Chapter 5

Field Instrumentation

Stanley D. Wilson and P. Erik Mikkelsen

The development of field instrumentation has had a significant impact on geotechnical engineering in the past 15 years. Of particular importance is the notable contribution of instrumentation to landslide-oriented problems (*5.50*). The usefulness of field instrumentation for the identification of landslide movements, for the monitoring of slides that have been remedially treated, and for other geotechnically oriented measurements has been described in the literature (*5.1, 5.8, 5.18, 5.36, 5.45, 5.48, 5.50, 5.56*). Although the earlier Highway Research Board Special Report on landslides (*5.20*) mentioned the use of simple instrumentation (which is by no means obsolete and is described in this chapter), its publication generally preceded the introduction of modern instruments. The economic impact of slope failures created a demand for better instrumentation and led to its general acceptance and use. This chapter describes the types of instrumentation currently available, their limitations, and their specific applications to slope instability problems. In addition, traditional approaches to the determination of slide movements and groundwater levels are reviewed.

The need to gather certain quantitative data to analyze slope-stability problems and to design remedial measures is discussed in other chapters. Topographic mapping, geologic mapping, subsurface soil and groundwater investigations, and laboratory strength testing are normally performed to aid in determining the cause of the slide, the mode of failure, and the physical and engineering characteristics of the soil and rock involved. Such investigations are necessarily performed on already developed landslides, the characteristics of which may be observed and noted. However, small movements of a soil mass prior to or even at incipient failure are usually not visually evident; so the value of information that can be obtained at the ground surface is limited. However, instrumentation can provide valuable information on incipient, as well as fully developed, landslides. In this respect, instrumentation is not intended to replace field observations (*5.38*) and investigative procedures discussed in other chapters. Instead, it augments other data by providing supplementary information and by warning of impending major movements. Typical situations for which various instruments have been used are

1. To determine the depth and shape of the sliding surface in a developed slide so that calculations can be made to determine the available soil strength parameters at failure and so that remedial treatments can be designed;
2. To determine absolute lateral and vertical movements within a sliding mass;
3. To determine the rate of sliding (accelerating or decelerating movements) and thus warn of impending dangers;
4. To monitor the activity of marginally stable natural slopes or cut slopes and the effects of construction activity or precipitation on them;
5. To monitor groundwater levels or pore pressures normally associated with landslide activity to enable effective stress analyses to be performed;
6. To provide remote digital readout or a remote alarm system that would warn of possible dangers; and
7. To monitor and evaluate the effectiveness of various control measures.

In the last situation, savings are often realized in remedial treatment by a planned and monitored sequence of construction. For example, drainage might be initially installed and its effect monitored to determine whether a planned buttress is actually necessary.

INSTRUMENTATION PLANNING

Adequate planning is required before a specific landslide is instrumented. The steps are (a) determine what types of measurements are required, (b) select the specific types of instruments best suited to make the required measurements,

(c) plan the location, number, and depth of instrumentation, and (d) develop the recording techniques.

Initially the planning process requires the development of ideas on the causes of the landslide and the probable limits of the depth and outer boundaries of the movements. Of course, to have the answers before the system is planned would be helpful. Reconnaissance of the area, study of the geology, review of rainfall records, and observation of topographic features, especially recent topographic changes, will often provide clues. Unfortunately, no two slides are alike in all details, and experience alone without the application of judgment may lead to erroneous concepts.

An instrumentation system in which the instruments do not extend below the zone of movement, were installed at the wrong locations, or are unsuitable is a waste of time and money. Loss of time may mean that corrective treatment is started too late to save the project.

Types of Measurements Required

Landslides, by definition, involve movement, and the magnitude, rate, and distribution of this movement are generally the most important measurements required. Equally important in many slide problems, however, are measurements of pore-water pressures within the slide area, particularly in layered systems in which excess hydrostatic pressures may exist between layers.

If the depth of sliding is readily apparent from visual observations, surface measurements may be sufficient for obtaining the rate of movement. The surface measurements should extend beyond the uppermost limit of visual movement so that possible extension in advance of cracking can be monitored. Vertical and horizontal measurements of movement of the ground surface at various locations within the slide area should be obtained. Vertical offsets, widening of cracks, and toe heave should be monitored. The direction of movement can often be inferred from the pattern of cracking, particularly by the matching of the irregular edges of the cracks. If the depth and thickness of the zone of movement are not apparent, inclinometers or similar devices that can detect the movement with depth must be used. Pore pressures at or near the sliding surface must be measured to enable an effective stress analysis to be performed and to assess the adequacy of drainage measures. Rapid-response piezometers are advantageous, particularly in impervious soils.

Selection of Instrument Types

Many types and models of instruments are available for measuring the changing conditions in a landslide; they vary in degree of sophistication, particularly in regard to readout capabilities. Instruments have been developed to measure vertical and horizontal earth movements, pore-water pressures, in situ stresses and strains, dynamic responses, and many other parameters. However, in most landslide problems, the measurement of horizontal movement of the foundation soil or rock and the measurement of pore-water pressures are of primary concern. Instruments commonly used for these purposes are described in this chapter.

If the movement is known to be along a well-defined shear plane, such as a bedding plane or fracture, simple probe pipes will suffice to determine the depth. If the movements are large and rapid, accuracy is not an essential requirement and even relatively crude inclinometers may suffice. When the rate of movement is small and the depth and distribution are not known, more precise instrumentation is required. Carefully installed precision inclinometers are best in such instances, although there may be cases in which extensometers or strain meters can be used to advantage.

The Casagrande type of piezometer is the most useful general purpose pore-pressure measuring device, but may give too slow an initial response in fine-grained soils; therefore, pneumatic or electric types may be preferable. High-air-entry, low-flow piezometers should be used in clays or clay-shales in which permeabilities are low or suctions may be present because of unloading (*5.53*).

The types of instruments, layout, and monitoring schedules are usually determined by the specific needs of a project. Several basics, however, should be thoroughly evaluated for any system. Instruments should be reliable, rugged, and capable of functioning for long periods of time without repair or replacement. They must also be capable of responding rapidly and precisely to changes so that a true picture of events can be maintained at all times. High sensitivity is usually a prerequisite when performance is monitored during construction, since it is often the rate of change rather than the absolute value that provides the key to proper interpretation. The location of instruments requires a thorough understanding of the geologic and subsurface conditions if meaningful data are to be obtained. This is particularly true of pore-pressure recording devices that are intended to measure pressures in specific zones of weakness or potential instability. Since most measurements are relative, a stable base or datum must be provided so that absolute movements can be determined.

In this chapter, the primary emphasis is on measurement techniques and instruments for landslides; less emphasis is placed on conventional surface surveying techniques. Conventional surveying methods are fundamental to displacement measurements at the surface (*5.23*); this has been discussed more fully in Chapter 4. Modern deformation instruments for landslides have augmented the methods of surface measurements, but more important they have expanded capabilities in terms of accuracy, simplicity, and ease or convenience of operation. In addition, an ever-expanding range of instruments makes possible the measurements of subsurface deformations (*5.13, 5.17, 5.21, 5.25, 5.44*). Table 5.1 gives summaries of various survey methods, instruments for measuring subsurface deformations, and piezometers.

SURFACE SURVEYING

Conventional Surveying

In an active slide area, surface movements are normally monitored to determine the extent of slide activity and the rate of movement (*5.21, 5.32*). Optical instrument surveys and tape measurements are used to determine lateral and vertical movements. Bench marks and transit stations, located on stable ground, provide the basis for which subsequent movements of hubs can be determined optically and

Table 5.1. Surveying methods, horizontal movement devices, extensometers, and piezometers (*5.13*).

Method or Measurement Instrument	Type	Range	Accuracy	Advantages	Limitations and Precautions	Reliability
Method	Chaining	Variable		Is simple and inexpensive; has direct observation	Requires clear, relatively flat surface between points and stable reference monuments Corrections for temperature and slope should be applied and a standard chain tension used	Excellent
	Ordinary, third order		±1/5000 to 1/10 000 of distance			
	Precise, first order		±1/20 000 to 1/200 000 of distance			
	Electronic distance measurement (EDM)	20 to 3000 m	±1/50 000 to 1/300 000 of distance	Is precise, long range, fast, usable over rough terrain	Accuracy is influenced by atmospheric conditons; accuracy at short ranges (<30 to 90 m) is limited for most instruments	Good
	Optical leveling					
	Ordinary, second and third order		±3 to 6 mm	Is simple, fast, particularly with self-leveling instruments	Has limited precision; requires good bench mark nearby	Excellent
	Precise, first order (parallel plate micrometer attachment, special rod)		±0.6 to 1.2 mm	Is more precise	Requires good bench mark and reference points and careful adherence to standard procedures	Excellent
	Offsets from a baseline					
	Theodolite and scale	0 to 1.5 m	±0.6 to 1.5 mm	Is simple; has direct observation	Requires baseline unaffected by movements and good monuments; accuracy can be improved by using a target with a vernier and by repeating the sight from the opposite end of the baseline	Excellent
	Laser and photocell detector	0 to 1.5 m	±1.5 mm	Is faster than transit	Is seriously affected by atmospheric conditions	Good
	Triangulation		±0.6 to 12 mm	Is usable when direct measurements are not possible; is good for tying into points outside of construction area	Requires precise measurement of base distance and angles; requires good reference monuments	Good
	Photogrammetric		±1/5000 to 1/50 000 of distance	Can record hundreds of potential movements at one time for determination of overall displacement pattern	Weather conditions can limit use	Good
Horizontal movement instrument	Long base strain meter with electrical readout	150 mm	0.3 mm	Is precise; can be used to check horizontal movement at top of other devices, such as inclinometers	Has risk of electrical failure; has limited application for tunnels	Good
	Fixed multipoint borehole deflectometers		±20 arc s[a]	Is available in portable version and double pivot version to measure movement along two axes	Is complex; does not measure continuous profile	
	Fixed multipoint borehole inclinometers		±0.03 mm in 3 m	Is precise; can be removed for repairs or reuse; uses standard inclinometer casing	Is complex; does not measure continuous profile Accuracy is lost when removed and replaced	
	Portable borehole inclinometers					
	Wheatstone bridge pendulum	±12°, optional to ±25°	±20 mm in 30 m	Has long experience record; is not sensitive to temperature	Requires lengthy calculations; reads one axis at a time; has no provisions for automatic readout	Very good

Table 5.1. Continued.

Method or Measurement Instrument	Type	Range	Accuracy	Advantages	Limitations and Precautions	Reliability
	Accelerometer	±30°, optional to ±90°	±5 mm in 30 m	Reads two axes at a time; has automatic readout and recording provisions	Requires lengthy calculations without automatic readout; requires manual check of data for errors with automatic readout	Good
	Vibrating wire	±15° or 20°	±10 mm in 30 m	Is available in single or double axis models	Requires lengthy calculations Errors due to zero drift are possible	Good
	Bonded resistance strain gauge	±20°	±10 mm in 30 m	Has adjustable range on some models; uses ordinary square tubing for casing on one model	Requires lengthy calculations; reads one axis at a time Errors due to zero drift, temperature, or electrical connections are possible	Fair
Extensometer	Tape	1 to 30 m	±0.03 to 0.3 mm	Is simple, precise, portable; is good for measuring tunnel diameter changes	Has accuracy limited by tension adjustment; requires temperature correction	Excellent
	Portable rod	1 to 8 m	±0.03 to 0.3 mm	Is simple, precise, portable	Has limited span; has accuracy limited by sag; invar tubes can be used to minimize temperature corrections	Excellent
	Weight-tensioned wire	Variable	±5 to 20 mm	Is simple	Has creep in wire that leads to errors; is vulnerable to damage in tunnel	Fair
	University of Illinois rod	150 mm	±0.03 to 0.13 mm	Is simple, precise, and large range (some models can be reset and extended through linings placed later); can be quickly installed; is more blast and damage resistant; has easily adjusted anchors	Is not adaptable to remote reading; has two anchor units only	Good
	Interfels rod		Variable	Is simple, precise; can have multiple anchors; can accept remote readout transducers	Has projecting head that is vulnerable to damage	Good
	Variable-tensioned wire	15 to 90 mm[b]	±0.05 to 0.13 mm	Has multiple anchors, up to 6 or 8 (some models are designed for remote readings with transducers using bonded resistance or vibrating-wire strain gauges)	Has variable tension that requires varying calibration factors, wire friction and hysteresis that can seriously affect accuracy, risk of electrical failure, and projecting head that is vulnerable to damage	Fair; short term
	Constant-tensioned wire	50 mm[b]	±0.05 to 0.13 mm	Has multiple anchors; is designed for remote reading using potentiometers; has constant calibration factor	Has wire friction and hysteresis that can seriously affect accuracy, risk of electrical failure, and projecting head; is complex mechanically	Fair; short term
Piezometer	Open system		±3 mm head	Is simple, inexpensive, and adequate for most earth problems	Central observation system cannot be used	Excellent
	Well point			Is simple, inexpensive, and universally available; can be driven in place	Has large time lag in low porosity materials ($k < 1$ μm/s) and metallic elements that may corrode; cannot measure negative pore pressures	
	Casagrande			Is simple, inexpensive; has no metallic elements, long service life, and provisions for offset riser pipe and flushing tip	Cannot measure negative pore pressures; requires borehole and carefully placed bentonite or grout seal	

Table 5.1. Continued.

Method or Measurement Instrument	Type	Range	Accuracy	Advantages	Limitations and Precautions	Reliability
	Geonor			Can be pushed or driven into soft ground; can be placed in borehole with filter zone to reduce time lag	Cannot measure negative pore pressures	
	Cambridge			Has simple drivable piezometer with inexpensive tip and shield to protect tip during driving	Cannot measure negative pore pressures; has metallic elements that may corrode	
	Closed system		—[c]	Allows central observation system to be used; can measure negative pore pressures; is usable in low permeability soils	Is more difficult to install than open system piezometers; requires frequent and careful deairing	Excellent
	USBR			Is simple, inexpensive, and readily available; has long experience record		
	Bishop			Is simple, and designed for less frequent deairing than USBR type; can be pushed into soil from bottom of borehole		
	Diaphragm		±1% of full scale[c]	Has small time lag; is usable in low permeability situations; allows central observation system to be used; can measure negative pore pressures	Is costly and difficult to install and operate	Good to fair
	Pneumatic			Is not subject to freezing; uses smaller, less expensive tubing	Cannot measure negative pore pressures; has leakage and moisture in lines	
	Hydraulic			Is simple and easy to seal against leakage	Cannot measure negative pore pressures; requires constant volume pumps or flow control valves	
	Electrical re-resistance strain gauge		±1% of range[d]	Can often be locally fabricated from commercially available parts; is adaptable to automatic data recording; can measure negative pore pressures	Has often limited service life; is susceptible to wiring damage	
	Vibrating-wire strain gauge			Is more reliable than resistance strain gauge type; is adaptable to automatic data recording		

Note: 1 m = 3.3 ft; 1 mm = 0.39 in.

[a] Total displacement accuracy depends on pivot spacing.

[b] Can be reset.

[c] Depends on pressure gauge used.

[d] Depends on transducer.

by tape measurement. As shown in Figure 5.1, transit lines can be established so that the vertical and horizontal displacements at the center and toe of the slide can be observed. Lateral motions can be detected by transit and tape measurements from each hub. When a tension crack has opened above the top of a slide, simple daily measurements across the crack can be made between two hubs driven into the ground. In many cases, the outer limit of the ground movements is not known, and establishing instrument setups on stable ground may be a problem.

Various techniques and accuracy achieved in optical leveling, offset measurements from transit lines, chaining distances, and triangulation have been discussed extensively in the literature (*5.8, 5.23*), particularly for dams, embankments, and buildings. Although conventional surveys, particularly higher order surveys, can define the area of movement, more accurate measurements may be required in many cases. Terzaghi (*5.47*) stated that "if a landslide comes as a surprise to the eyewitness, it would be more accurate to say that the observers failed to detect the phenomena which preceded the slide." The implication is that the smallest movements possible should be measured at the earliest possible time. Terzaghi (*5.47*) also describes the movements that precede a landslide.

The detection of small surface movements when cracking is not apparent requires a trained observer. If the ground surface is covered with rocks, or if there is a rock embankment, horizontal stretching will result in local instability of individual rocks such that walking over the slope gives one a sense of insecurity. Overturned rocks can be detected by a change in coloring or surface weathering. Trees inclined at the base but changing to vertical trunks a meter or so above the ground may indicate old slide movements. Inclined but straight trunks indicate recent movements. Cracks covered over with leaves or surface duff can be detected by walking over the area and noting the firmness of support. Frequently animals will avoid grazing in a potential landslide area because of uncertain support or hidden fissures. Small openings on the downhill side of structures, or next to tree trunks, may indicate creep. Overtaut or excessively sagging utility lines or misalignment of fence posts or utility poles are excellent indicators of ground movements. Such movements, when accurately monitored, serve as an important tool in assessing the potential hazard to structures, nearby residents, and the public.

Other Types of Surface Surveying

There are three rapidly developing surveying techniques today, and these will undoubtedly find increasing use in field measurements. Some are already in extensive use (e.g., electronic distance measurement and lasers); others are in limited use or are in the experimental stage (e.g., terrestrial photogrammetry). Electronic distance measurement (EDM) has changed surveying practices more than anything else in the last 100 years (*5.15*). EDM devices have proved particularly suited for use over rugged terrain; they perform more accurately and much faster than ordinary surveying techniques and require fewer personnel. Lightweight EDM instruments can be used efficiently under ideal conditions for distances as short as 20 m (66 ft) and as long as 3 km (2 miles); errors are as small as ±0.0032 m (±0.010 ft) (*5.28, 5.43*). Larger instruments using light waves or microwaves can be used at much longer distances. The accuracy of EDM is influenced by weather and atmospheric conditions; comparative readings with three different instruments are described by Penman and Charles (*5.40*).

EDM can be used to monitor large slides with large movements and provide a rapid way to survey many points on the mass from a single, readily accessible location. An example of such an installation involves an ancient landslide in Washington along the Columbia River, where the boundaries of the active slide are more than 0.6 km (1 mile) in width and 4.8 km (3 miles) in length. Yearly movements vary from only 1 m to 6 to 9 m (0.3 ft to 20 to 30 ft) and depend on the time of year and the rainfall. A permanent station, readily accessible all year, has been set up on the opposite side of the river, and monthly distance readings are taken to 14 points on the slide area and to 2 points located outside the slide zone. The distances involved vary from about 1500 to 6000 m (5000 to 20 000 ft). Figure 5.2a shows the movements (changes in distance) recorded by the EDM (electrotape) during a 1-year period for 2 selected points at the Columbia River slide, based on monthly readings. At the end of the year, the points were resurveyed by triangulation. The discrepancy is about 10 cm (4 in), which, although larger than anticipated, is quite satisfactory considering the total movements. Figure 5.2b shows recorded changes for 2 points believed to be on stable ground. The variation of monthly readings is seen to be ±0.06 m (±0.2 ft); variation is no greater for a 4765-m (15 630-ft) length than for a 1844-m (6050-ft) length.

Laser instruments are already widely used for setting alignments, and they are well suited for setting a reference line for offset measurements to surface monuments. Laser beams are also used with some EDM instruments. It should be possible to measure offsets with an error no greater than 0.003 to 0.006 m (0.01 to 0.02 ft) (*5.23*).

Terrestrial photogrammetry has been used in some cases on dam projects (*5.35*) and in mines, but no landslide measurements using this method have been found in the literature. Phototheodolites are used to take successive stereophotographs from a fixed station along a fixed camera axis; movements are identified in a stereocomparator, and accuracies of 0.006 to 0.009 m (0.02 to 0.03 ft) have been

Figure 5.1. Movement measurements in a typical slide area.

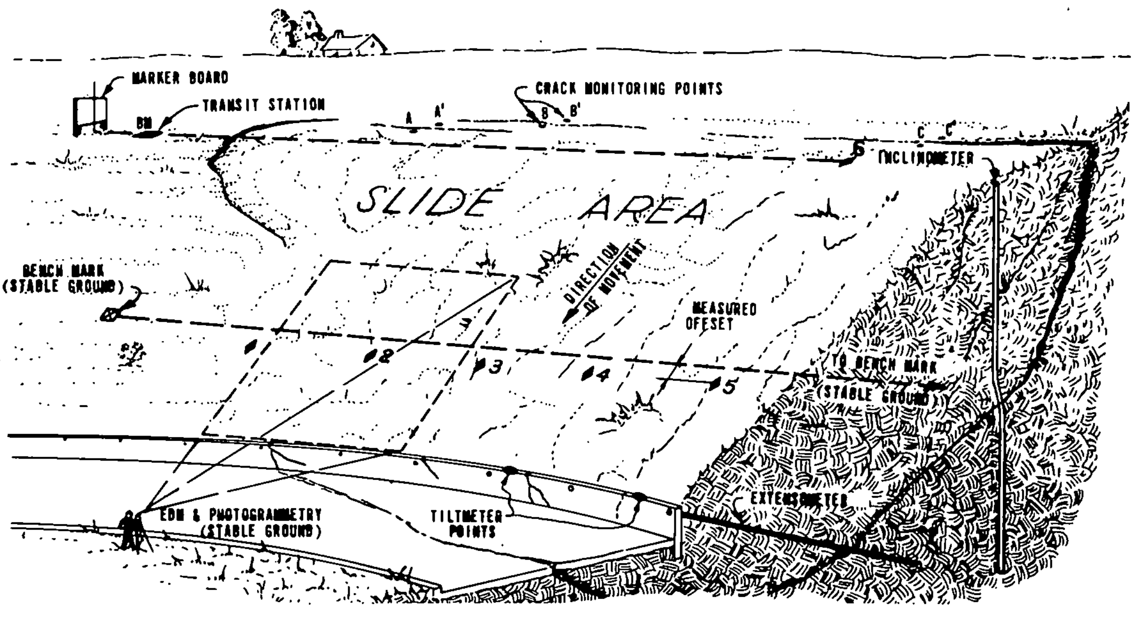
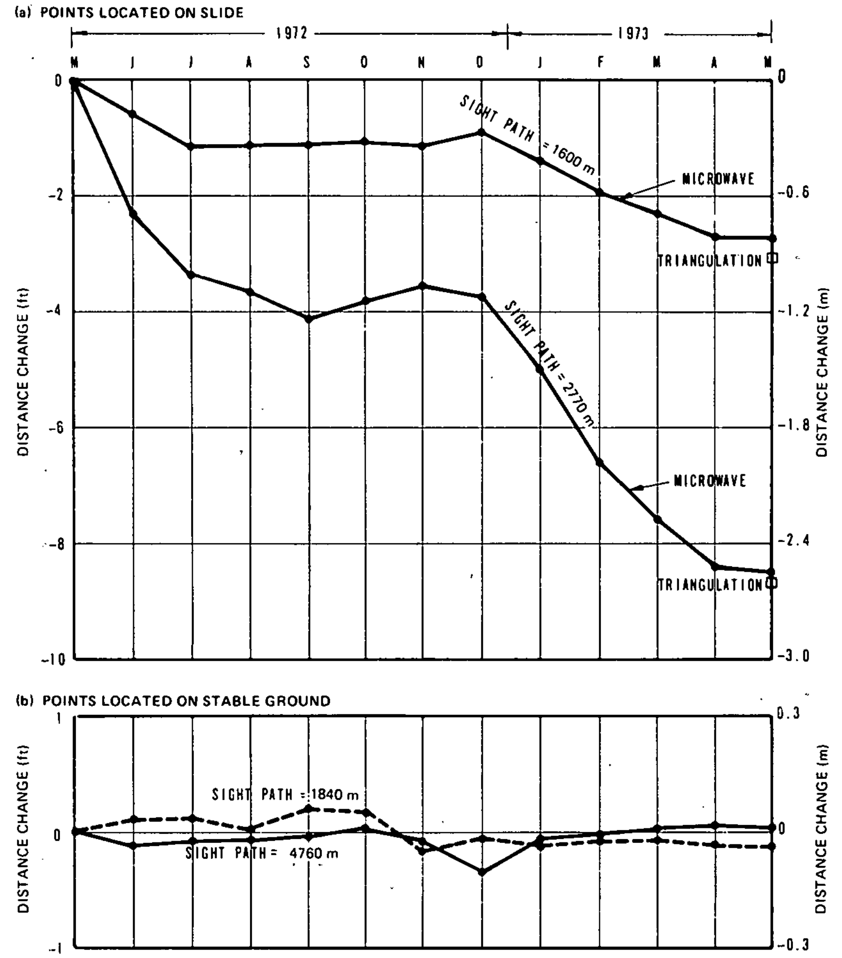


Figure 5.2. Microwave measurements of slide movements along Columbia River, Washington.



reported. Moore (*5.35*) reports average errors of about one order of magnitude greater than those from precise ground surveying.

Crack Gauges

When geologic mapping is considered adequate to describe the area affected by a landslide, simple qualitative measurements can provide knowledge of the activity. Movements on cracks, particularly those uphill and those downhill from the well-defined zone of movement, give clues to the increasing size associated with many landslides. Therefore, it is desirable to monitor the change in width, as well as the change in elevation, across the cracks. This can easily be done by direct measurement from hubs set on both sides of the cracks. Crude, simple gauges can be constructed in the field to provide a more accurate and continuing indication of crack movement. Periodic tape measurements can detect movements (and rates) not visually apparent.

In the areas of shear along the sides of the sliding area, a similar crack indicator system is used, except that three-point stations are often installed. However, because of the greater width of the shear zone, such simple devices may not be satisfactory. In jointed rock, the change in joint spacing can be determined by scribing marks or bonding washers on the rock surface on both sides of the joint. These indicator marks should be joined by a straight line so that both shear and opening or closing of the joint can be monitored. Another simple device is a hardwood wedge lightly forced into the joint crack with a mark placed on it at the level of the rock. If the joint opens, then the wedge will fall to a deeper level in the rock. However, if the joint is closing or subject to shear movement, such a wedge indicator may not be helpful.

Tiltmeters

Tiltmeters can be used to detect tilt (rotation) of a surface point, but such devices are relatively new and have had fairly limited use. They have been used mostly to monitor slope movements in open pit mines and highway and railway cuts, but they have potential application in landslide areas. They may be used in any area where the failure mode of a mass of soil or rock can be expected to contain a rotational component. One type of tiltmeter is shown in Figure 5.3, and sample tiltmeter data from a mine slope are shown in Figure 5.4. The same types of servo accelerometers are used with the tiltmeter as with sensitive inclinometers (a fairly rugged transducer with a similar range and sensitivity). The prime advantages are light weight, simple operation, and compactness at a relatively low cost.

INCLINOMETERS

The development of inclinometers has been the most important contribution to the analysis and detection of landslide movements in the past 2 decades. Although it has been used most extensively to monitor landslides, the inclinometer has gained widespread use as a monitoring instrument for dams, bulkheads, and other earth-retaining structures and in various areas of research (*5.1, 5.13, 5.17, 5.34, 5.44, 5.55, 5.59*). Since the introduction in the early 1950s of the pendulum-actuated inclinometer operating in grooved plastic casing (*5.45, 5.50*), the same basic concepts have been applied to instruments manufactured by at least 12 U.S. and foreign firms. Although the principle of operation has remained unchanged (Figure 5.5), inclinometers have been improved in accuracy and ease of operation and have undergone numerous modifications to adapt them to individual projects.

The inclinometer measures the change in inclination (or tilt) of a casing in a borehole (Figure 5.5) and thus allows the distribution of lateral movements to be determined as a function of depth below the ground surface and as a function of time (Figure 5.6). The application of the inclinometer to landslides is readily apparent, namely, to define the slip surface or zones of movement relative to the stable zones. Inclinometers have undergone rapid development to improve reliability, provide accuracy, reduce weight and bulk of instruments, lessen data acquisition and reduction time, and improve versatility of operations under adverse conditions. Automatic data-recording devices, power cable reels, and other features are now available.

An inclinometer system has four main components.

1. A guide casing is permanently installed in a near vertical borehole in the ground. The casing may be made of plastic, steel, or aluminum. Circular sections generally have longitudinal slots or grooves for orientation of the sensor unit, but square sections are used with some types.

2. A portable sensor unit is commonly mounted in a

Figure 5.3. Portable tiltmeter.

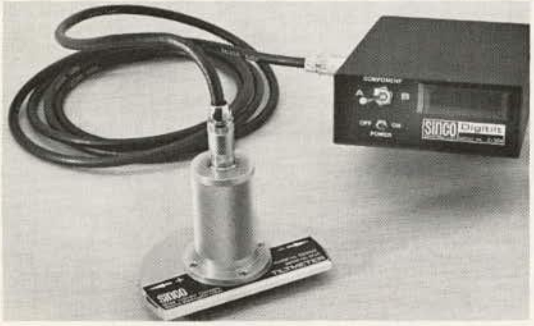


Figure 5.4. Tilt at ground surface due to advance of longwall mining face.

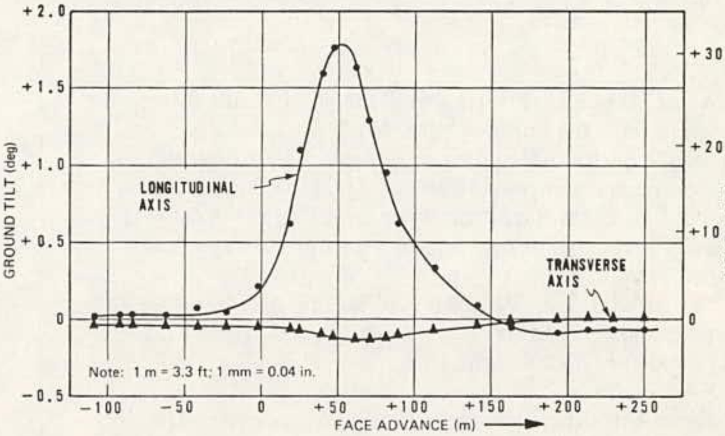


Figure 5.5. Principle of inclinometer operation.

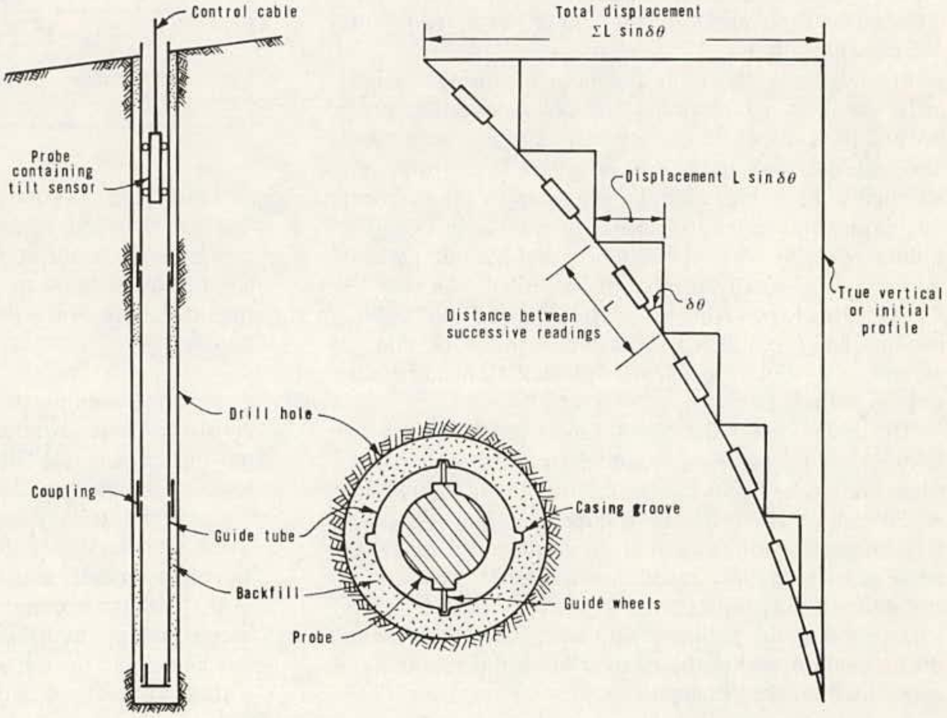
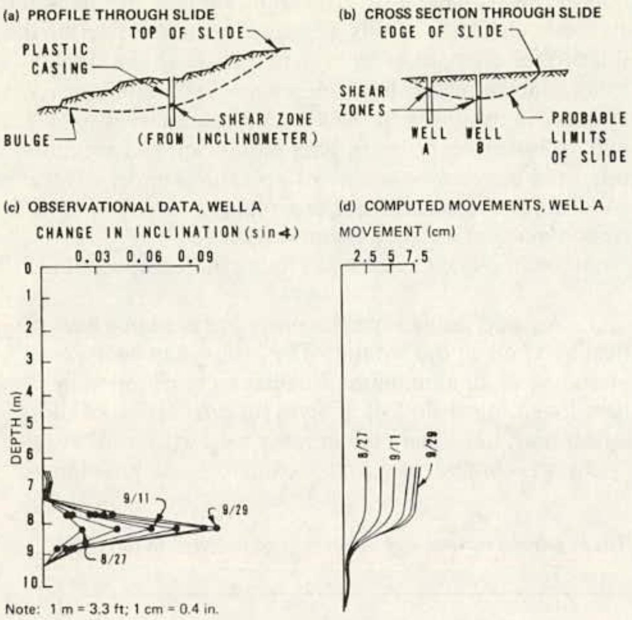


Figure 5.6. Measurement of slide movements with inclinometer.



Note: 1 m = 3.3 ft; 1 cm = 0.4 in.

carriage designed for operation in the guide casing (often referred to as the probe or torpedo).

3. A control cable raises and lowers the sensor unit in the casing and transmits electrical signals to the surface. For accurate depth control of the sensor unit, the cable is usually graduated or lowered on a separate surveyor's steel tape.

4. A portable control and readout unit at the surface supplies power, receives electric signals, and displays readings in dial or digital format.

Components other than these four are accessories. The need for accessories increases with the volume of output data (depth and number of inclinometers installed) and the need for rapid and efficient data retrieval, reduction, and presentation.

In landslide applications, inclinometer casings are normally installed in exploration drill holes extending through soil and rock suspected of movement and preferably well into materials that, in the best judgment at the time, are assumed to be stable. Samples are generally taken from the borings and tested to confirm such assumptions. The annular space between the drill hole and the outer wall of the casing is generally grouted or backfilled with sand. The success of the casing installation depends on the experience and the skill of the personnel; soil, rock, and groundwater conditions; depth of installation; and accessibility to the area.

The inclinometer instrument sensor unit (Figure 5.7) is lowered and raised on an accurately marked cable, its wheels or guides following the oriented, longitudinal slots of the casing. The response to slope changes in the casing are monitored and recorded at the surface. Readings are taken at fixed, usually equal, increments throughout the entire depth. Instruments differ primarily in the type of sensor used, in the accuracy with which the sensor detects the inclination, and in the method of alignment and depth control within the borehole.

Figure 5.7. Inclinometer.

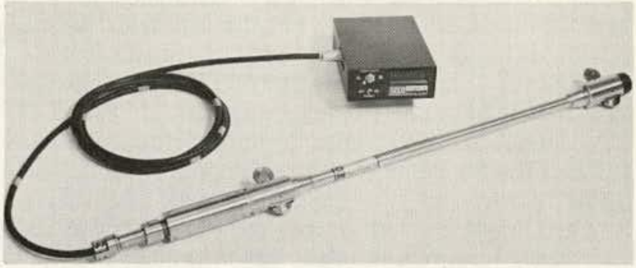
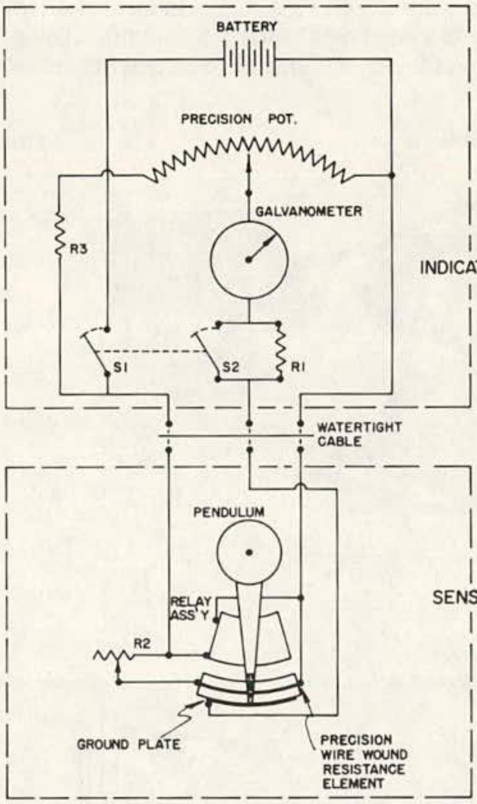


Figure 5.8. Pendulum-activated Wheatstone bridge circuit for typical inclinometer.



Four basic categories of inclinometers are in use in the United States today, and within each category are instruments of several different manufacturers. These categories are distinguished by the type of sensor selected to measure the inclination of the probe relative to the direction of gravity.

1. The pendulum-activated Wheatstone bridge circuit consists of a free-swinging, magnetically damped pendulum moving across a resistance coil; measurement for each sensor is in one plane (*5.24, 5.33, 5.54*).

2. In the strain-gauge type, bonded or vibrating-wire strain gauges are mounted around a stiff pendulum; measurement for each sensor is in one plane (*5.24*).

3. The accelerometer type is a closed-loop, servo-accelerometer circuit; measurement for each sensor is in one plane, and the use of two sensors per unit (biaxial) is common (*5.34*).

4. The photographic type photographically records the projection of a pendulum with two degrees of freedom on a target oriented by a magnetic compass; no orientation is needed within the casing or borehole. This type is not often used because its accuracy is generally much lower than that of previous types and data reduction is much more time consuming. Measurements with this instrument are described by Hanna (*5.25*).

With the exception of the photographic type, most inclinometers measure the inclination of the casing in two mutually perpendicular near-vertical planes. Thus, horizontal components of movement, both transverse and parallel to any assumed direction of sliding, can be computed from the inclinometer measurements. Deviations from this ideal occur because of limitations and compromises in manufacturing and because of installation circumstances. Instruments of different manufacture cannot generally be used interchangeably. In the interest of accuracy, the interchangeable use of instruments (probes) of the same manufacture also should be avoided.

The function of the inclinometer is to detect the change in inclination of the casing from its original near-vertical position. Readings taken at regular preestablished depths inside the casing allow the change in slope at various points to be determined; integration of the slope changes between any two points yields the relative deflection between those points. Repeating such measurements periodically provides data on the location, magnitude, direction, and rate of casing movement. The integration is normally performed from the bottom of the boring, since the bottom is assumed to be fixed in position and inclination (Figure 5.5).

All too often, confusing or unexplainable results appear because of instrumentation problems. These problems are common to all inclinometers; therefore, they are discussed in some detail so that, even without a detailed understanding of the inner workings of the instruments, one will be able to recognize these anomalies, the limits of accuracy, and the meaning of the results. In simple terms, the person closest to the job should be able to answer the basic questions: Is the slide active? How fast is it moving? How deep is it?

Inclinometer Sensors

Before a description is given of some of the sophisticated types of inclinometers, one device deserves mention. The borehole probe pipe, also called the slip indicator or poor boy, is one of the crudest of the subsurface measuring devices. It typically consists of a 25-mm (1-in) diameter semirigid plastic tube in a borehole. Metal rods of increasing length are lowered inside the tubing in turn, and the rod length that is just unable to pass a given depth gives a measure of the curvature of the tubing in the vicinity of that point. Toms and Bartlett (*5.49*) describe the use of this technique for the location of slip surfaces in unstable slopes. This type of measurement can easily be performed in the riser pipe of an observation well or open piezometer. If there are several shear planes, or if the shear zone is thick, a section of rod hung on a thin wire initially can be left at the bottom of the pipe; subsequently, the rod is pulled up to detect the lower limit of movement.

As originally conceived, the first modern inclinometers used pendulum-activated resistors to convert inclinations into electrical measurements; instruments of this type have had long and successful experience records (*5.45*). A typical inclinometer of this type consists of a pendulum, the tip of which makes contact with a resistance coil, subdividing the coil into two resistances that form one-half of a Wheatstone bridge (Figure 5.8). The other half of the bridge, contained in the portable control box, includes a precision potentiometer. The potentiometer readings are proportional to the inclination of the torpedo. The system has a sensitivity of about 3 min of arc and generally demonstrates a precision of ±13 to 25 mm (±0.50 to 1.0 in) over a 33-m (100-ft) casing (1:1000). The instrument is adaptable to most measurement programs; however, the data acquisition time is at least twice that of the more recent accelerometer types. Cornforth (*5.14*) and Green (*5.24*) describe the performance of this type of inclinometer in more detail.

The most sensitive transducer commercially available is the servo accelerometer. A servo accelerometer is composed of a pendulous "proof mass," that is free to swing within a magnetic field (Figure 5.9). The proof mass is provided with a coil or "torquer," which allows a linear force to be applied to the proof mass in response to current passed through the coil. A special sensing unit, called a pick-off, detects movement of the proof mass from a vertical position. A signal is then generated and converted by a restorer circuit, or servo, into a current through the coil, which balances the proof mass in its original position. In this manner, the current developed by the servo becomes an exact measure of the inertial force and thus of the transducer's inclination. The proof mass consists of either a pendulum with jewel-bearing support or a flexure unit that operates on the cantilever principle. Jewel-bearing accelerometers are fragile and subject to frictional interference at their bearings. Flexure accelerometers generally offer the same precision and have greater durability. Accelerometer systems have a sensitivity of approximately 18 s of arc in ranges from 30° to 90°, and they generally demonstrate a precision of ±1.3 to 2.5 mm (±0.05 to 0.10 in) over a 33-m (100-ft) casing (1:10 000). One excellent example of the precision of such measurements is shown in Figure 5.10. These data were taken with a servo-accelerometer sensor by the U.K. Transport and Road Research Laboratory during the advancement of a tunnel (*5.6*).

Temperature has only a negligible effect on readings taken with pendulum or accelerometer transducers. However, a marked variation of reading with temperature has been demonstrated for transducers using bonded resistance strain gauges (*5.24*), and errors resulting from temperature changes, as well as variations in zero drift, have been reported for inclinometers using vibrating-wire transducers (*5.17*). Even so, Burland and Moore (*5.11*) used a strain-gauge-based inclinometer to obtain some precise results for a diaphragm wall.

A recently developed inclinometer that is also used in a square casing (45-mm, 1.8-in, internal dimension) is described by Phillips and James (*5.41*). The unit has a cantilevered pendulum device with a four-arm resistance gauge bridge. Over a 5° range, the repeatability under ideal conditions is reported to be ±5 s of arc. However, the relatively small range may render it impractical for use in a

Figure 5.9. Servo accelerometer for inclinometer measurements (*5.13*).

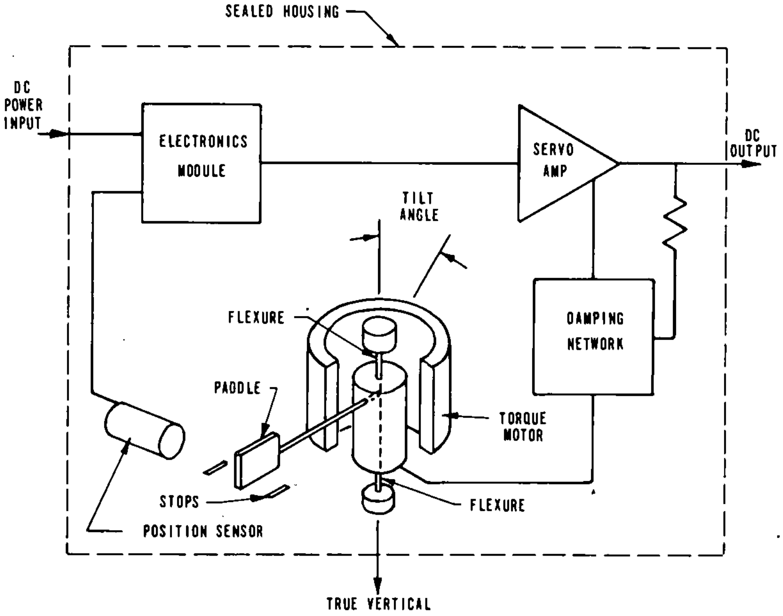
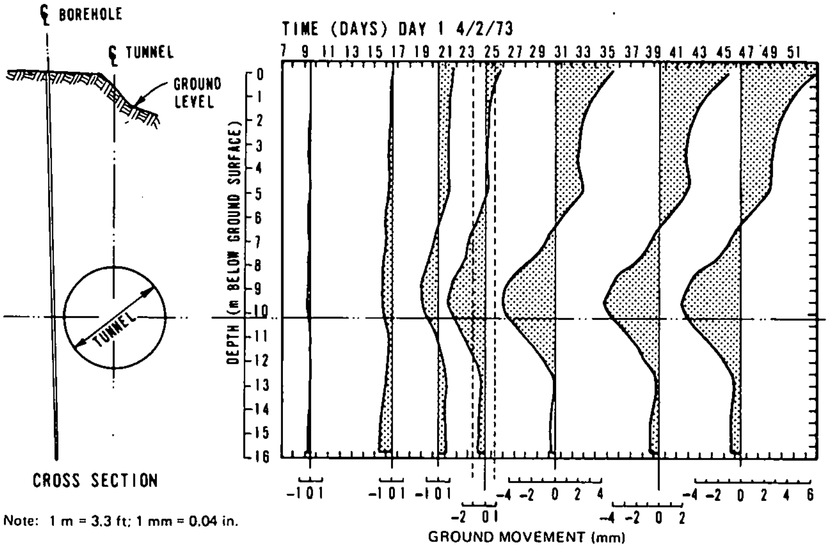


Figure 5.10. Lateral ground movement at borehole C1 of bentonite tunneling experiment (*5.6*).



Note: 1 m = 3.3 ft; 1 mm = 0.04 in.

landslide area where boreholes often have significant inclinations.

Casing Installation

Inclinometer casings generally may be installed for the full depth normally encountered in landslides. Some known installations are close to 300 m (1000 ft) deep (*5.7*). Since the measurement is referenced with respect to the bottom of the casing, the casing bottom must be extended about 6 m (20 ft) or more into soil that will not undergo lateral displacement. If any doubt arises as to the stability of the casing bottom, movement of the casing top should be checked by precise surveying methods. The accuracy of the observations is generally limited not by the sensitivity of the inclinometer, but by the requirement that successive readings be made with the same orientation of the instrument at the same point in the casing. Because the casing must provide a reliable orientation and a continuous tracking for the sensor unit, proper casing installation is of paramount importance.

It is important for the observation accuracy that the original installation be as close to vertical as practicable. The error in inclinometer surveys is proportional to the product of the casing inclination and angular changes in the sensor alignment. For inclined casings, angular changes in sensor alignment on the order of 1° to 2° may produce errors of several centimeters per 10 m (several inches per 100 ft) of casing, as shown in Figure 5.11. Sensor-alignment change occurs from time to time because of one or several factors, such as wheel play in the groove, wear of the sensor carriage (particularly wheel assemblies), internal change in the sensor itself, and change in the alignment between sensor and carriage.

The casing for the inclinometer is usually installed in 1.5 and 3.0-m (5 and 10-ft) lengths, which are joined together with couplings and either riveted or cemented or both to ensure a firm connection (Figure 5.12). Each coupling represents a possible source of leakage for grout or mud, which can seep into the casing and be deposited along the internal tracking system. For this reason, couplings should be sealed with tape or glue. The bottom section of the casing is closed with an aluminum, plastic, or wooden plug, which also should be sealed. If the drill hole is filled with water or drilling mud, the inclinometer casing must be filled with water to overcome buoyancy. Sometimes, extra weight (sand bags, drill stem) may be needed.

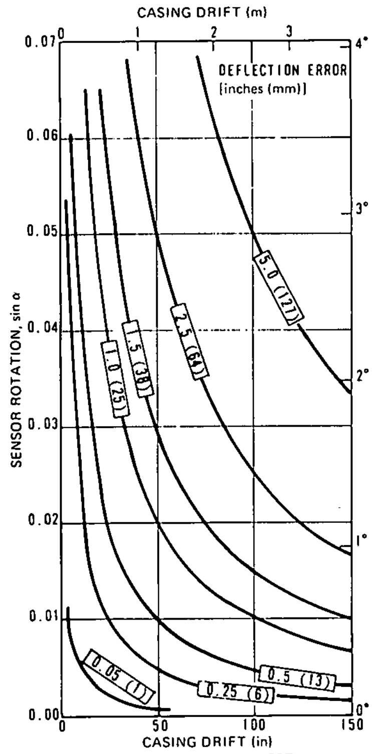
The annular space between the boring wall and the inclinometer casing may be backfilled with sand, pea gravel, or grout. The selection of backfill depends mostly on soil, rock, and groundwater conditions (i.e., whether the borehole is dry, wet, caving, stable without mud). The type of drilling technique (e.g., rotary, hollow-stem, cased holes) is also important. Poorly backfilled casings can introduce a scatter in initial measurements, and, if maximum precision is required, great care should be taken in the selection of the proper backfilling material. Grouting is generally preferred, but may not always be possible, particularly in pervious geologic materials, such as talus. Grouting may be facilitated by use of a 20 to 25-mm (¾ to 1-in) diameter plastic tube firmly attached to the casing bottom through which grout is pumped from the ground surface until the entire hole is filled. In a small clearance, drill-hole grout can be tremied through drill rods via a one-way valve at the inside bottom of the casing.

The as-manufactured casing may have some spiral to the grooves. During installation, the casing can become even more twisted or spiraled so that, at some depth, the casing grooves may not have the same orientation as at the ground surface. Spiraling as great as 18° in a 24-m (80-ft) plastic casing has been reported (*5.24*); 1°/3-m (10-ft) section is not uncommon. Because significant errors in the assumed direction of movement may result, deep inclinometer casings should be checked after installation by using spiral indicators available from the inclinometer manufacturer. For a particular installation, groove spiraling is generally a systematic error occurring for each section in the same amount and the same direction. In any event, spiraling, whether it is due to manufacture or to torque of the casing during installation, can be measured and is thus not really an "error" in inclinometer measurement. Spiraling is important only when the true direction of the movement is determined and at that time can and should be measured and the results adjusted accordingly.

Small irregularities in the tracking surface of the casing can lead to errors in observation, especially if careful repetition of the depth to each previous reading is not exercised. If plastic casing has been stored in the sun before installation, each section may be locally warped (opposite grooves not parallel) and large measurement errors can occur with minor variations at the depths at which the readings are taken (*5.23*).

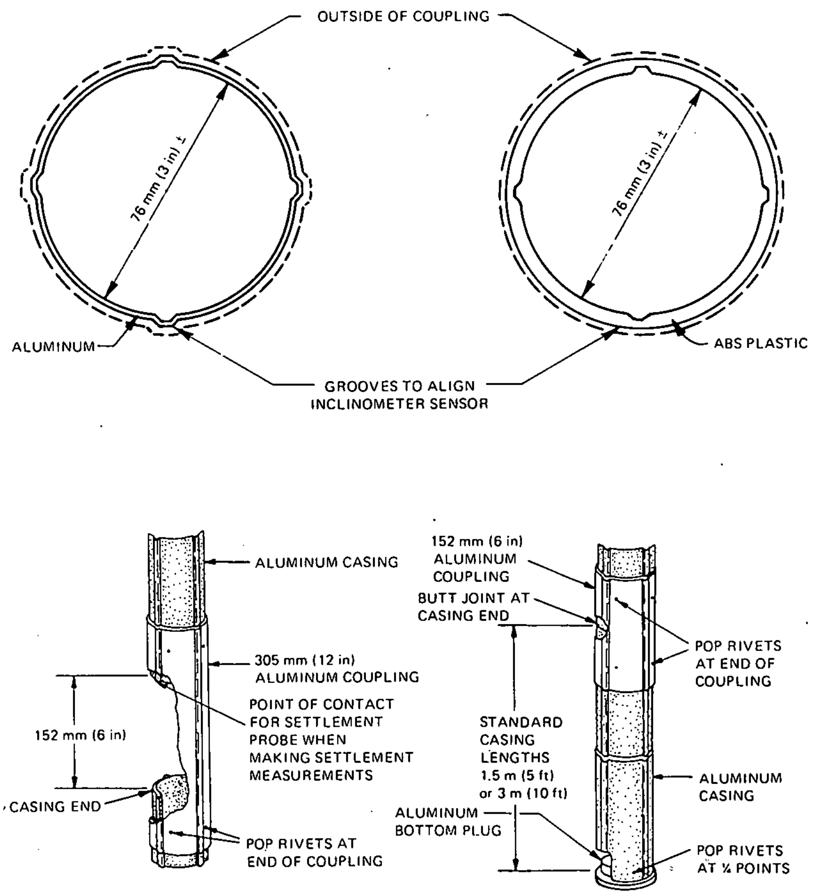
Aluminum casings should be used with some caution because severe corrosion may occur if the casing is exposed

Figure 5.11. Measurement error as a result of casing inclination and sensor rotation.



NOTE: CASING DRIFT OVER ANY DEPTH INCREMENT IS MEASURED AS HORIZONTAL DEVIATION FROM TRUE VERTICAL AXIS IN TWO MUTUALLY PERPENDICULAR PLANES. DEFLECTION ERROR IN ONE PLANE IS RESULT OF DRIFT IN OTHER PERPENDICULAR PLANE.

Figure 5.12. Details of inclinometer casing.



to alkaline soil, corrosive groundwater, or grout. Epoxy coatings help to minimize this problem. Burland and Moore (*5.11*) recommend cleaning the casing with a stiff brush before taking readings. Frequent flushing with water is also helpful. The top of the casing is generally capped with a tight-fitting plug to prevent intrusion of debris. In addition, the inclinometer casing should be protected and padlocked at the ground surface. When selecting protective casing or monument covers for use around the top, one should keep in mind the casing-mounted pulley or similar device.

In-Place Inclinometers

An inclinometer that remains in place in the borehole to continually monitor displacements normal to the borehole axis at discrete points along the borehole is called an in-place inclinometer. The sensors are permanently positioned at intervals along the borehole axis and may be more closely spaced (to increase resolution of the displacement profile) in zones of expected movement. Total movements are determined by summing the relative movements measured at each sensor along the borehole. Since fixed units consist of a number of sensors, they are more expensive and complex than portable inclinometers.

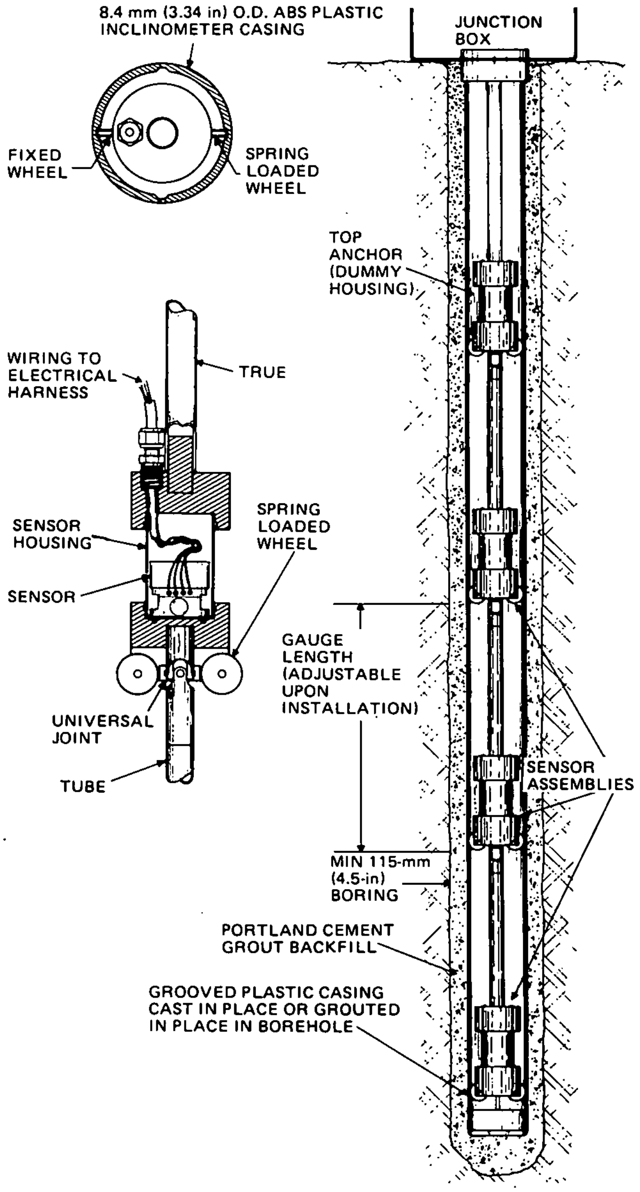
The major advantage of the fixed borehole inclinometer over the portable unit is the elimination of problems of tracking inaccuracies and repeatable positioning. If the fixed unit is removed for repairs, the overall accuracy will be reduced to that of a portable unit or less. Also, accelerometers may have long-term drift. With portable units, these and other effects are canceled by taking readings in opposite directions. Fixed units can be monitored remotely or connected to alarm systems, which are additional advantages. In some systems, a standard grooved inclinometer casing is used so that a portable inclinometer can be used to check readings of the fixed system.

Because of its accuracy, the fixed borehole inclinometer may be used to measure small movements in rock. Its adaptability to various remote-monitoring systems allows for a continuous record of displacement. As designed, most systems are not intended to obtain a continuous profile of deformation. Rather, they measure critical movements of a few sections within the borehole inclinometer. This instrument, therefore, is not necessarily a replacement for the portable units, but rather has its own purpose and is worthy of consideration for continuous monitoring of movements during critical construction stages.

A recently developed in-place inclinometer (Figure 5.13) employs a series of servo-accelerometer sensors; it is available with both one- and two-axis sensors (*5.13*). The sealed accelerometer packages are spaced along standard grooved inclinometer casing by a series of rods. The rods and sensors are linked by universal joints so that they can deflect freely as the soil and the casing move. The sensors are aligned and secured in the casing by spring-loaded fins or wheels, which fit the casing grooves. Use of the grooved casing and guide wheels allows removal of the instrument for maintenance, adjustments, and salvage. Since the casing is standard inclinometer casing, portable inclinometers can be used in the casing when the fixed instrument is removed.

Readings are obtained by determining the change in tilt of the accelerometer and multiplying by the gauge length or spacing between sensors. This gives the relative displacement of each sensor, and these relative displacements can be summed to determine the total displacement at each sensor. Since the grooves in the casing may spiral, the spiral should be checked after the casing is installed so that the exact direction of displacement measurement is known. Any number of sensors at any spacing can be used in one borehole. Maximum deflection range is ±30°, and sensitivity is reported to be ±0.01 mm in 1 m (±0.001 in in 10 ft), but this is not usually attained because of long-term drift. Monitor and alarm consoles and telemetry systems are available.

Figure 5.13. Installation and detail of multiposition in-place inclinometer.



EXTENSOMETERS AND STRAIN METERS

Extensometers and strain meters measure the increase or decrease in the length of a wire or rod connecting two points that are anchored to the soil in the borehole and whose distance apart is approximately known. One commercially available device is shown in Figure 5.14. When gauge lengths are on the order of a meter or less, these devices are often referred to as strain meters rather than as extensometers. When they are used as extensometers, the accuracy and repeatability depend on the type of sensing element and its range of travel and also on the type of connecting wire or rod and the methods used to control the tension. Dead weights are best for maintaining constant tension in wires; if these cannot be used, constant tension springs are acceptable, although there may be some hysteresis. Relatively inexpensive wire-wound or conducting plastic linear potentiometers are often used as sensors, and relatively simple, battery-operated Wheatstone bridge circuits are used for manual readout. Sensitivity is on the order of 0.1 percent of the range of travel, but repeatability and accuracy may be no better than 0.5 mm (0.02 in), depending on the type of anchor and connecting member; this is usually sufficient, however.

When the devices are used as strain meters, the repeatability and accuracy are essentially the same as the sensitivity. Thus, for a grouted-in-place assembly 3 m (10 ft) long with an invar rod and a range of 25 mm (1 in), unit strains as low as 0.000 01 may be detected with relatively inexpensive instrumentation. Horizontal stretching of embankments has been observed by installing anchors at various positions at a given elevation and attaching horizontal wires to dead weights on the downstream face; this was done at Oroville Dam (*5.55*). Care is required to ensure that the wires (or rods) do not get pinched off if localized vertical shear movements occur. Other cases of application are described by Dutro and Dickinson (*5.19*) and Heinz (*5.26*).

PORE-PRESSURE AND GROUNDWATER MEASUREMENTS

Observation Well

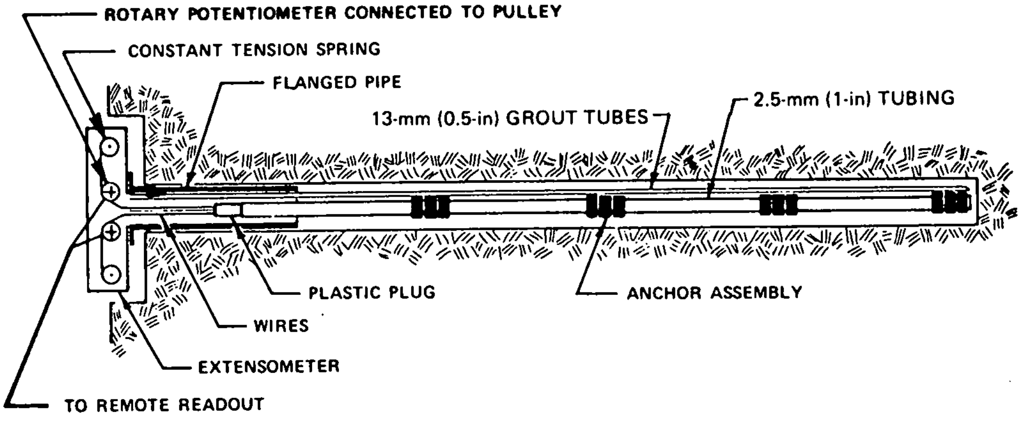
The most common water-level recording technique, despite more sophisticated methods, is the observation of the water level in an uncased borehole or observation well. A particular disadvantage of this system is that a perched water table or artesian pressure can occur in specific strata that may be interconnected by the borehole so that the recorded water level may be of little significance.

Piezometers

Pore pressures and groundwater levels in a slide area can be measured by a variety of commercially available piezometers (*5.25*). The selection of the best type for a particular installation involves several considerations.

1. Piezometers are usually installed in a difficult environment and may become inoperative because of pinching off of the tubes, blockage by air bubbles, electrical short circuits, and malfunctions of buried moving parts. Therefore, reliability and durability are often of greater importance than sensitivity and precision. For example, in some cases it does not matter that the actual head may be in error by 0.3 m (1 ft) or so as a result of time lag, provided the piezometer is functioning properly. If a malfunction occurs, the fact that the apparent head can be recorded to a millimeter is of little importance.
2. The basic problem with a piezometer is that the energy required to operate it prevents the instrument from recording a pore-pressure change immediately. For a given pressure change, this energy is proportional to the volume of pore water that must flow into the instrument. A piezometer with an open standpipe, for example, requires a much greater volume of water than one with a stiff diaphragm and therefore requires a correspondingly greater time to respond to a change of pore pressure. Hvorslev (*5.27*) discusses time-lag effects for open-standpipe piezometers; Brooker and Lindberg (*5.10*) evaluated these effects for closed types; and Penman (*5.39*) and Vaughan (*5.52*) studied the response times of various types of piezometers. In slides in which piezometers are placed in boreholes, it is usually possible to provide a large collecting volume of porous material (sand) around the tip and thus to reduce the time lag in open standpipes.
3. Partially saturated soils (fills, soils above the water table, and soils in which gas is generated from organic matter) pose particular problems because the gas in the voids exists at higher pressures than the water as a result of surface tension (capillarity). Thus, one must know whether the piezometer is reading the pore-air pressure or the pore-water pressure. To prevent the entry of air into the piezometer tip and thus to ensure that pore-water pressure

Figure 5.14. Extensometer.



is being measured, fine ceramic tips, with high air-entry values, can be used (*5.2*). Even with a high air-entry value filter, air will slowly enter the piezometer cavity by diffusion and, if reliable long-term readings are required, deairing facilities must be provided (*5.52*).

4. Additional problems resulting from diffusion of either air or water vapor through the walls of connecting tubing are described by Bishop, Kennard, and Penman (*5.2*). Furthermore, either settlement or horizontal movement may pinch off the connecting tubing or, in the case of open standpipes, may prevent the lowering of a probe to detect the water level.

The features and uses of various types of piezometers are discussed below.

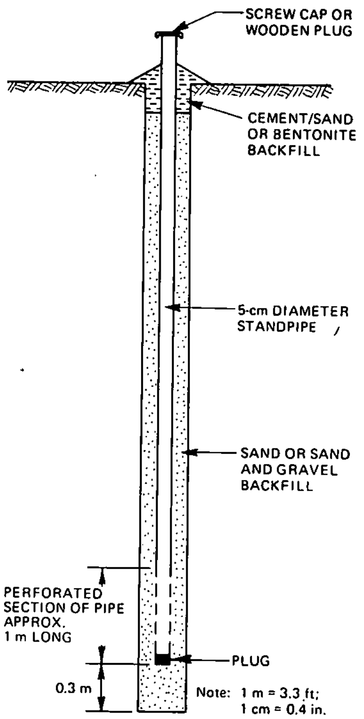
Open Standpipe

Open-standpipe piezometers vary mainly in diameter of standpipe and type and volume of collecting chamber. The simplest type (Figure 5.15) is merely a cased or open observation well in which the elevation to which the water rises is measured directly by means of a small probe. In this case, the static head is the average head that exists over the depth of the inflow part of the well below the water table. This measured head may be higher or lower than the free water table and, in the case of moderately impervious soils, may be subject to a large time lag. Although the open standpipe is not satisfactory in impervious soils because of time lag or in partially saturated soils because the significance of the measured head may be difficult to evaluate, its simplicity, ruggedness, and overall reliability dictate its use in many installations.

Casagrande

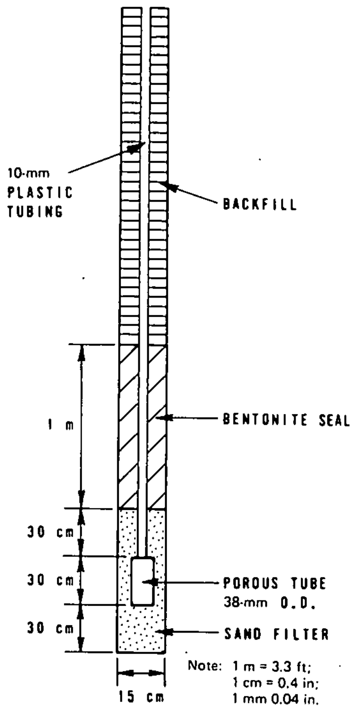
The Casagrande type of piezometer (Figure 5.16) consists of a porous stone tip embedded in sand in a sealed-off portion of a boring and connected with a 1-cm ($^3/_8$-in) diameter plastic riser tube (*5.45*). When properly installed, this type of piezometer has proved successful for many materials, particularly in the long term, for it is self-deairing and its nonmetallic construction is corrosion resistant. The reliability of unproven piezometers is usually evaluated on the basis of how well the results agree with those of adjacent Casagrande piezometers.

Figure 5.15. Open-standpipe piezometer.



Note: 1 m = 3.3 ft;
1 cm = 0.4 in.

Figure 5.16. Casagrande borehole piezometer.



Note: 1 m = 3.3 ft;
1 cm = 0.4 in;
1 mm 0.04 in.

Pneumatic

The pneumatic piezometer consists of a sealed tip containing a pressure-sensitive valve. The valve opens or closes the connection between two tubes that lead to the surface, or the slope face, at any convenient location and elevation. In the piezometer shown in Figure 5.17, flow of air through the outlet tube is established as soon as the inlet-tube pressure equals the pore-water pressure. In the hydraulic piezometer (Figure 5.18), hydraulic fluid is used instead of gas, but the basic principle remains the same. Pneumatic piezometers have the following advantages: (a) negligible time lag because of the small volume change required to operate the valve, (b) simplicity of operation, (c) capability of purging the lines, (d) minimum interference with construction, and (e) long-term stability. Their main disadvantage is the absence of a deairing facility.

Electric

Electric piezometers have a diaphragm that is deflected by the pore pressure against one face. The deflection of the diaphragm is proportional to the pressure and is measured by means of various electric transducers. A typical design is shown in Figure 5.19. Such devices have negligi-

Figure 5.17. Pneumatic piezometer.

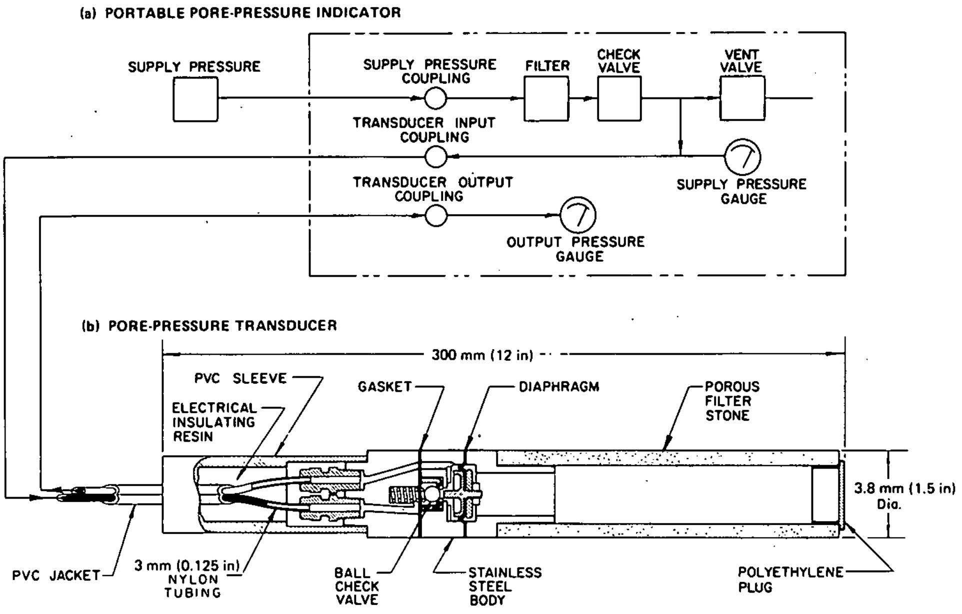
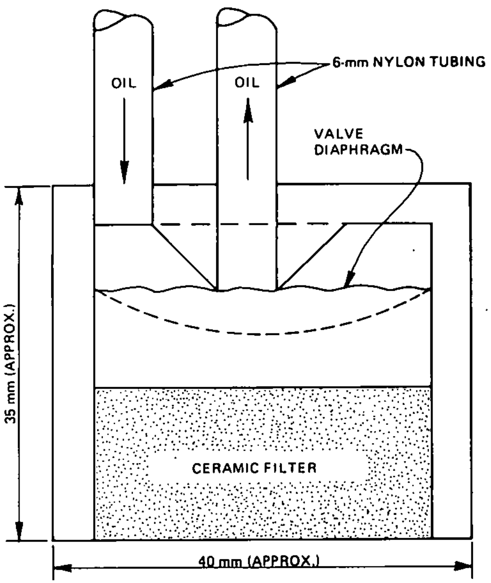


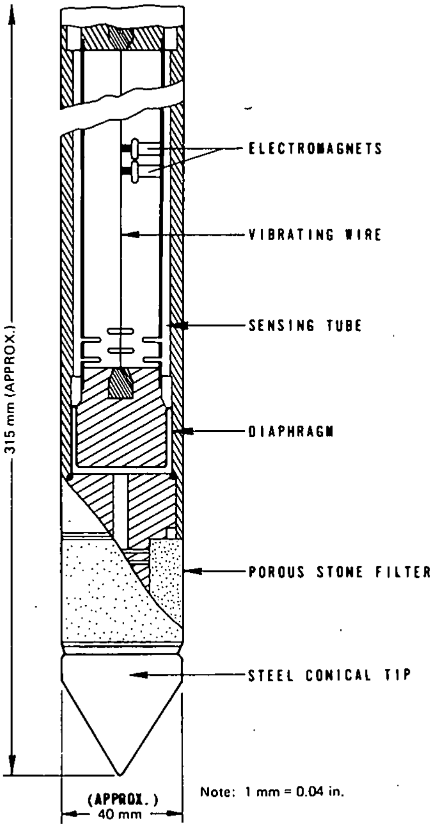
Figure 5.18. Hydraulic piezometer.



Note: 1 mm = 0.04 in.

ble time lag and are extremely sensitive. Piezometers of this type are described by Shannon, Wilson, and Meese (*5.45*), Cooling (*5.12*), Brooker and Lindberg (*5.10*), Penman (*5.39*), Bishop, Kennard, and Vaughan (*5.3*), and Hanna (*5.25*). Because they are affected by the environment and have poor long-term stability, electric piezometers are not generally recommended for installations in which reliable readings are required during an extended period of time. Generally, they cannot be deaired nor can the sensitive electric transducers be recalibrated in situ, although there are exceptions (*5.16*). For short-term observations at installations in which transmission is over limited distances, standard resistance strain-gauge pressure transducers are suitable. A new type of electric piezometer, using a pressure-sensitive crystal, has been developed, but its long-term stability has not yet been verified.

Figure 5.19. Electric piezometer.



Piezometer Sealing

Piezometers in landslides can be installed in boreholes advanced into soil or rock. Based on the drill log of materials encountered and the estimated position of the water table and of the sliding surface, the depth of each piezometer is selected. A simple open-standpipe piezometer device consists of a tube placed in the drilled hole extending to the ground surface. The bottom of the tube should be slotted or made of a porous material or a sufficiently fine screen that it will restrict movement of soil into the piezometer but at the same time permit free access of water. Only that part of the tube that is actually within the stratum in question should be pervious. When the tube is centered in the drilled hole, a known volume of clean sand is placed around the piezometer; the purpose is to create a sand filter between the soil and the piezometer tip.

Construction of an impervious barrier above the piezometer tip and sand pocket is essential. A well-established procedure is to drop balls of soft bentonite into the hole and then tamp those balls around the piezometer tubes by using an annular hammer. Prepared bentonite in ball or pellet form is now available commercially; it has a specific gravity that is sufficiently large to allow it to sink through the water in the piezometer hole so that a hammer is not necessary. Alternatively, a cement-bentonite grout can be tremied into the hole above the sand filter. Such a seal can be pumped through a 1.3 to 1.9-cm (½ to ¾-in) diameter pipe adjacent to the piezometer tube. Installing only one, or at most two, piezometers in a borehole has generally been found preferable because bentonite seals are sometimes difficult to construct and may leak slightly despite precautions. However, special-purpose, multiple-point piezometers (with four tips) have been used successfully by Vaughan (*5.51*) and merit consideration.

Both pneumatic and electric piezometers can be sealed in a similar manner to an open-standpipe or a Casagrande piezometer in a borehole. Since the piezometer tip cannot be deaired after installation, it should be soaked in deaired water beforehand and kept in the water until it is lowered into the borehole. A low air-entry filter tip can be placed in a saturated sand pocket, but a high air-entry tip should be pushed into the soil beyond the base of the hole or surrounded by a porous grout, such as plaster of paris. This will ensure more rapid equalization, which may be of considerable value in obtaining reliable pore-pressure measurements soon after installation in low-permeability clay. Both electric and pneumatic piezometers should be checked for malfunction before and during installation, and particular care is needed in driving electric piezometers to prevent overpressure.

In soft and medium-stiff soils, it may be preferable to push the piezometer directly into the ground. Flush-coupled heavy steel water pipe will be required, and the piezometer tip must be robust enough to resist damage to the well point and should be designed to minimize disturbance around the tip (*5.34*). It may be of the Casagrande, pneumatic, or, less commonly, electric type. This drive-in piezometer is self-sealing and rapidly installed and, if pneumatic, has a rapid response. Recently, an improved method of sealing piezometers in boreholes was described by Vaughan (*5.51*). Instead of compacting bentonite balls just above the sand filter, the borehole is completely grouted with cement-bentonite grout. Even if the permeability of the grout is significantly higher than that of the surrounding soil, little error will result because of the relatively large grouted length. In many cases, a sand filter need not be included, and the piezometer tip can be grouted directly with little error.

SYSTEMS FOR MONITORING ROCK NOISE

Use of the rock-noise detection method for observing the stability of soil and rock materials is a relatively new concept, and most practical applications so far have been in mines (*5.4, 5.5, 5.30, 5.60*). More work has been done in basic research than in practical application of the technique, although sufficient laboratory and field tests have been performed to demonstrate the importance and usefulness of the method. The ability to detect the occurrence of distress in a rock mass before the development of measurable movement is obviously a significant technological advance. Experimental work that has led to the development of techniques and instrumentation for measuring microacoustic, microseismic, or subaudible rock noise has been carried on in the United States since the early 1940s (*5.5*). All of this work, regardless of terminology, has been related to measuring transient noise disturbances in earth (soil and rock) materials for the purpose of establishing the relative stability and, in some instances, locating zones of weakness in earth materials (*5.29, 5.31, 5.42, 5.60*). In the mid-1960s, Goodman, Blake, and others (*5.4, 5.22, 5.31*) adapted the techniques developed by the U.S. Bureau of Mines to civil engineering applications and the study of soil and rock slides. Particular emphasis was placed on the determination of relative stability, location of failure planes, determination of the epicentral location of specific disturbances, and improvement of field equipment. Rock-noise detection systems can be divided into three basic elements: sensors, signal-conditioning equipment, and recording and data-acquisition equipment.

Sensors

The frequency response of the sensors is a critical design parameter. Rock noise is reported to occur over a broad range of frequencies between about 50 and 10 000 Hz (*5.4*); however, disturbances at lower frequencies (20 to 40 Hz) have been observed at Downie slide, British Columbia, and are believed to be important. The occurrence of rock noise at frequencies above 10 kHz has been observed in the laboratory when rock specimens are placed under high compressive loads (*5.30*). Because the frequency range of noise phenomena at a particular site is unknown, the frequency response of the sensors must be sufficiently broad to ensure that the system will respond properly to any meaningful noises that occur. In addition, the sensors must be capable of (a) producing a high-level signal that is proportional to the amplitude and frequency of the exciting noise and is without spurious response and (b) transmitting the signal through long electric cables. For this reason, the output impedance of the sensor must be relatively low to prevent the cable from acting as an antenna that could pick up radio, television, and ignition interference.

Signal-Conditioning Equipment

Signal-conditioning equipment includes amplifiers and filters for the enhancement of the rock-noise signals and rejection of unwanted noise or interference. It also includes the alteration of the received signal to whatever form is required to operate data acquisition equipment, usually analogue or event pulse form. Rock-noise events are minute bursts of energy; therefore, high amplification is required by the signal-conditioning equipment to make the signals useful. Further, the amplifiers must be of a type that generates little internal noise and responds accurately to rock-noise signals over the entire frequency range specified for the sensors. Sharp cutoff filtering for both high and low frequency must be incorporated into the signal-conditioning equipment to reject unwanted noise that might interfere with the rock-noise signals.

Recording and Data-Acquisition Equipment

After rock-noise signals are received, amplified, and conditioned, they must then be recorded or analyzed or both. A rock-noise signal is generally a sharp burst of acoustic energy that attenuates rapidly. It typically sounds like a snap, click, or grind and, in analogue form, has the appearance of the record of a miniature earthquake of short duration. To be able to record the signals in analogue form is important in their identification as rock noise rather than as noise generated from other sources. Recording in analogue form is usually not necessary on a continuous basis, but can be done only periodically to ascertain that interfering signals are not being received.

Data recorded in analogue form, which is the form in which rock noise data are usually presented, are generally inconvenient to use for routine analysis of noise rate. A convenient method of determining noise rate is to electronically convert each rock noise burst to a pulse signal, which is counted mechanically or electronically on a unit-of-time basis. The number of pulses or events per unit of time can then be recorded to document the time-rate history of rock-noise activity during a long period of time. Although time-rate data may need to be accumulated continuously in active areas, they are generally sampled during selected intervals of time, such as several minutes or an hour, and sampled periodically rather than continuously. Ambient background rock-noise data from a specific area should be studied before a reasonable estimate of recording duration and intervals can be recommended. Time-rate data can be accumulated and recorded by a variety of methods. The selection of the method depends on the cost and the required convenience to the user.

AUTOMATIC WARNING AND ALARM SYSTEMS

Installing a slide-warning system may be desirable in some instances. Such systems vary from simple slide fences, sometimes used by railroads (*5.9*), to more complicated in-place inclinometers, extensometers, and piezometers. Their purpose is to provide automatic warning in the event of a sudden change that could be indicative of an impending earth movement. The mechanics of such systems are relatively simple to devise. An in-place inclinometer or extensometer can be used to actuate a red light in a central location when the movement exceeds a certain threshold, which, for example, may occur along a specific shear plane in soil or a bedding plane in rock. A similar alarm could be actuated when the piezometric level exceeds a certain elevation.

One problem with such warning systems is to determine in advance the boundary between tolerable and intolerable change. Should the extensometer be set for 0.25, 2.5, or 25 mm (0.01, 0.1, or 1.0 in)? Should each instrument be monitored individually, or do only a few key instruments need such signals? Probably no computer or automatic warning system will ever replace engineering judgment in the evaluation of data from field instrumentation systems. With respect to advance warning, impending failure is normally signaled by a long-term change in rate rather than by a short-term localized change. The data must be reviewed periodically by a geotechnical engineer experienced both in field measurements and in the design requirements of that particular project.

DATA ACQUISITION AND EVALUATION

Data-Acquisition Methods

The data obtained from the instrumentation described consist of a sequence of numbers that are converted by means of a calibration chart or other relation to length, volume change, pressure, or other parameter. These numbers, plus other information, such as date, depth, or instrument number, must be recorded in the field in some form that can later be properly identified and retrieved for analysis. There are several ways in which this can be accomplished.

Manual Recording at Sensor Location

Each individual sensor or cluster of sensors can be connected to its own gauge or panel at a nearby location (Figure 5.20a). The field crew carries a portable control box to that specific location, connects it to the individual sensor, obtains and records the pertinent data, and forwards the data sheets to the field office or home office for reduction and analysis. This procedure is time consuming on large jobs and requires a substantial amount of labor, and the resulting data sheets are voluminous and subject to errors in reading, recording, and subsequent retrieval and analysis. However, the initial capital investment is low.

Automatic Recording at Sensor Locations

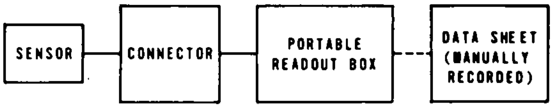
Significant reduction in the time required for data acquisition and subsequent savings in analysis of the data can be effected by use of automatic recording devices that can be transported to the sensor location or a nearby panel. For example, inclinometer surveys require that the sensing torpedo be transported to the hole and lowered manually to the desired depths. However, if the site is accessible by panel truck, the cable, reel, and associated readout equipment can be transported readily to the site and the data can be recorded on magnetic or punched tape in response to manual command (Figure 5.20b).

Manual Recording at Central Terminal

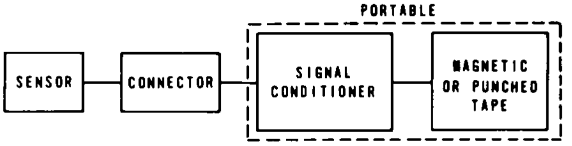
If the sensors are of a type whose output signal can be

Figure 5.20. Data acquisition systems.

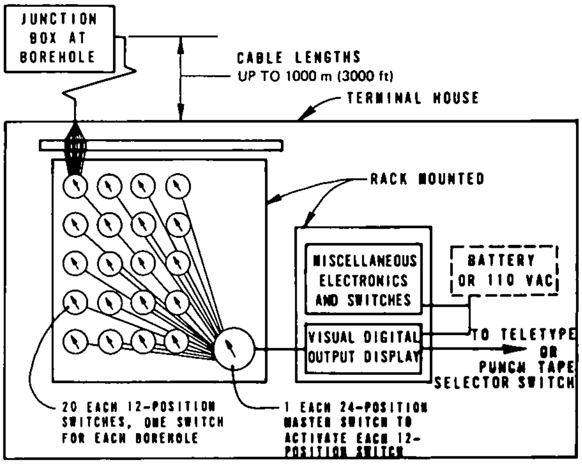
(a) MANUAL READOUT AT SENSOR LOCATION



(b) PORTABLE AUTOMATIC RECORDING AT SENSOR LOCATION



(c) COMBINED MANUAL AND SEMIAUTOMATIC READOUT AT CENTRAL TERMINAL HOUSE



transmitted for some distance over an electric cable or other direct connection or can be transmitted by wireless telemetry, all leads can often be brought into one or more central terminals. At those terminals, the signals are switched sequentially to the appropriate control boxes to produce a digital output. In the simplest form, the operator goes to the control terminal, activates the proper instruments, switches one at a time, and records the data manually. Additional savings in time and increased accuracy will result if the operator merely pushes a button to direct the output data to be recorded on punched tape (Figure 5.20c).

Completely Automatic Operation at Central Terminal

If extensive instrumentation is installed and frequent or continuous recording of the data is required, a completely automatic system may be necessary or desirable. This may vary from a comparatively simple continuous, slow-speed, strip-chart recorder to a system in which an event, such as an earthquake, triggers a mechanism that starts a recorder, which records the event for a predetermined interval of time. Several recent applications involve automatic systems in which accumulated totals or specific readings are systematically sampled at preset intervals and recorded on punched tape or some other form of record. One such arrangement systematically records at desired preset intervals the number of bursts of subaudible rock noise picked up remotely by sensors at varying depths in several holes (*5.26*). Similar systems can readily be devised to record, for example, the readings of individual extensometers.

Inclinometer Observations

Since all inclinometer readings are referenced to an original set of measurements, extreme care must be taken to procure an initial set of reliable observations. Measurements of the original profile, to which all subsequent observations will be related, should be established by a double set of data. If any set of readings deviates from the previous or anticipated pattern, the inclinometer should be checked and the readings repeated. A successful record of measurements generally requires at least two trained technicians who are allowed the proper amount of time for setting up, recording observations, and undertaking maintenance of the equipment. Use of the same technicians and instrument for all measurements on a particular project is highly desirable.

General Recording Procedures

The wheels of the inclinometer torpedo provide points of measurement between which the inclination of the instrument is measured. If the reading interval is greater or less than the torpedo wheel base, correspondence between the position and orientation of the instrument will be only approximate in determining the total lateral displacement profile. Optimum accuracy is achieved only if the distance between each reading interval equals the distance between the upper and the lower wheels of the torpedo. Gould and Dunnicliff (*5.23*) report that readings at depth intervals as large as 1.5 m (5 ft) result in poor accuracy.

Inclinometer measurements generally are recorded as the algebraic sums or differences of 180° readings. For each depth of measurement, the reading in one vertical plane is taken and then repeated with the instrument turned through 180°. Computing the differences of these readings minimizes errors contributed by irregularities in the casing and instrument calibration. One excellent check on the reliability of each measurement is to compute the algebraic sum of 180° readings. The sum of 180° readings should be approximately constant for all measurements, except those readings made while one set of wheels is influenced by a casing joint. Differences between the sum of two readings and the observed constant sum usually indicate that an error has occurred or that opposite sides of the casing wall are not parallel. If 180° readings are summed during observation, errors resulting from mistaken transcription, faulty equipment, or improper technique can be eliminated. When the algebraic sum does not remain nearly constant, the sensor unit, readout, and casing should be rechecked before further use.

When an inclinometer is surveyed for the first time (initial set of data), a fixed reference for the torpedo should be selected so that each time a survey is repeated the torpedo will have the same orientation in the casing (i.e., each time the same set of grooves is used for alignment orientation). For the typical biaxial inclinometer (Figure 5.7), it is generally recommended that the A-component (or sensor) be oriented so that it will register the principal component of the anticipated deformation as a positive change. For example, in an area suspected of landslide activity, the first set of readings is taken by orienting the fixed wheels of the torpedo in the casing groove closest to the downhill position. For a complete survey, the torpedo is reversed 180° after the first set and the readings are repeated. The algebraic difference of the two sets of readings is used to compute the profile of the casing or, more important, to determine any change compared to other surveys; using the algebraic difference is, in effect, the same as using the average of the two sets of readings.

Maintenance

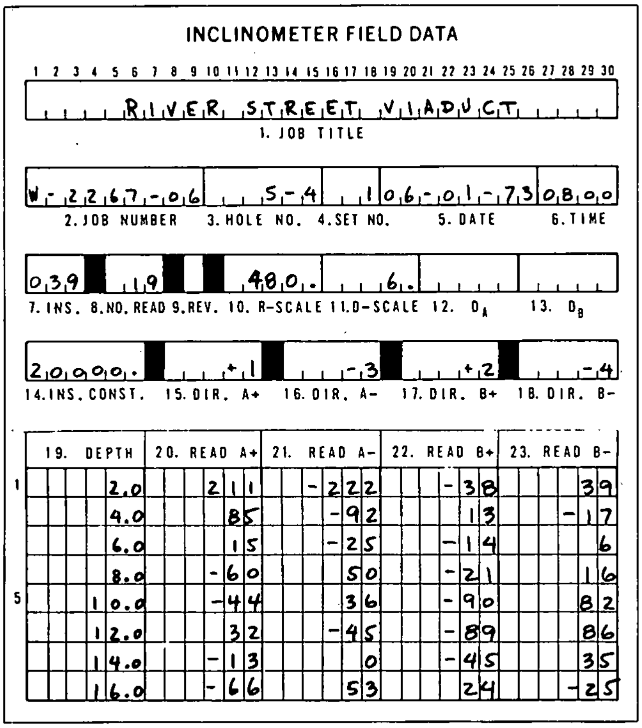
Inclinometers are specialty items; consequently, the number of suppliers is limited. Repair or replacement of an inclinometer can be expensive and time-consuming and result in loss of important data. The best insurance against damage is careful use and systematic maintenance. The sensor unit should be checked frequently during operation and its wheel fixtures and bearings tightened and replaced as necessary. After each casing has been read, the guide wheels should be cleaned and oiled. Most important, the electric readout should be protected against water at all times. The introduction of a few drops of water into the readout circuitry can cause the galvanometer to drift in a pendulum inclinometer or can induce a drift of numbers on the digital voltage display in an accelerometer instrument. If readings are made too rapidly or if an automatic recorder is used, this drift may not be detected and erroneous measurements may result. When the data are analyzed at a later time, the unexpected results cannot be related to the cause.

Inclinometer Data Reduction

Hand reduction of inclinometer data is a tedious and time-consuming operation. A single movement profile for a 33-m (100-ft) casing can involve more than 200 separate computations. Because of the amount of effort involved in taking readings and, subsequently, in computing displacements, a successful measurement program depends primarily on organization and discipline. Field readings must be transposed to discernible measurement, preferably in the form of summary plots indicating successive movement profiles, as soon as possible after the field observation. This also provides perhaps the best check of instrument reliability. If a record of successive movement profiles is established, the consistency of new measurements can be referenced to previous readings and judged in light of anticipated soil behavior. Because of occasional error in the accumulation of field readings, the data reduction for a particular casing must be performed with a knowledge of both former movement and expected soil behavior. Excessively large 180° sums, which occur without precedence and are not explainable, should be discarded or the readings should be repeated.

Recording the data manually is adequate for many projects of small and intermediate size. A special form of data sheet is used and is turned over to operators who transcribe the data into computer language. Once this is done, the punched cards or tape can be handled by the computer with any desired degree of sophistication. One must realize, however, that, in addition to the time delay (perhaps only a day or so), there are two potential sources of error: The first is in recording the data manually, and the second is in transcribing the data from digital form to punched card. Figure 5.21 shows a portion of a special field data sheet used for recording inclinometer data, which are to be subsequently transcribed onto punched cards.

Figure 5.21. Sample field data sheet for transcribing inclinometer data to punched cards.



INCLINOMETER FIELD DATA

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

RIVER STREET VIADUCT

1. JOB TITLE

W-2267-06	5-4	1	06-01-73	0800
2. JOB NUMBER	3. HOLE NO.	4. SET NO.	5. DATE	6. TIME

039	19		480.	6.		
7. INS.	8. NO. READ	9. REV.	10. R-SCALE	11. D-SCALE	12. D_A	13. D_B

20000.	+1	-3	+2	-4
14. INS. CONST.	15. DIR. A+	16. DIR. A-	17. DIR. B+	18. DIR. B-

	19. DEPTH	20. READ A+	21. READ A-	22. READ B+	23. READ B-
1	2.0	211	-222	-38	39
	4.0	85	-92	13	-17
	6.0	15	-25	-14	6
	8.0	-60	50	-21	16
5	10.0	-44	36	-90	82
	12.0	32	-45	-89	86
	14.0	-13	0	-45	35
	16.0	-66	53	24	-25

Sources of error in recording and transcribing can be reduced by automatic printout directly onto punched tape or magnetic tape. Provided the recordings are without extraneous data, these tapes can then be returned to the office and fed directly into the computer, which plots the movement. An example of this application is shown in Figure 5.22 for an inclinometer survey. The printed output from the computer is obtained within a few hours after the data are taken. In the case of a central terminal unit with automatic printout, the end result is the same; the only difference is in the ease and speed of operation. The computer programs vary from those that simply reduce data and provide a digital printout of results to extremely sophisticated programs that reduce the data, compare the results with initial readings, plot the changes, and compare those changes with previous changes.

Many inclinometer systems, particularly servo-accelerometer types, can be adapted to automatic data recording for computerized data processing. Inclinometer manufacturers produce different systems that use punched paper tape, dual image tape, or magnetic tape for data recording. The inclinometer control boxes are modified to contain keys for recording the inclinometer well number and depth and an automatic switch to transfer the reading directly to the data tape. Automatic data recording and processing are mainly useful in reducing the time and labor involved in office computation of the data.

Automatic data recording contributes to the complexity of the measurement operation and introduces an additional source of error. With manual data recording, the technician can scan the data for face errors and make corrections or reread the casing on the spot. Most data-recording systems do not allow this advantage; the data must be scanned for errors after being printed out in the office and before computer processing. During instrumentation of the Washington, D.C., subway construction (*5.13*), an automatic data recording and processing system for inclinometer data from 34 installations was used briefly, but it produced no increase in efficiency because of the need for individual screening and interpretation to detect errors.

Evaluation and Interpretation

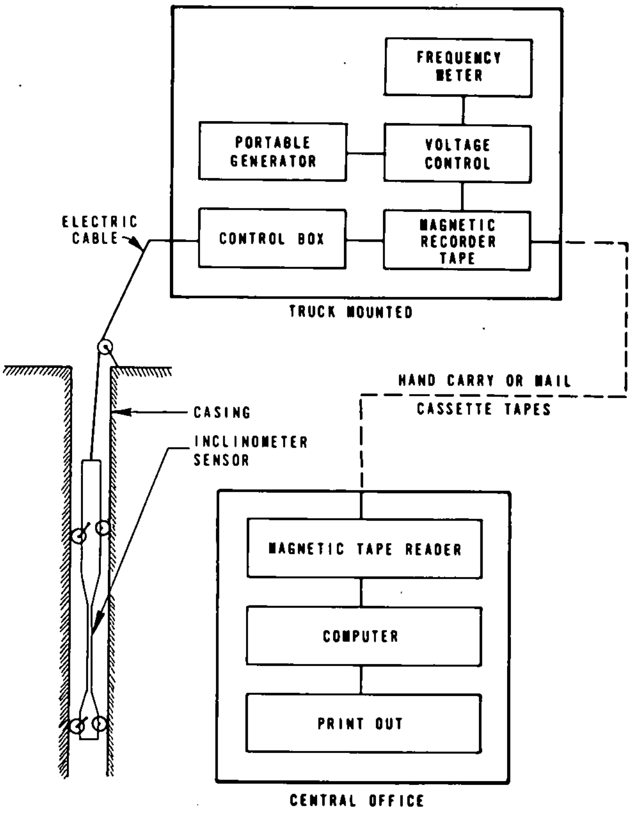
Inclinometers

One particular principle in movement monitoring with inclinometers should be emphasized: The instrument measures the change in slope over a certain depth interval during a period of time. An inclinometer will record this change in slope at any depth within the limitations of the cable on which the probe is lowered (i.e., weight, strength, and elongation characteristics of cable). Once the active zone has been detected from successive sets of data, the rate of deformation can be determined by plotting the change versus time. Usually, the slide zone is only a few feet thick; hence, the sum of the changes over a few consecutive intervals will often be representative of the magnitude and rate of movement of the entire slide.

Time plots of change at each reading depth are normally unwarranted and would be extremely time consuming and costly. As the data develop, however, such plots should be made for a selected number of intervals at which progressive change is evident. Despite its usefulness, this technique has seldom been used. Instead, the deformation, slope change, and casing profile most frequently are plotted versus depth. These plots, particularly those of change (difference) and cumulative change (deflection) versus depth, are important steps in detecting movement and visualizing what is occurring. As increasing numbers of data sets are plotted, the diagrams usually become increasingly cluttered by scatter of data and are difficult to interpret accurately. The most useful plots are those that show changes (differences) in inclination; the zones of movement can most readily be detected and time plots at each zone can be initiated.

Accuracy is usually discussed in terms of the repeatability of an integrated curve of deformation for a depth increment of 33 m (100 ft); this relates to the deflection versus depth plots. However, on deeper installations, this plot may be misleading to the interpreter. Although the instrument may be operating within its range of accuracy, over a period of time it may suggest tilting of several centimeters back and forth and perhaps a small kink may begin to develop somewhere in the curve. This situation is somewhat similar to an open-ended traverse. Primary concern should lie with the developing kink in the curve rather than with the overall tilt, which is probably related to instrument accuracy at the time of measurement. One must remember that the greatest asset of the inclinometer is its ability to measure change in inclination at a specific depth rather than to survey an exact profile of the borehole.

Figure 5.22. Schematic diagram of magnetic tape recording of inclinometer data.

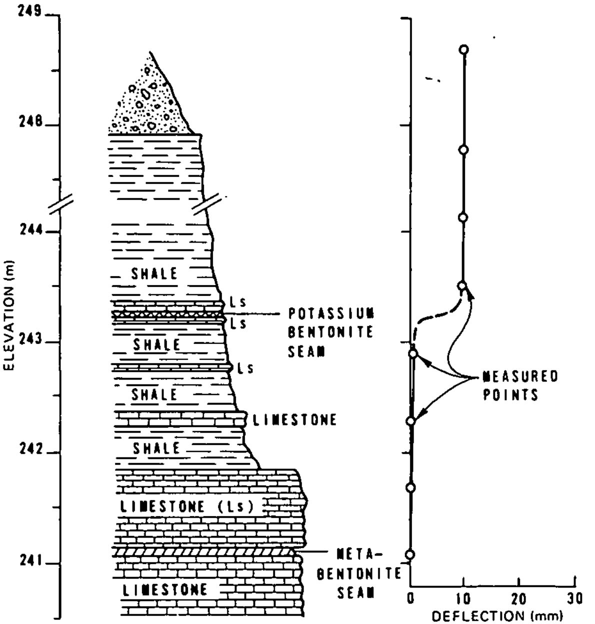


Extensometers

Up-to-date plots should be kept of changes in length from each anchor to the sensor and of computed changes between anchors. Particular attention should be given to the rate of change of length, for any increase in this rate may be an indication of impending failure. Extensometer readings are usually sensitive to temperature changes. If the connecting rods or wires are made of steel, any increase in temperature will result in an increase in its length and thus a reduction in the actual extensometer reading. Daily and seasonal temperature variations are likely to show similar variations in the gauge readings.

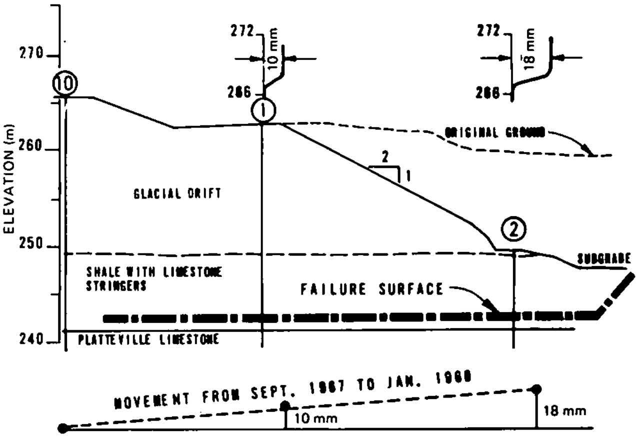
Wire extensometers, especially those with long distances between the anchor and the sensor, are particularly sensitive to changes in wire tension resulting from friction along the wire and hysteresis effects in the sensor or constant tension spring. For example, assume that a 16-gauge stainless wire 33 m (100 ft) long is subjected to a 67-N (15-lbf) pull. If the sensor requires 1.11 N (0.25 lbf) to actuate it, the change in length of the wire required to change the direction of movement of the sensor is 0.15 mm (0.006 in). An average change in temperature of -12°C (10°F) in the same wire will change its length by 3.0 mm (0.120 in). Not only does the length of the wire change with temperature, but

Figure 5.23. Geologic profile and inclinometer observations at failure plane of landslide on I-94, Minneapolis (*5.57*).



Note: 1 m = 3.3 ft; 1 mm = 0.04 in.

Figure 5.24. Section showing failure plane and movement distribution of landslide on I-94 (*5.57*).



Note: 1 m = 3.3 ft; 1 mm = 0.04 in.

the ground itself will expand and contract as its temperature varies. At a hillside stability project in Montana, rock outcrops were found to expand and contract seasonally by as much as 1.5 mm (0.06 in). Electric lead wires also change their electric resistance with temperature and will cause erroneous readings unless such a change is properly taken into account.

Piezometers

Piezometric heads should be plotted on time graphs showing rainfall and other data that may influence the pore pressure. If drainage has been installed, the quantity of seepage should also be recorded and plotted. If possible, the response of each piezometer should be checked periodically to determine its recovery rate.

EXAMPLES

This section briefly describes several projects that were successfully instrumented to measure pore pressures and to detect the depth and rate of movement before corrective treatment was designed. The emphasis is on selection of instrumentation and analysis of data rather than on corrective treatment.

Minneapolis Freeway

In 1967, a landslide developed along a 335-m (1100-ft) section of I-94 on the east side of Minneapolis (*5.56, 5.57*). The movements were evidenced by upheaval of the subgrade and by cracking and slumping of the cut slope. Although the area had been investigated before construction, no instability problems had been anticipated. Therefore, determining the depth of movement was necessary before investigating the cause of the movements could begin. Fifteen borings were drilled to serve the dual purpose of providing subsurface soil information and holes for the installation of field instrumentation. Two sections in the vicinity of the slide were instrumented rather extensively. To measure the depth and rate of movement, 10 inclinometers were installed, and to evaluate groundwater conditions, 7 Casagrande piezometers were installed.

Several of the inclinometers detected movement within a few days, and within a month it was apparent that slope failure was occurring as a result of sliding along a near-horizontal plane approximately at elevation 243 m (798 ft). Figure 5.23 shows the geologic profile and the detailed movements recorded by one of the inclinometers, and Figure 5.24 shows the distribution of those movements along a cross section and the location of the failure plane. The piezometric data obtained during 2 months of observation showed that there were several perched water tables, but none with high uplift pressures. Water levels also were observed in the 10 inclinometer casings. However, this is not always a reliable means of groundwater observation because grout may tend to seal water either in or out of the aluminum well casing.

The data from the inclinometers demonstrated conclusively that slope failure was occurring along a near-horizontal plane only 3.3 to 4.5 m (10 to 15 ft) below the bottom of the subcut elevation, and, since the borings had failed to detect any unusual material at this depth, a decision was made to undertake a test-pit program to observe the materials along the failure surface. Eight test pits were dug, varying in depth from 3.3 to 5.5 m (10 to 18 ft). During test-pit inspection, the soil within the failure zone could be observed, sampled, and tested. The slide was found to be taking place on a thin seam of potassium bentonite, averaging less than 2.5 cm (1 in) in thickness; this material had been removed by the wash water during sampling and therefore had not been detected previously.

Stability computations were conducted on three representative cross sections where the limits of sliding were clearly defined by inclinometer data and by field observation of the heaving and cracking. In addition, the geology was more completely mapped at those locations. Since the movement developed along a horizontal plane surface, a sliding wedge analysis was used. Hydrostatic pressures, based on available piezometric data, were assumed. After extensive analysis and review, the remedial scheme finally recommended and adopted was reinforced concrete buttresses cast in narrow slit trenches excavated normal to the roadway center line (*5.57*).

Potrero Tunnel Movements

The Potrero Tunnel of the Southern Pacific Railroad was constructed in 1906 and underlies a ridge that crosses the only rail entrance into San Francisco (*5.46, 5.56*). Excavation for a freeway above the tunnel in 1967 initiated movements that cracked the lining and caused the two walls of the tunnel to move closer together. However, visual observation and surface measurements did not indicate clearly (a) whether a massive landslide was involved and (b) what the relation was between freeway construction and tunnel movements. Those questions had to be answered in order to resolve legal responsibilities and to design and construct remedial measures. The following types of instruments and measuring systems were installed:

1. Inclinometers to measure horizontal ground movements,
2. Extensometers to measure ground extension adjacent to the tunnel,
3. Portable extensometers to measure closing of the tunnel sides and heave of the roof,
4. Bench marks inside the tunnel to measure changes in alignment and elevation of the track, and
5. Bench marks to measure surface movement.

The distribution and magnitude of the ground movements at a typical section are shown in Figure 5.25. Analysis of the data indicated that the ground downhill from the tunnel was stable and that the movements uphill were evenly distributed over a 9-m (30-ft) thick zone. The increased lateral earth pressure against the sides of the tunnel resulted from the freeway construction and was the cause of the tunnel distress. Corrective treatment consisted of placing two rows of heavily reinforced cast-in-place concrete piles alongside the tunnel with a connecting strut above the tunnel; these were installed from the subgrade elevation of the freeway. The same instruments that were used to determine the displacements were also read during and after construction to verify the effectiveness of the remedial measurements.

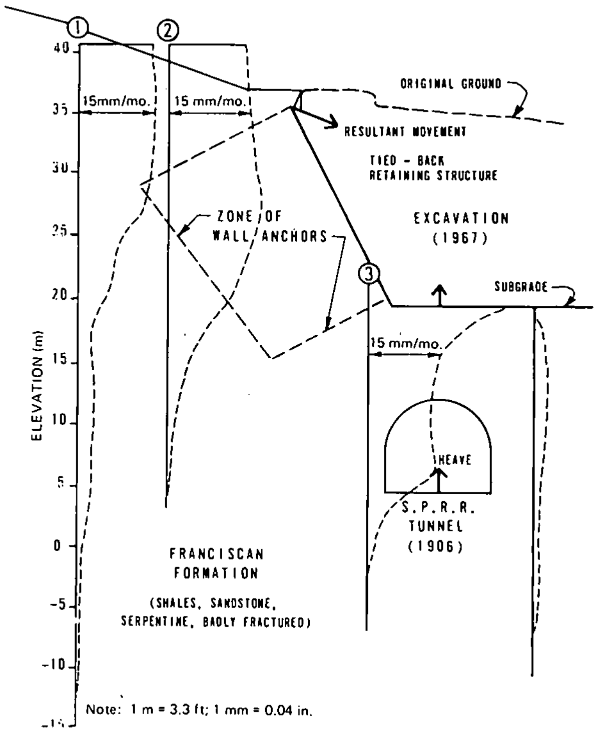
Seattle Freeway

In early 1960, a freeway constructed through downtown Seattle required the excavation of cuts into a gently sloping sidehill. A discovery was made at an early stage that even relatively shallow cuts, extending some distance along the hillside, could initiate serious ground movements. Extensive measurements, obtained primarily from inclinometers, demonstrated that the slides were progressive (*5.56*). After detailed study of the problem, the Washington State Highway Department developed the concept of cylinder-pile retaining walls, which consist of large-diameter concrete caissons cast in prebored, closely spaced holes placed uphill from the proposed cut before excavation. In each caisson, a massive H-beam was inserted to provide the necessary structural strength and resistance to bending. Next, a curtain wall was hung on the exterior of the piles to provide a finished surface. These cylinder pile walls, which act as cantilever beams, were designed to limit the maximum deflection of the top of the wall to approximately 5 cm (2 in).

Figure 5.26 shows the complexity of modern freeway construction in urban development. After the material was partially removed, as shown in the upper portion, movement occurred in the lower slope and cracks were detected in the apartment house. Inclinometers were then installed, as shown in the lower portion. Most of the inclinometers detected horizontal offsets in various layers well below the bottom of the excavation. To stabilize this section and complete the project required the installation of cylinder piles below the lower retaining wall, which had already been completed, and two rows of cylinder piles below the upper retaining wall, which also had been completed. The cylinder piles penetrated well below the movement zones. The effectiveness of the cylinder piles in stopping the movements is shown by the detailed records of movement of inclinometer 5 in Figure 5.27. During January and February 1963, the two rows of cylinder piles were installed and had an immediate influence in slowing down the rate of movement of inclinometer 5. After completion of the pile installation, the movement stopped and the section has since been stable (*5.37*).

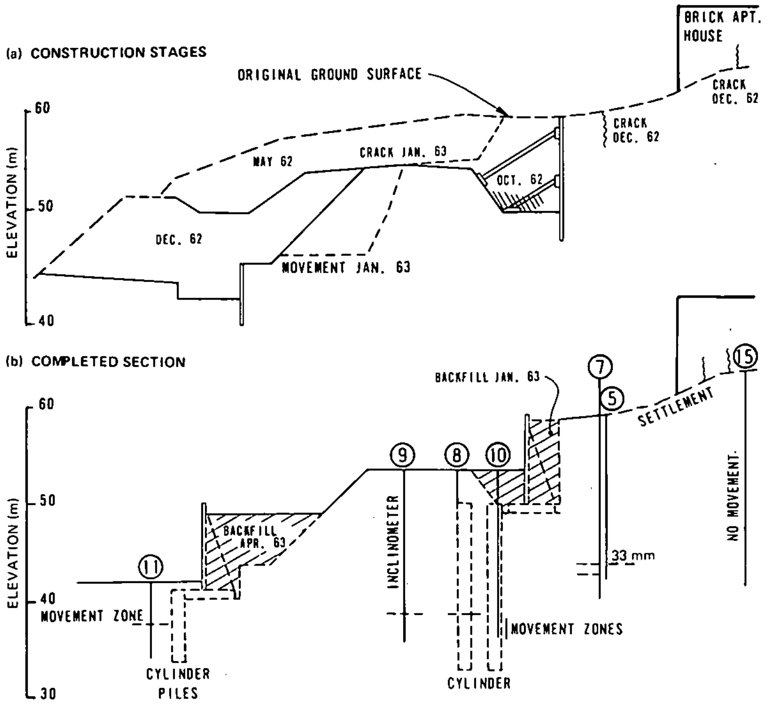
At the Tukwila interchange south of Seattle, inclinometers were installed in a completed wall to verify its performance after construction. Similar installations elsewhere showed that, in general, wall deflections seldom exceeded the design criteria of 5 cm (2 in). In this wall, however, small but gradually increasing horizontal movements were

Figure 5.25. Movements at Potrero Tunnel, San Francisco (*5.56*).



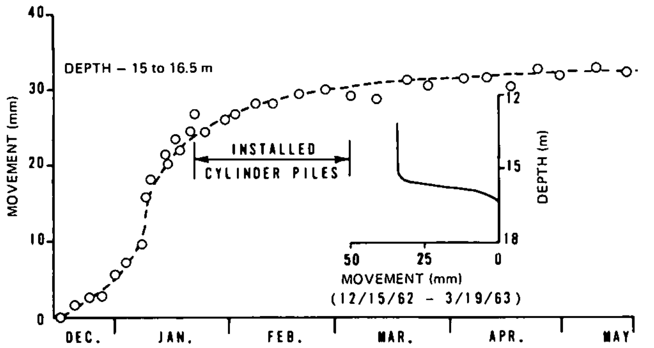
Note: 1 m = 3.3 ft; 1 mm = 0.04 in.

Figure 5.26. Seattle freeway problem (*5.56*).



Notes: 1 m = 3.3 ft; 1 mm = 0.04 in.
Circled numbers refer to inclinometer locations.

Figure 5.27. Movement of inclinometer 5, Seattle freeway (*5.56*).



Note: 1 m = 3.3 ft; 1 mm = 0.04 in.

detected in two zones well below the tip of the cylinder piles. Fortunately, the movements were detected at an early stage, thereby permitting the construction of a deeper wall before a failure condition developed. In this instance, there was no visual indication of a potential problem and surface alignment surveys would have been difficult because the wall was on a long sweeping curve.

Fort Benton Slide

Along the Burlington Northern rail line from Great Falls to Havre in western Montana, the tracks are situated on a sidehill fill some 12 to 15 m (40 to 50 ft) above the floodplain of the Missouri River. During a period of many years, the fill has experienced vertical and horizontal movement. These continuing movements have necessitated almost continuous maintenance, involving both realignment and restoration. In 1969, an investigation was undertaken to determine the probable cause of the continuing movements so that remedial treatment to improve stability could be initiated (*5.58*). To determine subsurface conditions beneath the site, several test borings were made and inclinometers, designated S-4 and S-5, were installed at the crest and toe of the slope respectively to determine the slide plane or zone of movement (Figure 5.28). Later in this study, for reasons described subsequently, a second inclinometer (S-6) was installed at the toe of the slope.

The borings for inclinometers S-4 and S-5 disclosed that the soils beneath the slope consisted of stiff to hard silts and clays of low to medium plasticity with intermittent zones or pockets of sand and gravel. These soils extended to a depth of about 16 m (52 ft) beneath the crest and 5 m (16 ft) beneath the toe. Below these depths, the soil encountered in both borings consisted of hard, mottled dark gray clay. It was first believed that the upper irregular zone of clay, silt, and sand was ancient slide debris or soil that had sloughed from the bluffs above and that the underlying hard clay was "original" ground. However, subsequent data obtained from inclinometer S-6 proved this was not the case. It was later determined that slide debris beneath the toe extended to a depth of about 11 m (37 ft) and was underlain by hard clay-shale that extended beyond the 26-m (85-ft) depth drilled. Although no additional drilling was accomplished at the top of the slope, movement recorded by inclinometer S-4 indicated that the surface of the hard clay-shale was probably at a depth of about 18 m (60 ft) beneath the tracks, about a meter below the bottom of the casing.

As stated earlier, movement of the sidehill fill had been occurring regularly for many years. Although no accurate

Figure 5.28. Section through Fort Benton slide, Montana (*5.58*).

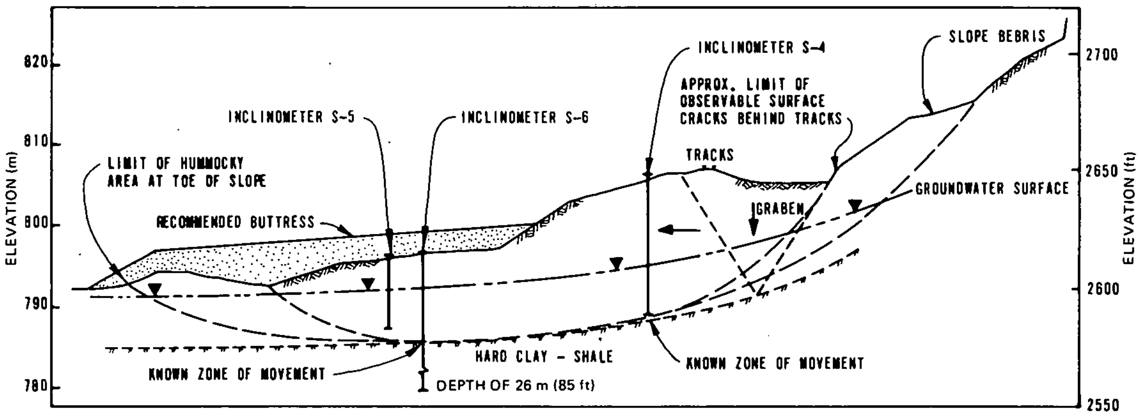


Figure 5.29. Movement of inclinometer S-5 at toe of slope, Fort Benton slide (*5.58*).

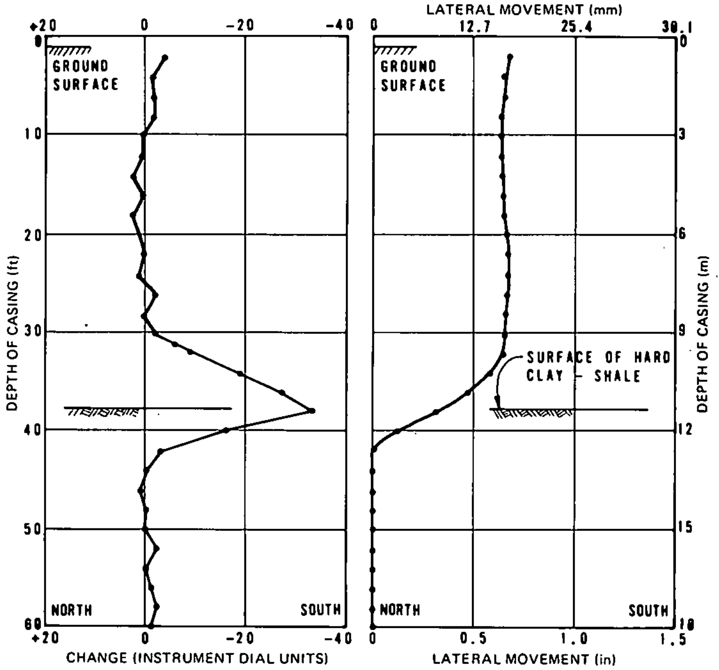
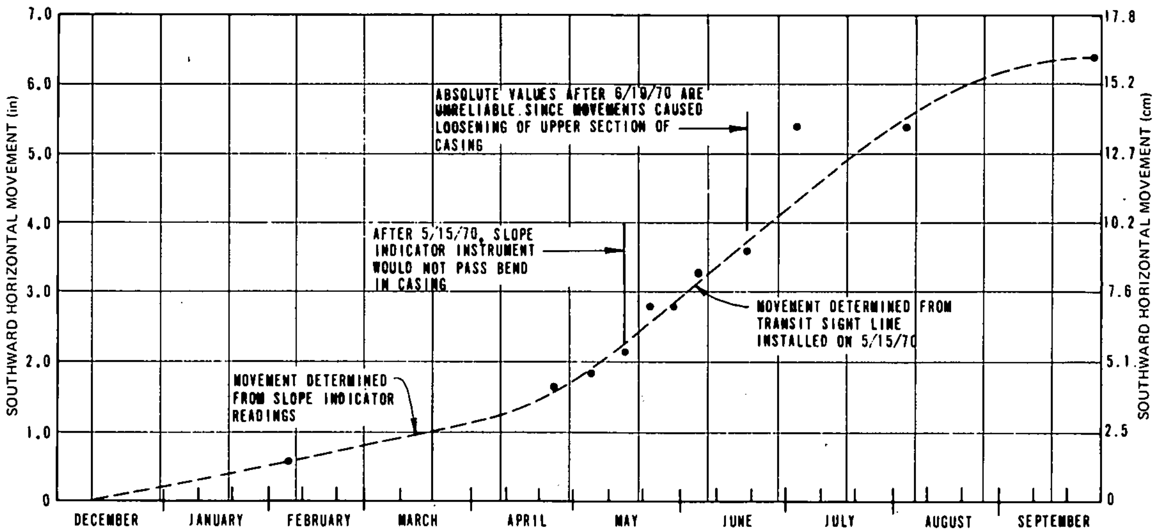


Figure 5.30. Movement of top of inclinometer S-4, Fort Benton slide (*5.58*).



records of the magnitude and direction had been kept, the predominant track movement had been vertical and accumulated settlement during the years amounted to nearly a meter. In addition, the movement rate varied seasonally, being greatest in spring and summer and least during late fall and winter. By mid-February 1970 (installation was in November 1969), the inclinometer at the crest of the slope indicated about 1.5 cm (0.5 in) of lateral (southward) movement; however, no change was recorded at the toe of the slope during this 3-month period. Movement of inclinometer S-4 continued, accelerating somewhat in early May and decreasing in late July and August. Data from this same installation showed that movement was occurring at a depth of 18 m (59 ft) and, because the bend in the casing was relatively abrupt, was occurring within a relatively thin zone, probably only a few centimeters thick. Although reliable data were being obtained from the installation at the top of the embankment, no changes were recorded at the toe of the slope, and it became apparent that the zone of movement beneath the toe of the slope was passing below the 9-m (30-ft) depth of the casing. Hence, a second, deeper casing (S-6) was installed.

Soon after the additional installation was made, the data showed definite displacement at the toe of the slope; movement was occurring within a relatively narrow zone some 11 m (37 ft) below the ground surface. These data are shown in Figures 5.29 and 5.30. The zone of movement correlated with the surface of the hard clay-shale layer that was encountered at the same depth. Once the failure mechanism had been established from the instrumentation data, the design of corrective measures could be started. Buttressing was selected as the most practicable means of stabilizing the hillside (Figure 5.28). Had the field data revealed movement was occurring within a cohesive zone (e.g., through a thin seam of bentonite clay), some other approach to stabilization would have been necessary.

REFERENCES

5.1 American Society of Civil Engineers. Proc., Specialty Conference on Performance of Earth and Earth-Supported Structures, Purdue Univ., Lafayette, Ind., American Society of Civil Engineers, New York, Vol. 1, Part 1, 883 pp., Part 2, 1555 pp.; Vol. 2, 154 pp.; Vol. 3, 385 pp., 1972.

5.2 Bishop, A. W., Kennard, M. F., and Penman, A. D. M. Pore-Pressure Observations at Selset Dam. In Pore Pressure and Suction in Soils, Butterworths, London, 1961, pp. 91-102.

5.3 Bishop, A. W., Kennard, M. F., and Vaughan, P. R. Developments in the Measurement and Interpretation of Pore Pressure in Earth Dams. Proc., 8th International Congress on Large Dams, Edinburgh, Vol. 2, 1964, pp. 47-72.

5.4 Blake, W., and Leighton, F. Recent Developments and Applications of the Microseismic Method in Deep Mines. In Rock Mechanics: Theory and Practice, Proc., 11th Symposium on Rock Mechanics, American Institute of Mining, Metallurgy and Petroleum Engineers, New York, 1970, pp. 429-443.

5.5 Blake, W., Leighton, F., and Duvall, W. I. Techniques for Monitoring the Behavior of Rock Structures. U.S. Bureau of Mines, Bulletin 665, 1974, 65 pp.

5.6 Boden, J. B., and McCaul, D. Measurements of Ground Movements During a Bentonite Tunneling Experiment. U.K. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. 653, 1974, 8 pp.

5.7 British Columbia Hydro and Power Authority. Inclinometer Data, Downie Slide. British Columbia Hydro and Power Authority, Vancouver, job file, 1976.

5.8 British Geotechnical Society. Field Instrumentation in Geotechnical Engineering. Wiley, New York, 1974, 720 pp.

5.9 Broms, B. B. Landslides. In Foundation Engineering Handbook (Winterkorn, H. F., and Fang, H. Y., eds.), Van Nostrand-Reinhold, New York, 1975, pp. 373-401.

5.10 Brooker, E. W., and Lindberg, D. A. Field Measurement of Pore Pressure in High Plasticity Soils. In Engineering Effects of Moisture Changes in Soils (Aitchison, G. D., ed.), Proc., International Research and Engineering Conference on Expansive Clay Soils, Texas A&M Univ., College Station, 1965, pp. 57-63.

5.11 Burland, J. B., and Moore, J. F. A. The Measurement of Ground Displacement Around Deep Excavations. In Field Instrumentation in Geotechnical Engineering (British Geotechnical Society), Wiley, New York, 1974, pp. 70-84.

5.12 Cooling, L. F. Second Rankine Lecture: Field Measurements in Soil Mechanics. Geotechnique, Vol. 12, No. 2, 1962, pp. 75-104.

5.13 Cording, E. J., Hendron, A. J., Jr., Hansmire, W. H., Mahar, J. W., MacPherson, H. H., Jones, R. A., and O'Rourk, T. D. Methods for Geotechnical Observations and Instrumentation in Tunneling. Department of Civil Engineering, Univ. of Illinois, Urbana-Champaign; National Science Foundation, Vol. 2, 1975, pp. 293-566.

5.14 Cornforth, D. H. Performance Characteristics of the Slope Indicator Series 200-B Inclinometer. In Field Instrumentation in Geotechnical Engineering (British Geotechnical Society), Wiley, New York, 1974, pp. 126-135.

5.15 Dallaire, E. E. Electronic Distance Measuring Revolution Well Under Way. Civil Engineering, Vol. 44, No. 10, 1974, pp. 66-71.

5.16 Dibiagio, E. Discussion: Session 2—Equipment. In Field Instrumentation in Geotechnical Engineering (British Geotechnical Society), Wiley, New York, 1974, pp. 565-566.

5.17 Dunnicliff, C. J. Equipment for Field Deformation Measurements. Proc., 4th Pan-American Conference on Soil Mechanics and Foundation Engineering, San Juan, American Society of Civil Engineers, New York, Vol. 2, 1971, pp. 319-332.

5.18 Durr, D. L. An Embankment Saved by Instrumentation. Transportation Research Board, Transportation Research Record 482, 1974, pp. 43-50.

5.19 Dutro, H. B., and Dickinson, R. O. Slope Instrumentation Using Multiple-Position Borehole Extensometers. Transportation Research Board, Transportation Research Record 482, 1974, pp. 9-17.

5.20 Eckel, E. B., ed. Landslides and Engineering Practice. Highway Research Board, Special Rept. 29, 1958, 232 pp.

5.21 Franklin, J. A., and Denton, P. E. The Monitoring of Rock Slopes. Quarterly Journal of Engineering Geology, Vol. 6, No. 3, 1973, pp. 259-286.

5.22 Goodman, R. E., and Blake, W. Rock Noise in Landslides and Slope Failures. Highway Research Board, Highway Research Record 119, 1966, pp. 50-60.

5.23 Gould, J. P., and Dunnicliff, C. J. Accuracy of Field Deformation Measurements. Proc., 4th Pan-American Conference on Soil Mechanics and Foundation Engineering, San Juan, American Society of Civil Engineers, New York, Vol. 1, 1971, pp. 313-366.

5.24 Green, G. E. Principles and Performance of Two Inclinometers for Measuring Horizontal Ground Movements. In Field Instrumentation in Geotechnical Engineering (British Geotechnical Society), Wiley, New York, 1974, pp. 166-179.

5.25 Hanna, T. H. Foundation Instrumentation. Trans Tech Publications, Clausthal, Germany, Series on Rock and Soil Mechanics, Vol. 1, No. 3, 1973, 372 pp.

5.26 Heinz, R. A. In Situ Soils Measuring Devices. Civil Engineering, Vol. 45, No. 10, 1975, pp. 62-65.

5.27 Hvorslev, M. J. Time Lag and Soil Permeability in Groundwater Observations. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Bulletin 36, 1951, 50 pp.

5.28 Kern and Company, Ltd. The Mekometer, Kern ME3000, Electro-Optical Precision Distance Meter. Kern and Co., Ltd., Aarau, Switzerland, Technical Information 10, 1974, 4 pp.

5.29 McCauley, M. L. Microsonic Detection of Landslides. Transportation Research Board, Transportation Research Record 581, 1976, pp. 25-30.

5.30 Malone, A. W. Elastic Wave Measurement in Rock Engineering. Imperial College, Univ. of London, PhD thesis, 1968, 299 pp.

5.31 Mearns, R., and Hoover, T. Subaudible Rock Noise (SARN) as a Measure of Slope Stability. Transportation Laboratory, California Department of Transportation, Sacramento, Final Rept. CA-DOT-TL-2537-1-73-24, 1973, 16 pp.

5.32 Merriam, R. Portuguese Bend Landslide, Palos Verdes Hills, California. Journal of Geology, Vol. 68, No. 2, 1960, pp. 140-153.

5.33 Mexico Ministry of Public Works. Failures at the Tijuana-Ensenada Highway. Mexico Ministry of Public Works, Mexico City, 1975, 220 pp.

5.34 Mikkelsen, P. E., and Bestwick, L. K. Instrumentation and Performance: Urban Arterial Embankments on Soft Foundation Soil. Proc., 14th Symposium on Engineering Geology and Soils Engineering, Boise, Idaho, Idaho Department of Highways, Univ. of Idaho, and Idaho State Univ., 1976, pp. 1-18.

5.35 Moore, J. F. A. The Photogrammetric Measurement of Constructional Displacements of a Rockfill Dam. Photogrammetric Record, Vol. 7, No. 42, 1973, pp. 628-648.

5.36 Muñoz, A., Jr., and Gano, D. The Role of Field Instrumentation in Correction of the "Fountain Slide." Transportation Research Board, Transportation Research Record 482, 1974, pp. 1-8.

5.37 Palladino, D. J., and Peck, R. B. Slope Failures in an Overconsolidated Clay, Seattle, Washington. Geotechnique, Vol. 22, No. 4, 1972, pp. 563-595.

5.38 Peck, R. B. Observation and Instrumentation: Some Elementary Considerations. Highway Focus, Vol. 4, No. 2, 1972, pp. 1-5.

5.39 Penman, A. D. M. A Study of the Response Time of Various Types of Piezometer. In Pore Pressure and Suction in Soils, Butterworths, London, 1961, pp. 53-58.

5.40 Penman, A. D. M., and Charles, J. A. Measuring Movements of Embankment Dams. In Field Instrumentation in Geotechnical Engineering (British Geotechnical Society), Wiley, New York, 1974, pp. 341-358.

5.41 Phillips, S. H. E., and James, E. L. An Inclinometer for Measuring the Deformation of Buried Structures With Reference to Multitied Diaphragm Walls. In Field Instrumentation in Geotechnical Engineering (British Geotechnical Society), Wiley, New York, 1974, pp. 359-369.

5.42 Piteau, D. R., Mylrea, F. H., and Blown, J. G. The Downie Slide, Columbia River, British Columbia. In Natural Phenomena: Developments in Geotechnical Engineering–Rockslides and Avalanches (Voight, B., ed.), Elsevier, New York, Vol. 1, 1978.

5.43 St. John, C. M., and Thomas, T. L. The N.P.L. Mekometer and Its Application in Mine Surveying and Rock Mechanics. Trans., Institution of Mining and Metallurgy, London, Vol. 79, Sec. A, 1970, pp. A31-A36.

5.44 Schmidt, B., and Dunnicliff, J. C. Construction Monitoring of Soft Ground Rapid Transit Tunnels. Urban Mass Transportation Administration, U.S. Department of Transportation, Vols. 1 and 2, 1974; NTIS, Springfield, Va., PB 241 536.

5.45 Shannon, W. L., Wilson, S. D., and Meese, R. H. Field Problems: Field Measurements. In Foundation Engineering (Leonards, G. A., ed.), McGraw-Hill, New York, 1962, pp. 1025-1080.

5.46 Smith, T. W., and Forsyth, R. A. Potrero Hill Slide and Correction. Journal of Soil Mechanics and Foundations Division, American Society of Civil Engineers, New York, Vol. 97, No. SM3, 1971, pp. 541-564.

5.47 Terzaghi, K. Mechanism of Landslides. In Application of Geology to Engineering Practice (Paige, S., ed.), Geological Society of America, Berkey Vol., 1950, pp. 83-123.

5.48 Tice, J. A., and Sams, C. E. Experiences With Landslide Instrumentation in the Southeast. Transportation Research Board, Transportation Research Record 482, 1974, pp. 18-29.

5.49 Toms, A. H., and Bartlett, D. L. Applications of Soil Mechanics in the Design of Stabilizing Works for Embankments, Cuttings and Track Formations. Proc., Institution of Civil Engineers, London, Vol. 21, 1962, pp. 705-711.

5.50 Transportation Research Board. Landslide Instrumentation. TRB, Transportation Research Record 482, 1974, 51 pp.

5.51 Vaughan, P. R. A Note on Sealing Piezometers in Boreholes. Geotechnique, Vol. 19, No. 3, 1969, pp. 405-413.

5.52 Vaughan, P. R. The Measurement of Pore Pressures With Piezometers. In Field Instrumentation in Geotechnical Engineering (British Geotechnical Society), Wiley, New York, 1974, pp. 411-422.

5.53 Vaughan, P. R., and Walbancke, H. J. Pore Pressure Changes and the Delayed Failure of Cutting Slopes in Overconsolidated Clay. Geotechnique, Vol. 23, No. 4, 1973, pp. 531-539.

5.54 Wilson, S. D. The Use of Slope Measuring Devices to Determine Movements in Earth Masses. In Field Testing of Soils, American Society for Testing and Materials, Philadelphia, Special Technical Publ. 322, 1962, pp. 187-197.

5.55 Wilson, S. D. Investigation of Embankment Performance. Journal of Soil Mechanics and Foundations Division, American Society of Civil Engineers, New York, Vol. 93, No. SM4, 1967, pp. 135-156.

5.56 Wilson, S. D. Observational Data on Ground Movements Related to Slope Instability. Journal of Soil Mechanics and Foundations Division, American Society of Civil Engineers, New York, Vol. 96, No. SM5, 1970, pp. 1521-1544.

5.57 Wilson, S. D. Landslide Instrumentation for the Minneapolis Freeway. Transportation Research Board, Transportation Research Record 482, 1974, pp. 30-42.

5.58 Wilson, S. D., and Hilts, D. E. Application of Instrumentation to Highway Stability Problems. Proc., Joint ASCE-ASME National Transportation Engineering Meeting, Seattle, American Society of Civil Engineers, New York, and American Society of Mechanical Engineers, New York, 1971.

5.59 Wilson, S. D., and Squier, R. Earth and Rockfill Dams. 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, State-of-the-Art Vol., 1969, pp. 137-223.

5.60 Wisecarver, D. W., Merrill, R. H., and Stateham, R. M. The Microseismic Technique Applied to Slope Stability. Trans., Society of Mining Engineers, American Institute of Mining, Metallurgical and Petroleum Engineers, New York, Vol. 244, 1969, pp. 378-385.