberro and Borbón, 1985).

The second general observation relates to the effect of nonuniform bedding and leads to a recommendation for selection of type of cell. Diaphragm cells are designed and calibrated for a uniformly distributed load on the active faces, and point loads, stress nonuniformities, or arching will cause significant errors. Hydraulic cells are also subject to errors from these causes but to a lesser extent than diaphragm cells. The best choice appears to be a flat hydraulic cell with thick active face, preferably with grooves to increase flexibility, and a thin layer of liquid.

Thomas and Ward (1969) state that their diaphragm cell, with two active faces and vibrating wire transducers, was designed for use in clay, therefore the effect of non-uniform bedding was not considered important. Uff (1970) demonstrated the high sensitivity of the same cell to uniformity of bedding by installing it as a contact cell, as shown in Figure 10.9, and loading the outer face with a 50 lb (23 kg) point load. Uff found that measurements at the outer face were highly dependent on the point of application of the load, whereas measurements at

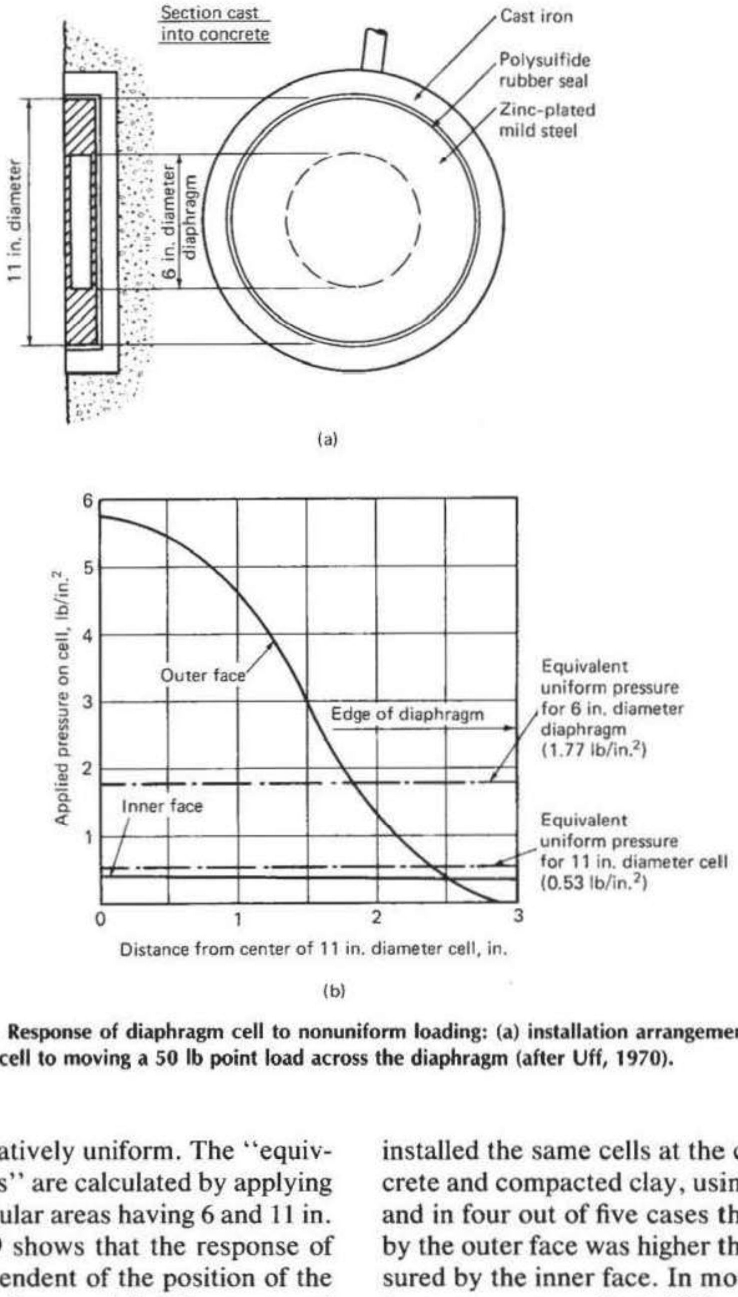



**Figure 10.9. Response of diaphragm cell to nonuniform loading: (a) installation arrangement and (b) response of cell to moving a 50 lb point load across the diaphragm (after Uff, 1970).**

the inner face were relatively uniform. The "equivalent uniform pressures" are calculated by applying the 50 lb load over circular areas having 6 and 11 in. diameters. Figure 10.9 shows that the response of the inner face is independent of the position of the point load on the outer face, and that the measured pressure is close to that calculated assuming the point load is uniformly distributed over a circular area of 11 in. diameter. Binnie et al. (1967) report on substantial underregistration and differences between the two faces when the same diaphragm cells were installed as embedment cells in gravel. Thomas and Ward (1969) report on smaller underregistration and smaller differences between the two faces when the cells were installed as embedment cells in clay. Vaughan and Kennard (1972) installed the same cells at the contact between concrete and compacted clay, using Uff's arrangement, and in four out of five cases the pressure measured by the outer face was higher than the pressure measured by the inner face. In most cases the pore water pressure was about 80% of the measured total stress, and therefore the two faces indicated very significant differences in effective stress. The writers comment that the inner face may well give the more accurate measurement.

This review of reported placement effects on diaphragm cells with vibrating wire transducers makes it clear that they are very sensitive to nonuniformity of bedding.

In contrast, Figure 10.10 shows the results of tests by McRae and Sellers (1986), similar to those

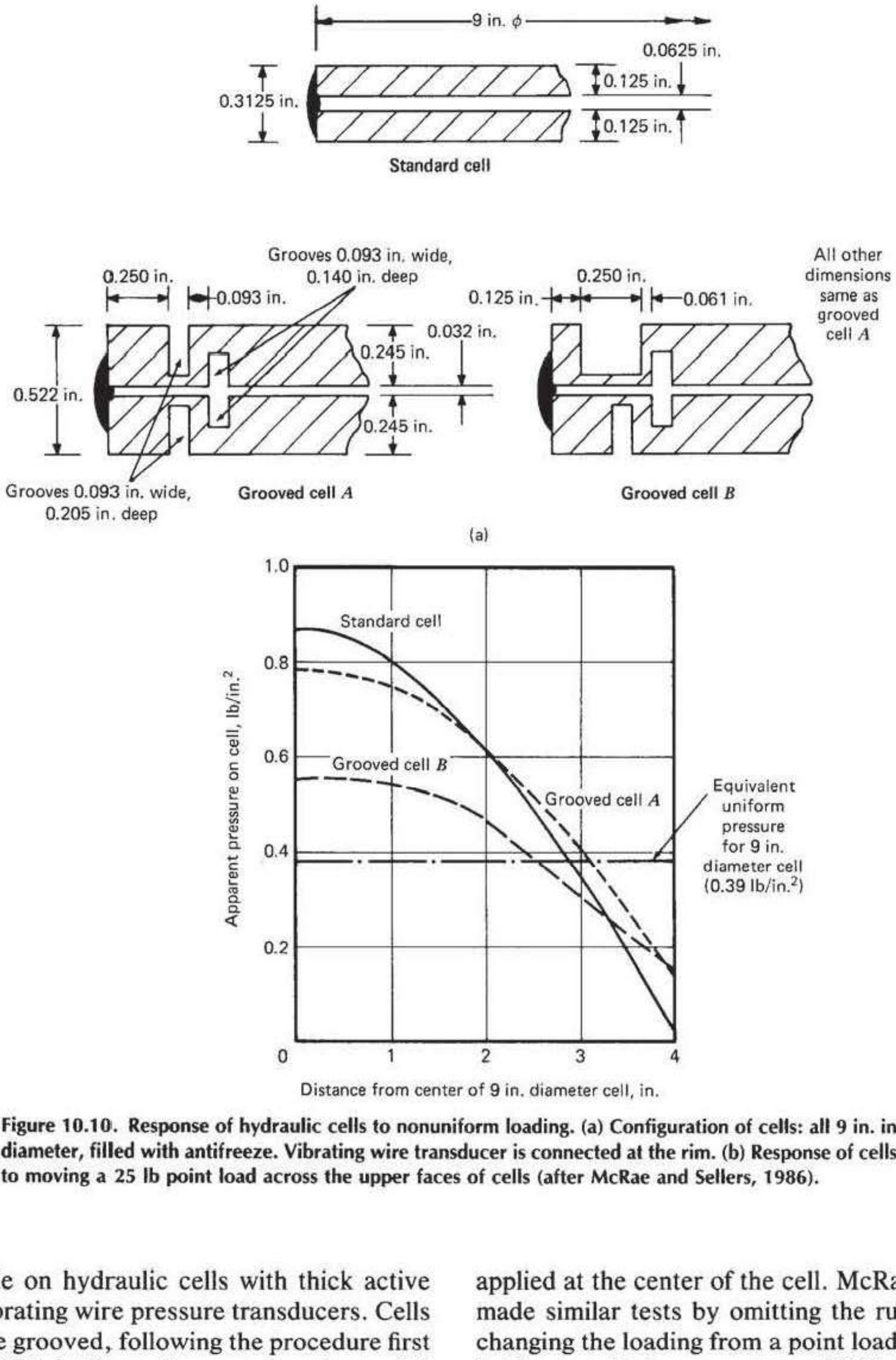



**Figure 10.10. Response of hydraulic cells to nonuniform loading. (a) Configuration of cells: all 9 in. in diameter, filled with antifreeze. Vibrating wire transducer is connected at the rim. (b) Response of cells to moving a 25 lb point load across the upper faces of cells (after McRae and Sellers, 1986).**

by Uff, made on hydraulic cells with thick active faces and vibrating wire pressure transducers. Cells *A* and *B* were grooved, following the procedure first adopted by Glötzl. The cells were placed on a 0.1 in. (2.5 mm) thick rubber pad, resting on a firm flat horizontal surface and loaded on the upper face with a 25 lb (11 kg) point load. When compared with Uff's test results on diaphragm cells, all the hydraulic cells showed less deviation from the "equivalent uniform pressure" when the point load was applied at the center of the cell. McRae and Sellers made similar tests by omitting the rubber pad, by changing the loading from a point load to a 25 lb (11 kg) load applied over an area of 0.5 in.$^2$ (320 mm$^2$), and by using point loads of 13 and 67 lb (6 and 30 kg), all with similar results to those shown in Figure 10.10. The wide groove in the face of cell *B* evidently reduces bending when the cell is subjected to nonuniform loading and clearly is a worthwhile feature. A reasonable minimum thickness of the active face, when grooves are included, appears to be 0.25 in. (6.3 mm).

It is appreciated that all the point loading tests described above are extreme tests and that in practice great efforts are made to minimize nonuniformity of bedding by appropriate field installation procedures. However, since uniformity of bedding cannot be assured, it is apparent that hydraulic cells with grooved thick active faces are preferable to diaphragm cells. More tests are needed to define the optimum grooving pattern. In practice this observation limits the application of diaphragm cells to installations in clay. Even for installations in clay it cannot be said with certainty that the two independent measurements with a diaphragm cell should be preferred to the single measurement with a hydraulic cell. Comparative tests are needed to establish this preference. The cost of a hydraulic cell is typically less than half the cost of a diaphragm cell. Until further comparative data are available, the author favors selection of hydraulic cells with grooved thick active faces for all applications.

Green (1986) suggests that a "best of both worlds" type of hydraulic cell could be created with two back-to-back hydraulic chambers, each with its own pressure transducer. Each outer face would be grooved, and the stiff disk between the two hydraulic chambers would have flat faces. The cell would therefore provide two independent measurements and would have minimum sensitivity to nonuniform bedding.

### *Design of Cell Rim: Diaphragm Cells*

For a diaphragm cell it is important to minimize the effect on the diaphragm of stress concentrations normal to the cell near the rim. An inactive integral stiff edge ring is used at the rim. Peattie and Sparrow (1954) recommend that the ratio of the sensitive area to the total facial area should be less than about 0.45. Typical diaphragm cells with vibrating wire transducers, such as the cell shown in Figure 10.9a, have a ratio of about 0.3.

The diaphragm must deflect, so that a measurable strain is achieved, and thus the effective modulus of the center of a diaphragm cell is much lower than that of the solid steel edge ring. Stress therefore tends to be concentrated on the edge ring.

### *Design of Cell Rim: Hydraulic Cells*

The two versions of hydraulic cell shown in Figure 10.1 will be considered separately.

The cell with a grooved thick active face has a uniform modulus across most of its face, and if the liquid film is thin the modulus can be close to that of the soil. Under these conditions there is no need for an edge ring, and in fact an edge ring would distort the stress normal to the face of the cell.

The cell with thin active faces is generally provided with an edge ring (also called a *guard ring, stiffening ring,* or *confining ring*) for three reasons. First, it adds strength to an otherwise weak and flexible cell. Second, it tends to reduce the sensitivity to temperature changes. Third, it reduces the effect of inward radial stresses on the edge of the cell and the possibility of buckling the thin sheet metal faces.

### *Orientation of Cell*

An additional factor affecting measurements arises if the orientation of a cell changes as overlying fill is placed. DiBiagio (1977) reports on changing orientation of cells installed in a vertical plane in the moraine core of a dam, as indicated by biaxial tiltmeters mounted on the cells. As overlying fill was placed and compacted, the inclination of a cell to vertical typically varied up to 18 degrees by the time the fill surface was 11 feet (3.4 m) above the cell.

### *Consequence of Factors Affecting Measurements: An Overview*

Weiler and Kulhawy (1982) conclude:

> A stress cell which performs well mechanically as installed does not guarantee that the measured stress is representative of the correct free-field stress. In-soil calibration of the stress cells under the conditions expected in the field combined with an understanding of the cell–soil system behavior is essential to achieve good results. The time and expense [and difficulty] of in-soil calibration make the procedure unattractive to most engineers, so the number and accuracy of stress measurements are not high. Further research is needed before the understanding of stress cell behavior is complete, but it is believed that successful stress measurements may be made even now, in soils placed by man, if sufficient time and care are taken in making the stress measurement and in interpreting the results.

The author agrees that successful stress measurements can be made in clayey soils but has doubts about success in sands and rockfill. In contrast to

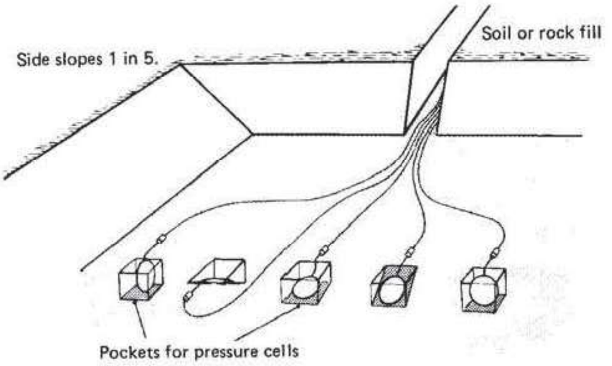



**Figure 10.11. Typical layout of embedment earth pressure cells (courtesy of Soil Instruments Ltd., Uckfield, England).**

the view of Weiler and Kulhawy, Wilson (1984) believes that the error caused by arching of stresses around embedment earth pressure cells can be so great that the very expensive in-soil calibrations (which cannot match site conditions) are often of marginal value.

In summary, the many factors that affect measurements can result in substantial errors, so that measurements with embedment earth pressure cells can rarely be made with high accuracy.

### 10.2.4. Installation of Embedment Earth Pressure Cells

A report by the International Society for Rock Mechanics (ISRM, 1981c) includes detailed recommendations for installation of embedment earth pressure cells. The report recommends the following procedure:

> An excavation to accommodate a cluster of five cells should be as shown in Figure 10.11, with dimensions not less than 13 × 13 × 6 ft deep (4 × 4 × 2 m), with side slopes not steeper than 1 vertical in 5 horizontal. The base of the excavation should be compacted and level. The cells should be individually installed in small pockets at the base of the excavation, each being approximately twice the size of the cell. Pockets should be separated from each other by at least 3 ft (1 m). Cell locations should be marked out, the pockets hand dug very carefully, and trimmed. Protruding stones should be removed and the holes filled with compacted stone-free soil. Each cell should be positioned in its pocket and checked for correct functioning and for alignment and level. The pocket should then be backfilled, whenever possible using the excavated soil, stone-free and at unchanged water content, to a density similar to that of the surrounding soil. The main excavation should then be backfilled with embankment material at unchanged water content, having removed rocks larger than the size of the cell. Three lifts of 4–8 in. (100–200 mm) each should first be placed and compacted by hand-operated equipment before completing the backfill with light mechanical equipment. No heavy vibratory rollers should be used until at least 6 ft (2 m) of fill has been so placed. When installing cells in rockfill, the pocket for each cell should be larger than in soil and should be backfilled with thoroughly compacted material of progressively smaller size, until the soil in contact with the cell is of a grain size less than 0.2 in. (5 mm).

Although the above procedure appears to prevent damage to the cells and to be accepted by many users, the author believes that several improvements are warranted in an attempt to increase accuracy. First, a 3 ft (1 m) deep excavation is preferable to an excavation twice that depth, and vibratory rollers can generally be used after 3 ft (1 m) of fill has been placed over the cells if they have been designed for high short-term overload capacity. Second, the procedure of installing cells in small pockets at the base of an excavation clearly runs the risk of decreasing measurement accuracy. Although no clear recommendations can be made, perhaps a procedure similar to that described in Chapter 17 (Clements, 1982) for installing horizontal tubes and cables in fill by mounding rather than excavating may be workable. Cells would be required to withstand temporary stress caused during compaction, and a robust hydraulic cell appears to be the best choice. The transducer should be housed within a thick-walled steel cylinder and a thick-walled annealed steel tube used to connect the transducer to the cell. The tests currently planned in Mexico (Alberro and Borbón, 1985) may result in improved placement procedures. Third, Wilson (1984) suggests that if a layer of plaster of Paris or weak cement is used as bedding for each cell and the cell worked into the bedding, errors caused by nonuniform bedding may be reduced, but the improvement, if any, resulting from this modification has not yet been verified. Fourth, where knowledge of cell orientation is critical, consideration should be given to attaching two electrolytic levels to the cell, mounted 90° apart.

The author believes that, for installations in rockfill, the free-field stress may be so perturbed by installation that measurements made with cells as small as 9 in. (230 mm) in diameter may not be worthwhile. Larger cells up to 3 or 4 ft (1 or 1.2 m) in diameter appear to be preferable, despite their substantial installation difficulties and high cost.

A variation on the procedure described by ISRM (1981c) involves placing the earth pressure cell within a hole in the middle of a plate approximately 2 ft (600 mm) square. Diaphragm cells with a vibrating wire transducer mounted on one active face, and with a diameter to thickness ratio of 4.5, are customarily installed in this way in Norway (DiBiagio and Myrvoll, 1985). Each cell is mounted in the middle of either a steel or a segmented concrete plate, flush front and back, as shown in Figure 10.12. The segmented concrete plate creates flexibility and best possible conformance with the prepared surface of the fill, both immediately after installation and when compaction equipment passes over the plate. The diameter to thickness ratio is increased to 15, and it is claimed that because the edge effects shown in Figure 10.4 now occur near the edges of the plate rather than the cell, the stress against the cell will be more uniform and representative of free-field conditions. However, it appears to the author that use of an appropriately designed hydraulic cell, of uniform stiffness across the face of the cell, would make the plate unnecessary.

As indicated in Section 10.2.2, some success has been achieved in measuring horizontal stress in soft soils by using specially designed "push-in" earth pressure cells, stepped blades, pressuremeters, and flat plate dilatometers. Installation procedures for these instruments are described in the references cited in Section 10.2.2.

## 10.3. CONTACT EARTH PRESSURE CELLS

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

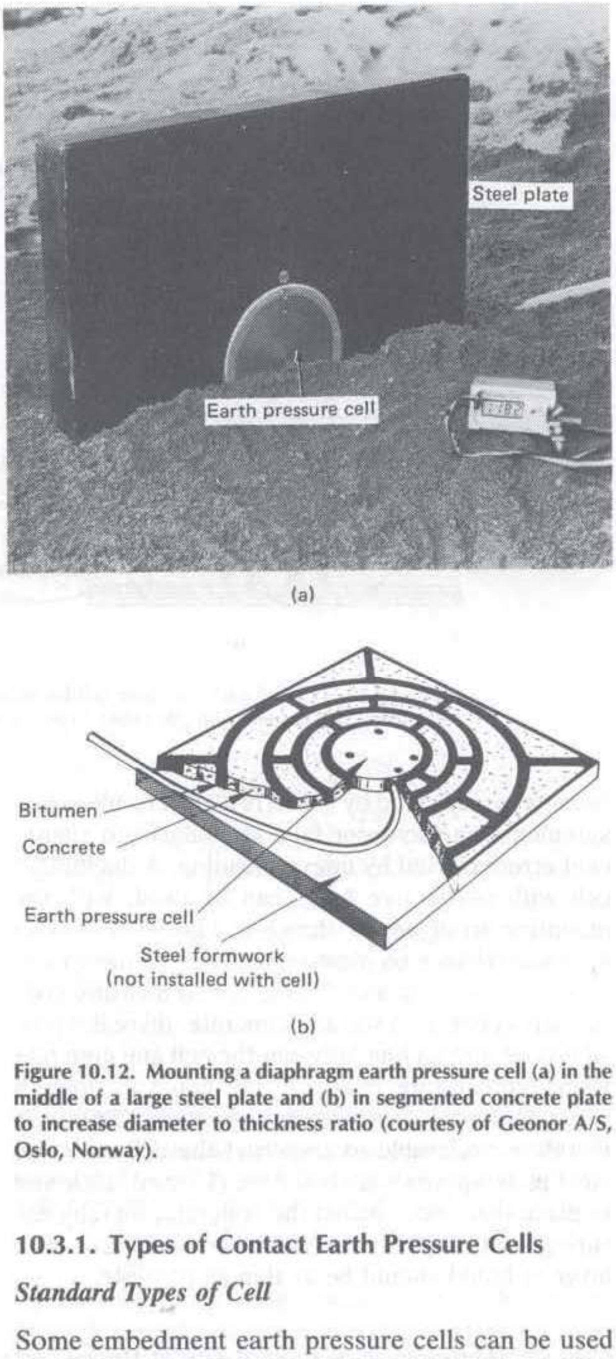



**Figure 10.12. Mounting a diaphragm earth pressure cell (a) in the middle of a large steel plate and (b) in segmented concrete plate to increase diameter to thickness ratio (courtesy of Geonor A/S, Oslo, Norway).**

### 10.3.1. Types of Contact Earth Pressure Cells

#### *Standard Types of Cell*

Some embedment earth pressure cells can be used directly as contact cells. All the cells shown in Figure 10.1 are suitable, with two exceptions. First, a hydraulic cell with a thin active face is unsuitable, because it is not readily possible to install this type of cell completely flush with the surface of a structure. Its stiffness is too low, and is also sensitive to the temperature changes that often occur at the locations of contact cells. Second, a diaphragm cell with a single active face is not likely to be suitable

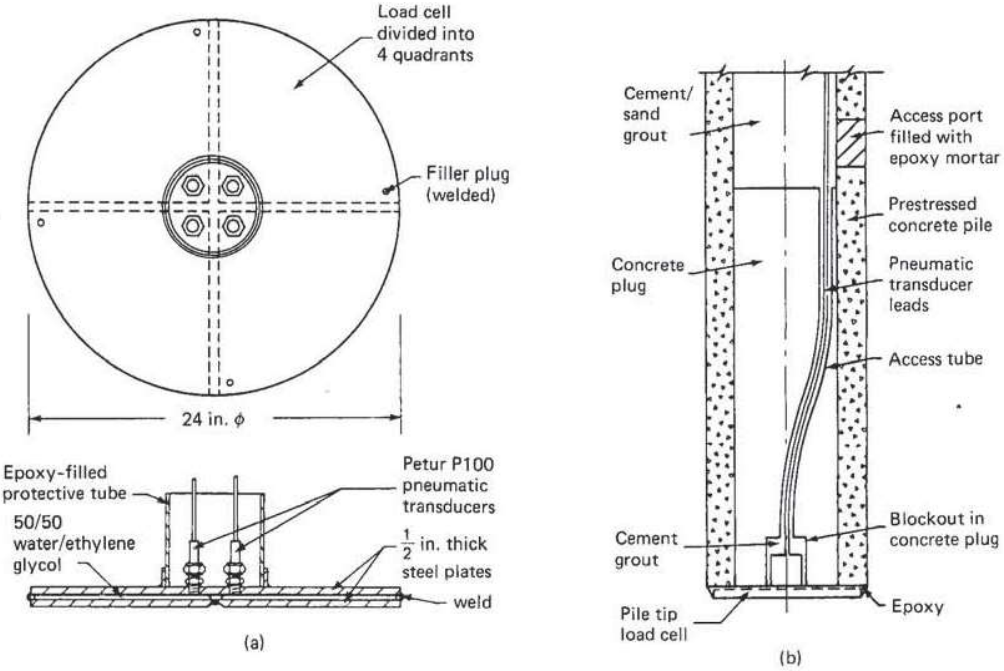



**Figure 10.13. Contact earth pressure cell for measurement of pile tip load: (a) construction details of cell and (b) cell mounted on pile (after Green et al., 1983).**

because, as indicated by Uff's (1970) test results, measurements on the outer face are subject to significant errors caused by uneven loading. A diaphragm cell with two active faces can be used, with the mounting arrangement shown in Figure 10.9, placing more reliance on measurements at the inner face.

When installing a hydraulic cell to measure contact stress between soil and concrete, there is a possibility of uncoupling between the cell and concrete when temperature rises and the liquid in the cell expands during concrete cure (Section 13.6). It is therefore preferable to construct the cell with one steel plate approximately 0.5 in. (13 mm) thick and to place that plate against the concrete, thereby ensuring that any expansion occurs outward. The layer of liquid should be as thin as possible.

### *Cells for Measurement of Load at Tips of Driven Piles or Drilled Shafts*

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

A hydraulic cell with pneumatic transducers is described by Green et al. (1983) for measurement of tip load on driven prestressed concrete piles. The cell is divided into four independent quadrants as shown in Figure 10.13, each with a pneumatic pressure transducer, and covers almost the entire area of the pile tip. Design criteria included adequate robustness to withstand driving forces and a thin enough active face to transmit tip pressure to the liquid in the cell. High-quality welding and leak testing procedures were used. The piles were 24 in. (610 mm) octagonal, with a 15 in. (380 mm) diameter hollow core. A 5 ft (1.5 m) long solid concrete plug was cast into the tip of each instrumented pile and the pneumatic transducer tubing carried up through the core. The writers discuss the difficulties of calibrating such a cell and justify their adopted check calibration procedure, whereby the pile was first lifted into a vertical position and cell readings taken in air, then lowered on to four wood blocks, one centered on each quadrant. The measured load was within 10% of the dead weight of the pile. Three instrumented piles were driven as part of a bridge foundation, through loose to medium dense sand into dense to very dense sand. Tubing to one of the 12 quadrants was damaged during driving, but the remaining quadrants have provided consistent and apparently reliable data during a 5 year monitoring period.

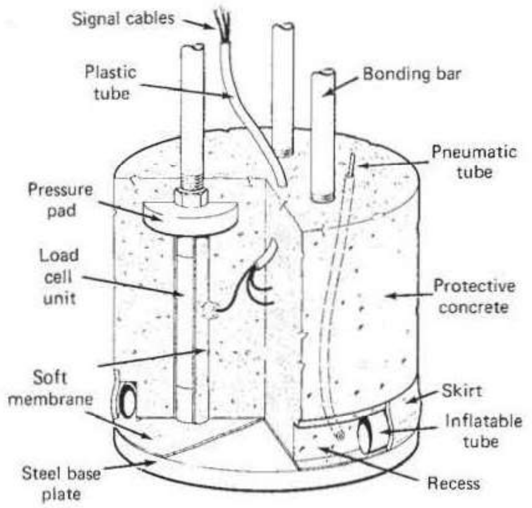



**Figure 10.14. Contact earth pressure cell for measurement of load at the tip of a drilled shaft (after Price and Wardle, 1983). By courtesy of Building Research Establishment, Crown copyright.**

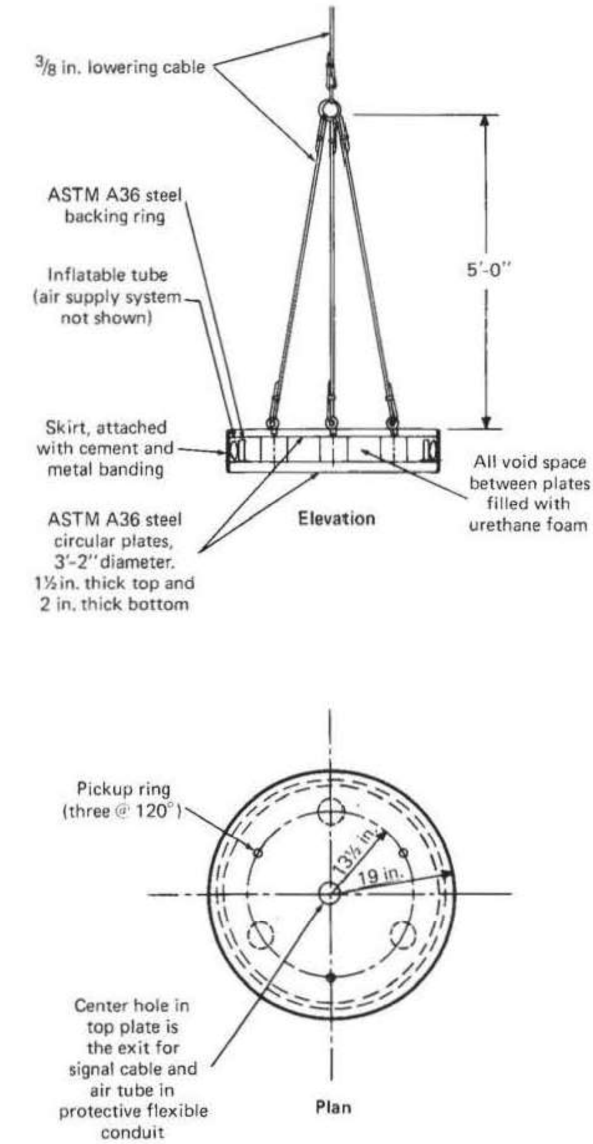



**Figure 10.15. Schematic of contact earth pressure cell for measurement of load at the tip of a drilled shaft: vibrating wire load cells mounted between two thick circular plates.**

Figure 10.14 shows a cell designed at the Building Research Establishment in England (Price and Wardle, 1983) for measurement of tip loads in drilled shafts. The load cell units are steel tubes with internal strain gages, usually the vibrating wire type. The steel base plate is isolated from the concrete shaft by a soft membrane sheet, so that the tip load is transferred entirely into the load cells. The load is prevented from bridging the cell by the inflated tube. Load from the load cells is then transferred to the concrete shaft through pressure pads and bonding bars, located above the load cells. The cell is lowered into place on a small bed of concrete, the inflatable tube pressurized to form a seal, and the shaft concrete poured. After 24 hours the air pressure is released.

Barker and Reese (1969) describe a "bottomhole cell," also designed for measuring tip loads in drilled shafts, consisting of electrical resistance strain gaged load cells mounted between two thick circular steel plates. The cell is similar to one designed by Whitaker (1964) at the Building Research Station, which has been superseded by the Price–Wardle cell.

Nowack and Gartung (1983) used a hydraulic cell with a pneumatic transducer for measuring tip loads in a drilled shaft. The cell was bedded on a concrete pad and the annular space around the cell filled with a soft material to ensure that the entire tip load was transmitted through the cell. Horvath (1985) used two flatjacks for the same purpose, positioned in series with intervening bearing plates. The second flatjack served as a backup in the event of malfunction.

The author has recently been involved in planning an instrumented load test of a drilled shaft in sand and gravel, with cobbles and boulders, to be conducted by the State of Ohio Department of Transportation, that requires measurement of tip load. Steel casing and drilling mud will be used to drill the shaft, and the bottom cannot be inspected. The selected cell is shown in Figure 10.15, and will be manufactured by Geokon. The circular steel plates will be held together by three steel bolts, one through the hollow-core of each load cell, with heads flush with the bottom face of the lower steel plate. The bolts will be tightened only hand tight, and steel domed covers (not shown in the figure) will be attached to the top plate over each bolt to ensure that no load will be carried by the bolts. This provision allows use of a calibration based on the sum of the three separate load cell calibrations. The skirt and inflatable tube ensure that no load can bridge the cell during the load test. The cell will be placed on a 6 in. (150 mm) layer of either stiff grout or leveled and compacted sand, the inflatable tube pressurized to center the cell, the tube depressurized, the reinforcing cage inserted over the lowering cable and conduit, the shaft concrete poured and casing raised. Early during pouring concrete and raising casing the inflatable tube will again be pressurized to form a seal, and the air pressure will be released after 24 hours.

*Cells for Measurement of Stress on Sides of Driven Piles*

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

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

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

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

### 10.3.2. Factors Affecting Measurements

Overall requirements for contact earth pressure cells are the same as given in Section 10.2.2 for embedment cells: good conformance, adequate sensing area, minimum sensitivity to nonuniform bedding, and a method of installation that will not seriously change the state of stress. These requirements can usually be accomplished by installing a flat-faced cell of adequate size with its sensitive face absolutely flush with the surface of the structure and by attention to correction methods for minimizing various sources of error. These methods are identified by a superscript *b* in Table 10.1. Various additional factors that affect measurements are discussed in the following subsections.

*Number of Cells*

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

As indicated in Section 10.1, it is sometimes preferable to isolate a portion of the structure and to determine stresses by use of load cells and strain gages within the structure.

*Laboratory Calibrations*

Each cell should be calibrated under fluid pressure to be sure that it is functioning correctly and not leaking, and the calibration chamber shown in Figure 10.6 can be adapted for performing fluid calibrations of contact earth pressure cells. Selig (1980) presents details of the procedure, whereby the bottom half of the chamber is replaced with a concrete block containing the cell cast in place. The cell can be removed and the concrete block reused by casting an oversized cavity in the block and seating each cell in the cavity with a thin layer of high-modulus gypsum plaster.

Selig also describes calibrations in contact with soil, by placing the cell and concrete block within a 3 ft (910 mm) diameter chamber used for soil calibration of embedment cells. A similar procedure is described by Felio and Bauer (1986), but it is believed that their 2 ft (610 mm) diameter chamber is too small for calibrating their 12 in. (300 mm) diameter cells.

*Temperature Effects on Cell*

Embedment earth pressure cells are not usually subject to temperature changes, but contact cells are often near the atmosphere or near concrete pours that create changes in temperature.

As discussed in Section 10.3.1, hydraulic cells used to measure contact stress between soil and concrete should have an inner thick inactive face, and the layer of liquid within the cell should be as thin as possible. The active face should also be thick and grooved. Examples of the significant influence of temperature on hydraulic cells are given by Coyle and Bartoskewitz (1976) and Felio and Bauer (1986), but the cells in both examples did not satisfy the above guidelines. Calibrations to examine temperature sensitivity under unconfined conditions, for example, in a heated water bath, are sometimes quoted but are of limited value. When confined, the cell is likely to be much more susceptible to temperature effects that cause expansion or contraction of the layer of liquid, and this condition is very difficult to simulate correctly in a laboratory test. In both the laboratory and in the field there may also be real stress changes occurring at the interface, owing to temperature effects on the soil and the structure. The best solution to this dilemma is to design the cell for minimum sensitivity to temperature.

A properly designed diaphragm cell is not significantly affected by temperature changes and, if temperature changes are great, diaphragm cells may be more appropriate than hydraulic cells. However, the cells should have two active faces, the mounting arrangements shown in Figure 10.9 should be used, care should be taken to minimize nonuniform loading, and more reliance should be placed on measurements at the inner face.

*Stiffness of Cell*

As for embedment cells, errors will normally be caused by excessive diaphragm displacement. When installed at a contact between concrete and all materials other than soft clay, it is recommended that the same modulus criterion should be adopted as for concrete stress cells (Chapter 13): the modulus of elasticity of the cell should be at least one-half that of the concrete.

*Irregularity of Surface of Structure*

When contact earth pressure cells are installed at an irregular surface, significant measurement errors can be expected. As an example, DiBiagio (1977) describes measurements on a sheet pile wall. Because of the corrugations in the wall, the cells must be placed either on the protruding or indented corrugation, as shown in Figure 10.16. Cells were placed at both locations at the same depth in soft clay. Measurements were almost equal immediately after pile driving but later followed the pattern shown in Figure 10.16, presumably because of sheeting movements or consolidation and arching. A "best estimate" of the average total stress per unit length of the wall can be made by averaging $P_1$ and $P_2$, but clearly this average may be significantly in error.

### 10.3.3. Installation of Contact Earth Pressure Cells

*Installation at Interface Between Soil and Concrete*

As indicated in Section 10.3.1, when a hydraulic cell is to be installed at an interface between soil and concrete, a thick steel plate should be used for the face of the cell adjacent to concrete.

ISRM (1981c) indicates that, when placing cells to measure contact stress between soil and concrete, the cells may be installed by any one of three methods: (1) attached to the formwork and placed in the structure before concreting, (2) fastened to the structure after concreting and prior to backfilling, or (3) embedded in the backfill a short distance away from the structure. In the view of the author, the overriding need is to ensure that the sensitive face of the cell is **absolutely** flush with the interface. The first method is therefore suitable, and also the second if the cell is grouted into a blockout. The second method is not suitable if the cell protrudes outside the profile of the structure, because

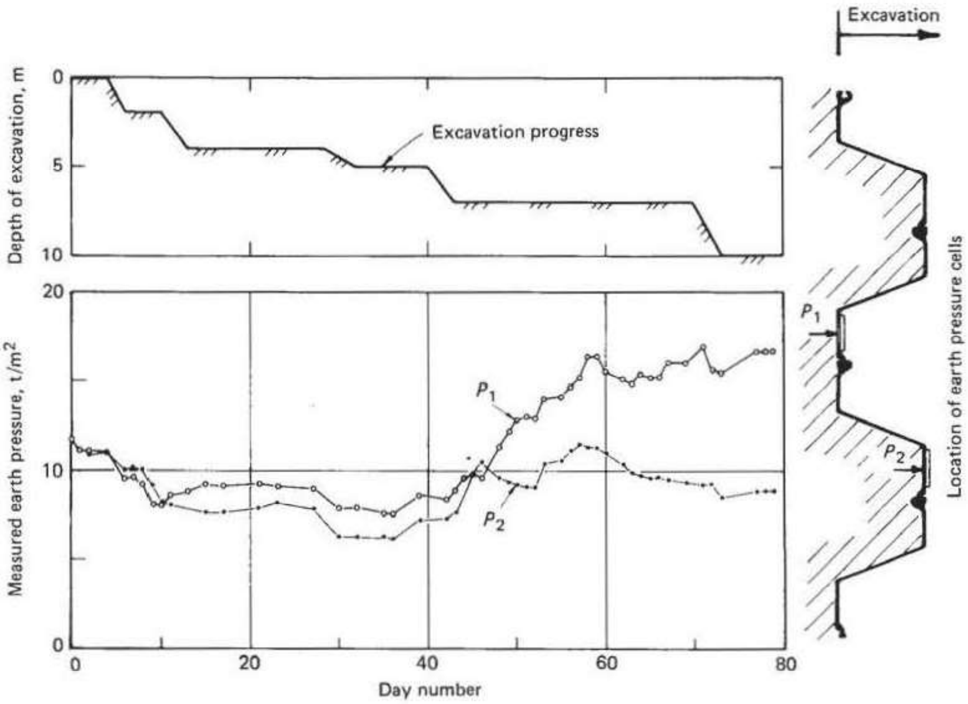



**Figure 10.16. Variation in measured earth pressure on the corrugated surface of a sheet pile wall (after DiBiagio, 1977).**

conformance errors may be significant. The third method creates the potential for all the errors described above for embedment cells. Also, leads may be sheared if they are installed through the concrete, and leads are exposed to damage if they are installed in the soil. It is therefore not recommended.

The first method, whereby cells are attached to the formwork and placed in the structure, can generally be used for measurements against retaining walls, culverts, precast concrete piles, and precast concrete tunnel linings. The cell can sometimes be held in place during concreting by light bracing against the form at the opposite face, or alternatively, the cell can be attached by bolts with heads on the outside of the form. The back face of the cell should be cleaned and degreased to ensure good contact between cell and concrete. A ring of soft sealing material, about 0.1 in. (2.5 mm) thick, around the edge of the cell is useful to prevent concrete fouling the outside face of the cell and to limit the influence of stresses that may act on the edge of the cell as the concrete cures. The same soft sealing material should be used during calibration. When using diaphragm cells, the mounting arrangement shown in Figure 10.9 is suitable. Concrete vibrators should not be allowed to touch the cell. In cases where cells cannot be installed in this way, a blockout can sometimes be made in the concrete and the cell subsequently embedded within the blockout.

Where concrete is cast directly against soil, for slurry walls, drilled shafts, base slabs, spread footings, cast-in-place tunnel linings, and other structures, installation is less straightforward, and a method must be devised whereby the cell is held against a suitably flat surface of soil and protected during concreting. DiBiagio and Roti (1972) describe installation of cells in a slurry trench by using an expendable hydraulic jack to hold the cell in position against the soft clay. Each cell was first mounted in the center of a flat steel plate such that its sensitive face was flush with the surface of the plate. The cell and plate were attached to one end of the hydraulic jack with a flexible coupling and a reaction plate of similar size attached to the other end. The cells and hydraulic jacks were attached to the reinforcing cage before lowering into the trench and, to avoid exerting forces on the cage during installation of the cells, the body of each jack was placed inside a short length of oversized pipe welded to the cage. Actuation of jacks forced the cells in contact with the sides of the trench; pressure was maintained during concreting and released after the concrete had set. A similar installation procedure is described by Uff (1970), and this approach can be used in other applications where concrete is cast directly against a vertical soil face. Bierschwale et al. (1981) describe use of steel pins to attach contact earth pressure cells to the soil in the wall of a dry hole for a drilled shaft.

Where there is concern for damage or disturbance to cells during concreting, each cell may be cast in a briquette of concrete identical to the parent concrete. The sensitive face of the cell should be **absolutely** flush with the face of the briquette that will be in contact with the soil. It is **essential** that the briquette be cast not more than 24–48 hours ahead of the main concrete pour, otherwise, accuracy will be severely reduced because of poor conformance. When cells are installed against a horizontal soil surface, for example, at the base of a foundation slab or spread footing, it is best to use a briquette over the entire cell, and to cast it in place so that good contact with the soil is ensured. A typical briquette size for a 9 in. (230 mm) diameter cell with transducer attached to a tube emerging from the rim of the cell is 2 ft × 1 ft × 3 in. (600 × 300 × 76 mm). When cells are installed in situations where the briquette cannot be cast in place, it should be 6 in. (150 mm) thick to ensure adequate strength for moving.

Wherever possible, grains larger than about 0.2 in. (5 mm) should not be allowed in contact with the cell, and if necessary a thin layer of grout or fine-grained material should be placed against the face of the cell.

*Installation at Interface Between Soil and Steel*

Wherever possible, the cell should be mounted such that its face is absolutely flush with the surface of the steel structure. If a hole can be cut in the structure, the cell can be attached to studs welded on the back face, and any irregularities in the front face can be smoothed with a suitable filler material such as epoxy resin.

*Installation at Tips of Driven Piles or Drilled Shafts and on Sides of Driven Piles*

Installation for these special applications is discussed in Section 10.3.1.

*Installation at Interface Between Soil and Rock*

Three approaches are possible, depending on the irregularity of the rock surface at and near the place of installation and the strength of the rock.

First, if the surface at and near the place of installation is generally planar and the rock is strong, a location should be selected for the cell where the surface is flat within about ± 0.4 in. (± 10 mm), and any loose material should be removed. A cement mortar or epoxy resin pad should be troweled on to the rock surface and the cell pushed into the pad, squeezing out mortar or resin until a layer no thicker than 0.2–0.4 in. (5–10 mm) remains beneath the cell. Entrapment of air bubbles must be avoided, and the cell may need to be secured in position by tying to pins in the rock. The area around the cell should be blended into the surrounding rock surface by troweling cement or epoxy after the pad has hardened.

Second, if the surface at and near the place of installation is generally planar and the rock is weak, a hollow can be excavated in the rock and the cell installed with its sensitive face outward, flush with the surface of the rock, as described for installation at an interface between soil and concrete.

Third, if the surface of the rock is irregular, such that either of the above methods might result in measurement anomalies, an embedment earth pressure cell should be used and installed entirely within the soil as described in Section 10.2.4.

*Installation at Interface Between Rock and Concrete*

The three approaches decribed above for an interface between soil and rock are generally applicable. However, additional precautions are necessary to minimize the problem of cell/concrete uncoupling when temperature rises and falls during concrete cure (Section 13.6), and a stiff concrete stress cell rather than a contact earth pressure cell should be used.

# CHAPTER 11

# MEASUREMENT OF STRESS CHANGE IN ROCK*

## 11.1. APPLICATIONS

The primary application for measurement of stress change in rock is to monitor the stability of pillars and walls formed during underground mining.

Similar measurements are occasionally made during excavation of underground openings for civil engineering purposes, but most civil engineers prefer to monitor opening stability by measuring deformation. The mining engineer is accustomed to working with much lower factors of safety than the civil engineer and is therefore generally more concerned with critical stress conditions. However, the current and frequent use of design tools such as finite and boundary element modeling techniques has greatly increased the importance of monitoring stress change in rock. The primary output from these design aids are estimates of rock deformation and stress, and the effectiveness of an adopted design can only be confirmed by monitoring one or both of these variables in situ.

*Coauthored by Robert J. Walton, Senior Experimental Scientist, Commonwealth Scientific and Industrial Research Organization (CSIRO), Victoria, Australia. J. Barrie Sellers, President, Geokon, Inc., Lebanon, NH, and Frank S. Shuri, Senior Engineer, Golder Associates, Inc., Seattle, WA, have also assisted in preparation of this chapter.

Stress change monitoring is used to diagnose critical stability in situations where the rock approaches a failure condition caused either by an increase or decrease in stress. A decrease in stress can lead to failure by reducing confinement, thereby causing rock blocks to slide or unravel. It is important to realize that the actual stress within a rock mass may vary significantly from point to point, depending on geologic features and the local stress concentrating effect of an opening. A single point measurement of stress may therefore be misleading, and if reliable data are required it is usually necessary to make measurements at numerous points throughout the rock mass of interest.

Extensive measurements of stress change are made during thermomechanical testing in underground excavations for studies relating to the disposal of high-level nuclear waste. This application requires devices capable of withstanding the high temperatures created to simulate anticipated repository conditions and is not discussed in this book.

## 11.2. INSTRUMENT CATEGORIES

The following three methods can be used for monitoring stress change in rock.

*Repeated Measurements of in Situ Stress*

Repeated measurement of rock stress can be made in situ by using one of the absolute stress measurement techniques such as borehole overcoring. However, this approach is not often used because the cost is high, absolute accuracy is low, and there is a possibility that important data may be missed between measurements. Even when the most accurate borehole overcoring measurements are made, the variation between tests is generally at least $\pm 25\%$ for magnitude and $\pm 15$ degrees for orientation of the principal stresses. These variations are caused both by measurement inaccuracy and by real variations in stress from one point in the rock mass to another.

*Geophysical Techniques*

Several geophysical techniques use the relationship between deformability and stress—hence the propagation of seismic waves—in an attempt to determine in situ stress. For example, the time can be measured for an energy pulse to travel through the rock, or the rise time of a similar pulse can be measured. Geophysical techniques for monitoring stress change in rock are still at an early stage of understanding and development and are not discussed in this book.

*Measurements in a Borehole*

An instrument can be installed in a borehole to measure displacement, strain, or pressure caused by changes in the near-field stress. This approach is the most viable method for monitoring stress change and is the subject of the remainder of this chapter.

Instruments can be divided into two general categories: *soft* and *hard (rigid) inclusions*.

Soft inclusion gages have small stiffness relative to the host rock, and stress determination requires knowledge of rock properties and constitutive behavior. In practice, a gage having an intact modulus of elasticity less than that of the surrounding rock by a factor of 3 or more is considered to be a soft inclusion gage. These gages therefore offer minimum resistance to rock deformation, so that gage readings depend on rock stress and elastic properties but are independent of the modulus of the gage.

Rigid inclusion gages are designed to be stiff relative to the host rock, and stress determination requires knowledge of rock properties and constitutive behavior only within broad bounds. For practical purposes, a rigid inclusion gage is a gage having a modulus of elasticity exceeding that of the surrounding intact rock by a factor of at least 3. Rock deformations are resisted by the gage so that strains in the gage have minimum sensitivity to rock modulus. Rigid inclusion gages are usually referred to as *stressmeters*.

It is evident that an instrument that is a soft inclusion when installed in one type of host rock could be deemed a rigid inclusion when installed in rock of lower modulus.

Comprehensive descriptions of measurement methods are given by IECO (1979), LBL (1982), Lemcoe et al. (1980), Lingle et al. (1981), RKE/PB (1984), and Schrauf and Pratt (1979). Methods in practical use are described and compared in Sections 11.3 and 11.4, and recommendations are given in Section 11.5.

## 11.3. SOFT INCLUSION GAGES

As discussed in Section 11.2, soft inclusion gages have small stiffness relative to the host rock, and stress determination requires knowledge of rock properties and constitutive behavior. This definition clearly places borehole deformation gages and biaxial and triaxial strain cells in the category of soft inclusions, but the categorization may be less appropriate for borehole pressure cells. Borehole pressure cells have been used to measure stress change in a wide variety of rock types; in some cases stress determination requires knowledge of rock properties, while in other cases it does not. Some borehole pressure cells can be filled with mercury specifically to increase their stiffness so that they can be considered as rigid inclusions when used in low-modulus rocks. Despite this qualification, borehole pressure cells are classified in this book as soft inclusion gages.

### 11.3.1. Flat and Cylindrical Borehole Pressure Cells

A borehole pressure cell consists of a flat or cylindrical metal chamber, filled with a liquid and fitted with pressure regulation and monitoring capabilities. The term *borehole pressure cell* (BPC) is normally used for a flat cell and the term *cylindrical pressure cell* (CPC) for a cylindrical cell.

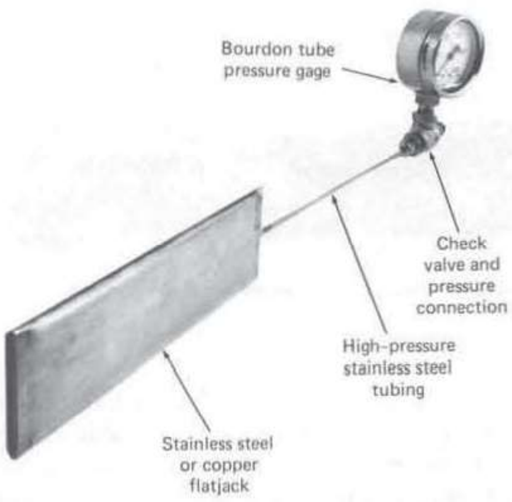



**Figure 11.1. Flat borehole pressure cell (courtesy of Geokon, Inc., Lebanon, NH).**

Flat cells are essentially miniature flatjacks and are typically about 2 in. wide, 0.25 in. thick, and 8 in. long (50 × 6 × 200 mm). A BPC is shown in Figure 11.1. A remote reading pressure transducer can replace the Bourdon tube pressure gage.

The U.S. Bureau of Mines BPC is described by Panek and Stock (1964) and Smith (1972). Embedment earth pressure cells with pneumatic transducers have been used as BPCs (e.g., Sauer and Sharma, 1977).

A BPC can be installed in any one of four ways. First, the cell can be inserted into a borehole already filled with grout. Second, grout can be pumped into the borehole after the cell is in place. Third, the cell may be preencapsulated inside an appropriately sized cylinder of mortar or epoxy, or sandwiched between two curved aluminum platens, inserted into a borehole, and pumped up to create adequate contact with the wall of the borehole. Fourth, a preencapsulated cell can be grouted within a borehole. A method that includes grouting is highly recommended, because it results in more intimate contact between the cell and borehole wall. After the grout has set, the cell is pumped up until the hydraulic pressure is somewhat greater than the estimated rock stress, a valve in the pressure line is closed, and the system is allowed to stabilize.

In elastic (the terms *elastic* and *viscoelastic* are defined in Chapter 2) rocks the relationship between change in BPC pressure gage reading and change in rock stress can be determined by using the pump-up procedure described by Sellers (1970), but rock deformability values must be known. The BPC responds primarily to stresses acting perpendicular to the plane of the flatjack but also has a small sensitivity to stress changes in the plane of the flatjack. Two BPCs oriented at right angles to each other in the same borehole will, provided the orientations of the principal stresses in the plane perpendicular to the axis of the borehole are known, allow determination of changes in these stresses. It is generally assumed that in viscoelastic rocks the BPC will achieve an equilibrium pressure that is close to the actual rock stress.

CPCs are typically 1.5 in. (38 mm) in diameter and 8 in. (200 mm) long. The U.S. Bureau of Mines CPC (Hall and Hoskins, 1972; Panek et al., 1964; Smith, 1972) consists of a cylindrical copper jacketed inflatable probe connected to a screw pump for pressure application. The Colorado School of Mines dilatometer, which is 3 in. (76 mm) in diameter and has a flexible plastic jacket (Hustrulid and Hustrulid, 1973), can also be used as a CPC.

For a CPC the diameter and smoothness of the borehole are critical, and every effort should be made to keep the tolerance within approximately 0.02 in. (0.5 mm). The borehole should preferably be drilled with a rotary diamond bit, and if percussion drilling is used a reaming shell should be inserted behind the bit. The CPC is inserted into the borehole and pressure is applied with the screw pump to seat the cell against the wall of the borehole. As for the BPC, the cell is then pumped up until the hydraulic pressure is somewhat greater than the estimated rock stress, a valve in the pressure line is closed, and the system is allowed to stabilize. The procedure for calculation of stress change is described by Sellers (1970). It should be noted that CPC data provide a measurement of the **average** change of the two principal stresses in the plane perpendicular to the axis of the borehole, and this may not be appropriate for strongly anisotropic stress conditions.

A typical range of commercially available BPCs and CPCs is 0–10,000 lb/in.$^2$ (0–70 MPa), with a sensitivity of 40 lb/in.$^2$ (300 kPa).

If biaxial stress measurements are required, three BPCs can be installed in the same borehole. Lu (1981) describes determination of stress changes in viscoelastic rock by using one CPC and two BPCs in the same borehole.


**Figure 11.2. U.S. Bureau of Mines borehole deformation gage (courtesy of Rogers Arms and Machine Company, Inc., Grand Junction, CO).**

### 11.3.2. Borehole Deformation Gages

A borehole deformation gage (BDG) is designed to measure diametral changes in a small-diameter borehole, using remotely read electrical transducers.

#### *U.S. Bureau of Mines Gage*

The most well-known BDG is the U.S. Bureau of Mines (USBM) instrument, shown in Figure 11.2, allowing measurement of changes in three diameters 120 degrees apart. The gage has three pairs of contact points, the outer ends contacting the borehole wall and the inner ends bearing against cantilevers, each of which is clamped to the gage body. Electrical resistance strain gages are bonded to both faces of each cantilever and the strain gages on opposite cantilevers are wired into a full Wheatstone bridge configuration. A conventional strain indicator is used as a readout unit. Changes in borehole diameter cause bending of appropriate cantilevers and changes in strain gage resistances. Borehole deformations are related, via an assumed or measured deformability and Poisson's ratio, to stress changes in the plane perpendicular to the axis of the borehole. Most versions of the USBM BDG are designed for measurements within 1.5 in. (38 mm) diameter boreholes, but larger and smaller sizes are available.

The USBM BDG was developed for determination of absolute in situ stress by using the overcoring procedure (Hooker and Bickel, 1974; Hooker et al., 1974; Obert, 1966), a test that involves reading the gage for less than 30 minutes. Most commercially available USBM BDGs have transducers that are insufficiently stable for monitoring stress change over a longer period, and the design makes it very difficult to prevent ingress of moisture into the transducer area. The version shown in Figure 11.2 is believed to be the most stable version available and has been used for stress change monitoring during in situ tests related to underground high-level nuclear waste repositories. However, even this version has been proved reliable for stress change monitoring only for short-term applications at ambient temperature.

The USBM gage is designed as a reusable device and is held in place in the borehole by frictional forces. For long-term monitoring purposes, the instrument would have to be held in place by grouting to prevent movement caused by blasting or other vibrations. However, the cost of the instrument usually prevents grouting in place.

The USBM BDG is not recommended for monitoring stress change in rock.

#### *CSIRO Yoke Gage*

The yoke BDG has been designed by the CSIRO (Commonwealth Scientific and Industrial Research Organization) in Australia specifically for stress change monitoring. It has been used to monitor stress changes in the roof of a coal mine (Walton and Fuller, 1980) and in pillars of metalliferous mines (Walton and Worotnicki, 1986). As shown in Figure 11.3, three strain gaged yokes are used to measure changes in three diameters 120 degrees apart. As for the USBM gage, output is related, via an assumed or measured deformability and Poisson's ratio, to stress changes in the plane perpendicular to the axis of the borehole.

The instrument is held in place in a borehole by epoxy cement, but the cement merely fills the annular space and plays no part in transmitting rock deformations to the yokes. The tips of the yokes are rounded, so that they slide freely along the wall of the borehole during installation. A front-to-rear

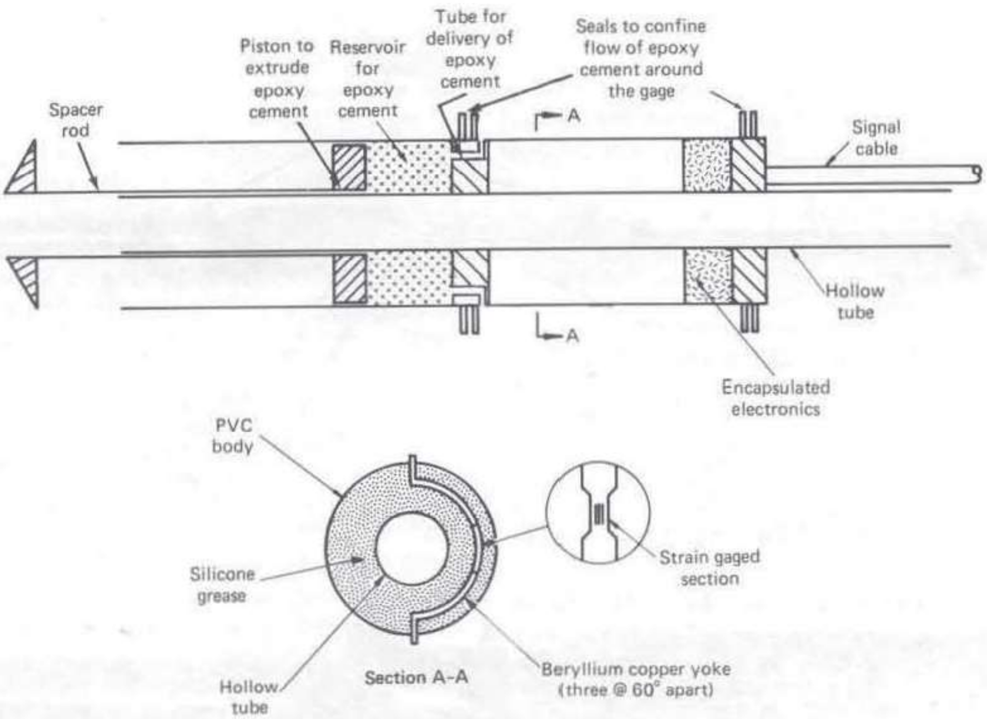



**Figure 11.3. CSIRO yoke borehole deformation gage (courtesy of Commonwealth Scientific and Industrial Research Organization, Australia).**

opening allows passage of signal cables from instruments installed deeper in the borehole.

To date, instruments have been installed in 56.5 mm (2.22 in.) diameter boreholes. The instrument has a large range in both tension and compression. For example, the tensile range is 100 MPa ($1.4 \times 10^4$ lb/in.$^2$) in rock of modulus 70 GPa ($10 \times 10^6$ lb/in.$^2$). The compressive range is greater than the tensile range.

### 11.3.3. Biaxial and Triaxial Strain Cells

Biaxial and triaxial strain cells are designed to measure strains on the wall or end of a borehole. The cells include electrical resistance strain gage transducers and are bonded to the rock. Their primary application, as for the USBM borehole deformation gage, is the determination of absolute in situ stress by using the overcoring procedure, and five versions are in practical use for this very short-term application.

The South African CSIR (Council for Scientific and Industrial Research) doorstopper biaxial strain cell (Gregory et al., 1983; Leeman, 1971; Stickney et al., 1984) is bonded to the end of a borehole. Four versions of triaxial cell are bonded to the wall of a borehole: CSIR, South Africa (Herget, 1973; Leeman, 1971); Luleå, Sweden (Gregory et al., 1983; Hiltscher et al., 1979; Stillborg and Leijon, 1982); LNEC, Portugal (Rocha and Silverio, 1969; Rocha et al., 1974); and CSIRO, Australia (Jagger and Enever, 1978; Walton and Worotnicki, 1978; Worotnicki and Walton, 1976).

The CSIRO cell has been used for stress change monitoring on more occasions than the other four cells. Likely reasons are its high degree of waterproofing of the strain gages, use of a hard-wired cable, which eliminates the need for downhole connectors, and ready commercial availability.

The CSIR doorstopper and triaxial strain cells and the Luleå cell all require bonding of strain gages directly to the rock surface. For analytical reasons it is preferable for the strain gages to be on the rock surface, but moisture can cause severe degradation of the resistance signal, and in practice it is better to ensure complete waterproofing by full encapsulation of the strain gages, accepting that a correction factor is required to account for the annulus of en-

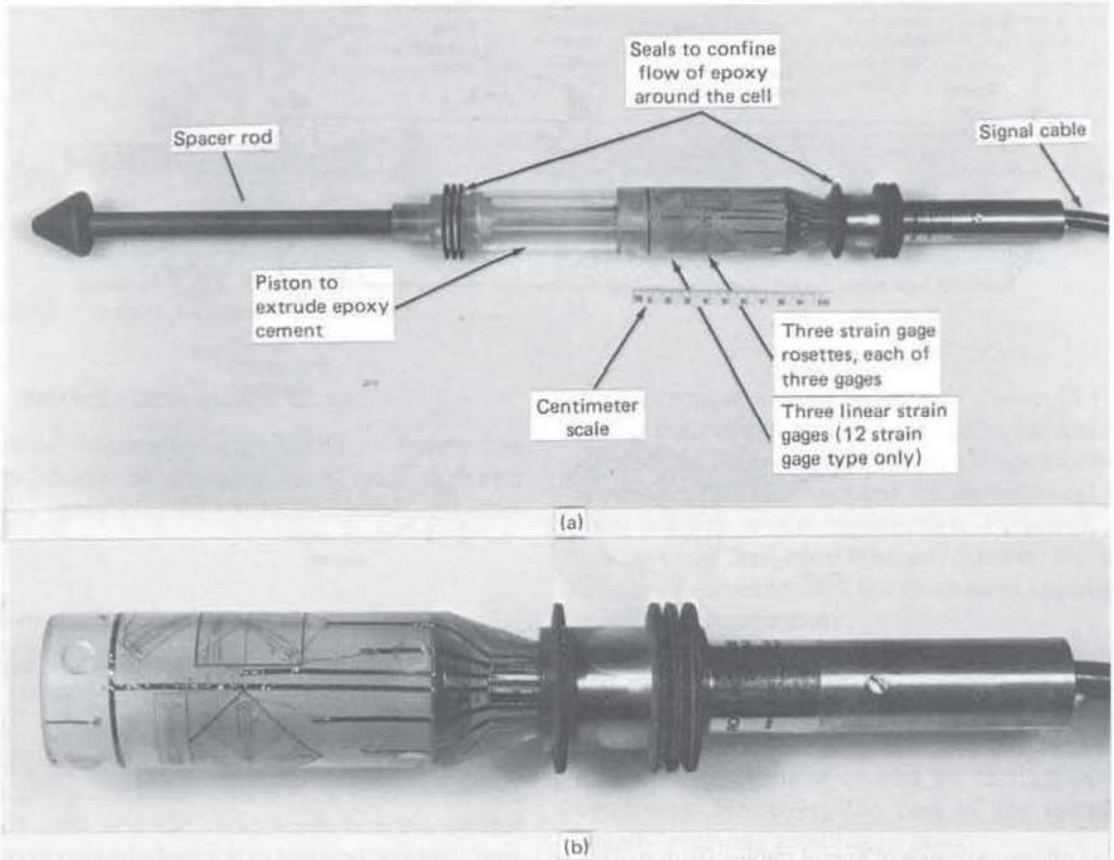



**Figure 11.4. CSIRO hollow inclusion stress cell—12 strain gage type: (a) general arrangement and (b) detail of epoxy body of cell, containing strain gages (courtesy of Commonwealth Scientific and Industrial Research Organization, Australia).**

capsulating material. The LNEC and CSIRO cells have strain gages that are fully encapsulated in epoxy resin.

The CSIRO cell, referred to as a *hollow inclusion (H.I.) cell*, is shown Figure 11.4. It consists of a thin epoxy tube, with three strain gage rosettes, each of three gages, embedded within the epoxy. The strain gage layout provides for some duplication of gages, so that in the event of malfunction of two and sometimes three of the nine gages it is still possible to determine the full stress tensor. When all nine gages are functional, the extra readings are a significant benefit, in that they allow data evaluation by least-squares fitting methods. Gages are wired in three-wire quarter bridge circuits. A version with 12 strain gages is available for use in anisotropic rock or to give greater redundancy of measured strains.

The cell is installed in a 1.5 in. (38 mm) diameter borehole by filling the hollow core with specially formulated epoxy cement, attaching the piston and spacer rod, and pushing with a setting tool to the end of the borehole. The cement is extruded between the cell and rock and allowed to set before initial strain readings are taken.

Methods of converting measured strain to rock stress are described by Duncan Fama and Pender (1980) and by Worotnicki and Walton (1976). An example of successful measurements is given by Kohlbeck and Scheidegger (1986).

The cell can be used to monitor changes in compressive stress up to the point where nonlinear conditions occur in the rock around the borehole. The ultimate sensitivity of the instrument is dependent on the resolution and accuracy of the readout equipment used: between 1 and 5 microstrain is common.

Despite the advantages of the CSIRO cell when compared with the other four versions, the user should be aware of four sources of error.

First, the cell may exhibit an output that implies a radial biaxial compression, which is in fact caused by a slight volumetric expansion of the epoxy as moisture is absorbed. The effect is largest soon after the cell is installed and decreases significantly after 30–100 days (Walton and Worotnicki, 1986). Windsor and Worotnicki (1986) state that cells used for long-term monitoring should preferably be installed at least 1 month before reliable data are required. The writers also present more than 1 year of data for cells that were installed 6–12 months before the monitoring period commenced, indicating apparent stability of the cells.

Second, the cell is sensitive to temperature changes, particularly in the circumferential direction, but the magnitude of the temperature response is known (Walton and Worotnicki, 1986). The cells are supplied with a thermistor to monitor temperature; therefore, a correction can be applied, but this is often unnecessary because many installed locations are deep underground where the rock undergoes little change in temperature.

Third, the epoxy body of the cell could continue to polymerize after the cell has been installed, which would cause a small volume change of the epoxy to occur. This effect is significantly reduced by curing the instrument at a temperature in excess of the field temperature. All commercially available instruments are cured by the manufacturer at 50°C (122°F) for 6 days, and most experience with long-term use of the cell has been at temperatures below 40°C (104°F). Long-term performance above this temperature is unknown, although the cell has been used for overcoring at temperatures up to 60°C (140°F).

Fourth, the body of the cell and the cement may creep. Walton and Worotnicki (1986) have demonstrated that, over a 1 year period and at a temperature of 18–25°C (64–77°F), the creep rate is approximately 0.3 microstrain/day radial compression and almost zero in the axial direction. They have also carried out a long-term laboratory test of a conventional and prototype thin-wall version of the cell, showing that the thin-wall version had superior long-term stability. However, no field experience was reported.

### 11.3.4. Miscellaneous Soft Inclusion Gages

Blackwood (1977), Rocha and Silverio (1969), and Worotnicki and Walton (1976) all describe variations of a solid soft inclusion cell for use when measuring in situ stress by the overcoring procedure. When used as stress monitoring instruments, solid soft inclusion cells appear to offer no advantages over hollow soft inclusion cells and in fact have a disadvantage when monitoring reduction in compressive stress. In this application the solid cells require a more effective cell-to-rock bond than the hollow cells, because higher tensile stresses must be maintained across the interface.

### 11.3.5. Comparison Among Soft Inclusion Gages

Comparative information is summarized in Table 11.1.

## 11.4. RIGID INCLUSION GAGES

Section 11.2 indicates that rigid inclusion gages, usually referred to as *stressmeters*, are designed to be stiff relative to the host rock and that stress determination does not require accurate knowledge of rock properties and constitutive behavior. Clearly, this is an advantage when comparing rigid gages with soft inclusion gages. The definition of a rigid inclusion gage, also given in Section 11.2, sets a practical upper limit of intact rock modulus for which rigid inclusions remain "rigid" as one-third the modulus of the gage. Steel, which has a modulus of approximately $30 \times 10^6$ lb/in.$^2$ (200 GPa), is the most practical material from which to manufacture rigid inclusion gages. If a steel inclusion totally fills the cross section of the borehole, the upper limit of intact rock modulus for which a rigid gage can be considered "rigid" is therefore $10 \times 10^6$ lb/in.$^2$ (70 GPa). When used in rock with a higher modulus, the gage must be calibrated in the host rock if maximum accuracy is required.

### 11.4.1. Vibrating Wire Stressmeters

The vibrating wire transducer, together with methods for minimizing errors caused by zero drift and by corrosion of the vibrating wire, is described in Chapter 8.

#### *Uniaxial Stressmeter*

The most commonly used and commercially available rigid inclusion gage is the uniaxial version of the vibrating wire stressmeter, which was developed for measuring stress change in underground

**Table 11.1. Soft Inclusion Gages**

| Method | Advantages | Limitations[a] |
|---|---|---|
| Flat (e.g., Figure 11.1) and cylindrical borehole pressure cells | Economical | Small measurement scale[b]<br>Rock deformability needed for pressure/stress conversion in elastic rocks |
| USBM borehole deformation gage (e.g., Figure 11.2) | Recoverable<br>Suitable for monitoring reductions in compressive stress | Small measurement scale[b]<br>Low electrical output; lead wire effects; errors resulting from moisture, temperature, and electrical connections are possible<br>Not proved for other than short-term monitoring<br>Movement of gage causes false readings: expensive to grout in place<br>Rock deformability needed for strain/stress conversion<br>Not recommended for use |
| CSIRO yoke borehole deformation gage (Figure 11.3) | Suitable for monitoring reductions in compressive stress | Small measurement scale[b]<br>Low electrical output; lead wire effects; errors resulting from moisture, temperature, and electrical connections are possible<br>Rock deformability needed for strain/stress conversion<br>Limited field experience |
| CSIRO hollow inclusion triaxial strain cell (Figure 11.4)[c] | Triaxial data<br>Strain gages encapsulated in epoxy resin<br>Suitable for monitoring changes in tensile stress | Small measurement scale[b]<br>Low electrical output; lead wire effects; errors resulting from moisture, temperature, and electrical connections are possible<br>Rock deformability needed for strain/stress conversion<br>Cementing difficulties in wet holes<br>Potential errors caused by creep and moisture absorption of the epoxy<br>Should be installed at least 1 month before requiring data |

[a] Requires knowledge of rock properties and constitutive behavior.
[b] Limitation of small measurement scale can be minimized by installing several instruments in the rock mass of interest.
[c] When compared with other four versions of biaxial and triaxial strain cell, CSIRO cell is preferred: see Section 11.3.3. Other versions are therefore not included in this table.

coal mines (Hawkes and Bailey, 1973; Sellers, 1977). As shown in Figures 11.5 and 11.6, the stressmeter consists of a thick-walled steel cylinder with a vibrating wire transducer mounted across a diameter at approximately midlength. The cylinder is wedged in the borehole and acts as a proving ring. Figure 11.5 shows the vibrating wire in line with the wedge, a configuration adopted during the original development of the stressmeter. This configuration causes a range limitation, because the tension in the wire decreases as stress across the proving ring increases, and the wire can eventually become too

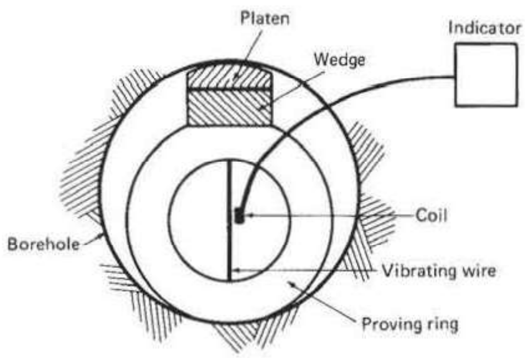



**Figure 11.5. Schematic of uniaxial vibrating wire stressmeter.**

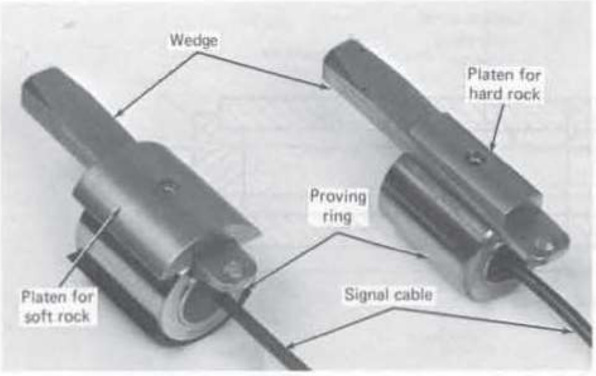



**Figure 11.6. Uniaxial vibrating wire stressmeter (courtesy of Irad Gage, a Division of Klein Associates, Inc., Salem, NH).**

slack. The device is now available in an alternative configuration, with the vibrating wire perpendicular to the alignment shown, such that the tension in the wire increases as stress increases. The range of this version is 15,000 lb/in.$^2$ (100 MPa) compression and, by setting the wedge to create a high preload, it can be used to monitor a stress reduction of about 5000 lb/in.$^2$ (35 MPa). Sensitivity is 0.5–5 lb/in.$^2$ (3–30 kPa).

The stressmeter is available for installation in boreholes of nominal diameter 1.5, 2.4, and 3.0 in. (38, 60, and 76 mm), using special setting tools, and is preloaded across a diameter by pulling the wedge between the body and platen. A high preload is necessary so that both positive and negative stress changes can be monitored. Various platen sizes are available, and in soft rocks wide platens are used so that stress at the borehole wall is minimized. If this stress is excessive, local crushing of the rock can be caused at the line of contact between platen and rock. For rock moduli less than about $3 \times 10^6$ lb/in.$^2$ (20 GPa), the soft rock platens should be used. The borehole should preferably be drilled with a rotary diamond bit, because if the contact between the stressmeter and the wall of the borehole is not uniform, the in-place loading will not match the loading during stressmeter calibration. If percussion drilling is used, a reaming shell should be inserted behind the bit. Installation details are given by Hawkes and Bailey (1973), Sellers (1977), and manufacturers of the instrument.

Early work with the stressmeter suggested that readings are insensitive to rock deformability when deformability of the intact rock is less than about $2 \times 10^6$ lb/in.$^2$ (15 GPa), but recent tests indicate that the device is somewhat sensitive to rock deformability throughout its range. Dutta et al. (1981) and others have determined that the amount of surface contact between the gage platen and the borehole wall has a major influence on gage readings, and it is therefore important to control borehole size, shape, and surface condition at gage locations. When these factors are controlled carefully, the sensitivity to rock deformability approximately halves when intact rock deformability changes from 1 to $10 \times 10^6$ lb/in.$^2$ (7–70 GPa). Rock deformability should therefore be measured or estimated and the calibration determined from data supplied by the instrument manufacturer.

When using the stressmeters in rocks with intact modulus higher than about $2 \times 10^6$ lb/in.$^2$ (15 GPa), two cautions are applicable. First, if the modulus of the gage is taken as about $7.5 \times 10^6$ lb/in.$^2$ (50 GPa) (Dutta et al., 1981), the definition given in Section 11.2 for a rigid inclusion indicates that the gage is no longer a true *stressmeter* when used in a host rock with an intact modulus greater than about $2.5 \times 10^6$ lb/in.$^2$ (17 GPa). The higher the modulus of the host rock, the greater is the need for calibrations specific to the rock type in which the gage will be used. Second, Dutta et al. (1981) have established that, in stiffer rock, it is not always possible to reproduce the amount of surface contact between the gage and borehole for each installation. Because the amount of surface contact has a major influence on gage readings, it is desirable to install several gages in one location if maximum accuracy is required.

When a uniaxial vibrating wire stressmeter is installed in rock subject to biaxial or triaxial stress changes, it may not give a correct indication of stress changes in the direction of measurement. When a complete evaluation is required of stress change in the plane normal to the borehole axis, three uniaxial stressmeters can be set at known orientations to each other (Pariseau, 1985). Alternatively, a biaxial stressmeter can be used.

### *Biaxial Stressmeters*

A biaxial stressmeter, shown in Figure 11.7, has been developed by the Soil and Rock Instrumentation Division of Goldberg-Zoino and Associates, Inc., Newton, MA, in conjunction with Geokon, Inc., Lebanon, NH, for thermomechanical testing in an underground excavation for studies relating to disposal of high-level nuclear waste in salt rock (Shuri and Green, 1987).

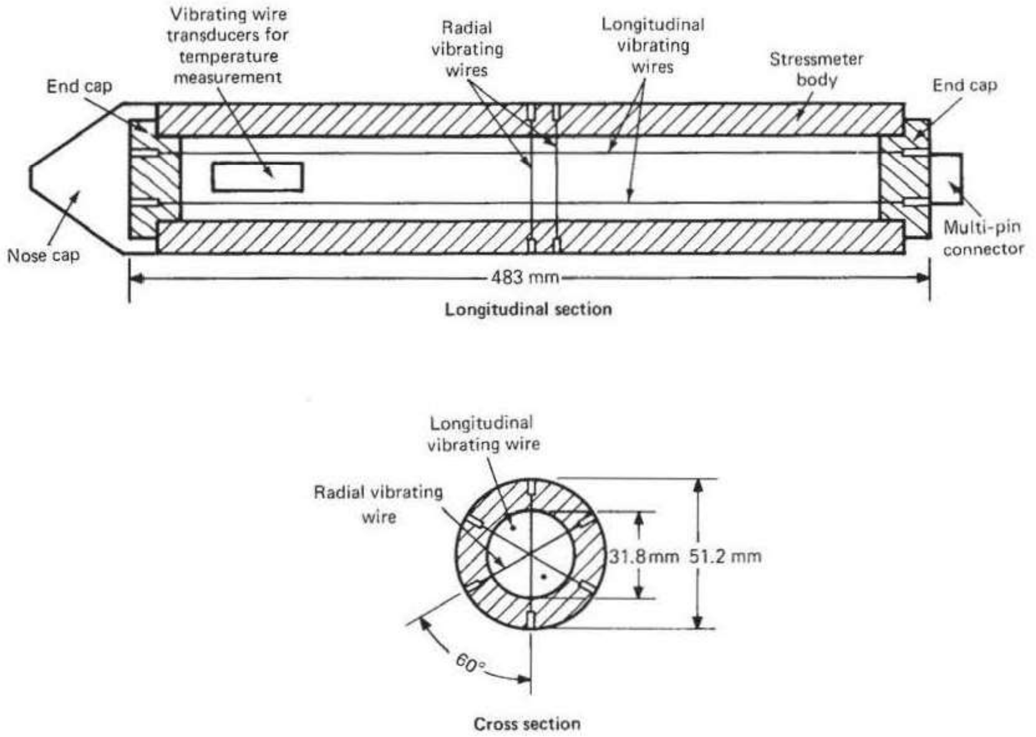



**Figure 11.7. Biaxial vibrating wire stressmeter (courtesy of Soil & Rock Instrumentation Division, Goldberg-Zoino & Associates, Inc., Newton, MA, and Geokon, Inc., Lebanon, NH).**

Radial deformation is measured using three vibrating wire transducers mounted at 60 degree intervals, and two such sets are included to provide redundancy. Two longitudinal transducers are also included to allow for correction of extensional effects, and two additional vibrating wire transducers are incorporated to provide data for thermal corrections. The stressmeter is grouted into the borehole using a cement/water mix. Extensive large-scale laboratory testing has been performed under triaxial stress conditions and at elevated temperatures.

There is currently no field experience with this stressmeter, but a similar instrument has been used successfully in the field to monitor stress in ice. The scale and extent of the laboratory testing program are both large, and the development appears promising. The results of the testing program indicate that, when installed in salt rock, the stressmeter has only minor dependence on characteristics of the rock and of the grout between stressmeter and rock. Test results were also satisfactory under compressive loading in granite with an intact modulus up to $10 \times 10^6$ lb/in.$^2$ (70 GPa).

### 11.4.2. Photoelastic Stressmeters

A photoelastic stressmeter (Hall and Hoskins, 1972; Roberts and Hawkes, 1979) consists of a cylindrical glass plug with an axial hole, together with a light source and polarizing filter. The meter is bonded around its periphery into a borehole as shown in Figure 11.8. When the glass plug is subjected to stress, light and dark areas are visible when the plug is illuminated with polarized light and viewed through a hand analyzer. The light and dark areas are referred to as *photoelastic interference fringes*, and the change in the number of fringes is proportional to shear strain in the glass. Methods for installation and reading photoelastic stressmeters are described by Roberts and Hawkes (1979).

Use of photoelastic stressmeters is limited by the need for experienced reading personnel, and they are not viable instruments for general usage.

### 11.4.3. Tapered Plugs

Several solid metal instruments with a tapered outer surface have evolved in an attempt to create a posi-

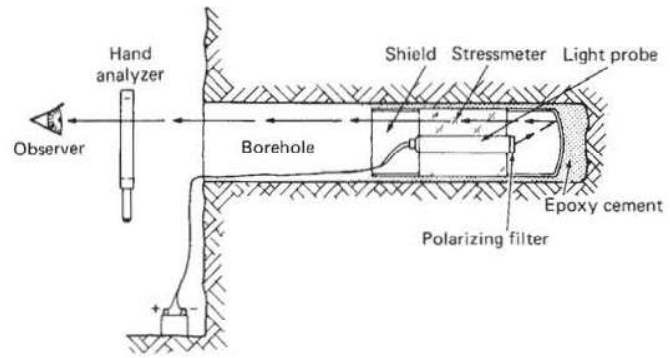



**Figure 11.8. Photoelastic stressmeter (after Roberts and Hawkes, 1979).**

tive contact between the instrument and the rock. They are forced into a matching tapered socket drilled at the bottom of a borehole.

Most designs are variations on those of Potts and Tomlin (1960) and of Wilson (1961). Wilson's gage consists of a tapered brass plug, split longitudinally to form a plane to which resistance strain gages are bonded, then rejoined. The instrument is designed for measuring stress change in coal and is forced into a matching tapered socket drilled at the bottom of a borehole. It can be used for determination of stress change across one diameter of the borehole only; that is, it is a uniaxial device. Enever et al. (1977) describe a similar uniaxial instrument, manufactured from steel, for determining stress changes in rock of higher modulus. Variations of this instrument, with strain gage rosettes mounted on planes transverse to the axis of the instrument to create biaxial gages, are described by Truong (1977) and Walton and Matthews (1978).

The main disadvantage of tapered plugs is their requirement for nonstandard boreholes and the consequent increased drilling cost. Walton and Matthews (1978) report that it is sometimes difficult to achieve adequate and reproducible transfer of stress from the rock to the gage, owing to uneven contact with the rock. It is also difficult to install these gages into a borehole in a way that produces moderate radial compression within the gage. They can therefore be used to monitor increase in stress but not reduction in stress, unless first radially compressed by a stress increase.

### 11.4.4. Miscellaneous Rigid Inclusion Gages

A rigid inclusion stressmeter with mechanical components similar to the vibrating wire stressmeter but with a bonded electrical resistance strain gage transducer is described by Cook and Ames (1979). Peng et al. (1982) describe a biaxial stressmeter consisting of a thick steel pipe with a transverse central portion on which a resistance strain gage rosette is mounted. It is bonded into a borehole with epoxy resin.

Spathis and Truong (1987) have analyzed the effect of placing a layer of in-fill material between a cylindrical plug and the wall of the borehole. A biaxial inclusion design that is bonded to the rock using a thick layer of high-modulus cement grout has been proposed, but no laboratory or field trials have been reported. Percussion drilled boreholes could be used with this design.

### 11.4.5. Comparison Among Rigid Inclusion Gages

Comparative information is summarized in Table 11.2.

Several of the gages fill the borehole completely. This is considered an advantage in all rock types, because when compressive stress is high, the support provided by the gage will help to minimize stress concentrations around the borehole. In contrast, with the "open" borehole condition that exists with most soft inclusion gages, failure of the rock around the borehole may occur because of concentration of stress. In viscoelastic rocks, if the gage does not fill the borehole completely, the rock will creep around the gage and results are likely to be incorrect.

## 11.5. RECOMMENDED PROCEDURES FOR MEASUREMENT OF STRESS CHANGE IN ROCK

Methods for the determination of stress change in rock masses have been a major area of research for

**Table 11.2. Rigid Inclusion Gages**

| Method | Advantages[a] | Limitations[b] |
|---|---|---|
| Uniaxial vibrating wire stressmeter (Figure 11.5) | Lead wire effects minimal<br>Installation is relatively simple<br>Can be used to monitor reduction in compressive stress of up to about 5000 lb/in.$^2$ (35 MPa) | Calibration dependent on contact geometry and initial preload<br>Calibration somewhat dependent on rock deformability in high-modulus rock<br>Gage does not fill borehole completely<br>Single measurement axis<br>Risk of wedge slippage if blasting nearby |
| Biaxial vibrating wire stressmeter (e.g., Figure 11.7) | Biaxial<br>Lead wire effects minimal<br>Gage fills borehole completely | Cannot be used to monitor large reductions in compressive stress[c]<br>No field experience in rock |
| Photoelastic stressmeter (Figure 11.8) | Biaxial<br>Gage fills borehole completely<br>Poor cementing is immediately apparent<br>Shows principal stress directions very well | Cannot be used to monitor large reductions in compressive stress[c]<br>Lack of wide commercial availability<br>Readings subjective<br>Not amenable to remote readout |
| Tapered plugs | Gage fills borehole completely | Lack of wide commercial availability<br>Custom drilling equipment required<br>Low electrical output; lead wire effects; errors resulting from moisture, temperature, and electrical connections are possible<br>Cannot be used to monitor large reductions in compressive stress[c]<br>Not recommended for use |

[a] Requires knowledge of rock properties and constitutive behavior only within broad bounds.
[b] Small measurement scale, but limitation can be minimized by installing several instruments in the rock mass of interest.
[c] Unless first subjected to a significant increase in compressive stress.

many years. Because it is impossible to measure stress or stress change directly, indirect techniques are required, and theoretical and practical difficulties are common. Monitoring stress changes involves the accurate measurement of small quantities over a long time period, often in a harsh environment, and at this stage of development there is no universal technique that can be applied to every rock type, rock mass condition, and measurement situation.

As indicated in Section 11.1, it is important to realize that the actual stress within a rock mass may vary significantly from point to point, depending on geologic features and the local stress concentrating effect of an opening. A single point measurement of stress may therefore be misleading, and if reliable data are required it is usually necessary to make measurements at numerous points throughout the rock mass of interest. Whenever possible, attempts should be made to determine the locations of discontinuities and to install the instruments in zones of intact rock.

When deciding on a technique to monitor stress change, consideration must be given to the type and condition of the rock mass, its stiffness, and whether uniaxial, biaxial, or triaxial measurements are required. The problem of attaching a measuring instrument to the rock and ensuring that the instrument experiences the correct stress is a general problem, common to nearly all instruments. Firm recommendations are impossible and the following recommendations are intended as guidelines only, to be used while bearing in mind the advantages and limitations given earlier. The guidelines are summarized in Table 11.3.

When considering accuracy of measurements, the authors concur with the following views expressed by IECO (1979):

> Evaluation of the accuracy of in situ stress-change measurement devices is difficult. Many types of gages . . . have produced data for periods of a year or more after emplacement. In most cases, the consistency of the stress change readings has led inves-

**Table 11.3. Guide to Recommended Procedures for Measurement of Stress Change in Rock**[a]

| Number of Measurement Axes | Elastic Rock | Viscoelastic Rock |
|---|---|---|
| Uniaxial | Uniaxial vibrating wire stressmeter | Borehole pressure cell |
| Biaxial | Three uniaxial vibrating wire stressmeters<br>Biaxial vibrating wire stressmeter<br>CSIRO H.I. triaxial strain cell<br>CSIRO yoke gage | Three borehole pressure cells<br>Biaxial vibrating wire stressmeter |
| Triaxial | CSIRO H.I. triaxial strain cell | No instrument available |

[a]Comparative information among options is given in Sections 11.3 and 11.4, and summarized in Tables 11.1 and 11.2.

tigators to assume the results are reliable to at least ±25 percent. . . . But no detailed effort has been reported to document either gage accuracy or the rock and gage characteristics that affect the gage accuracy.

### 11.5.1. Uniaxial Measurements in Elastic Rock

The uniaxial vibrating wire stressmeter is well suited to applications in this category. However, the guidelines given in Section 11.4.1 should be followed carefully.

### 11.5.2. Biaxial Measurements in Elastic Rock

The choice for biaxial measurements is among four gages: a group of three uniaxial vibrating wire stressmeters, the biaxial vibrating wire stressmeter, the CSIRO H.I. cell, and the CSIRO yoke gage.

Provided that the Section 11.4.1 guidelines are followed, three uniaxial vibrating wire stressmeters, installed at different but known orientations in a single borehole, can be used in an attempt to make biaxial measurements. The dependence on deformability characteristics is within reasonable limits, and the vibrating wire transducers provide a stable signal that cannot be degraded readily. However, recognizing that contact geometry and initial preload cause uncertainties in calibration, substantial errors may arise from the combination of data from three stressmeters. Also, the capability for measuring large reductions in compressive stress depends greatly on the amount of preload created during installation.

The biaxial vibrating wire stressmeter, although having no field experience in rock, is an option for use in elastic rock with an intact modulus up to about $10 \times 10^6$ lb/in.$^2$ (70 GPa), provided that the stress change is known to be compressive. As for the uniaxial stressmeter, the vibrating wire transducers provide a stable signal.

The CSIRO H.I. cell has been used successfully for measuring stress change, but there is a potential for errors caused by creep and moisture absorption. The errors can be minimized by installing the cell at least 1 month before requiring data. Measurement accuracy depends on the accuracy of deformability estimates or measurements, and electrical resistance transducers are subject to lead wire effects, and to the possibility of errors caused by moisture, temperature, and electrical connections.

The CSIRO yoke gage, although having limited field experience, has also been used successfully for monitoring stress change. As for the H.I. cell, measurement accuracy depends on the accuracy of deformability estimates or measurements, and the electrical resistance transducers can be sources of error.

It is important that readers should not make a selection among these four options, based merely on a reading of this summary section. The factors discussed earlier and summarized in Tables 11.1 and 11.2 should also be studied. Any one of the four options, depending on the specifics of each case and on comparative costs, may be the instrument of choice.

### 11.5.3. Triaxial Measurements in Elastic Rock

When monitoring stress changes in elastic rock, use of nontriaxial instruments involves assumptions for the magnitude of stress changes that occur in directions or planes in which there is no measurement. A triaxial instrument is required to measure any rotation of the principal stresses and to monitor stress changes in a situation where a borehole cannot be drilled perpendicular to the stress direction or plane of interest. Although the CSIRO hollow inclusion triaxial strain cell was not originally designed for monitoring stress change, with due care and consideration it can be used in elastic rocks when triaxial data are required. Because of the limitations reviewed in the previous section, it is best suited to monitoring three-dimensional stress changes that occur over short time periods (approximately 1–2 months), rather than a slow buildup or decrease in stress levels.

### 11.5.4. Measurements in Viscoelastic Rock

Borehole pressure cells are suitable for measurement of stress change in viscoelastic rock. The biaxial vibrating wire stressmeter, although having no field experience in rock, is an attractive alternative for biaxial measurements, and of course it can also be used if only uniaxial data are required.

# CHAPTER 12

# MEASUREMENT OF DEFORMATION

## 12.1. INSTRUMENT CATEGORIES

Instruments for measuring deformation can be grouped in the categories listed in Table 12.1. Definitions of each category, together with an indication of typical applications, are given in later sections of this chapter. It can be seen that there is a vast array of instruments for monitoring deformation, but Peck (1972) warns:

> An instrument too often overlooked in our technical world is a human eye connected to the brain of an intelligent human being. It can detect most of what we need to know about subsurface construction. Only when the eye cannot directly obtain the necessary data is there a need to supplement it by more specialized instruments. Few are the instances in which measurements by themselves furnish a sufficiently complete picture to warrant useful conclusions.

## 12.2. SURVEYING METHODS*

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

Surveying methods are described briefly in the following subsections, and comparative information is given in Table 12.2. Reference datums and measuring points for monitoring surface deformation are also described in this section.

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

Discussions of surveying methods by Cording et al. (1975), Gould and Dunnicliff (1971), and Senne

*Written with the assistance of Thomas S. McGrath, Land Surveyor, Upper Montclair, NJ, and Joseph H. Senne, Professor and Chairman, Department of Civil Engineering, University of Missouri–Rolla, MO.

**Table 12.1. Categories of Instruments for Measuring Deformation**

| Category | Type of Measured Deformation | | | | | | Section |
|---|---|---|---|---|---|---|---|
| | ↔ | ↕ | ⤢ | ↻ | ∸ | ⨪ | |
| SURVEYING METHODS | • | • | • | | • | | 12.2 |
| Optical and other methods | | | | | | | |
| Benchmarks | | | | | | | |
| Horizontal control stations | | | | | | | |
| Surface measuring points | | | | | | | |
| SURFACE EXTENSOMETERS | • | • | • | | • | | 12.3 |
| Crack gages | | | | | | | |
| Convergence gages | | | | | | | |
| TILTMETERS | | | | • | • | • | 12.4 |
| PROBE EXTENSOMETERS | • | • | • | | | • | 12.5 |
| Mechanical heave gage | | | | | | | |
| Mechanical probe gages | | | | | | | |
| Electrical probe gages | | | | | | | |
| Combined probe extensometers and inclinometer casings | | | | | | | |
| FIXED EMBANKMENT EXTENSOMETERS | • | • | • | | | • | 12.6 |
| Settlement platform | | | | | | | |
| Buried plate | | | | | | | |
| Mechanical gage with tensioned wires | | | | | | | |
| Gages with electrical linear displacement transducers | | | | | | | |
| Soil strain gage | | | | | | | |
| FIXED BOREHOLE EXTENSOMETERS | • | • | • | | | • | 12.7 |
| Single-point and multipoint extensometers | | | | | | | |
| Subsurface settlement points | | | | | | | |
| Rod settlement gage | | | | | | | |
| INCLINOMETERS | • | • | • | • | | • | 12.8 |
| TRANSVERSE DEFORMATION GAGES | • | • | • | | | • | 12.9 |
| Shear plane indicators | | | | | | | |
| Plumb lines | | | | | | | |
| Inverted pendulums | | | | | | | |
| In-place inclinometers | | | | | | | |
| Deflectometers | | | | | | | |
| Borehole directional survey instruments | | | | | | | |
| LIQUID LEVEL GAGES | | • | | | | • | 12.10 |
| Single-point and multipoint gages | | | | | | | |
| Full-profile gages | | | | | | | |
| MISCELLANEOUS DEFORMATION GAGES | | | | | | | 12.11 |
| Telltales | • | • | • | | • | • | |
| Convergence gages for slurry trenches | • | | | | | • | |
| Time domain reflectometry | • | • | • | | • | • | |
| Fiber-optic sensors | • | • | • | | • | • | |
| Acoustic emission monitoring | • | • | • | | | • | |

Key: ↔ horizontal deformation; ↕ vertical deformation; ⤢ axial deformation (↔ or ↕ or in between); ↻ rotational deformation; ∸ surface deformation; ⨪ subsurface deformation.