Table 19.3. Summary of Selected Case Histories of Braced Excavations

Project	Principal Concerns	Type of Support and Ground	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Harvard Square Subway Station, Cambridge, MA, U.S.A.	Stability of excavation Integrity of nearby structures Development of improved design procedures for slurry wall systems	Externally braced slurry walls in fill, sand, and glacial till	D D D D D G L S TS	Surveying methods Tiltmeters Inclinometers In-place inclinometers Probe extensometers Fixed borehole extensometers Open standpipe piezometers Load cells on external bracing Surface-mounted and embedment strain gages in slurry wall Contact earth pressure cells at face of wall	Very comprehensive instrumentation	Rawnsley et al. (1985) Hansmire et al. (1984)
Good Samaritan Hospital, Cincinnati, OH, U.S.A.	Stability of excavation	Externally braced soldier piles in residual and unweathered shale and limestone	D D D L	Surveying methods Inclinometers Fixed borehole extensometers Load cells on external bracing	Instrumentation planned for monitoring long-term performance	Anderson et al. (1984)
Sewer culvert in Islais Creek Basin, San Francisco, CA, U.S.A.	Stability of excavation Bottom heave	Internally braced sheet pile excavation in very soft clay	D D G S	Surveying methods Inclinometers Pneumatic piezometers Vibrating wire strain gages on internal bracing	Large movements measured	Clough and Reed (1984)
Two excavations for buildings in Taipei, Taiwan	Stability of excavations	Internally braced slurry walls in silty clay and silty sand	D D D G G G S	Surveying methods Inclinometers Heave gages Observation wells Open standpipe piezometers Pneumatic piezometers Vibrating wire strain gages on internal bracing		Moh and Song (1984)

Table 19.3. Summary of Selected Case Histories of Braced Excavations (*Continued*)

Project	Principal Concerns	Type of Support and Ground	Measurement[a]	Instrumentation Discussed	Special Features	Reference
			S	Strain gages on reinforcement in slurry wall		
			TS	Contact earth pressure cells on slurry wall		
Six-story building, Westminster, London, England	Stability of excavation	Internally braced slurry walls in London clay	D	Inclinometers		Wood and Perrin (1984)
			D	Probe extensometers		
			G	Open standpipe piezometers		
			S	Vibrating wire strain gages on internal bracing and on reinforcement in slurry wall		
Charles Center Station, Baltimore Metro, Baltimore, MD, U.S.A.	Integrity of nearby structures	Internally braced slurry walls in silty sand, gravel, and residual soil	D	Surveying methods		Zeigler et al. (1984)
			D	Subsurface settlement points		
			D	Inclinometers		
			G	Observation wells		
			S	Vibrating wire strain gages on internal bracing		
Rio de Janeiro Underground, Brazil	Development of improved design procedures for slurry wall systems	Internally braced slurry walls in sandy and silty clay and sand	D	Inclinometers in soil and slurry wall	Inclinometer data used to estimate bending moments in slurry wall	Soares (1983)
			D	Probe extensometers for monitoring bottom heave		
			D	Subsurface settlement points		
			G	Pneumatic piezometers		
			S	Vibrating wire strain gages in concrete and on reinforcing steel in walls		
			S	Vibrating wire strain gages on bracing		
			TS	Contact earth pressure cells on slurry walls		

Table 19.3. Summary of Selected Case Histories of Braced Excavations (*Continued*)

Project	Principal Concerns	Type of Support and Ground	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Underpass in Edmonton, Alberta, Canada	Loading on external bracing in winter conditions	Externally braced sheet piles in stiff fissured clay and till	L	Electrical resistance strain gage load cells on external bracing	Effect of temperature change on loading in external bracing	Morgenstern and Sego (1981)
University of Quebec, Montreal, Canada	Integrity of nearby structures	Externally braced slurry walls in glacial till	D D D L	Surveying methods Crack gages Inclinometers Hydraulic load cells on external bracing		Rosenberg et al. (1977)
Thorold Tunnel, St. Catharines, Ontario, Canada	Define cause of damage	Reinforced concrete highway tunnel in dolomite and limestone	D D D G S T	Surveying methods Convergence gages Inclinometers Piezometers Mechanical strain gages Thermocouples	Cracking of 6 ft (1.8 m) thick reinforced concrete side walls	Bowen and Hewson (1976)
Railway Tunnel, Oslo, Norway	Stability of excavation Development of improved design procedures	Internally braced sheet pile excavation in quick clay	D D G S TS	Surveying methods Inclinometers Open standpipe and vibrating wire piezometers Vibrating wire strain gages on bracing Contact earth pressure cells on sheeting	Excessive movement arrested by casting concrete beams across excavation below bottom	Karlsrud and Myrvoll (1976)
Washington Metro, Washington, DC, U.S.A.	Stability of excavation Development of improved design procedures Performance of strain gages	Two internally braced excavations	S	Vibrating wire strain gages on internal bracing		O'Rourke and Cording (1975)
Three excavations in Toronto, Canada	Improvement of design procedures	Externally braced excavations in sand, silt, and till	L	Load cells on external bracing		Trow (1974)
Embarcadero Subway Station, BARTD, San Francisco, CA, U.S.A.	Stability of excavation	Internally braced slurry walls in clay and sand	D D G S	Surveying methods Inclinometers Observation wells Vibrating wire strain gages on internal bracing		Armento (1973)

Table 19.3. Summary of Selected Case Histories of Braced Excavations (*Continued*)

Project	Principal Concerns	Type of Support and Ground	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Bank of California Center, Seattle, WA, U.S.A.	Stability of excavation Verification of design methods	Soldier piles and external bracing in overconsolidated clay	D D D L	Surveying methods Inclinometers Heave gages (fixed borehole extensometers) Load cells on external bracing		Clough et al. (1972)
CNA Center and Standard Oil Building, Chicago, IL, U.S.A.	Stability of excavation	Externally braced slurry walls in silty clay Internally braced slurry walls in medium clay	D D	Surveying methods Inclinometers		Cunningham and Fernandez (1972)
Test Site at Studenterlunden, Oslo, Norway	Behavior of slurry trench excavation: research test	Slurry trench excavation in soft clay: full-scale test	D D D G	Surveying methods Inclinometers Slurry trench convergence gages Vibrating wire piezometers		DiBiagio and Myrvoll (1972)
Subway extension, South Cove, Boston, MA, U.S.A.	Integrity of adjacent building	Internally braced slurry walls in clay	D D G S	Surveying methods Inclinometers Open standpipe and vibrating wire piezometers Vibrating wire strain gages on internal bracing		Lambe et al. (1972)
Office Building, Ottawa, Canada	Stability of excavation Comparison between measured and predicted movements	Externally braced sheeting in fill, stiff clay, and glacial till	D D G L	Surveying methods Fixed borehole extensometers Open standpipe and vibrating wire piezometers Load cells on external bracing		McRostie et al. (1972)
Greenway Pollution Control Centre, London, Ontario, Canada	Stability of excavation	Internally braced sheeting in fine sand	D G S	Surveying methods Open standpipe piezometers Mechanical strain gages on internal bracing		Scott et al. (1972)

Table 19.3. Summary of Selected Case Histories of Braced Excavations (*Continued*)

Project	Principal Concerns	Type of Support and Ground	Measurement[a]	Instrumentation Discussed	Special Features	Reference
			TS	Contact earth pressure cells on sheeting		
Various	Various	Internally and externally braced excavations	D G L S TS	Deformation gages Piezometers Load cells Strain gages Contact earth pressure calls	State-of-the-art summary of measurements	Gould (1970)
Pierre Laclede Building, Clayton, MO, U.S.A.	Stability of excavation	Soldier piles and external bracing in silty clay and clayey silt	S S	Mechanical strain gages on soldier piles Mechanical and electrical resistance strain gages on external bracing		Mansur and Alizadeh (1970)
Various	Various	Internally and externally braced excavations	D S	Deformation gages Strain gages on internal bracing	State-of-the-art summary of measurements	Peck (1969b)
Harris Bank and Trust Company, Chicago, IL, U.S.A.	Load in internal bracing	Internally braced excavation	S	Mechanical strain gages on internal bracing	Measurements initiated by construction contractor to justify reduced number of struts	Peck (1969a)

[a]D: deformation L: load in bracing T: temperature
G: groundwater pressure S: strain in support TS: total stress between supporting wall and soil

CHAPTER 20

EMBANKMENTS ON SOFT GROUND*

This chapter deals with instrumentation of embankments that do not retain water. Guidelines for water-retaining embankments are given in Chapter 21.

In many cases, selection of soil parameters for the foundation soil is reliably conservative. The embankment is therefore designed with confidence that performance will be satisfactory, and "comfortable" factors of safety are used. In such cases, many projects will proceed without the use of instrumentation. However, some uncertainties always exist.

Where design uncertainties are great, factors of safety small, or the consequences of poor performance severe, a prudent designer will include a performance monitoring program in the design.

20.1. GENERAL ROLE OF INSTRUMENTATION

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

Figure 20.1 shows a typical embankment on soft ground and conveys the contrast between how the project will usually appear on engineering drawings and what may actually happen in the field. Primary mechanisms that control the behavior of embankments on soft ground are described in Chapter 2.

The most frequent uses of instrumentation for embankments on soft ground are to monitor the progress of consolidation and to determine whether the embankment is stable. For example, instrumentation will normally be used to indicate the progress of consolidation beneath a single-stage embankment, a surcharged embankment, or staged construction, so that construction schedules can be determined. When vertical drains are installed, instrumentation will normally be used to evaluate whether they are being effective in accelerating consolidation. If the calculated factor of safety is likely to approach unity, instrumentation will generally be installed to provide a warning of any instability, thereby allowing remedial measures to be implemented before critical situations arise.

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

20.2. PRINCIPAL GEOTECHNICAL QUESTIONS

The following principal questions are presented in the normal order of occurrence for an embankment on soft ground. The order does not reflect a rating of importance.

*Written with the assistance of James R. Lambrechts, Senior Engineer, Haley & Aldrich, Inc., Cambridge, MA.

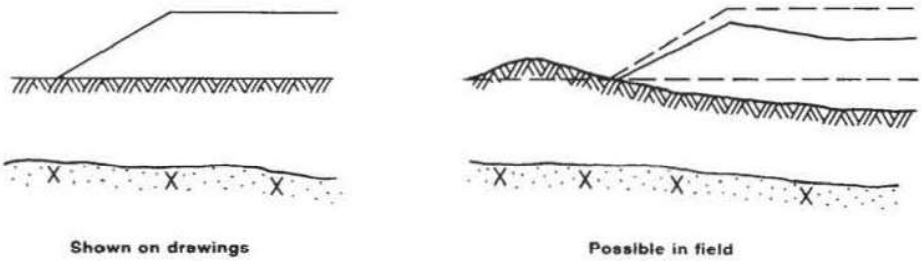

Figure 20.1. Embankment on soft ground (after DiBiagio and Myrvoll, 1981).

20.2.1. What Are the Initial Site Conditions?

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

20.2.2. What Information Can Be Provided by Instrumenting a Test Embankment?

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

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

Figure 20.2 shows a possible layout of instrumentation for monitoring the progress of consolidation beneath a test embankment when vertical drains have been installed. If vertical drains have not been installed, a similar layout may be appropriate. This is only one example, and it must be stressed that many other configurations are possible, as indicated by the case histories described later in this chapter.

When the test embankment is designed to fail, a knowledge of the location of the failure plane is required. For this purpose, shear plane indicators usually provide adequate data, and it is not always necessary to install inclinometer casings.

20.2.3. What Is the Progress of Consolidation?

When an embankment is constructed on a soft compressible foundation, the added weight of the em-

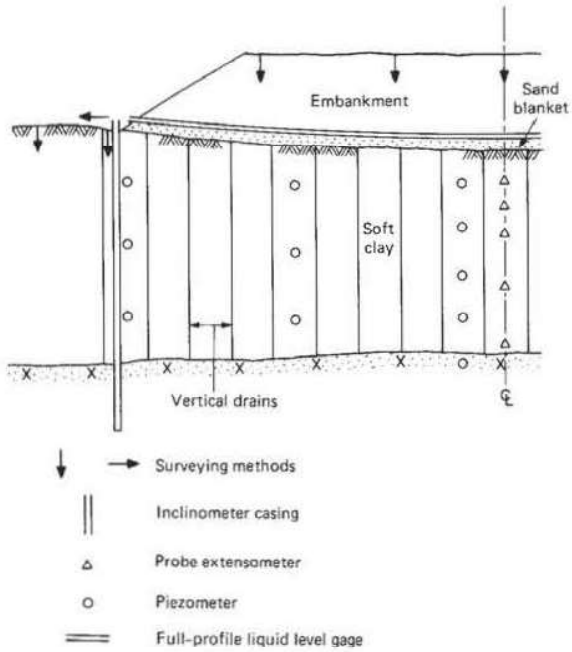

Figure 20.2. Possible layout of instrumentation for monitoring progress of consolidation beneath a test embankment when vertical drains have been installed. Note: Use of surveying methods requires benchmark and horizontal control station, remote from embankment. Piezometers also required, remote from embankment.

bankment causes consolidation in the foundation. The process of consolidation is described in Chapter 2. Construction of a facility on the top of the embankment, such as a building or a highway pavement, will normally be delayed until there is assurance that the facility will not be damaged by settlement that occurs during subsequent consolidation. If the predicted delay is unacceptable, the time required for consolidation can be reduced by *surcharging* or by installation of *vertical drains*.

Surcharging involves placement of fill material to a level higher than required for the embankment, so that consolidation proceeds more quickly. The excess material is removed when adequate consolidation has occurred.

Vertical drains are highly permeable columns of material installed at close intervals within the foundation soil. A continuous sand blanket is placed over the tops of the drains, so that the drains communicate with the blanket and allow pore water pressures to dissipate more rapidly, thus minimizing the time required for consolidation.

If the foundation might become unstable by placing the entire embankment rapidly, *staged construction* can be used. Staged construction involves placement of a first stage of fill material, waiting for consolidation and strength gain, and repeating the process until the embankment is complete. Staged construction can be used in conjunction with surcharging or vertical drains or can be used alone when sufficient time is available for consolidation.

When surcharging, vertical drains, or staged construction is employed, instrumentation will normally be used as a tool to indicate the progress of consolidation. Instrumentation data allow schedules to be determined for surcharge removal and for placement of stages during staged construction and also allow an evaluation of the effectiveness of vertical drains.

When evaluating the progress of consolidation beneath a single-stage embankment, a surcharged embankment, or staged construction, both settlement and pore water pressure measurements are normally made. Whenever piezometers are installed in the vicinity of the embankment, reference piezometers should also be installed, remote from the embankment, to monitor any variation in groundwater pressure that may result from other causes.

A predictive extrapolation can be made from a plot of pore water pressure versus time because the equilibrium piezometric level corresponding to full consolidation is known to be equal to the reference value remote from the embankment. Prediction from a plot of settlement versus time is less reliable, because the magnitude of ultimate settlement cannot be known with certainty. Ladd (1984) provides guidelines on the interpretation of field data when vertical drains are used.

Instruments for monitoring the progress of consolidation can be selected from the list given in Table 20.1. The list is not intended to be exclusive: it merely indicates instruments that have provided satisfactory data on projects with which the author has been involved. Selection among the various options will depend on the factors discussed in Chapters 9 and 12, including the feasibility of extending instrument pipes and leads up through the embankment without risk of damage and the need for redundancy. In all cases predictions should be made of maximum vertical foundation compression and horizontal spreading, and a special effort must be made to select instruments that are both capable of surviving the deformations and capable of providing reliable data as deformation occurs. Installation procedures must also be planned in recognition of the future deformations, following the guidelines given in Chapter 17.

The arrangement shown in Figure 20.2 for a test embankment is also a possible layout for monitoring the progress of consolidation beneath a prototype embankment.

When planning a program to monitor the performance of vertical drains, a decision must be made on whether to install instruments before or after installation of drains. Installing instruments well before drains ensures that baseline data are obtained but runs the risk of damage to instruments as drains are installed. The author suggests installing up to half of the instruments well before installing drains, and the remainder afterward.

20.2.4. Is the Embankment Stable?

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

Table 20.1. Instruments Suitable for Monitoring Progress of Consolidation

Measurement	Suitable Instruments
Vertical deformation of embankment surface and ground surface at and beyond toe of embankment	Surveying methods
Vertical deformation of original ground surface below embankment	Settlement platforms Buried plates Single-point liquid level gages Full-profile liquid level gages with pressure transducers and attached liquid-filled tubes[a] Horizontal inclinometer casings[a]
Vertical deformation and compression of subsurface	Spiral-foot settlement gages Probe extensometers with induction coil or magnet/reed switch transducers[b] Single-point liquid level gages
Groundwater pressure	Open standpipe "push-in" piezometers, as in Figure 9.23 Pneumatic and vibrating wire piezometers pushed in place below bottom of boreholes, as in Figures 9.31 and 9.32

[a]Casing or pipe should either have telescoping couplings or be surrounded with a corrugated plastic pipe.
[b]Requires careful selection of method for anchoring the measuring points to the soil, depending on the predicted vertical compression (see Chapter 12).

When a monitoring program is required to provide an indication of incipient failure, horizontal deformation measurements will normally provide the most direct data. An inclinometer is the primary tool, supplemented with surface deformation monitoring by surveying methods. However, measurements of pore water pressures may give a better first indication of failure conditions developing than can be derived from deformation data (Clausen et al., 1984). Rapidly responding diaphragm piezometers are required and, to be certain of having measurements in the zones of initial yielding, a relatively large number of piezometers may be required unless experience and proven methods of analysis can be used to locate the critical zones.

20.2.5. What Are the Fill Quantities?

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

20.3. OVERVIEW OF ROUTINE AND SPECIAL APPLICATIONS

For those readers who seek general guidelines on what is typically *routine* monitoring and what is monitoring for *special applications*, Table 20.2 presents a broad generalization.

20.4. SELECTED CASE HISTORIES

Table 20.3 summarizes selected case histories of instrumented embankments on soft ground. A study of these case histories reveals major similarities among many of the instrumentation programs, indicating that the general aspects of the state of the practice are well established. The general approach to instrumenting test and prototype embankments is similar, although the intensity of instrumentation is usually greater for test embankments.

Table 20.2. Overview of Routine and Special Monitoring

Application	Measurement
Routine monitoring	Vertical deformation of the embankment surface Vertical and horizontal deformations of the ground surface at and beyond the toe of the embankment Vertical deformation of the original ground surface below the embankment Groundwater pressure
Special applications	Subsurface vertical and horizontal deformations of the prototype Deformation and groundwater pressure during performance monitoring of a test embankment

Table 20.3. Summary of Selected Case Histories of Embankments on Soft Ground

Project	Principal Concerns	Type of Embankment	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Airport at Chek Lap Kok, Hong Kong	Feasibility of construction Need to establish construction procedures Performance of wick and sand drains	Test, for airport	G D D D D	Open standpipe and pneumatic piezometers (push-in type) Surveying methods Settlement platforms Probe extensometers Single-point and full-profile liquid level gages Inclinometers	Offshore reclamation Staged construction used Instrumentation designed to accommodate up to 30% vertical strain in foundation, large lateral deformation, over-water installation	Handfelt et al. (1987)
Geotextile-reinforced embankment, Bloomington Road, Aurora, Ontario, Canada	Performance of embankment during construction	Prototype, for highway	G D D S	Open standpipe piezometers (push-in type) Full-profile liquid level gages Inclinometers Strain gages on fabric (hydraulic and induction coil type)		Rowe et al. (1984a,b)
Highway B5, Brunsbüttel-Itzehoe, West Germany	Performance of embankment during construction: staged construction and surcharge used	Prototype, for highway	D D	Settlement platforms Combined probe extensometers and inclinometers		Thamm (1984)
Jourdan Road Terminal, New Orleans, LA, U.S.A.	Performance of stone columns and wick drains	Prototype, for wharf	G D D D	Pneumatic piezometers Settlement platforms Probe extensometers Inclinometers		Castelli et al. (1983)
Jourdan Road Terminal, New Orleans, LA, U.S.A.	Performance of stone columns in stabilizing clay deposit Magnitude of load carried by stone columns	Test, for wharf	G D D D TS	Pneumatic piezometers Probe extensometers Inclinometers Shear plane indicators Embedment earth pressure cells	Test embankment purposely failed by excavating adjacent ground Pneumatic piezometers fitted with two hydraulic tubes for flushing gas from cavity between diaphragm and filter tip	Munfakh et al. (1983)

Table 20.3. Summary of Selected Case Histories of Embankments on Soft Ground (*Continued*)

Project	Principal Concerns	Type of Embankment	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Site near Rio de Janeiro, Brazil	Progress of consolidation Stability Improvement of design procedures	Test	G D D D D D	Twin-tube hydraulic piezometers Surveying methods Settlement platforms Inclinometers Probe extensometers (installed horizontally) Fixed embankment extensometers (installed horizontally)	Failure of test embankment	Ramalho-Ortigão et al. (1983)
Site near Szolnok, Hungary	Progress of consolidation	Prototype, for highway	D	Buried plates		Rózsa and Vidacs (1983)
Queenborough Bypass, Isle of Sheppey, Kent, England	Progress of consolidation Stability: staged construction controlled by measurements Performance of wick and sand-wick drains	Prototype, for highway	G D	Twin-tube hydraulic piezometers (push-in type) Probe extensometers	Vertical drains installed after first stage measurements indicated need to accelerate consolidation	Nicholson and Jardine (1981)
River Street Viaduct, Tacoma, WA, U.S.A.	Progress of consolidation Stability: staged construction controlled by measurements	Prototype, for highway	G D D	Pneumatic piezometers (push-in type) Settlement platforms Inclinometers	Embankment constructed alongside railroad tracks and water main	Mikkelsen and Bestwick (1976)
I95, Saugus, MA, U.S.A.	Pore water pressures, deformation, and stability, as additional fill was placed	Test, for highway	G D D	Piezometers Settlement platforms Inclinometers	Ten teams of geotechnical engineers predicted behavior (before fill placement) Predictions and actual behavior are reported Failure of test embankment	MIT (1975)
Various	Various (numerous case histories)	Test, for highways, levees, research, dams	G D TS	Numerous types of instruments for measurement of groundwater pressure, deformation, and total stress	Discussion of role of test embankments	Bishop and Green (1974)

Table 20.3. Summary of Selected Case Histories of Embankments on Soft Ground (*Continued*)

Project	Principal Concerns	Type of Embankment	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Thames Oil Refinery, Canvey Island, Essex, England	Need to establish design criteria	Test, for oil storage tanks	G	Open standpipe piezometers		George and Parry (1974)
			D	Surveying methods		
			D	Probe extensometers		
			D	Single-point and full-profile liquid level gages		
			D	Inclinometers		
Flood Defense Banks, River Thames, England	Progress of consolidation Stability Comparison with predictions	Prototype, for flood defense	G	Pneumatic and electrical resistance piezometers	Pneumatic piezometers in organic deposits fitted with two hydraulic tubes for flushing gas from cavity between diaphragm and filter tip	Marsland (1974b)
			D	Surveying methods		
			D	Probe extensometers		
			D	Inclinometers		
M5 Motorway, Edithmead and Huntworth, England	Progress of consolidation: surcharge used Stability	Prototype, for highway	G	Twin-tube hydraulic piezometers		McKenna and Roy (1974)
			D	Surveying methods		
			D	Settlement platforms		
			D	Probe extensometers (installed horizontally and vertically)		
			D	Full-profile liquid level gages		
			D	Inclinometers		
Embankments at Saint-Alban, Quebec, Canada	Need to develop methods for predicting magnitude and rate of settlement for high embankments	Test, for research	G	Pneumatic and vibrating wire piezometers (push-in type)	Three test embankments, with same height but different side slopes	Tavenas et al. (1974)
			D	Surveying methods		
			D	Settlement platforms		
			D	Probe extensometers		
			D	Inclinometers		
Hampton Roads Crossing, VA, U.S.A.	Progress of consolidation: jetted sand drains installed Stability Measurement of fill quantity	Prototype, for highway	G	Open standpipe and pneumatic piezometers	Up to 12 ft (3.7 m) of settlement was measured	Kuesel et al. (1973)
			D	Surveying methods		
			D	Settlement platforms		

Table 20.3. Summary of Selected Case Histories of Embankments on Soft Ground (*Continued*)

Project	Principal Concerns	Type of Embankment	Measurement[a]	Instrumentation Discussed	Special Features	Reference
			D	Subsurface settlement points		
			D	Inclinometers		
I295, Portland, ME, U.S.A.	Performance of driven, augered, and jetted drains	Test, for highway	G	Vibrating wire piezometers		Aldrich and Johnson (1972)
			D	Settlement platforms		
			D	Subsurface settlement points		
Gloucester Test Fill, Ottawa, Canada	Comparison with predictions	Test, for research	G	Open standpipe piezometers		Bozozuk and Leonards (1972)
			D	Probe extensometers		
			D	Full-profile liquid level gages		
			TS	Embedment earth pressure cells		
Bangkok-Siracha Highway, Thailand	Bearing capacity of soft clay Need for stabilizing berm Performance of sand drains	Test, for highway	D	Settlement platforms		Eide and Holmberg (1972)
			D	Shear plane indicators		
Embankments at Skå-Edeby, Stockholm, Sweden	Performance of sand drains in varved clay Comparison with predictions	Test, for research	G	Hydraulic piezometers	Summary of 14 years of observations	Holtz and Broms (1972)
			D	Subsurface settlement points		
I95, Portsmouth, NH, U.S.A.	Need to establish design criteria	Test, for highway	G	Open standpipe and vibrating wire piezometers		Ladd (1972)
			D	Surveying methods		
			D	Settlement platforms		
			D	Subsurface settlement points		
			D	Inclinometers		
I95, Portsmouth, NH, U.S.A.	Progress of consolidation: sand drains and surcharge installed Stability: staged construction and stabilizing berms used	Prototype, for highway	G	Observation wells		Ladd et al. (1972)
			G	Open standpipe piezometers (push-in type)		
			G	Pneumatic and twin-tube hydraulic piezometers		
			D	Settlement platforms		

Table 20.3. Summary of Selected Case Histories of Embankments on Soft Ground (*Continued*)

Project	Principal Concerns	Type of Embankment	Measurement[a]	Instrumentation Discussed	Special Features	Reference
			D	Subsurface settlement points		
			D	Inclinometers		
Watergate Complex, Emeryville, CA, U.S.A.	Progress of consolidation: jetted sand drains installed	Prototype, for apartment and office complex	G	Pneumatic piezometers		Margason and Arango (1972)
King's Lynn Bypass, England	Need to establish design criteria: augered sand drains installed	Test, for highway	G	Twin-tube hydraulic piezometers	Failure of test embankment	Wilkes (1972)
			D	Surveying methods		
			D	Settlement platforms		
			D	Probe extensometers		
			D	Liquid level gages		
			D	Inclinometers		
			D	Shear plane indicators		
I95, Saugus, MA, U.S.A.	Need to develop methods for predicting deformation and stability	Test, for highway	G	Observation wells		Wolfskill and Soydemir (1971)
			G	Twin-tube hydraulic and vibrating wire piezometers		
			D	Settlement platforms		
			D	Subsurface settlement points		
			D	Inclinometers		
			TS	Embedment earth pressure cells		
Atchafalaya Levees, Mississippi River, LA, U.S.A.	Feasibility of using stabilizing berms	Test, for levee	G	Open standpipe, twin-tube hydraulic and pneumatic piezometers		Kaufman and Weaver (1967)
			D	Surveying methods		
			D	Settlement platforms		
			D	Subsurface settlement points		
			D	Inclinometers		
			D	Shear plane indicators		

[a]D: deformation
G: groundwater pressure
S: strain in structural member
TS: total stress in soil

CHAPTER 21

EMBANKMENT DAMS*

Embankment dams generally include earthfill dams of various types, rockfill dams with impervious earth cores, and rockfill dams with upstream concrete or asphaltic concrete facings. Primary mechanisms that control the behavior of embankment dams are described in Chapter 2.

21.1. GENERAL ROLE OF INSTRUMENTATION

The main purpose of instrumentation installed within an embankment dam is to study whether or not the dam is behaving according to design predictions. This general statement can be subdivided into two categories: first, the study of special problems at individual sites that are related to special foundation conditions or uncommon design features and second, the study of behavior when there are no such special problems.

21.1.1. Dams with Special Foundation Conditions or Uncommon Design Features

When instrumentation is used for this purpose, it helps to determine whether design assumptions for the special conditions and features are being realized during construction and operation. The design of the monitoring program is tuned directly for the special conditions and features and, because the designers of the dam know the weaknesses of the particular site and the sensitive features of the design, they play a leading role in the choice of type and location of instruments.

21.1.2. Dams Without Special Foundation Conditions or Uncommon Design Features

The practice of placing instruments routinely in embankment dams where there is no special problem to be studied, for example, a conventional dam of moderate height on a good foundation, has seen several ups and downs in the last 30 years. In the 1960s, many important dams were constructed with essentially no internal instruments. Each failure or major problem tends to be widely publicized, and dam engineers feel pressure to install instruments routinely for their own protection, even if they do not believe it absolutely necessary. The situation is similar to the dilemma of medical doctors who, fearing lawsuits if they do otherwise, prescribe suites of laboratory tests that may not be justified on medical grounds alone. At the present time, the failure of the Teton Dam is so recent that most engineers would hesitate to eliminate instruments in an important dam, for fear of criticism.

Most older dams of less than 50 ft (15 m) height have no instruments other than V-notch weirs or other means of measuring leakage. There are very many up to 300 ft (90 m) built during 1960–1975 without additional instruments. Although this is often not considered by inspecting engineers to be a

*Written with the assistance of Arthur D. M. Penman, Consulting Geotechnical Engineer, Harpenden, England, and James L. Sherard, Consulting Engineer, San Diego, CA.

shortcoming, the importance of many higher dams built during the last two decades has resulted in significant use of instrumentation.

Peck (1985) comments:

> Instrumentation, vital for obtaining quantitative answers to significant questions, is too often misused, especially in earth and rockfill dams. In some countries regulations concerning the safety of dams demand the incorporation of inclinometers, settlement indicators, and piezometers in the cores of virtually all new dams, but for what purpose? Not for research, because the patterns of deformation and pore pressure development for ordinary geometrics and materials are now well known and can be predicted by calculation. Only under unusual circumstances can it be said that design assumptions in these regards require verification. Yet, installation of instruments, even under the best of circumstances, introduces inhomogeneities into the cores, and occasionally is the direct cause of such local defects as sinkholes. The potential weakness introduced by an installation should be balanced against the potential benefit from the observations. In contrast to those located in cores, piezometers in foundation materials near the downstream toes detect upward seepage pressures that cannot be predicted reliably, and can thus give timely warning if measures are needed to ensure safety. There is a danger that instrumentation may be discredited because of indiscriminate use.

Of all the internal instruments in embankment dams, piezometers are by far the most common, and an installation consisting only of piezometers can serve two purposes. First piezometers in a clay core indicate rates of dissipation of pore water pressure and the approach of equilibrium conditions after impounding. Second, piezometers may be appropriate where it is desired to avoid criticism when designing a dam of conventional design and moderate height on a good foundation

21.1.3. Instruments that Cause Problems

Instruments can be a direct cause of problems. Peck refers above to instruments causing local defects such as sinkholes, and these cases are generally where inferior compaction has resulted from carrying tubes, cables, or pipes upward as filling proceeds. The solution to this problem is to avoid vertical risers (except within filters as described in Section 21.4), following the guidelines given in Section 21.4 and Chapter 17.

There is another category of problems, which can be classified as *rogue instruments*. In this category the dam is performing perfectly well but concern arises because a malfunctioning instrument indicates unexpected or alarming measurements. Often it is impossible to examine the health of the instrument directly and, after intensive study and worry, a final conclusion is reached that the instrument is not functioning correctly.

There have been a sufficient number of cases in which instruments have been the cause of problems, so that the designer of an embankment dam is likely to be very wary of geotechnical instrumentation. The solution to this dilemma has two parts. First, instruments should be used only when there is a valid reason that can be defended. Second, the overriding feature when selecting instruments should be *reliability*. A recipe for reliability is given in Chapter 15.

21.1.4. Summary to the General Role of Instrumentation

Despite the reservations given above, instrumentation **can** indeed be used to assist in obtaining quantitative answers to significant questions during the design, construction, and performance phases, and guidelines are given in the remainder of this chapter.

The general role of instrumentation for embankment dams is described by ICOLD (1969). The International Commission on Large Dams, Committee on Monitoring Dams and their Foundations, is currently assembling a series of national state-of-the-art reports presenting the instrumentation practices in various countries, which are intended to supersede ICOLD (1969). USCOLD (1986) is the U.S. contribution.

21.2. PRINCIPAL GEOTECHNICAL QUESTIONS

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

21.2.1. What Are the Initial Site Conditions?

Initial site conditions are determined by use of conventional site investigation procedures, sometimes supplemented by in situ testing. However, piezometers also play a role.

It is usually of value to install piezometers, preferably of the open standpipe type, in the exploratory boreholes made during the investigation of a dam site, rather than simply backfilling the boreholes. Regular readings can reveal much about groundwater pressure conditions on the site before dam construction. They are particularly useful in the abutments, where knowledge of groundwater pressures and their seasonal variations is vital to an understanding of abutment permeability and the need for grouting in rock abutments. Selected piezometers can later be converted, as described in Chapter 9, to be read remotely and form part of the long-term monitoring system.

21.2.2. Is Performance Satisfactory During Construction?

Dams with Special Foundation Conditions or Uncommon Design Features

If the site includes special foundation conditions or the design of the dam has uncommon features, instrumentation is a valuable tool to assist with construction control.

For example, soft foundation conditions may lead to the use of instrumentation for monitoring foundation stability and the progress of consolidation, as described for embankments on soft ground in Chapter 20. A similar need may arise for monitoring within the embankment itself, when there is concern that pore water pressures may become excessive during construction.

As another example, if there is concern that tensile strains may cause adverse behavior, such as in high dams with irregular foundations or steep abutments, instrumentation can be used to provide performance data and indicate the need for remedial action. Figure 21.1 illustrates such an application.

It is not reasonable to make a complete list of recommended instruments for monitoring such special conditions or uncommon features, because the selection depends on the details of each case. The primary monitored parameter will generally be either deformation or pore water pressure, but on occasion measurements of total stress may be included.

Dams Without Special Foundation Conditions or Uncommon Design Features

When the dam does not have special foundation conditions or uncommon design features, instrumentation can be used to determine whether or not the dam as a whole is behaving according to design predictions.

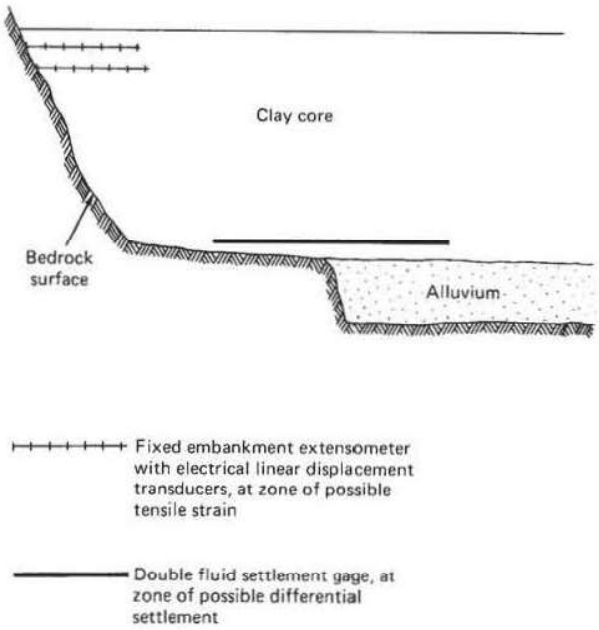

Figure 21.1. Possible layout of instrumentation for monitoring zones of potential tensile strain and differential settlement.

As an example, Figure 21.2 shows a possible arrangement of instrumentation for monitoring deformation of a dam that has been designed in accordance with acceptable deformation criteria. If measured deformations exceed acceptable values, construction procedures can be amended, for example, by building berms, flattening slopes, or slowing down the rate of construction.

One proven combination of instruments shown in Figure 21.2 is a probe extensometer with a current-displacement induction coil transducer (sometimes called a *horizontal plate gage*) and an overflow-type full-profile liquid level gage, both of which are described in Chapter 12. Figure 21.2 shows the system installed within a rockfill shell, but it can also be installed within downstream shells of other materials, provided that drainage is provided downstream of the core. The combined version for use in embankment dams is described by Penman and Charles (1982), and a precision of ± 0.02 in. (± 0.5 mm) is possible for both horizontal and vertical deformation measurements. The pipes are placed at the same inclination as lifts of fill, thereby minimizing interference to construction, and no delicate components are embedded. When used, such an ar-

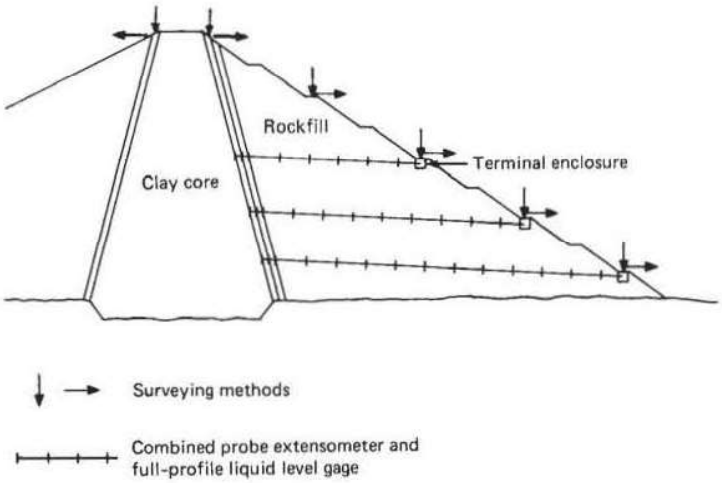

Figure 21.2. Possible layout of instrumentation for monitoring vertical and horizontal deformation during construction. Note: Use of surveying methods requires benchmarks and horizontal control stations, remote from dam.

rangement is typically installed at two cross sections, one at or near the maximum height, and permits comparison with a two-dimensional analysis.

It should be noted that the instruments shown in Figure 21.2 do not require extending leads or pipes upward through the fill. They are therefore preferable to use of vertically installed inclinometers and probe extensometers, both of which cause interruption to construction and often inferior compaction around the pipes. As indicated in Section 21.1, this practice can cause significant problems. Despite these comments, inclinometers are being used with increasing frequency, and the author recommends against this trend. However, when horizontal deformation data cannot be obtained by alternative instruments, such as for the special problem of potential shear sliding on weak horizontal clay or claystone beds in the foundation, inclinometers are the instruments of choice but, as discussed later, they should be installed only within filter zones.

On occasion, designers opt for monitoring horizontal and vertical deformations in a plane parallel to the axis of the dam, so that analyses can be made in two perpendicular planes. However, the types of instruments shown in Figure 21.2 can be read only from galleries in the abutments, and galleries are not normally justified for instrumentation alone. When such an instrumentation program is warranted, settlement measurements should be made with liquid level gages, with a preference for either the single-point overflow version with duplicate weirs or the double fluid settlement gage. Measurements of horizontal deformation can be made with inclinometers, but this practice is discouraged. Fixed embankment extensometers with electrical linear displacement transducers can also be used for measuring horizontal deformation, but these are expensive and much more prone to malfunction than the instruments shown in Figure 21.2.

21.2.3. Is Performance Satisfactory During First Filling?

Instrumentation data during first filling of the reservoir provide verification that the performance of the dam is within acceptable limits or that an unexpected event is occurring.

Regular visual observations and monitoring of leakage that emerges downstream are essential to assessing the behavior of a dam during first filling. The first indication of a potential problem is often given by an observed change of leakage rate, and monitoring the solids content in the leakage water can provide important information. Effectiveness of drains, relief wells, and grout curtains can be evaluated by measuring pore water pressures.

Equipment to measure leakage quantities formed some of the earliest instrumentation installed at dam sites. Where underdrains or relief wells are connected to manholes, flows are often measured with V-notch weirs or, when rates are low, by diverting the flow temporarily into containers of known volume. More sophisticated equipment in-

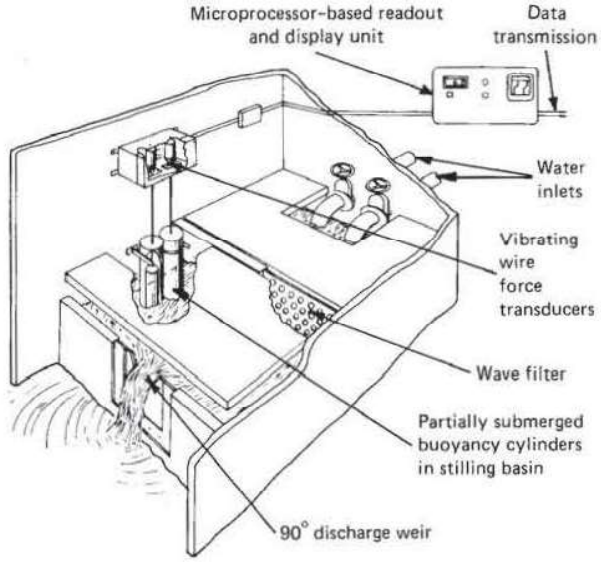

Figure 21.3. Remote-reading weir for monitoring leakage quantities (courtesy of Geonor A/S, Oslo, Norway).

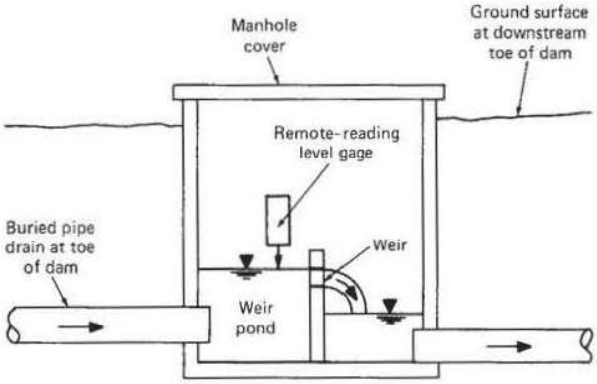

Figure 21.4. Arrangement for monitoring water flows in a buried pipe drain near the toe of a dam.

cludes remote-reading level sensors to record water levels and hence flows over weirs. Figure 21.3 shows a useful version that uses vibrating wire transducers to measure buoyancy of a partially submerged cylinder in the stilling basin. Duplicate cylinders and transducers are included to provide redundancy. However, DiBiagio and Myrvoll (1985) describe false measurements with this device, caused both by blockage of water flow and by freezing, and recommend heating of the weir station, temperature monitoring of both water and air, and electrical level switches in the stilling basin. Bartholomew et al. (1987) describe various methods for monitoring the quantity and quality of leakage water.

Figure 21.4 illustrates an arrangement for monitoring water flows in a buried pipe drain near the toe of a dam, and the addition of the remote-reading system greatly reduces the reading effort. At Svartevann Dam (DiBiagio et al., 1982), a small seepage barrier was built on the impervious bedrock directly under the crest of the sloping core so as to catch core leakage but reduce the contribution from rainfall. The arrangement, shown in Figure 21.5, in-

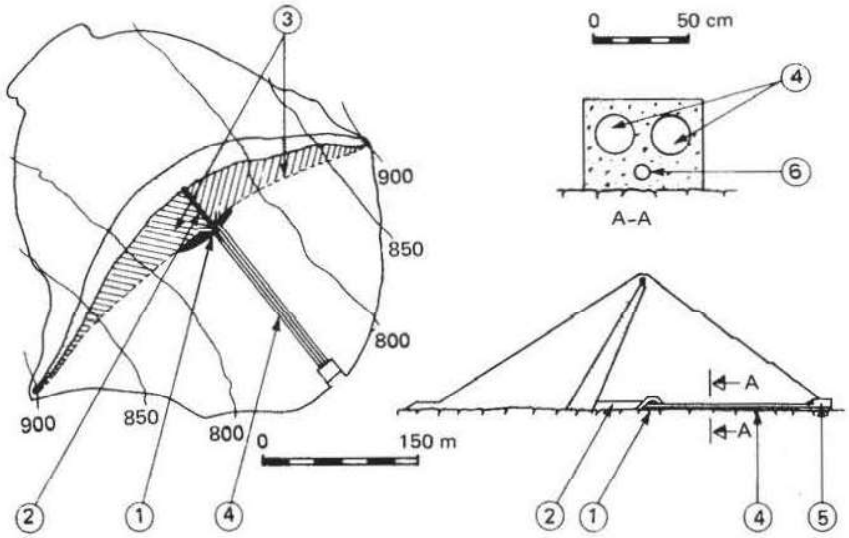

Figure 21.5. Arrangement for measurement of leakage water at Svartevann Dam: 1 seepage barrier, 2 partition wall, 3 catchment area, 4 pipelines encased in concrete, 5 instrument house with weir station, 6 conduit for instrument cables (after DiBiagio and Myrvoll, 1985).

cluded a longitudinal concrete partition wall between the core and the seepage barrier to separate right- and left-side leakage flows. Pipes were laid from the seepage barrier to a terminal enclosure at the downstream toe, and leakage was measured with the system shown in Figure 21.3.

When interpreting measurement data from downstream weirs, one must remember that the weirs also capture rain runoff. DiBiagio and Myrvoll (1985) report on the development of a precipitation gage suitable for remote measurements at dam sites.

When internal instrumentation has been installed to monitor performance during construction or drawdown or to monitor long-term performance, the same instrumentation will be monitored during first filling to determine whether or not the dam is behaving according to design predictions. Apart from instrumentation to monitor leakage, internal instruments installed specifically for monitoring performance during first filling are generally limited to piezometers for determining effectiveness of drains, relief wells, and grout curtains.

For a rockfill dam with upstream concrete facing, additional instrumentation is typically used to monitor deformation at perimeter joints in the facing. An electrical crack gage with a variable reluctance transducer is typically used.

21.2.4. Is Performance Satisfactory During Drawdown?

Rapid drawdown may cause instability because of the removal of stabilizing water forces. For dams with upstream slopes of impervious soil, it is difficult to predict pore water pressures following rapid reservoir drawdown. For any such dam retaining a reservoir that will be subject to drawdown at periodic intervals, a piezometer installation is desirable. This applies particularly to upper reservoirs for pumped storage hydro projects and to reservoirs that will be emptied partially or completely at periodic intervals for any purpose. When evaluating the need for piezometer installations, one must consider the rate of drawdown in light of the properties of the upstream shell material. With a clay, "rapid" may be a drawdown taking several weeks.

Penman (1971) gives examples of pore water pressures measured during drawdown.

21.2.5. Is Long-Term Performance Satisfactory?

A perspective on the use of instrumentation for monitoring long-term performance has been given in Section 21.1.

The subject of long-term performance monitoring of embankment dams merits a main section in this chapter and is included as Section 21.3.

21.2.6. Can the State of the Art Be Improved?

Instrumentation has provided and will continue to provide basic data for improvement of design practices, and without the knowledge gained from instruments during the past 20 years we would not be able to design and build embankment dams as we do today. Past measurements have strengthened our confidence in building dams with increasing height at sites with steep abutments and narrow valleys. Past measurements have also shown that well-designed dams are safe despite the important transfer of stress that is caued by differential settlements. Measurements with piezometers have revealed construction pore water pressures and provided essential data for the design and construction control of embankment dams with "wet" cores. As more instrumentation data become available, future design improvements will be made with confidence.

In recent years, the instrumentation of concrete face dams has contributed significantly to the data base for the design of future dams of greater height. A typical layout for a dam exceeding 400 ft (120 m) in height is shown in Figure 21.6. The joint meters are used to measure joint opening, offset, and parallel movement below normal minimum water level. The joint pins are above minimum water level to measure joint opening and are located where opening is anticipated. For lower dams, but above about 200 ft (60 m) in height, a typical program would consist of surface settlement points on the crest, two levels of three liquid level gages at the maximum section, and a leakage weir. Ninety percent of the face movements typically take place during first filling, and crest settlement is essentially complete in 2 years.

The profession has no reliable method for predicting total stresses between earth and concrete dams at connections, except with a soft clay core. The same difficulty exists for total stresses on the surfaces of concrete conduits passing through the

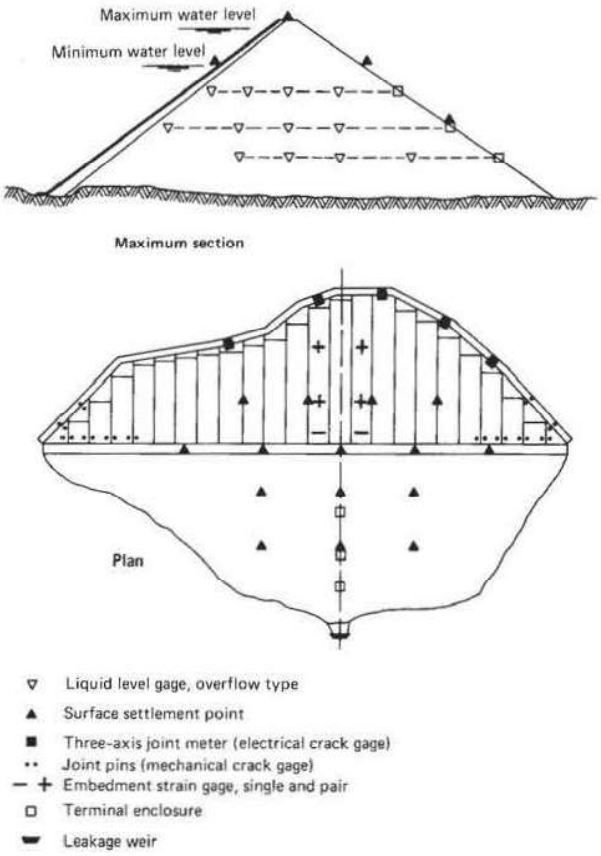

Figure 21.6. Typical layout of instrumentation to contribute to the data base for the design of future concrete face rockfill dams (after Cooke, 1986). Note: Use of surveying methods requires benchmark, remote from dam. For dams higher than about 450 ft (140 m), typical layout includes inclinometer casing on or below face slab.

bases of dams. Contact earth pressure cells are being used to provide case history data, and piezometers are usually installed nearby so that effective stresses can be determined.

21.3. LONG-TERM PERFORMANCE MONITORING OF EMBANKMENT DAMS

Once an embankment dam has reached full height, most failures are the result of internal erosion, overtopping, or other unexpected event, and a designer should be reluctant to depend on instrumentation for an advance warning of adverse behavior.

The need for long-term instrumentation depends greatly on the basic conservatism and foundation conditions of the dam. For example, a concrete face rockfill dam on a sound rock foundation forms one extreme, and an argument can be supported that instrument readings are only a formality, that the dam cannot fail except by severe overtopping. A dam on a horizontally bedded shale is an example of the other extreme, where measurements during construction show shear movement in the shale, and where calculations indicate a marginal safety factor against shear movement. In this case measurements of deformation and pore water pressure may be justified for several decades.

Table 21.1 gives a broad generalization, in priority order, of measurements that may be made for monitoring long-term performance. However, the priority must be tempered by the perspective given in the previous paragraph and in Section 21.1. The table also indicates recommended instruments and additional instruments for use in special cases: justifications for these recommendations are given later in this section.

Monitoring emphasis should be on regular visual observations and leakage quantities. Observations and instrumentation complement each other. When reviewing the performance of an existing dam on the basis of visual observations only, one may miss important behavioral trends. However, if judgments are made using instrumentation records without visual observation, incorrect conclusions may be drawn if instruments are not at the critical location or if they are not read at the critical time.

Instrumentation to monitor seismic events is now routine for important dams in areas of potentially strong earthquakes.

The need for additional external or internal instrumentation depends on the individual characteristics of the dam. The selection of instrument quantities and types must consider both technical factors and the capability of the owner to read instruments and interpret the data and will often be a compromise between criteria put forward by the designer, the owner, and regulating agencies.

If additional instrumentation is used, piezometers should be the first choice, and deformation monitoring should be the second choice. If instrumentation has been installed to monitor deformations during construction, instrument readings are often continued routinely in the long term. Measurements of total stress within the embankment are rarely reliable enough for long-term performance monitoring, but contact earth pressure cells can be used to monitor total stresses between earth

Table 21.1. Measurements and Instruments for Long-Term Performance Monitoring

Measurement, in Priority Order	Recommended Instruments	Additional Instruments for Special Cases
Condition of entire structure	Visual observations	
Leakage emerging downstream	Leakage weirs Precipitation gage	
Performance of relief wells	Leakage weirs Open standpipe piezometers	
Seismic events	Strong motion accelerographs Microseismographs	
Pore water pressure within the embankment	Open standpipe piezometers[a] Twin-tube hydraulic piezometers	Vibrating wire piezometers Pneumatic piezometers
Vertical movement of the embankment surface	Optical leveling Trigonometric leveling Satellite-based system Benchmarks	
Lateral movement of the embankment surface	Electronic distance measurements Triangulation Satellite-based system Horizontal control stations	
Vertical deformation within the embankment	Single-point and full-profile liquid level gages, overflow type Double fluid settlement gages Horizontal inclinometers Benchmarks	Probe extensometers, installed vertically[a]
Lateral deformation within the embankment	Probe extensometers with multiple induction coil or magnet/reed switch transducers, connected by rods and installed horizontally	Fixed embankment extensometers with vibrating wire transducers, or induction coil transducers with frequency output
	Horizontal control stations	Inclinometers[a]
Total stress at contact between the embankment and a structure	Contact earth pressure cells	

[a]If carried up through fill, there will be significant interruption to construction and the probability of inferior compaction.

and concrete dams at connections and on the surfaces of concrete conduits passing through the bases of dams.

Instrumentation installed for monitoring long-term behavior during the life of a dam will in general impose selection criteria different from short-term instrumentation, and a dam designer is advised to view each of the two needs separately. Although in both cases the overriding desirable feature is reliability, instruments for short-term monitoring may be called on to provide extensive data in a short time period and, provided appropriate contractual arrangements have been made, will be read and maintained by competent personnel. This is generally not the case for long-term instruments, when durability, simplicity, and ease of reading are primary criteria.

Recommendations for monitoring various parameters during service life of an embankment dam are given in Sections 21.3.1–21.3.10. Recommendations for monitoring the reservoir rim are given by USCOLD (1979).

21.3.1. Regular Visual Observations

In many countries, requirements for regular visual observation are laid down by regulation. A detailed discussion of regular visual observations of em-

bankment dams is beyond the scope of this book. They are mentioned here to remind readers that regular visual observations are an essential aspect of a program for monitoring long-term performance. In fact, visual observations by reservoir staff, trained to look for seeps, boils, shallow sloughing, cracks, or any other signs of distress, to log their observations, and to contact the responsible engineer when appropriate, is the primary approach to monitoring long-term performance of embankment dams. The visual observations should include the spillway and the abutments. If visual observations indicate a potential problem, it may be necessary to initiate a quantitative monitoring program to define the problem and assist in selecting a solution.

21.3.2. Monitoring of Leakage

Recommendations for monitoring leakage are given in Section 21.2.3.

Long-term performance of relief wells deserves special attention. Their efficiency can be reduced by plugging of filter packs or by accumulation of carbonates or bacteria, and it is often worthwhile to install open standpipe piezometers midway between selected wells and also directly adjacent to some wells. If indicated pressures become excessive, wells should be cleaned out or replaced.

21.3.3. Monitoring of Seismic Events*

The outdated concept that seismic instrumentation of embankment dams and reservoir sites is only a research tool has given way to the modern concept that seismic instrumentation is necessary for moderate to high dams in seismic areas. It is also desirable in traditionally nonseismic areas. This modern concept was graphically illustrated in the 1975 earthquake that subjected Oroville Dam, previously considered to be located in a nonseismic area, to significant levels of seismic shaking (Department of Water Resources, 1979).

Two classes of seismic instrumentation are used for monitoring dams and reservoirs: *strong motion monitoring* on and near the dam itself and *reservoir seismicity networks* in the area of the project site. USCOLD (1985) presents a compilation of U.S. dams with instruments for monitoring strong motion and reservoir seismicity.

*Written with the assistance of Marshall L. Silver, Marshall Silver & Associates Ltd., Chicago, IL.

Strong Motion Monitoring

Strong motion monitoring is used to measure the response of the dam to ground shaking. Although measurements can be used to validate seismic design assumptions, the most important benefit is to guide decisions on inspection and repair after the dam has been subjected to a seismic event. For example, if an earthquake has caused structural damage to the dam or to its appurtenant structures, the following questions must be answered immediately:

1. Was the earthquake larger or smaller than the design earthquake?
2. What will be the performance of the dam in the event of a larger shock?
3. What repair or strengthening is required?

If no *obvious* earthquake damage has occurred, a decision must be made on the extent of elaborate and expensive inspection operations, and data provided by strong motion monitoring equipment provide vital input to answering questions and reaching decisions. Deformation data provided by static instrumentation are also valuable, because seismic events often cause regional tectonic movements and also deformation within cohesionless soil deposits. On special occasions the pulses of seismically induced excess pore water pressures can be monitored during ground shaking, requiring the installation of piezometers with high dynamic response (Banister et al., 1976; Ishihara, 1981).

Instruments for strong motion monitoring are normally called *strong motion accelerographs* or *seismographs*. The key element of the instrument is an accelerometer, which can be thought of as a mass suspended in a case. The case is itself securely anchored to the dam. During an earthquake, relative movement between the mass and the case provides an electrical signal proportional to either the acceleration or velocity of the ground motion. An accelerograph also contains signal amplifiers, a recording device (paper, photographic film, or magnetic tape), a rechargeable battery power supply, a highly accurate clock, and a seismic trigger set to turn on the instrument when a preset level of ground shaking is exceeded.

Strong motion accelerographs should, as a minimum, be located at the base and crest of the dam, and additional instruments may be located on the downstream slope. If possible, an additional instru-

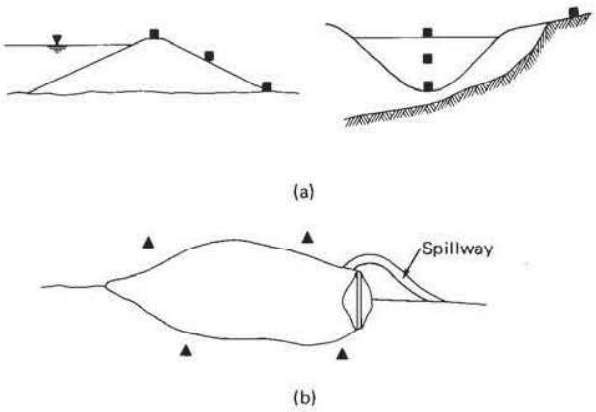

Figure 21.7. Recommended locations for (a) strong motion accelerographs and (b) microseismographs.

ment should be placed on a rock outcrop as close as possible to the dam to record bedrock motion. A recommended layout is shown in Figure 21.7a. Installations should be made as soon as possible after embankment construction.

Details of instrument selection, installation, and data evaluation are given by Bolt and Hudson (1975) and the Corps of Engineers (1981). Six important details are summarized below. First, because mains power will often fail during the seismic event being recorded, all instruments must be battery operated. Second, lightning protection must be provided. Third, it is important to ensure that any structures constructed to house the instruments do not influence dynamic measurements. Fourth, time history records of induced ground motions must be available at the site immediately after the earthquake, and little useful information results if the seismic record must be sent to the instrument manufacturer to be read. Fifth, regular maintenance of the instruments is vital. Sixth, all instruments and transmission devices must be secured against damage from earthquake shaking.

Reservoir Seismicity Networks

Seismicity networks are used to measure local small earthquakes in the area of the project site. Measured data provide information on the frequency of local earthquakes, the location and depth of seismic activity, and the magnitude and mechanisms of ground shaking.

Instruments for reservoir seismicity networks are normally called *microseismographs*. They should be located around the reservoir rim at intervals of from 3 to 20 miles (5–30 km), as shown in Figure 21.7b. Wherever possible, they should be located on rock in shallow pits away from noise from quarries, streams, and spillways. Microseismographs should be installed as soon as the project site has been selected, using the advice of a seismologist familiar with the seismic characteristics of the region.

21.3.4. Monitoring of Pore Water Pressure

Instruments for measurement of pore water pressure are described and evaluated in Chapter 9.

There is general agreement that open standpipe piezometers are the most reliable instruments for measurement of pore water pressure. When used for monitoring long-term performance of an embankment dam, they can often be installed in the foundation under the downstream slope, and at the downstream toe, after completion of the dam. They are generally unsuitable for installation during construction, because they interfere with fill placement. If open standpipe piezometers are not acceptable, the choice for long-term monitoring is among twin-tube hydraulic, pneumatic, and vibrating wire piezometers.

21.3.5. Diversity of Views on Selection of Remote-Reading Piezometer Type

During recent years several engineers have expressed their views in the technical literature on the advantages and limitations of twin-tube hydraulic, pneumatic, and vibrating wire piezometers for long-term monitoring of pore water pressure in embankment dams. There is no clear unanimity of view. The key points in each view are summarized in this section and followed by the author's suggestions for good practice.

Sherard (1981) generally favors the use of vibrating wire piezometers. He argues that although twin-tube hydraulic piezometers are good instruments, diaphragm piezometers are easier to use and require less skill and care in their maintenance. Sherard points out that some models of pneumatic piezometer have not given uniformly good service. When diaphragm piezometers are used with saturated high air entry filters and saturated interior cavity to measure pore water pressure, the vibrating wire piezometer has important advantages over the pneumatic piezometer because the initial saturation and air tightness of the piezometer can be tested before installation. The test is made by removing

the assembled and saturated piezometer from its container of water and taking periodic readings while evaporation of the water occurs in the pores of the filter. Sherard concludes that, although the vibrating wire piezometer and the better types of pneumatic piezometer are satisfactory instruments, the vibrating wire piezometer has substantial advantages and will probably be used more widely in the future.

Green (1982), in his discussion of Sherard's paper, presents some points in favor of twin-tube hydraulic piezometers and favors pneumatic over vibrating wire versions. Green points out that the twin-tube hydraulic piezometer is the simplest, but the pneumatic piezometer is also mechanically simple and can be made entirely of corrosion-resistant plastics. In contrast, the vibrating wire piezometer is a complicated electromechanical device in which the calibrated part of the measuring system is buried and is inherently susceptible to corrosion, zero drift, calibration slope changes, and grounding problems owing to water ingress to the electrical system via the buried leads. Green argues that, although these problems can be minimized by good design, defects are all too often present in any instrument owing to the manufacturer's lack of awareness, inadequately detailed design, or cost-cutting production methods forced on manufacturers by the user's low-bid procurement procedures. Green states that the order of increasing complexity is: twin-tube hydraulic, pneumatic, vibrating wire. Provided a pneumatic piezometer is designed to operate at subatmospheric pressure, its initial saturation and air tightness can be checked before installation as described by Sherard. Green concludes that the simpler pneumatic piezometer appears to have much more to offer the geotechnical profession and the dam designer than does the vibrating wire piezometer.

Mikkelsen (1982), in his discussion of Sherard's paper, also favors pneumatic over vibrating wire piezometers, citing many factors similar to those given by Green. Mikkelsen indicates that if leads over 1000 m (3300 ft) are necessary, pneumatic piezometers start to become impracticable, whereas cables over 4000 m (13,000 ft) long have been used on vibrating wire piezometers with success. Pneumatic piezometers are not subject to damage by lightning strikes, can be read automatically using high-precision electronic pressure gages and microprocessor controlled pneumatic flow systems, and are less expensive than vibrating wire piezometers. Mikkelsen also states that because the pneumatic piezometer has a simple valve that is either open or closed, there is little or no concern for stability of calibration. It is nearly an ideal gage in that when the gage functions, the reading will tend to be correct.

Sherard (1982) responds to the discussions by Green and Mikkelsen. He states that many years of experience have shown that currently available vibrating wire piezometers have not had significant problems because of zero drift, corrosion, calibration, slope changes, and water leakage. This generally good performance record has convinced a large fraction of engineers designing dams that the vibrating wire piezometer is a basically good instrument.

Londe (1982) states that vibrating wire instruments are more reliable and more durable than mechanical or hydraulic instruments. In Londe's view, vibrating wire instruments are not subject to rust, leaks, or wear and, whenever long-term readings are required, they are preferable to other types. They have the merit of accuracy, extreme reliability, and ability to transmit readings without alteration over long distances.

DiBiagio and Myrvoll (1985) describe a very robust vibrating wire piezometer, shown in Figures 9.33 and 9.34, that has been designed specifically for embankment dams and report on good performance.

Dunnicliff (1981) gives recommendations for selection among twin-tube hydraulic, pneumatic, and vibrating wire piezometers and contends that twin-tube hydraulic piezometers have the best chance of longevity. Houlsby (1982) endorses this view. Suggestions by both are included in the recommendations given in the following section. Kleiner and Logani (1982), in their discussion of Dunnicliff's paper, recommend use of open standpipe piezometers wherever possible and indicate that both pneumatic and vibrating wire piezometers can provide reliable data over long periods of time, but that a substantial number of instruments can fail.

Penman (1986) favors hydraulic piezometers. He believes that the primary application for vibrating wire piezometers is to provide rapid response and it seems unreasonable to expect the thin diaphragm of a pneumatic piezometer to last for the life of a dam. Penman comments that the de-airing process seems to have given twin-tube hydraulic piezometers a bad name. In the early days with polyethylene (polythene) tubing, air passed through the walls, but with polyethylene coated nylon tubing and Bishop

piezometer tips, de-airing needs are infrequent. With the recent development of automatic de-airing, de-airing is no longer an arduous procedure. Twin-tube hydraulic piezometer tubing will accommodate much more stretch than electrical cable.

21.3.6. Recommended Piezometer Types for Long-Term Monitoring

The author's suggestions for good practice are given in the following subsections.

Open Standpipe Piezometers

Open standpipe piezometers are the most reliable instruments for measurement of pore water pressure. In many countries they are commonly used for monitoring seepage pressures in the foundation under the downstream slope and at the downstream toe. Whenever possible, piezometers under the downstream toe should be installed in boreholes after completion of the dam so that there is no interference to fill placement.

When open standpipe piezometers are used, they should have low air entry filters. Standpipes should be of thick wall PVC or ABS plastic pipe, rather than steel pipe or plastic tubing. Standpipe inside diameter should be adequate for self-de-airing (minimum of 0.3 in., 8 mm) and, unless considerations of response time dictate otherwise, inside diameters of 0.75 or 1.0 in. (19 or 25 mm) are preferred so that strength is adequate and insertion of the reading device presents no problems. Standpipes should always be protected by an outer telescoping pipe to avoid ruptures of the types shown in Figure 17.10. When it is necessary to prevent leakage of water along the annulus between the pipes, the annulus should be filled with a slurry.

Friction problems and breakage of electrical dipmeter cables have been common features in very deep installations. When a standpipe is deeper than about 300 ft (90 m), the deflections of the standpipe owing to its own weight, within the protective pipe, can cause the dipmeter to be trapped by friction so that the effort required to extract it causes cable breakage (Wilson, 1982). This difficulty can be overcome by reading with the purge bubble device, leaving a small-diameter plastic tube within each standpipe. Because the tube cannot be pushed down a long standpipe after installation, it must be inserted as the standpipe is assembled.

If access to the top of the standpipes is difficult in the long term, readings can be made from a remote location by using one of the methods described in Chapter 9.

If open standpipe piezometers are installed in boreholes after completion of the dam, conformance between the instrument and fill should be ensured by selection of appropriate standpipes and backfill materials, and the potential for hydraulic fracturing should be recognized. The concern over hydraulic fracturing caused by drilling fluid is debatable. Sherard (1986) indicates that hydraulic fracturing probably occurs to some degree in most embankment dams, and usually no erosion is caused. It appears that the only certain method of preventing hydraulic fracturing during drilling is by using augers, either to advance the borehole directly or to clean soil from within driven casing. Hydraulic fracturing while grouting the borehole above a piezometer can be avoided by stage grouting.

Twin-Tube Hydraulic, Pneumatic, and Vibrating Wire Piezometers

If open standpipe piezometers are not acceptable, the choice is among twin-tube hydraulic, pneumatic, and vibrating wire piezometers. Based on his own experience and the diverse views summarized above, the author puts forward the following suggestions. There appear to be three primary questions that need to be answered when selecting among the three types of piezometer for long-term measurements. These are discussed in the following subsections.

First Question

Can I accept not knowing whether the measured pressure corresponds to the pore water pressure, pore gas pressure, or something in between?

As indicated in Section 9.13, the compacted fill in an embankment dam may remain unsaturated for a prolonged period after the reservoir is filled, and in fact the fill may never become permanently saturated by reservoir water. Increase of water pressure causes air to go into solution, and the air is then removed only when there is enough flow through the fill to bring in a supply of less saturated water. The pressure and time required to obtain saturation depend on the soil type, degree of compaction, and degree of initial saturation. Pore gas pressure may therefore remain significantly higher than pore water pressure for a substantial length of time, perhaps permanently. Despite use of saturated high air entry

filters, they may not remain saturated during this time because gas may enter the filters by diffusion, and flushing of the filter and cavity may be necessary to ensure that pore water pressure continues to be measured. Twin-tube hydraulic piezometers allow for flushing with de-aired liquid, but this is not possible with conventional diaphragm piezometers. Although it is possible to add flushing tubes to diaphragm piezometers, their addition dictates that the instruments must be installed, terminated, operated, and maintained as required for twin-tube hydraulic piezometers.

If the answer to this first question is **no**, twin-tube hydraulic piezometers are required. If the answer is **yes**, all three types are suitable.

Second Question

To what extent are skilled personnel available to maintain instruments during the operating life of the dam?

Maintenance of all types of piezometer requires regular checking of all components in the terminal enclosure. Twin-tube hydraulic piezometers may also require occasional flushing of the system with de-aired liquid as described in Appendix E. Some users will not consider using a twin-tube hydraulic piezometer system because they claim that the maintenance burden is unacceptable. However, the author believes that the maintenance burden has been exaggerated by improper selection of components, and flushing is seldom necessary in a properly designed and installed system, except when subatmospheric pore water pressures are being read. A properly designed system entails attention to the many details discussed in Appendix E.

Maintenance of pneumatic and vibrating wire piezometers also requires regular checking and calibration of readout units, which can if necessary be done at an appropriate facility away from the site. The need for skilled maintenance is therefore applicable to all piezometers, but the availability of skilled personnel (local, visiting on a regular basis, and/or at off-site facility) is likely to impact on the selection among the three types.

Third Question

Do connecting tubes or cables need to be longer than about 2000 ft (600 m)?

If **yes**, twin-tube hydraulic and pneumatic piezometer readings are sluggish and questionable. The length of tubing can sometimes be minimized by constructing terminal houses within drainage or inspection galleries or by use of almost vertical runs of tubing in filters as discussed in Section 21.4. Penman (1986) indicates that the sluggishness of twin-tube hydraulic piezometers ceases to be a problem if an automatic reading and de-airing system is used. The same argument can be advanced for automatically read pneumatic piezometers of the *normally closed* type. Thus, for very long connecting tubes or cables, the options are vibrating wire piezometers or the remaining two types with automatic reading equipment.

Summary of Recommendations for Selection Among Twin-Tube Hydraulic, Pneumatic, and Vibrating Wire Piezometers

Answers to the above three questions may dictate selection among the three piezometer types for long-term measurements. The selection of filter type, shape, size, and saturation procedure should follow the guidelines given in Chapter 9.

Sherard (1981) states that the likely useful life of any of the three types of piezometer is more related to the conservatism of the design and construction details of the specific piezometer, connecting lines, and installation care, and to the conditions in which it is placed, than it is related to differences among the three types. The author totally agrees with this view.

The author also agrees strongly with Green (1982) that cost-cutting production methods, forced on the manufacturers by use of the user's low-bid procurement procedures, tend to reduce conservatism of the design and construction details. Perhaps this last observation helps to explain the diversity of views presented in the previous section: the good experiences with vibrating wire piezometers that have been reported by Londe, Sherard, and DiBiagio and Myrvoll are generally with Telemac and Geonor instruments in situations where lowest cost does not dominate the choice of instruments.

Although some manufactured versions have a better success record than others, there appears to be no certain and intrinsic longevity difference between pneumatic and vibrating wire piezometers. The inaccessible parts of both are self-evidently more complex than the twin-tube hydraulic piezometer.

General requirements for maximizing reliability of performance monitoring are given in Chapter 15, including a listing of three major requirements re-

lating to the design of the instruments themselves: simplicity, self-verification, and durability in the installed environment. In the view of the author, these three requirements are satisfied by twin-tube hydraulic piezometers to a greater extent than by the other two types. Open standpipe piezometers installed in the foundation can be converted to remotely read twin-tube hydraulic piezometers by using the arrangement shown in Figure 9.10. **If engineers are truly committed to long-term readings that are as verifiable as possible, twin-tube hydraulic piezometers should be their first choice.**

Those who disagree with this preference for twin-tube hydraulic piezometers and favor use of pneumatic or vibrating wire piezometers for **long-term** measurements are encouraged to make a close study of recommendations by DiBiagio and Myrvoll (1985), Green (1982), Mikkelsen (1982), and Sherard (1981, 1982). They are also encouraged to consider very seriously the implications of low-bid procurement procedures and to favor heavy-duty instruments that are designed for long-term survivability, for example, the Geonor vibrating wire piezometer shown in Figure 9.34 or the heavy-duty version manufactured by Geokon. They should review published opinions on the likelihood of zero drift of vibrating wire piezometers: reference citations are given in Chapter 8. Chapter 8 also reports on two unpublished case histories, involving measurements with vibrating wire piezometers supplied by two different manufacturers and installed in two embankment dams. During first filling of each reservoir, one piezometer indicated a high piezometric level that caused concern, and filling was stopped. The piezometer reading continued to rise and, when the indicated piezometric level rose above pool level, the measurements were discounted and filling continued. These cases of a *rogue instrument* reinforce the author's preference for twin-tube hydraulic piezometers. They also emphasize the view expressed by LeFrancois (1986) (Section 8.4.9) that evaluations of data should not be made on the basis of readings from a single instrument.

There is a need for comparative evaluations on long-term performance of the three types of remote-reading piezometer, with care to avoid incorrect general conclusions because of commercial bias, unusual project conditions, improper installation, improper selection of components, or improper operation. Engineers who are in a position to pursue such practical comparisons are strongly encouraged to do so and to report their findings to the profession.

21.3.7. Recommended Instruments for Monitoring Long-Term Settlement

Measuring points on the crest and downstream face should be the first choice for long-term measurements of surface settlement. Measurements are normally made by optical or trigonometric leveling, but the satellite-based Navstar Global Positioning System is likely to be used with increasing frequency in the future.

If subsurface settlement measurements are required, the choice is among liquid level gages, horizontal inclinometers, and probe extensometers, and selection depends on the specific needs and details of each project. Liquid level gages and horizontal inclinometers are greatly preferable to vertically installed probe extensometers, because they cause much less interference to construction. Of the many available types of liquid level gage, the most suitable choices are the following:

1. Single-point overflow gages with duplicate weirs.
2. Full-profile overflow gages, often combined with probe extensometers installed horizontally as shown in Figure 21.2.
3. Double fluid settlement gages.

Details of all instrument types are given in Chapter 12.

A reliable benchmark is essential, beyond the influence of the dam and fluctuating reservoir. When benchmarks are located close to the dam or on steep slopes of native overburden soil, their uncertain reliability can invalidate the monitoring objective.

21.3.8. Recommended Instruments for Monitoring Long-Term Lateral Deformation

If lateral deformation measurements are required, measuring points on the crest and downstream face should be the first choice. Data are normally obtained by trilateration with electronic distance measuring equipment, but triangulation is also used and the satellite-based Navstar Global Positioning System is likely to be used more widely in the future. Stable horizontal control stations are critical and should be located as described in Section 21.3.7 for benchmarks.

If subsurface measurements of lateral deformation are required, the choice is among probe exten-

someters, fixed embankment extensometers, and inclinometers, and selection depends on the specific needs and details of each project. For measurements in the downstream shell, the best choice is usually a probe extensometer with a current-displacement induction coil transducer (sometimes called a *horizontal plate gage*) installed horizontally as shown in Figure 21.2. Precision is maximized by using the version with a primary induction coil near each steel plate and with interconnecting rods. Fixed embankment extensometers with electrical linear displacement transducers can be used to determine the pattern of horizontal strain in the core or elsewhere. Preferred transducers for long-term measurements are vibrating wire transducers or induction coil transducers with frequency output. Inclinometers can be installed to monitor lateral deformation throughout the length of a vertical casing but, unless the casing is extended upward within a filter as described in Section 21.4, this practice is discouraged. As discussed in Section 12.8.2, accuracy of inclinometer data in inclined casings is poor (e.g., casings within a filter alongside a sloping core).

Details of available instruments are given in Chapter 12.

21.3.9. Additional Approaches to Long-Term Monitoring

Automatic Monitoring

Recent developments in automatic data acquisition systems (ADASs), power supplies, and communication systems are having a major impact on long-term monitoring of embankment dams. These developments are described in Chapter 8, including a discussion of the applicability of ADASs for monitoring various transducers. Advantages and limitations of ADASs and automatic data processing and presentation systems are discussed in Chapter 18. ICOLD (1982) provides guidelines and examples for automatic monitoring, and the USCOLD Committee on Measurements is currently preparing a summary of U.S. experience (USCOLD, 1988).

One caution needs to be given. Adoption of an automated monitoring program is not usually a justifiable reason for selecting inaccessible electrical transducers. For example, the decision to read automatically should not in itself cause vibrating wire piezometers to be selected in favor of open standpipe or twin-tube hydraulic piezometers, because these last two types can also be read automatically by use of accessible hydraulic/electrical transducers. The selection of an inaccessible transducer should be based on inherent longevity criteria, not on the fact that it provides an electrical signal at the source.

Streaming Potential

Streaming potential (also called *self-potential*) appears to be a promising new method for long-term detection of seepage paths (Bartholomew et al., 1987; USCOLD, 1986). When water is forced to flow under laminar conditions through porous earth materials, it generates an electrical current. The current is monitored by embedding electrodes within the dam. However, there appears to be little practical experience to date.

Acoustic Emission Monitoring

Acoustic emission monitoring is being considered for detection of seepage characteristics (Chapter 12) but, as for streaming potential, there appears to be little practical experience to date.

Thermal Monitoring

Attempts have been made to monitor leakage through embankment dams by thermal monitoring, also called *thermotic surveys* (Bartholomew et al., 1987; Birman et al., 1971; USCOLD, 1986). The principle is based on embedding thermistors in the fill, drawing isotherms, and deducing flow patterns. The key to success of this approach appears to rest on having a significant temperature difference between the water in the reservoir and in the embankment fill.

21.3.10. Guidelines for Operating and Maintenance Personnel

Most major public agencies that are responsible for long-term performance of embankment dams have guidelines for emergency preparedness planning. These guidelines generally include regular visual inspections and the use of instrumentation for safety evaluation. For example, the U.S. Army Corps of Engineers has prepared several *Engineering Regulations* that are listed by the Corps of Engineers (1984). The Corps of Engineers (1985) provides additional guidelines. The U.S. Bureau of Reclamation has a similar list (Misterek, 1986), and the Australian National Committee on Large Dams has prepared guidelines (ANCOLD, 1983).

Dunnicliff (1981) attempts to answer the question:

> When instrumentation is installed on and in an embankment dam for the purpose of monitoring long-term performance, what guidelines should be given to operating and maintenance (O&M) personnel to assist them in judging satisfactory performance based on the measurement data?

The suggested guidelines relate to measured pore water pressures, settlements, lateral deformations, and seepage flows and to regular visual observations of the condition of exposed parts of the embankment and spillway.

An operating and maintenance manual should be prepared for the guidance of O&M personnel at all dams. If instrumentation has been installed for long-term performance monitoring, the manual should include guidelines on regular calibration and maintenance procedures (Chapter 16) and on procedures for collection, processing, presentation, interpretation, and reporting of instrumentation data (Chapter 18). The instrument installation report (Chapter 17) should be made available to O&M personnel.

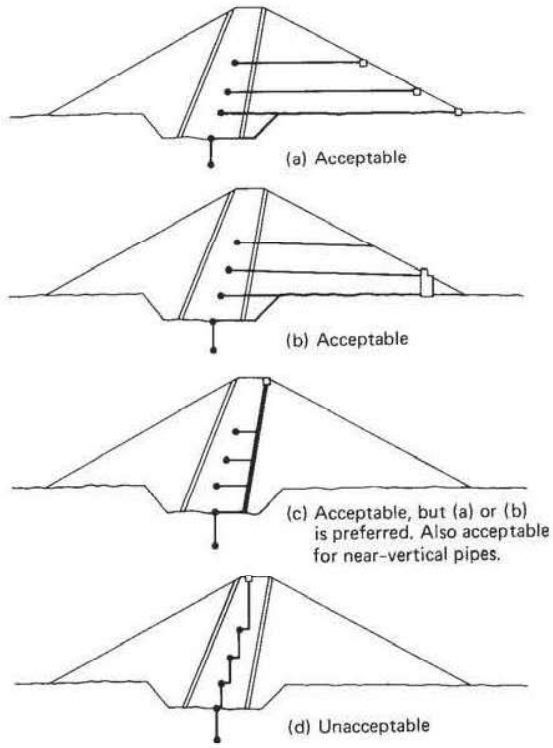

Figure 21.8. Examples of acceptable and unacceptable routings for instrumentation tubes and cables.

21.4. GENERAL GUIDELINES ON THE EXECUTION OF MONITORING PROGRAMS FOR EMBANKMENT DAMS

General guidelines on the execution of monitoring programs are given in Part 4 of this book. The guidelines include instrument calibration, maintenance and installation, and collection, processing, presentation, interpretation, and reporting of instrumentation data. This section provides general guidelines specific to execution of monitoring programs for embankment dams. Most of the guidelines relate to installation of instruments.

21.4.1. Installation of Pipes, Tubes, and Cables

Methods of installing pipes, tubes, and cables are described in Chapter 17. The following subsections elaborate on some important issues for embankment dams.

Comparison Between Vertical and Horizontal Runs

In embankment dams, vertical and near-vertical runs of pipes, tubes, and cables should be avoided wherever possible. There is one exception to this rule: installations can be extended upward through a filter as discussed later in this section. If installed other than in a filter, they impede the quality of compaction, and may cause significant problems with the performance of the dam.

It is preferable, wherever possible, to select instruments that do not require vertical runs. For example remote-reading piezometers can often be selected in preference to open standpipe piezometers. Liquid level gages can often be selected in preference to vertically installed probe extensometers, and horizontally installed probe extensometers can often be selected in preference to inclinometers. Pipes, tubes, and cables for these preferable instruments should be routed along the fill at the slope adopted for general placement purposes and terminated in an enclosure buried in the fill near the downstream face. Figure 21.2 shows an acceptable routing for horizontal pipes. Figure 21.8 shows acceptable and unacceptable routings for tubes and cables.

The arrangement shown in Figure 21.8d has been

used in several dams. Tubes or cables have been bundled together, and the vertical parts of the step have been protected by enclosing them within a temporary protective steel pipe, often about 3 ft (1 m) in diameter, jacked up vertically as filling proceeds. There is ample evidence that this method causes inadequate compaction around the bundle, damage to instrument leads, and a potential water pathway. Adverse behavior of major dams has been caused as a result of this practice (e.g., Harder, 1982; Mikkelsen and Wilson, 1983), but it is still considered acceptable by many engineers. The practice is **strongly** discouraged. However, if the pipes, tubes, or cables are extended upward through a filter as shown in Figure 21.8c, the result is much more satisfactory. Electrical cables were carried up the downstream filter of Srinagarind Dam in Thailand (Champa and Mahatharadol, 1982) and pneumatic tubes in the downstream filter of Wadaslintang Dam in Indonesia, using the jacked-up protective steel pipe arrangement described above. Filter material was added inside the pipe as it was jacked up, keeping the internal level close to the bottom end of the pipe to avoid creating tension in the leads as the pipe was jacked up.

For both horizontal and vertical runs, the engineering properties of fill material around pipes, tubes, and cables should match, as closely as possible, the properties of the surrounding fill. When installed in "impervious" fill, the installations should not provide a leakage zone for water.

Installation of Vertical Runs

When it is necessary to extend vertical or near-vertical runs of pipes, tubes, or cables upward through the fill, they should be restricted to installation within a filter as described above. Either the jacked-up protective steel pipe procedure, or alternatively, the *mounding* method or the *excavate and replace* method can be used, as described in Chapter 17.

Installation of Horizontal Runs

Mikkelsen (1986b) describes an innovative method for installing a horizontal pipe in rockfill during construction of a concrete face rockfill dam. The pipe was plastic inclinometer casing, for use with a horizontal inclinometer, and was installed on 6 in. (13 mm) of compacted sand as shown in Figure 21.9. Mikkelsen reports on several advantages of this method of installation when compared with a

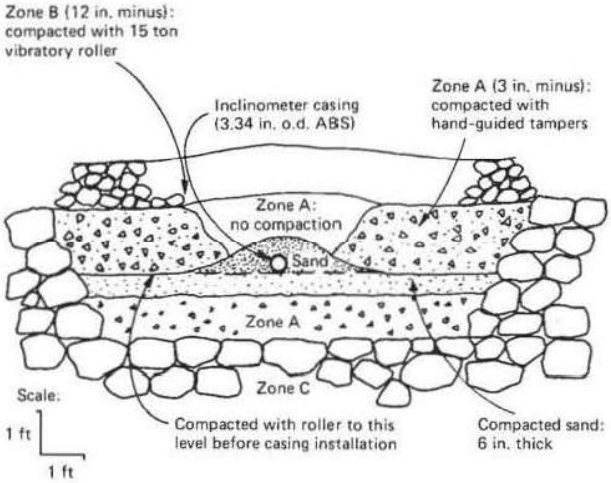

Figure 21.9. Installation of horizontal inclinometer casing in concrete face rockfill dam (after Mikkelsen, 1986b).

trenched-in method: maximized use of fill production and compaction equipment; no disturbance owing to trenching; improved initial grade and line; much faster installation; better access for compactors; less potential for casing damage; and better compaction.

Mikkelsen and Wilson (1983) maintain that when trenches are excavated for instrumentation tubes or cables, they should not extend in the upstream–downstream direction through more than one-half, or preferably not more than one-third, of the width of the core. It is worthwhile to consider three likely reasons for this prohibition.

First, some engineers are concerned that instrument leads may be broken where they cross zones of different compressibility. This issue is discussed in the following subsection and appears not to be a major concern. Second, leakage can develop along the route if instrument leads deteriorate and leave an open hole. This issue can be minimized by selection of the types of tubes and cables that are recommended in Chapter 8 and by appropriate design of the downstream filter. Third, leakage can develop along the route if fill around the leads is poorly placed, such that it does not develop a total pressure greater than the pressure from reservoir water at that elevation. If great care is taken with fill placement, there should be no potential for leakage. The guidelines given in Chapter 17 should be followed, using the trenching and backfilling method. In soft clay cores, excavated material should be recompacted into the trench at placement water content and with at least as much compactive effort as

used for the main fill. In more sandy materials it may be useful to add some bentonite to the excavated material to improve its plasticity and reduce its permeability. The "bentonite cutoffs" described by USBR (1974) are not necessary. Alternatively, the *surface installation method* described by Clements (1982) and in Chapter 17 can be used, provided that there is good supervision of general fill placement.

If a leak occurs through the core, of a type that can be imagined as the most extreme result of the actions of the above second and third points, the leak will be controlled and sealed safely by a good downstream filter (Sherard, 1985). For most dams it is necessary for the designer to assume that a small concentrated leak may develop through the core from other causes, and Sherard et al. (1984) give criteria for the design of appropriate filters. Others argue that leaks should be prevented to the extent possible, regardless of the filter design provided, because no filter design accounts for placement errors during construction.

In summary, if instrumentation of an embankment dam can be achieved without running tubes and cables through the core, this is the preferred method. However, if instruments are considered essential and if there are substantial economic advantages to running leads completely through a core, the practice is acceptable provided that great care is taken while placing surrounding fill material.

Installation of Tubes and Cables Across Zones of Different Compressibility.

The question of running piezometer tubes and cables through the boundaries between embankment zones is widely debated, particularly through the boundary between the downstream face of a central core and the downstream filter. Dogmatic guidelines cannot be given, but it appears to the author that the practice is acceptable.

Shear movements have been measured across zones of different compressibility in several dams, for example, between a vertical core and downstream filter, using double fluid settlement gages. Wilson (1982) reports on such measurements in Angostura Dam, a 500 ft (150 m) high rockfill dam with a central vertical clay core, showing that shear movements were not abrupt. This and other experiences convinced Wilson that, in general, it is satisfactory to route tubes and cables through the boundary between a vertical core and downstream filter, provided that they can accommodate minor tensile and compressive strains. Penman (1986) shares this view and comments that plastic tubing of up to 0.25 in. (6 mm) diameter survives the deformations. Suitable tubing and cable is recommended in Chapter 8. When using electrical cables, the water-blocked type should be selected to avoid water passing between the jacket and conductors. Because most electrical cables have limited ability to tolerate axial strain, it may be worthwhile to reduce concentrations of strain by threading individual cables within minimum-sized plastic tubing where they pass through zones that will experience significant strain and to fill the annular space with petroleum jelly. The tubing should extend at least 10 ft (3 m) on either side of the interface to help distribute the strain.

If designers predict more than normal differential movement between boundaries, an alternative routing will be more conservative. For example, it seems reasonable to expect a greater tendency for the face of an upstream sloping core to "slip" with respect to the first downstream filter layer than is the case for a vertical core, and more tendency near the top of a dam than near the bottom. For high dams with steep rock abutments, the core may slip abruptly downward with respect to the abutments as the core settles, and leads passing through the interface may be sheared. However, it is not usual to route instrument leads into abutments unless drainage galleries are being used for terminal arrangements. At New Don Pedro Dam (Taylor, 1968), where substantial differential movements between zones were anticipated, all tubes and cables were carried parallel to the axis of the dam, entirely within each zone either to the abutments or to a central backfilled shaft within the core. Today, a better practice would be to extend the leads upward in a filter, as described earlier.

21.4.2. Completion of Terminal Enclosure

On some projects, instruments are installed and leads are left for a significant time as coils on the fill. Many instruments, such as twin-tube hydraulic piezometers and liquid level settlement gages, cannot be read until terminal arrangements are completed, and if coils are left on the fill no initial readings are taken and early data are not available. Without initial readings, no calculations can be made of total changes. To avoid this shortcoming, terminal enclosures must be completed in a timely

manner so that initial instrument readings can be made immediately after instruments are installed. Other instruments, such as vibrating wire and pneumatic piezometers, can be read while leads are coiled, and this issue does not arise.

21.4.3. Interpretation of Settlement Measurements in the Core

When measurements of settlement are made within the central core of an embankment dam, the data are often used to compute the modulus of compression. However, the engineer rarely knows whether the "compression" measured between two points is vertical compression (decrease in volume) or if it may primarily be due to shear deformation at constant volume as the sides of the core spread outward. In any given case both actions occur. If the shear deformation is ignored, interpretations of data showing that the central core of a rockfill dam is more compressible than the shells may be incorrect. When intending to use settlement data for this purpose, one must include additional measurements to resolve the uncertainty. The extent of horizontal spreading of the downstream face of the core can be measured by using the arrangement shown in Figure 21.2. Double fluid settlement gages can be used to measure settlement in both the core and shell.

21.4.4. Data Collection Frequency

Chapter 18 gives general guidelines for the frequency of data collection, and suggested schedules for embankment dams are given by Bartholomew et al. (1987) and USCOLD (1986).

When instruments have been installed in an embankment dam to monitor performance during construction, first filling, or drawdown, a decision must be made on whether to continue readings in the long term. The decision should be based on long-term needs, **not** on the fact that instruments are **there.** The need for monitoring a typical dam, after the reservoir has been filled for 2 or 3 years, decreases rapidly with each passing year. In particular, during the first few years the settlements and internal strains usually come to equilibrium, after which there is little interest in continued monitoring of internal deformation.

When long-term instruments have been read intensively for a period after the dam is in operation, and when long-term reading costs must be reduced, data collection frequency will be reviewed. A good approach is to select key instruments, based either on their importance or on previous behavior, and to continue to read these at an unchanged frequency. Others will then be read at a greatly reduced frequency. For instruments requiring multiple readings for a single data set, such as probe extensometers or inclinometers, frequent readings can be limited to the zone of greatest importance or of greatest previous change.

21.5. SELECTED CASE HISTORIES

Table 21.2 summarizes selected case histories of instrumented embankment dams. Several of the case histories are selected from the *Transactions of the Fifteenth International Congress on Large Dams* (ICOLD), held in Lausanne in 1985: there are many more good case histories in Volume 1 of these *Transactions*. A summary is given by Post (1985) and a brief overview by Water Power & Dam Construction (1985). Post also lists other recent occasions when monitoring of dams and their foundations has been the subject of earlier ICOLD Congresses.

Table 21.2. Summary of Selected Case Histories of Embankment Dams

Project	Principal Concerns	Type of Dam	Measurement[a]	Instrumentation Discussed[b]	Special Features	Reference
Fifteen dams in Canada	Post-construction deformations	Various rockfill dams, 30–500 ft (10–170 m) high	D	Surveying methods	Summary of post-construction settlement and horizontal deformation data	Dascal (1987)
Shiroro Dam, Nigeria	Performance during construction and operation	410 ft (125 m) high concrete face rockfill	LE	Leakage weirs	Inclined inclinometer casings installed below concrete face.	Bodtman and Wyatt (1985)
			G	Pneumatic and vibrating wire piezometers		
			D	Surveying methods		
			D	Probe extensometers (H)		
			D	Full-profile liquid level gages		
			D	Crack gages		
			D	Inclinometers (I,V)		
			T	Thermistors		
Various dams in Norway	Various	Various	LE	Leakage weirs	Summary of types and quantities of instruments in Norwegian dams	DiBiagio and Kjaernsli (1985)
			G	Piezometers		
			D	Probe extensometers (V)		
			D	Liquid level gages		
			D	Fixed embankment extensometers		
			D	Inclinometers (H,I,V)		
			TS	Embedment earth pressure cells		
Salvajina Dam, Colombia	Performance during first filling and operation	485 ft (150 m) high concrete face gravel/rockfill	LE	Leakage weirs		Hacelas et al. (1985)
			G	Pneumatic piezometers		
			D	Surveying methods		
			D	Liquid level gages		
			D	Crack gages		
			S	Embedment strain gages		
			TS	Embedment earth pressure cells		
			SE	Strong motion accelerographs		
Monasavu Dam, Fiji	Performance during construction and operation	280 ft (85 m) high rockfill with central core	LE	Leakage weirs	Very wet residual clay core	Knight et al. (1985)
			G	Twin-tube hydraulic piezometers		
			D	Surveying methods		

Table 21.2. Summary of Selected Case Histories of Embankment Dams (*Continued*)

Project	Principal Concerns	Type of Dam	Measurement[a]	Instrumentation Discussed[b]	Special Features	Reference
			D	Liquid level gages		
			D	Fixed embankment extensometers		
			TS	Embedment earth pressure cells		
			SE	Strong motion accelerographs		
Foz do Areia Dam, Brazil	Deformations during and after construction	525 ft (160 m) high concrete face rockfill	LE	Leakage weirs		Lagos Marques et al. (1985)
			D	Surveying methods		
			D	Probe extensometers (H)		
			D	Liquid level gages		
			D	Crack gages		
			S	Embedment strain gages		
Khao Laem Dam, Thailand	Performance during construction and operation	430 ft (130 m) high concrete face rockfill	LE	Leakage weirs	Inclinometer casings attached to concrete face	Mahasandana and Mahatharadol (1985)
			G	Observation wells		
			D	Surveying methods		
			D	Settlement platforms		
			D	Liquid level gages		
			D	Crack gages		
			D	Inclinometers (I)		
			S	Embedment strain gages		
			TS	Contact earth pressure cells		
Elandsjagt Dam, South Africa	Performance during construction and operation	250 ft (75 m) high earth and rockfill	LE	Leakage weirs	Very rapid first filling	Melvill (1985)
			G	Twin-tube hydraulic and vibrating wire piezometers		
			D	Probe extensometers (H,V)		
			D	Liquid level gages		
			TS	Contact and embedment earth pressure cells		
Meggett Dam, Scotland	Performance during construction and operation	180 ft (55 m) high dam with central asphaltic core and gravel shells	D	Surveying methods	Comparison between observed and predicted deformations	Penman and Charles (1985)
			D	Probe extensometers (H)		
			D	Liquid level gages		

Table 21.2. Summary of Selected Case Histories of Embankment Dams (*Continued*)

Project	Principal Concerns	Type of Dam	Measurement[a]	Instrumentation Discussed[b]	Special Features	Reference
Nurek Dam, USSR	Performance during construction and operation	980 ft (300 m) high zoned earthfill	G	Piezometers	Height of dam (world's highest)	Sokolov et al. (1985)
			D	Fixed embankment extensometers		
			D	Inclinometers		
			TS	Contact and embedment earth pressure cells		
W. A. C. Bennett Dam (Portage Mountain Dam), BC, Canada	Performance during construction and operation	600 ft (185 m) high zoned earthfill	LE	Leakage weirs	16 years of performance records	Taylor et al. (1985) Taylor and Chow (1976)
			G	Open standpipe, twin-tube hydraulic, pneumatic, and vibrating wire piezometers		
			D	Surveying methods		
			D	Probe extensometers (V)		
			D	Inclinometers (V)		
			S	Embedment strain gages		
			TS	Embedment earth pressure cells		
Bath County Hydroelectric Pumped-Storage Project, Virginia, U.S.A.	Performance during construction and operation	460 ft (140 m) and 150 ft (45 m) high zoned earth and rockfill	G	Pneumatic piezometers	Dual piezometers	Kleiner (1983)
			G	Dual pneumatic and vibrating wire piezometers		
Svartevann Dam, Norway	Performance during construction and operation: height of dam substantially exceeded other dams in Norway	425 ft (130 m) high rockfill with central earth core	LE	Leakage weirs		DiBiagio et al. (1982)
			G	Vibrating wire piezometers		
			D	Surveying methods		
			D	Probe extensometers (I,H)		
			D	Fixed embankment extensometers		
			D	Inclinometers (I,H)		
			TS	Embedment earth pressure cells		
Cethana Dam, Tasmania, Australia	Performance during construction and operation	360 ft (110 m) high concrete face rockfill	LE	Leakage weirs	Inclinometer casings attached to concrete face	Fitzpatrick et al. (1982)
			D	Surveying methods		
			D	Liquid level gages		
			D	Crack gages		

Table 21.2. Summary of Selected Case Histories of Embankment Dams (*Continued*)

Project	Principal Concerns	Type of Dam	Measurement[a]	Instrumentation Discussed[b]	Special Features	Reference
			D	Inclinometers (I)		
			S	Embedment strain gages		
Various dams in Britain	Long-term performance	Various earthfill dams	G	Twin-tube hydraulic piezometers	Emphasis on avoiding vertical pipes and tubes	Penman and Kennard (1981)
			D	Probe extensometers (H)		
			D	Liquid level gages		
			D	Fixed embankment extensometers		
			TS	Embedment earth pressure cells		
Various dams in western Canada	Various	Various earthfill dams	LE	Leakage weirs	Discussion of needs for limited and comprehensive monitoring programs	Peters and Long (1981)
			G	Piezometers		
			D	Surveying methods		
			D	Probe extensometers (V)		
			D	Inclinometers (V)		
El Infiernillo Dam, Mexico	Long-term performance	480 ft (145 m) high rockfill with central earth core	G	Open standpipe piezometers	18 years of performance records	Rosati and Esquivel (1981)
			D	Surveying methods		
			D	Probe extensometers (V)		
			D	Fixed embankment extensometers		
			D	Inclinometers (V)		
			SE	Strong motion accelerographs		
Tarbela Dam, Pakistan	Performance during construction and operation	460 ft (140 m) high zoned earth and rockfill	LE	Leakage weirs and flumes		Szalay and Marino (1981)
			G	Open standpipe, twin-tube hydraulic, pneumatic, and vibrating wire piezometers		
			D	Surveying methods		
			D	Single-point and full-profile liquid level gages		
			D	Fixed embankment extensometers		

Table 21.2. Summary of Selected Case Histories of Embankment Dams (*Continued*)

Project	Principal Concerns	Type of Dam	Measurement[a]	Instrumentation Discussed[b]	Special Features	Reference
			D	Tiltmeters		
			TS	Embedment earth pressure cells		
			T	Thermometers		
			SE	Strong motion accelerographs		
Mica Dam, BC, Canada	Cracking Control of seepage	800 ft (245 m) high zoned earthfill	LE	Leakage weirs	Height of dam	Khilnani and Webster (1976) Webster (1970)
			G	Observation wells		
			G	Piezometers		
			D	Surveying methods		
			D	Probe extensometers (V)		
			D	Fixed embankment extensometers		
			D	Inclinometers (V)		
			TS	Embedment earth pressure cells		
Ilha Solteira Project, Brazil	Performance during construction and operation	230 ft (70 m) high earth and rockfill	G	Open standpipe, pneumatic, and vibrating wire piezometers	Water loss, caves, and cracks resulting from inferior compaction around vertical pipes	Vargas and Hsu (1975)
			D	Probe extensometers (V)		
			D	Inclinometers (V)		
			TS	Embedment earth pressure cells		
Oroville Dam, California, U.S.A.	Performance during construction and operation Comparison between measured and predicted data	770 ft (235 m) high zoned earthfill	LE	Leakage weirs		O'Rourke (1974) Kulhawy and Duncan (1972)
			G	Twin-tube hydraulic piezometers		
			D	Probe extensometers (V)		
			D	Liquid level gages		
			D	Fixed embankment extensometers		
			S	Embedment strain gages		
			TS	Embedment and contact earth pressure cells		
			SE	Strong motion accelerographs		

Table 21.2. Summary of Selected Case Histories of Embankment Dams (*Continued*)

Project	Principal Concerns	Type of Dam	Measurement[a]	Instrumentation Discussed[b]	Special Features	Reference
Various	Various	Various earth and rockfill dams	D	Surveying methods		Wilson (1973)
			D	Probe extensometers (V)		
			D	Liquid level gages		
			D	Fixed embankment extensometers		
			D	Inclinometers (V)		
Briones Dam, California, U.S.A.	Performance during construction and operation	280 ft (85 m) high zoned earthfill	LE	Leakage weirs	Sloping clay core placed directly on upstream face	Anton and Dayton (1972)
			G	Open standpipe, twin-tube hydraulic, and pneumatic piezometers		
			D	Surveying methods		
			D	Probe extensometers (V)		
Galisteo Dam, New Mexico, U.S.A.	Large lateral deformations during construction	160 ft (50 m) high zoned earthfill	D	Surveying methods		Catanach and McDaniel (1972)
			D	Inclinometers (V)		
			D	Tiltmeters		
New Don Pedro Dam, California, U.S.A.	Performance during construction and operation	585 ft (180 m) high zoned earth and rockfill	G	Pneumatic piezometers	No pipes, tubes, or cables crossing interfaces between zones	Taylor (1968)
			D	Surveying methods		
			D	Probe extensometers (V)		
			D	Liquid level gages		
			D	Fixed embankment extensometers		
			D	Inclinometers (V)		
Mangla Dam, Pakistan	Performance during construction and operation	450 ft (135 m) high zoned earthfill	G	Open standpipe, twin-tube hydraulic, and vibrating wire piezometers		Binnie et al. (1967)
			D	Probe extensometers (V)		
			D	Inclinometers (V)		
			D	Plumb lines		
			D	Inverted pendulums		
			TS	Embedment earth pressure cells		

[a]D: deformation; G: groundwater pressure; LE: leakage; S: strain in structural members; SE: seismic event monitoring; T: temperature; TS: total stress in soil or at contact with structure.

[b]H: horizontal; I: inclined; V: vertical.

CHAPTER 22

EXCAVATED AND NATURAL SLOPES*

22.1. GENERAL ROLE OF INSTRUMENTATION

Analysis of slope stability is the principal geotechnical design task for temporary or permanent excavated slopes. Factors influencing stability include stratigraphy, groundwater levels, seepage gradients, strength of the soil or rock mass, geometry, and driving forces.

Stability of slopes in soil is controlled by the ratio between the available shearing resistance along a potential failure surface and the shear stress on the surface. Circular or wedge-shaped surfaces are often used in analyses that attempt to model actual conditions. Available strength includes cohesion and frictional components; for long-term considerations, the contribution of cohesion is often reduced significantly.

Stability of slopes in rock is usually controlled by the presence of discontinuities in the rock mass and the presence of water under pressure in these discontinuities. Failures most frequently occur as a result of sliding or separation along discontinuities.

Primary mechanisms that control the behavior of excavated slopes are outlined in Chapter 2. It is imperative that, prior to planning an instrumentation program for a slope in soil or rock, an engineer first develop one or more working hypotheses for a potential behavior mechanism. The hypotheses must be based on a comprehensive knowledge of the locations and properties of discontinuities.

*Written with the assistance of Robin B. Dill, Senior Engineer, and Douglas G. Gifford, Associate and Vice President, Haley & Aldrich, Inc., Cambridge, MA.

Figures 22.1 and 22.2 show typical excavated slopes in clay and in rock. The figures convey the contrast between how the project will usually appear on engineering drawings and what may actually happen in the field. A discussion of the role of instrumentation for slopes in soil is given by Wilson and Mikkelsen (1978) and for slopes in rock by Franklin and Denton (1973), Hoek and Bray (1981), and Patton (1983).

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

Instrumentation programs for natural slopes are essentially the same as for excavated slopes.

22.2. PRINCIPAL GEOTECHNICAL QUESTIONS

The following principal questions are presented in the normal order of occurrence for an excavated or natural slope. The order does not reflect a rating of importance.

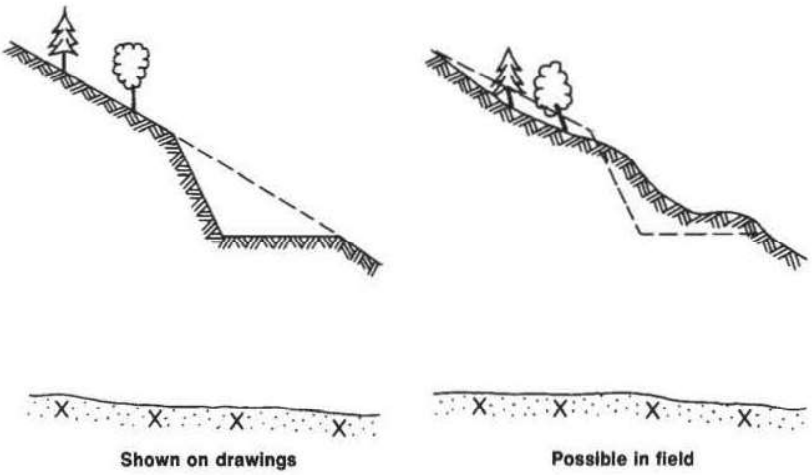

Figure 22.1. Excavated slope in clay (after DiBiagio and Myrvoll, 1981).

22.2.1. What Are the Initial Site Conditions?

Initial site conditions are determined by use of conventional site investigation procedures, sometimes supplemented by in situ testing. Special attention should be given to defining possible failure mechanisms. If potential failure zones are identified, the need for reinforcement or other methods of slope stabilization can then be addressed. For slopes in rock, comprehensive structural geologic mapping will indicate critical discontinuities. Particular attention should be given to persistent, adversely oriented joint sets, to possible low strength zones, and to continuous features such as faults and shears at the top of the slope that could allow release of potentially unstable blocks and wedges.

Performance monitoring instrumentation sometimes plays a role in defining initial site conditions. For example, groundwater pressures can have a large impact on the stability of slopes. Piezometers can be installed well before the start of excavation, to define the preconstruction groundwater pressure regime, including any perched or artesian water. If piezometers are installed sufficiently early, seasonal variations can be defined.

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

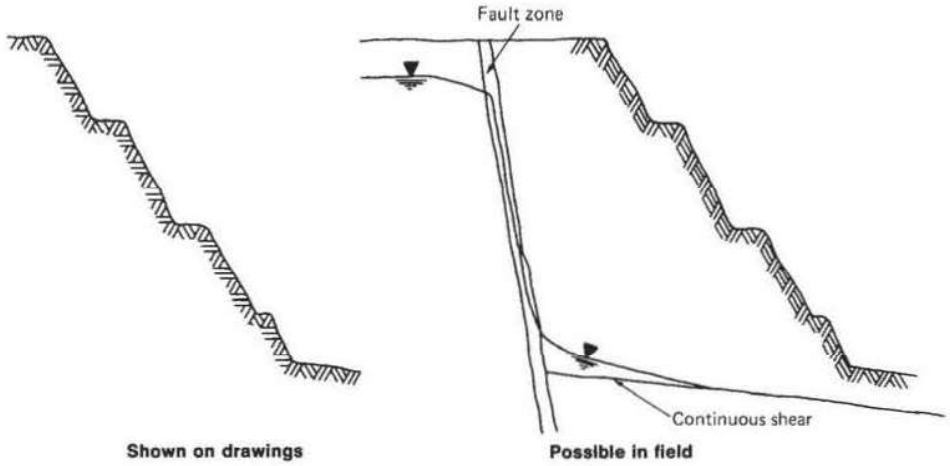

Figure 22.2. Excavated slope in rock.

22.2.2. Is the Slope Stable During Excavation?

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

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

Surface Monitoring of Deformation

Deformation monitoring will often be limited to surface measurements. Vertical and horizontal deformations are normally monitored by surveying methods, with electronic distance measurement playing a significant role. Surface measurements should extend beyond the uppermost limit of any possible movement zone to an area that is known to be stable, so that possible surface strain in advance of cracking can be monitored. Any toe heave should also be monitored. Tension cracks at the crest of the slope may be the first sign of instability. If cracks appear at the crest of the slope or elsewhere, their widths and vertical offsets should be monitored.

Crack measurements give clues to the behavior of the entire slope, and often the direction of movement may be inferred from the pattern of cracking, particularly by the matching of the irregular edges of the cracks.

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

For slopes in rock, monitoring the tilt of critical blocks can provide an assessment of stability if the deformation has a rotational component. Tiltmeters with electrolytic level transducers provide the most precise data, and the high precision allows trends to be determined in a minimum time period. Multipoint liquid level gages have been installed on benches of large excavations in rock where there is concern for a wedge failure (e.g., Stepanov, 1983): the instruments are used to monitor vertical deformation and are intended to provide a forewarning of any instability.

Subsurface Monitoring of Deformation

Subsurface deformation measurements will be required if sliding occurs and if the depth of sliding is not readily apparent from surface measurements and visual observations. Measurements of subsurface horizontal deformation are more important than measurements of subsurface vertical deformation.

For slopes in soil, inclinometers are the instruments of choice, although shear plane indicators can be used for crude measurements, and slope extensometers may be preferred if deformation is predicted to occur along thin shear zones.

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

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

Monitoring of Groundwater Pressure

Open standpipe piezometers are normally selected for slopes in soil, but diaphragm piezometers are appropriate if more rapid response is required. If any sliding is occurring, pore water pressures at or near the sliding surface must be measured to enable an effective stress analysis to be performed.

For slopes in rock, the heterogeneous nature of most rock masses results in a need for comprehensive monitoring of joint water pressure along, above, and below each possible failure plane. Patton (1983) presents a strong case for use of the movable probe type of multipoint piezometer, because it (1) provides a large number of measuring points, (2) does not create nonconformance, (3) allows the transducer to be calibrated at any time, (4) provides redundancy in field data, and (5) reduces the problems of creating multiple seals when more than

one conventional piezometer is installed in a single borehole. Patton comments:

> In my opinion most existing field piezometer installations for [rock] slope stability investigations are deficient in the number of piezometers (probably by a factor of 5 to 10) unless the geology and hydrology of the slope are very simple. However, simple geology and hydrology cannot be demonstrated without a significant number of drillholes to document the geologic and piezometric data.

The Westbay Instruments Ltd. combined piezometer–inclinometer system provides a comprehensive profile of both joint water pressure and horizontal deformation in a single borehole.

If stability has been increased by improving groundwater drainage before the start of excavation, effectiveness of the drainage will usually be monitored by measuring groundwater pressure.

Predicting Stability of Completed Slope

Measurements during excavation can be used to assess the stability of the completed slope. Back-computations are made to examine the validity of parameters used in the original design. On the basis of these back-computations, new design calculations are made to predict the performance of the final slope, and slope angles are modified if necessary. If this approach is taken, clearly the calculations must be made in a timely manner, and a microcomputer will normally be used.

Table 22.1. Instruments Suitable for Examining Slope Stability During Excavation

Measurement	Suitable Instruments
Surface deformation	Surveying methods Crack gages Tiltmeters Multipoint liquid level gages
Subsurface deformation	Inclinometers Fixed borehole extensometers Slope extensometers Shear plane indicators Multiple deflectometers In-place inclinometers Combined piezometer–inclinometer system Acoustic emission monitoring
Groundwater pressure	Single piezometers Multipoint piezometers Combined piezometer–inclinometer system

Summary of Possible Instrumentation

In summary, possible instrumentation for examining slope stability during excavation can be selected from the list given in Table 22.1.

To the degree possible, key instruments should be installed and initial measurements taken before excavation starts. Additional instruments can be installed as excavation progresses.

22.2.3. How Much Ground Is Moving?

If there is evidence of instability during excavation or after completion of the slope, its characteristics must be defined so that any necessary remedial measures may be taken. The question *how much ground is moving?* can be answered by use of instrumentation. The question *why is the ground moving?* will not be answered by instrumentation alone: the answer of course also requires a complete geotechnical investigation and analysis.

Methods of instrumentation have been described in Section 22.2.2. Possible layouts for determining how much ground is moving in excavated slopes are shown in Figures 22.3 and 22.4. It must be stressed

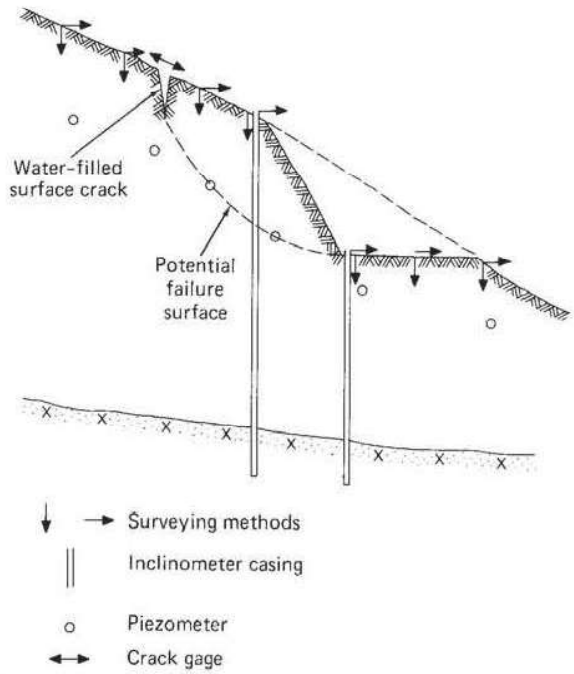

Figure 22.3. Possible layout of instrumentation for monitoring an excavated slope in soil when there is evidence of instability (after DiBiagio and Myrvoll, 1981). Note: Use of surveying methods requires benchmark and horizontal control station, remote from slope.

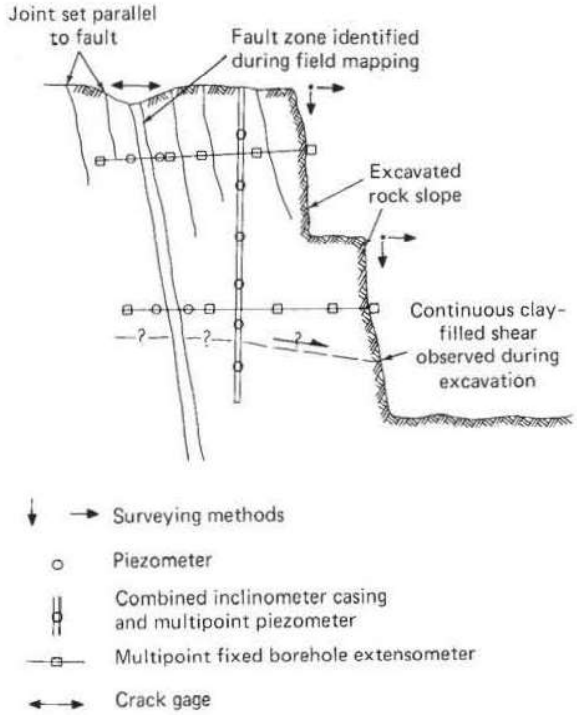

Figure 22.4. Possible layout of instrumentation for monitoring an excavated slope in rock when there is evidence of instability. Note: Use of surveying methods requires benchmark and horizontal control station, remote from slope.

that these layouts are only examples, and many other configurations are possible. Other configurations are indicated by the case histories described later in this chapter.

22.2.4. Is the Slope Stable in the Long Term?

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

In general, a choice must be made among:

1. doing nothing and accepting the consequences of slope failure;
2. monitoring to provide a forewarning of instability, so that remedial measures can be implemented before critical situations arise; and
3. stabilizing the slope, perhaps including a monitoring program to verify that stability has been achieved.

The choice will be based on many factors, including the consequences of failure and the economics of stabilizing. When planning a monitoring program for either of the second two options, a two-step approach is recommended: first, measurements of deformation on the surface of the entire slope to indicate the existence of any instability and second, additional measurements at any location shown to be unstable. Measurement methods are similar to those discussed in Section 22.2.2, and Chapter 15 provides guidelines on maximizing reliability of long-term instrumentation.

For the first step, regular visual observations play an important role. Electronic distance measurement is the primary method for monitoring any deformation on the surface of the entire slope, and crack gages are used to monitor any deformation at existing cracks. If necessary, crack gages can be connected to alarms, set to trigger after a predetermined deformation has occurred. The crack gage shown in Figure 12.14 is particularly useful. Acoustic emission techniques and tilt monitoring can sometimes be helpful when assessing stability. If stability has been increased by improving groundwater drainage, long-term effectiveness of the drainage will usually be monitored as part of the first step, by measuring groundwater pressure.

If data gathered during the first step indicate an unstable area, the same surface observations and measurements will be concentrated in that area during a second step, to define the instability more closely. The depth of sliding may become readily apparent, and it may therefore be possible to plan and implement remedial measures without the need for subsurface instrumentation. When the depth and thickness of the zone of sliding are not apparent, measurements of subsurface horizontal deformation and groundwater pressure will normally be required.

Two additional factors may cause instability in the long term and may merit a monitoring program. First, the wedging action of ice can cause serious instability, and freezing at the surface can cause ice dams to form, which hold unfrozen groundwater inside the ground. The buildup of internal water pressure, unless relieved by drainage, can lead to slope failure. The same effect is observed when sunlight acts on the upper slopes of deep valleys, melting the ice and snow at upper levels, which then percolate downward and exert pressure against the still frozen ground in the lower, more shaded regions. To combat this effect, drainage of the slope

may be required, and drains must be prevented from freezing. Temperature measurements will show the depth and extent of any freezing problems.

Second, loss of tension in rockbolts can cause instability. If rockbolts have been installed for slope reinforcement, the opportunity arises to measure load in the rockbolts, and these measurements may be useful for verifying assumed design loads. However, recognizing that the purpose of rockbolts is to restrain deformation, it is usually more effective to monitor deformation of the rock by using fixed borehole extensometers rather than to monitor stress in rockbolts. A typical approach is use of single-point fixed borehole extensometers, with rods and expansion shell anchors, anchored deeper than the rockbolts. If measurements of load in the rockbolts are also required for more comprehensive monitoring, load cells or strain gages can be used on end-anchored rockbolts but strain gages are the only option for use on fully grouted rockbolts.

Table 22.2. Overview of Routine and Special Monitoring

Application	Measurement
Routine monitoring	Surface deformation Groundwater pressure
Special applications	Subsurface deformation Load in rockbolts Temperature

22.3. OVERVIEW OF ROUTINE AND SPECIAL APPLICATIONS

For those readers who seek general guidelines on what is typically *routine* monitoring and what is monitoring for *special applications*, Table 22.2 presents a broad generalization.

In practice, the most frequent use of instrumentation for excavated and natural slopes is to investigate the characteristics of a slope that is observed to be moving.

22.4. SELECTED CASE HISTORIES

Table 22.3 summarizes selected case histories of instrumented excavated and natural slopes.

Table 22.3. Summary of Selected Case Histories of Excavated and Natural Slopes

Project	Principal Concerns	Type of Slope	Measurement[a]	Instrumentation Discussed	Special Features	Reference
New Liskeard College of Agriculture and Technology, New Liskeard, Ontario, Canada	Stability of slope after installation of horizontal drains	Varved clay	G	Vibrating wire piezometers		Lau and Kenney (1984)
Riverbank at Grand Coulee Dam, Washington, U.S.A.	Stability of slope during large fluctuations in river level	Granular alluvium, varved silts, and clays	G	Piezometers	Shafts and radial drains installed to reduce groundwater levels	Von Thun (1984)
Riverbank on Big Sunflower River, Clarksdale, MS, U.S.A.	Stability of slope, after stabilization with gravel trenches	Highly plastic clay	D	Inclinometers		Wardlaw et al. (1984)
Two sites in southern England	Rate of landside movement Effectiveness of trench drain remedy	Colluvium and clay	D D D G	Surveying methods Shear plane indicators Inclinometers Open standpipe piezometers		Barton and Coles (1983)

Table 22.3. Summary of Selected Case Histories of Excavated and Natural Slopes (*Continued*)

Project	Principal Concerns	Type of Slope	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Vaiont Slide, Italy	Cause, physical characteristics, activity status of slides	Rock	D	Surveying methods		Patton (1983)
Pillar Mountain Slide, Alaska			D	Crack gages		
Downie Slide, Canada	Planning remedial measures		D	Inclinometers		
			D	In-place inclinometers		
			D	Fixed borehole extensometers		
			D	Acoustic emission monitoring		
			G	Open standpipe piezometers		
			G,D	Combined multipoint piezometers and inclinometers		
Various hydroelectric power plants, USSR	Provide forewarning of failure	Rock	D	Tiltmeters	Use of multipoint liquid level gages to provide forewarning of failure	Stepanov (1983)
			D	Fixed borehole extensometers		
			D	Multipoint liquid level gages		
Downie Slide, Revelstoke, BC, Canada	Physical characteristics and activity status of slide	Rock	D	Surveying methods		Piteau et al. (1978)
			D	Inclinometers		
			D	Acoustic emission monitoring		
Saxon pit, Whittlesey, England	Stability of brickpit	Overconsolidated clay	D	Surveying methods		Burland et al. (1977)
			D	Mechanical crack gages		
			D	Subsurface settlement points		
			D	Inclinometers		
			D	Probe extensometers		
			D	Fixed borehole extensometers		
			G	Open standpipe piezometers		
Mine in western Canada	Stabilility of slopes	Rock	D	Electrical crack gages	Data transmitted to a remote location, using telemetry system	Weir-Jones and Bumala (1975)
Reservoir area of a dam			D	Tiltmeters		
			D	Inclinometers		
			D	In-place inclinometers		
			D	Fixed borehole extensometers		
			D	Acoustic emission monitoring		
			G	Vibrating wire and pneumatic piezometers		

Table 22.3. Summary of Selected Case Histories of Excavated and Natural Slopes (*Continued*)

Project	Principal Concerns	Type of Slope	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Five open pit mines	Stability of slopes	Rock	D	Surveying methods		Brawner (1974)
			D	Mechanical crack gages		
			D	Inclinometers		
			D	Fixed borehole extensometers		
			G	Piezometers		
Cabin Creek Hydroelectric Project, Colorado, U.S.A.	Stability of slope	Rock	D	Fixed borehole extensometers		Dutro (1974)
Eleven sites in Canada	Stability of natural slopes	Champlain Sea clays	D	Inclinometers	Analyses of failures in natural slopes	Lo and Lee (1974)
			G	Piezometers		
Fountain Slide, Oregon, U.S.A.	Stability of slope	Rock	D	Inclinometers		Muñoz (1974)
			G	Pneumatic piezometers		
I40, Tennessee, U.S.A.	Stability of slopes	Rock	D	Surveying methods		Tice and Sams (1974)
I26, North Carolina, U.S.A.			D	Inclinometers		
Minneapolis Freeway, Minneapolis, MN, U.S.A.	Cause and physical characteristics of slide	Rock	D	Inclinometers	Slope failure along bentonite seam	Wilson (1974)
	Planning remedial measures		G	Open standpipe piezometers		
Seattle Freeway, Seattle, WA, U.S.A.	Stability of eight slopes	Overconsolidated clay	D	Surveying methods		Palladino and Peck (1972)
			D	Inclinometers		
			G	Observation wells		
			G	Piezometers		
Chuquicamata Mine, Chile	Prediction of failure	Rock	D	Crack gages		Kennedy et al. (1971)
			D	Acoustic emission monitoring		
Potrero Hill Slide, San Francisco, CA, U.S.A.	Stability of slope	Rock	D	Surveying methods	Tunnel below toe of slope	Smith and Forsyth (1971)
			D	Inclinometers		
			D	Fixed borehole extensometers		
			D	Acoustic emission monitoring		
Tripp-Veteran Pitt, Ruth, NV, U.S.A.	Stability of slope	Rock	D	Mechanical crack gages		Stateham and Vanderpool (1971)
			D	Acoustic emission monitoring		

Table 22.3. Summary of Selected Case Histories of Excavated and Natural Slopes (*Continued*)

Project	Principal Concerns	Type of Slope	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Eight slope failures	Various	Soil and rock	D	Surveying methods		Wilson (1970)
			D	Inclinometers		
			G	Open standpipe piezometers		
Steel Plant Expansion, Weirton, WV, U.S.A.	Cause and physical characteristics of slide	Colluvium	D	Surveying methods	Slope stabilized with flexible sheet pile wall and tension ties	D'Appolonia et al. (1967)
	Planning remedial measures		D	Inclinometers		
			G	Open standpipe piezometers		
			S	Strain gages on tension ties		
Three slides alongside railroads in Japan	Prediction of failure	Soil and rock	D	Surveying methods	Recommendations for forecasting failure	Saito (1965)
			D	Crack gages		
Portugese Bend Landslide, Palos Verdes, CA, U.S.A.	Stability of slope	Rock	D	Surveying methods	Unsuccessful attempt to control sliding by installing caissons across sliding surface	Merriam (1960)
			D	Crack gages		
			D	Tiltmeters		

[a]D: deformation; G: groundwater pressure; S: strain in structural member.

CHAPTER 23

UNDERGROUND EXCAVATIONS*

Underground excavations include soft ground and rock tunnels, large caverns such as underground powerhouses, subway stations, underground repositories for nuclear waste, and mines.

Figures 23.1 and 23.2 show a typical soft ground tunnel and a typical tunnel in rock supported by rockbolts. The figures convey the contrast between how the projects will usually appear on engineering drawings, and what may actually happen in the field. The *field* figures are exaggerated to emphasize the contrast. Primary mechanisms that control the behavior of underground excavations are described in Chapter 2.

23.1. GENERAL ROLE OF INSTRUMENTATION

Analysis of stability is a principal geotechnical design task for underground excavations. Factors influencing stability include stratigraphy, groundwater levels, strength of the soil or rock mass, geometry, excavation method, type of support, sequence of excavation and support, and method of support installation.

Stability of underground excavations in rock is usually controlled by the presence and orientation of discontinuities in the rock mass and the presence of water under pressure in these discontinuities. Failures most frequently occur as a result of sliding or separation along discontinuities.

Stability of soft ground tunnels in soft clay is usually controlled by the undrained shear strength of the soil in relation to the total overburden stress at the depth of the tunnel. In silts and fine sands, stability is usually controlled by the effectiveness of construction dewatering or compressed air in controlling the inflow of groundwater. In desiccated hard clays stability is usually controlled by the presence of fissures and slickensides.

The consequence of poor performance of an underground excavation can be severe and may on occasion be catastrophic. A monitoring program may not be required if the design is very conservative, if there is previous experience with design and construction of similar facilities under similar conditions, or if the consequences of poor performance will not be severe. However, under other circumstances a monitoring program will normally be required to demonstrate that the excavation is stable and that nearby structures are not affected adversely.

As with all monitoring programs, planning of a monitoring program for an underground excavation should follow the systematic approach described in Chapter 4. Of particular importance is development of one or more working hypotheses for a mechanism that controls behavior, based on a comprehensive knowledge of the properties of the soil or intact rock material, and/or the locations, frequency, orientation, and properties of discontinuities. To

*Written with the assistance of Bruce E. Beverly, Associate & Vice President, Haley & Aldrich, Inc., Cambridge, MA.

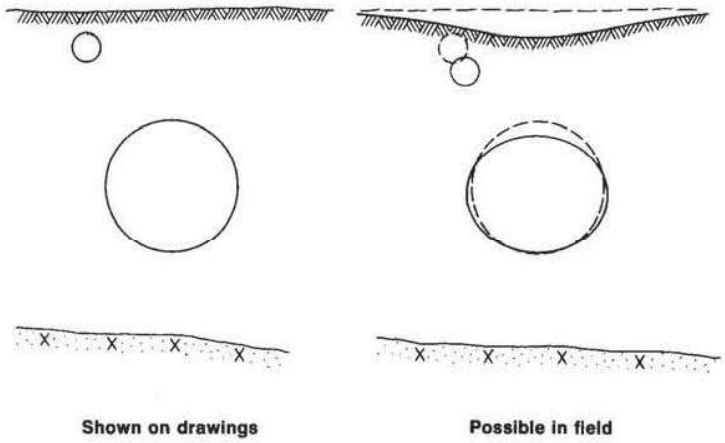

Figure 23.1. Soft ground tunnel.

the extent possible, planning of the program should also be based on construction experience with similar excavations in the same geologic formation, the complexity of the excavation geometry, the amount of rock or soil overburden, the location of the project with respect to structures and utilities, and the method of contracting for construction work.

Full-scale instrumented test sections can be very beneficial to underground excavation projects by confirming economical and safe construction procedures, and potentially large cost savings can be realized. For example, the understanding of the nature of earth pressures on tunnel liners has increased as a result of data obtained from instrumentation of soft ground tunnels during the past decade. Notable among these are U.S. subway tunnel projects in San Francisco, Washington, and Baltimore. Updating of the state of the art will continually occur from the accumulation of knowledge gained from these and similar future studies, thereby permitting better design and more efficient construction.

A program of instrumentation and observation can play a major role in evaluating stability of an underground excavation. When an excavation is **not** stable, underground safety and integrity of nearby structures become a major concern. When an excavation is marginally stable, early detection of instability is possible through use of instrumentation and observations such that supplemental support work can be implemented on a timely basis.

When the underground excavation encounters a known or unexpected major geologic feature such as a fault, shear zone, or a highly jointed or weathered rock zone, instrumentation can be used to monitor behavior, thereby confirming that the implemented construction methods address the major

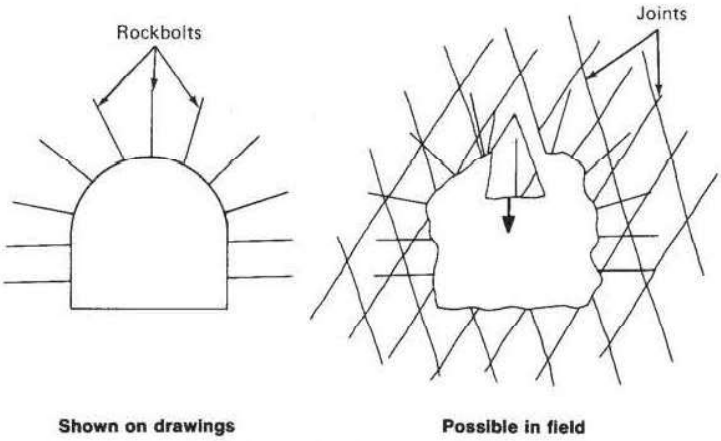

Figure 23.2. Tunnel in rock, supported by rockbolts.

geologic feature such that work proceeds safely and economically. Instrumentation can play an important and critical role where two or more excavations intersect, by confirming that the chosen excavation and support methods and sequence result in stable openings.

Where pillars in mines or other underground excavations are designed to carry high loads, and where integrity of pillars is crucial to stability, instrumentation can be used to monitor loads and deformations, thereby allowing a rational judgment of stability.

An instrumentation program can provide performance documentation for use in evaluating claims from the construction contractor associated with support requirements or "differing site conditions." A program can also be planned to obtain facts for use in legal proceedings if third parties claim that construction work has caused damage to their property. The public can obtain reassurance from a planned instrumentation program, which indicates that the construction will be carefully watched, and this can expedite the legal approval and public acceptance of a project.

When new construction techniques are conceived, their implementation is difficult to justify without monitoring the results to permit an adequate assessment. As an example, instrumentation was used to evaluate the effectiveness of compaction grouting to reduce building settlement caused by tunneling for the Baltimore subway. This gave confidence in the method, which permitted elimination of costly underpinning of many buildings in later parts of the tunnel construction.

More detailed discussions on the role of instrumentation for monitoring underground excavations are given by ASCE (1984), Cording et al. (1975), Dunnicliff et al. (1981), FHWA (1980), Lane (1975), and Schmidt (1976). A keyworded bibliography of about 300 publications describing instrumentation methods and applications for underground excavations has been prepared by FHWA (1980). Keyword groups include ground type, support method, excavation method, reason for monitoring, and monitored parameters.

23.2. PRINCIPAL GEOTECHNICAL QUESTIONS

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

23.2.1. What Are the Initial Site Conditions?

Information on anticipated subsurface conditions is determined by use of conventional site investigation procedures, sometimes supplemented by in situ testing. However, performance monitoring instrumentation sometimes plays a role. For example, piezometers play a role in defining the groundwater regime, including any perched or artesian water. If they are installed sufficiently early, seasonal variations can be defined. If highly permeable zones that may be connected to a source of groundwater recharge are anticipated, full-scale pumping tests may be made to assess dewatering requirements and methods, and these tests will include use of piezometers. Bonded resistance strain gage piezometer systems, with multichannel recorders, are available for this purpose on a rental basis from several manufacturers of geotechnical instruments.

When a shallow underground excavation is to be made near existing structures, a *preconstruction conditions survey* will normally be undertaken by representatives of the construction contractor's insurance company and sometimes also by the owner. The purpose of the preconstruction survey is to observe and document the conditions of structures that may be influenced by the excavation, using photographs, written descriptions, and/or videotapes of existing defects in the structures. The preconstruction conditions survey is frequently supplemented by use of surveying methods to define the elevations of reference points on the structures and on the ground surface. The widths of any existing cracks may also be measured.

Pilot tunnels or exploratory adits are sometimes excavated prior to construction of full-scale rock tunnels and large caverns such as underground powerhouses and subway stations. The primary purposes of these exploratory excavations are to study geologic conditions at excavation depth and to assess various excavation and support methods. In situ tests are sometimes made in holes drilled from the ground surface and also within the exploratory excavations to determine ground permeability and grouting requirements, rock mass deformation properties, and in situ stress conditions.

23.2.2. If an Exploratory Excavation Is Made, Can Field Measurements Assist with the Design of the Prototype?

Although the primary purposes of exploratory excavations are to study geologic conditions at excava-

tion depth and to assess various excavation and support methods, their construction provides an opportunity to use geotechnical field instrumentation. For example, when squeezing ground conditions are anticipated, measurements of ground deformations can provide useful information for design of the prototype. However, attempts to extrapolate ground behavior from instrumentation measurements within exploratory excavations are often hampered severely by scale effects. Extrapolations to the scale of the prototype may be unwarranted unless the size of the exploratory excavation is large compared with the spacing between discontinuities. If the size is large enough, measurements of ground deformation and support stress may provide valuable input to the design of the prototype, although local geologic factors and construction details have a large influence on measurements. Extrapolation of the data must always be tempered by engineering judgment and experience.

When squeezing ground conditions are monitored in an exploratory excavation, instrumentation normally consists of surveying methods, convergence gages, and single-point fixed borehole extensometers with rods and mechanical readout. When support stresses are monitored, strain gages and/or load cells are used, of the types recommended below for full-scale test sections.

23.2.3. What Information Can Be Provided by Instrumenting a Full-Scale Test Section?

The design of underground excavations usually requires the selection of the most economical excavation and support method. The primary goals when designing support are to control ground movements and to establish a stable excavation. Ground movements and support stress are of course interdependent, and both are affected by the method and sequence of excavation and support.

The designers may identify a combination of circumstances that lead them to opt for a full-scale test section, as a means of ensuring design adequacy and economy. The combination of the following four circumstances provides an example. First, there is little precedent for excavations under similar conditions. Second, new excavation or support methods are to be used. Third, it appears that cost savings or increased safety may result from increasing site-specific knowledge of the relationships between excavation and support method, sequence of excavation and support, and ground movements and support stresses. Fourth, scale effects severely hamper the use of exploratory excavations for developing design parameters for the prototype.

Most full-scale test sections are initiated by the designers of the excavation, but Fox (1980) describes three cases in which the construction contractor initiated a full-scale instrumented test in a soft ground tunnel. First, the contractor needed to determine how much reinforcement was needed to a light steel liner plate so that the shield could be advanced by shoving against the liner. Second, there was a need to assess whether compressed air could be taken off when all liner plates were installed, so that the final concrete liner could be placed in free air rather than in compressed air. Third, the contractor needed to determine at what distance along the tunnel the shield thrust was dissipated in ground friction, so that a jacking frame could be removed from a shaft to improve access.

If both an exploratory excavation and a full-scale test section have been selected, the test section will normally be constructed by enlarging the exploratory excavation. In other cases the test section will be excavated and supported under the same construction contract as the prototype, as early as possible during the construction period. Ground conditions at a test section should be representative of a significant length of the underground excavation. Ideally, the excavation should be made to the same size as the prototype and the length should be adequate so that end effects are minimized, typically requiring a minimum length of about four times the width of the excavation. However, this ideal must be tempered by cost considerations and by knowledge of site and project conditions. The methods and sequence of excavation and support should match, as closely as possible, the likely methods and sequence for the prototype.

The primary parameter monitored in a test section will generally be deformation. Often support stresses are also monitored, and measurements of groundwater pressure may be included. Suitable instruments are listed in Table 23.1, and more details are given in the following subsections.

Deformation of Ground Around Soft Ground Tunnels

Inclinometers and probe extensometers can be installed from the ground surface alongside a soft ground tunnel and sometimes are also installed on the centerline in front of the heading. A probe ex-

Table 23.1. Suitable Instruments for Monitoring a Full-Scale Test Section

Measurement	Suitable Instruments
Deformation of ground surface and structures	Surveying methods Rod settlement gages
Settlement of utilities	Subsurface settlement points
Change in width of cracks in structures	Crack gages
Deformation within excavation	Surveying methods Convergence gages
Deformation of ground around soft ground tunnels	Inclinometers Probe extensometers Fixed borehole extensometers
Deformation of ground around underground excavations in rock	Fixed borehole extensometers Acoustic emission monitoring
Stress in or loading on soft ground tunnel liners	Strain gages Load cells Concrete stress cells Contact earth pressure cells
Stress in or loading on supports for underground excavations in rock	Strain gages Load cells Concrete stress cells
Stress change in rock	Soft inclusion gages Rigid inclusion gages
Groundwater pressure	Piezometers

tensometer and inclinometer casing can be combined in a single borehole. Fixed borehole extensometers, installed from the ground surface or from within the tunnel can provide supplementary deformation data.

Deformation of Ground Around Underground Excavations in Rock

Multipoint fixed borehole extensometers are the usual instruments of choice for monitoring deformations around underground rock excavations. They will normally be installed above the crown and also at appropriate angles from within the test section to monitor rock deformation along critical discontinuities. If the overburden is less than about 200 ft (60 m) deep, extensometers above the crown can be installed from the ground surface in vertical or angled boreholes prior to excavation, using groutable anchors and mechanical transducers. Use of groutable anchors creates a simple installation procedure and avoids possible communication of groundwater into the excavation. Borehole directional surveys will normally be required to determine anchor locations with respect to the excavation, and a colored plastic tube should be attached to the deepest anchor to protrude into the path of the excavation to indicate the location of the borehole.

When the excavation is too deep for installation of extensometers from the ground surface, extensometers can be installed in boreholes drilled upward from the excavation as soon as access becomes available. If a pilot tunnel has been driven at the crown of the future excavation, these extensometers can be installed from the pilot tunnel before production downward excavation starts, thereby avoiding interruption to production excavation. Extensometers installed from within the excavation should initially have mechanical transducers, to minimize the potential for damage while excavation continues nearby. Later they can be converted for reading with electrical transducers, either when access difficulties arise or when a remote reading arrangement is preferred.

Stress in Soft Ground Tunnel Liners

When stress data are required in soft ground tunnel liners, strain gages and/or load cells can be used. Circumferential stresses within steel and precast concrete liners can be determined by using either vibrating wire or electrical resistance strain gages, with a preference for the vibrating wire type. Weldable gages are usually preferred on steel liners, but bonded resistance gages may be appropriate if available space is too limited for weldable gages. The type of vibrating wire gage with two wires at different distances from the liner surface can be used to monitor bending strains by attaching the gage to the inside face only. Embedment gages with vibrating wire transducers are usually preferred in precast concrete liners. Measurements in cast-in-place or shotcrete liners normally require use of concrete stress cells, because if strain gages are used the strain data cannot meaningfully be converted to stress.

Load cells have been interposed between segments of precast concrete liner to measure circumferential loading (e.g., Tattersall et al., 1955),

thereby overcoming the difficulties of converting strain in concrete to stress, but care must be taken to prevent the creation of a nontypical liner ring that would result in misleading data. When the total load and distribution of load imposed on the outside of the liner is required, circumferential strain gages can be placed at sufficiently close intervals so that imposed loads can be derived from stresses in the liner (Beloff et al., 1979). Alternatively, contact earth pressure cells with sensitive faces flush with the outside of the liner can be used, but data are very much affected by local contact irregularities, and a large number of cells are required for meaningful measurements. If there is a need to separate water loading from effective soil loading, piezometers can be installed in shallow holes, drilled through grout ports in the lining.

When the shield is advanced by shoving against the previously installed lining, the shoving jacks impose longitudinal stresses in the lining, and these stresses are often more critical than circumferential stresses caused by soil loading. Determination of longitudinal stress requires gages with adequate dynamic response, vibrating wire strain gages may not be suitable, and resistance gages may be required. Options are weldable or bonded gages on steel liners and embedment gages in concrete liners, monitored using a multichannel automatic data acquisition system with appropriate dynamic response. Correlations between longitudinal stress and jacking thrust can be made by including electrical resistance strain gage pressure transducers in the hydraulic lines for the jacks or by placing an electrical resistance strain gage load cell in series with each jack.

Stress in Supports for Underground Excavations in Rock

When stress data for steel support elements are required in underground rock excavations, strain gages and/or load cells can be used. As for soft ground tunnels, measurements in cast-in-place or shotcrete liners normally require use of concrete stress cells. However, although stress data can be useful, they are essentially *point* measurements, subject to variability in geologic or support characteristics, and may therefore not represent average or typical conditions on a larger scale. A large number of measurement points will usually be required before confidence can be placed in the data. In contrast, many deformation measuring devices respond to movements within a large and representative zone; data provided by a single instrument can therefore be meaningful, and deformation measurements are generally the most reliable and least ambiguous.

When load data are required for rockbolts, load cells can be used on end-anchored bolts, but fully grouted bolts require the use of strain gages. Weldable strain gages are normally used, with a preference for the vibrating wire rather than the resistance type.

Stresses in steel sets are normally calculated from strain data, using vibrating wire strain gages, either the arc welded or spot welded type. Load cells have been interposed between adjacent segments of a steel set, but care must be taken to prevent the creation of a nontypical set that would result in misleading data.

Figure 23.3 shows a possible layout of instrumentation in a shallow full-scale test section in rock. This is only one example, and it must be emphasized that many other configurations are possible. Other configurations are indicated by the case histories described later in this chapter.

23.2.4. Is the Excavation Stable?

The general role of instrumentation for stability monitoring has been described in Section 23.1. The types of instruments used are similar to those used in full-scale test sections, as listed in Table 23.1.

Routine Data for Stability Assessment

Measurements of crown settlement and wall-to-wall convergence, using surveying methods and convergence gages within the excavation, provide routine data for stability assessment. As an example, ovalling of the primary lining is a concern when tunneling in soft clay. These linings are generally thin and deform until all the earth load is carried in lining thrust. It is important to know when ovalling has stabilized, such that any secondary lining can be constructed and/or if continued ovalling is leading to overstress and failure of the lining.

When the underground excavation is shallow and the consequences of ground loss are significant, measurements of ground surface settlement will often be made above the excavation at several cross sections. Measurements of unacceptable settlement will indicate the need to alter the excavation or support method and/or sequence.

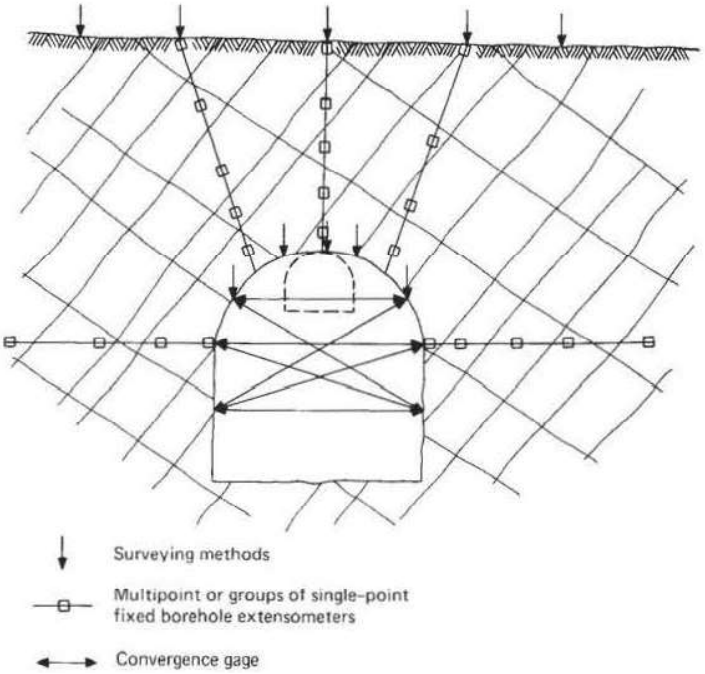

Figure 23.3. Possible layout of instrumentation for monitoring a shallow full-scale test section in rock. Note: Use of surveying methods requires benchmark, remote from excavation. Strain gages and/or load cells may be installed on steel supports, or concrete stress cells in shotcrete support.

Adverse Effects on Utilities and Buildings

Where there is a concern that ground movements may cause an adverse effect on nearby utilities, pavements, and buildings, additional measurements should be made. Instrumentation can provide a forewarning of adverse effects, thereby allowing remedial measures to be implemented before critical situations arise.

Settlement of important utilities can be measured using subsurface settlement points. A rod settlement gage alongside each important structure will provide accurate settlement data without need for a survey crew.

If the monitoring program consists only of surface and shallow subsurface settlement measurements and the measurements indicate adverse behavior, there will often be inadequate data for defining the cause of the problem and for developing an economical solution. A well-planned monitoring program for providing forewarning of adverse effects will therefore include instrumentation that indicates comprehensive causal data. Instruments for this purpose include inclinometers, probe extensometers, fixed borehole extensometers, crack gages, and piezometers. Acoustic emission monitoring can sometimes be used in shallow boreholes to provide an indication of deformation trends and locations.

Stability of Individual Rock Blocks

Concern for stability of individual rock blocks in underground rock excavations or of large zones bounded by adverse discontinuities is addressed using fixed borehole extensometers, convergence gages, and crack gages. For example, during construction of DuPont Circle Station in Washington, DC, in poor-quality rock, fixed borehole extensometer measurements provided essential input to daily decisions on excavation and support methods.

Areas of Special Concern

Instrumentation may be used effectively to monitor stability in areas of special concern, such as the intersections of the excavation and a major geologic feature or at locations where two or more openings intersect or cross in close proximity.

New Austrian Tunneling Method

Instrumentation plays an important role in the *New Austrian Tunneling Method (NATM)*. The method

takes maximum advantage of the capacity of the rock to support itself. As the excavation is made, forces in the surrounding rock readjust and are carefully and deliberately controlled by selecting appropriate support (Rabcewicz and Golser, 1973). Support requirements are a minimum when the ground has reacted to the point where loosening of rock is about to occur. Various categories of ground conditions are established prior to construction, together with commensurate types and amounts of support required to stabilize that category of ground. The actual support is jointly decided on at the heading by qualified representatives of the owner and construction contractor, and contracts provide for appropriate payment in accordance with these decisions.

Visual observations and convergence measurements provide primary input to the decisions on actual support. Additional input is often gained by use of fixed borehole extensometers, to provide more comprehensive data on ground movement with distance from the heading.

Comparison Between Measurements and Predictions

In large underground excavations in rock, such as underground powerhouses, measurements are often made to assess stability, by comparing actual rock deformations with predicted values. Instrumentation generally includes multipoint fixed borehole extensometers and convergence gages.

Stability of Underground Mines

Concern for overload of mine pillars is often addressed by monitoring stress change with rigid or soft inclusion gages, supplemented with deformation measurements using fixed borehole extensometers, convergence gages, and crack gages. Bauer (1985) presents a thorough summary of methods in current use for monitoring the stability of underground mines.

Overload of Individual Supports

Concern for overload of individual supports can be addressed by using the types of strain gage and/or load cell described in Section 23.2.3. However, stress data are essentially *point* measurements and, recognizing that the purpose of support is to restrain deformation, it is often more effective and economical to monitor deformation rather than support stress.

Effect of Dewatering

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

Effectiveness of Innovative Construction Techniques

When an innovative construction technique is being used, instrumentation normally plays a role in monitoring the effectiveness of the technique. Selection of measured parameters and instruments of course depends on the construction technique. As an example, Figure 23.4 shows a possible layout of instrumentation for monitoring the effectiveness of chemical grouting in soft ground, to increase face stability and stand-up time and to decrease permeability.

23.2.5. What Is the Nature and Source of Undesirable Ground Movement?

If there is evidence of instability during excavation, the depth of sliding may be readily apparent from visual observations, and the routine stability monitoring described in Section 23.2.4 is often sufficient for obtaining the rate of movement. When the ex-

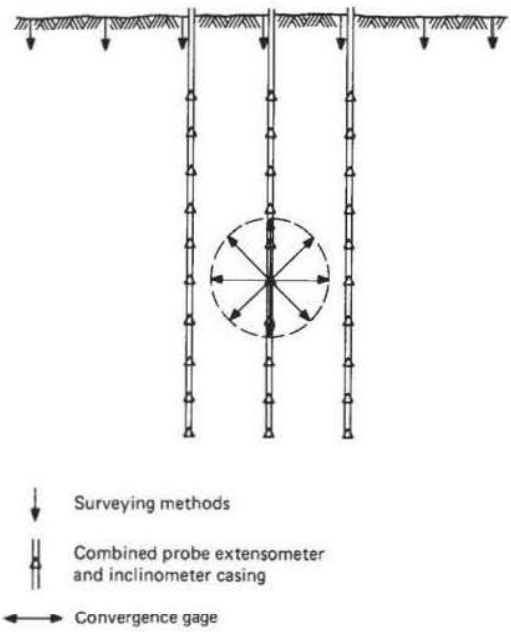

Figure 23.4. Possible layout of instrumentation for monitoring effectiveness of chemical grouting in soft ground (after Clough et al., 1977, 1979). Note: Use of surveying methods requires benchmark, remote from tunnel.

tent of the moving zone is not readily apparent, subsurface deformation measurements should be made so that an appropriate remedy can be planned. The question *how much ground is moving?* can be answered by the use of instrumentation, but the question *why is the ground moving?* will not be answered by instrumentation alone: the answer of course also requires a complete geotechnical investigation and analysis.

For soft ground tunnels, surveying methods, convergence gages, and inclinometers are the primary monitoring tools, supplemented if necessary by probe extensometers and fixed borehole extensometers. Deformation around the tunnel face is three dimensional, and measurements may include the use of inclinometer casings on the tunnel centerline ahead of the face.

For underground excavations in rock, surveying methods, convergence gages, and fixed borehole extensometers are the primary monitoring tools, supplemented as necessary by crack gages and acoustic emission monitoring.

If groundwater pressures are a possible cause of instability, they should be measured with piezometers.

23.3. OVERVIEW OF ROUTINE AND SPECIAL APPLICATIONS

For those readers who seek general guidelines on what is typically *routine* monitoring and what is monitoring for *special applications*, Table 23.2 presents a broad generalization.

In practice, the most frequent use of instrumentation for underground excavation projects is to determine whether stability of the excavation has been achieved.

Table 23.2. Overview of Routine and Special Monitoring

Application	Measurement
Routine monitoring	Surface settlement Settlement of structures and utilities Change in width of cracks in structures Convergence within excavation Groundwater pressure
Special applications: design phase	Groundwater pressure during full-scale pumping test Deformation of ground and support, within exploratory excavations Stress in or loading on support, within exploratory excavations Deformation of ground and support, within full-scale test section Stress in or loading on support, within full-scale test section
Special applications: construction phase	Deformation of ground and support, within excavation Stress in or loading on support, within excavation

23.4. SELECTED CASE HISTORIES

Approximately 300 case histories of instrumented underground excavations are included in a keyworded bibliography by FHWA (1980). Table 23.3 summarizes selected case histories from this and other sources.

Table 23.3. Summary of Selected Case Histories of Underground Excavations

Project	Principal Concerns	Type of Excavation	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Various	Stability	Underground mines	D	Convergence gages		Bauer (1985)
			D	Fixed borehole extensometers		
			SCR	Soft and rigid inclusion stress gages		
Lafayette Park, Washington Metro, Washington, DC, U.S.A.	Improvement of design and construction procedures	Subway tunnel in sand, gravel, and clay	G	Open standpipe piezometers	Full-scale test section	Hansmire and Cording (1985, 1972)
			D	Surveying methods		
			D	Fixed borehole extensometers		
			D	Inclinometers		
Stillwater Tunnel, Utah, U.S.A.	Binding of tunnel boring machine	9.6 ft (2.9 m) diameter tunnel in highly fractured and sheared shales	D	Convergence gages	Deformations measured ahead of heading	Robinson et al. (1985)
			D	Fixed borehole extensometers		
			D	Horizontal inclinometers		
			D	Deflectometers		
			S	Mechanical and vibrating wire strain gages		
Milchbunk Tunnel, Zürich, Switzerland	Effectiveness of ground freezing in limiting ground movements	Highway tunnel in moraine	D	Surveying methods	Measurements led to modification of freezing procedure	Mettier (1983)
			D	Convergence gages		
			D	Fixed borehole extensometers		
			D	Inclinometers		
			T	Temperature sensors		
Demonstration storage cavern for crude oil, Kikuma, Japan	Stability of cavern Hydraulic behavior of rock	Storage cavern for crude oil in granite	G	Electrical resistance piezometers		Miyashita et al. (1983)
			D	Convergence gages		
			D	Fixed borehole extensometers		
			S	Strain gages on rockbolts		
Armco No. 7 Mine, Montcoal, WV, U.S.A.	Stability and subsidence	Longwall coal mine in sandstone, shale, and coal	D	Surveying methods		O'Connor (1983)
			D	Convergence gages		
			D	Fixed borehole extensometers		

Table 23.3. Summary of Selected Case Histories of Underground Excavations (*Continued*)

Project	Principal Concerns	Type of Excavation	Measurement[a]	Instrumentation Discussed	Special Features	Reference
			D	Combined inclinometers and probe extensometers		
			SCR	Soft and rigid inclusion stress gages		
Research mine, Kiruna, Sweden	Stability	Open stope mine	D	Convergence gages	Automatic data acquisition system used	Stillborg et al. (1983)
			D	Fixed borehole extensometers		
			D	Probe extensometers		
			SCR	Soft inclusion stress gages		
Kielder Water Scheme, Northumbria, England	Improvement of design and construction procedures for water tunnels in mudstone	11 ft (3.3 m) diameter experimental tunnel in mudstone	D	Convergence gages	Various support systems used: e.g., steel ribs, steel liner, rockbolts	Ward et al. (1983, 1976)
			D	Probe extensometers		
			D	Fixed borehole extensometers		
			L	Telltale load cells		
			S	Mechanical and vibrating wire strain gages		
Bolton Hill Tunnels, Baltimore, MD, U.S.A.	Effectiveness of compaction grouting in limiting ground movements	Subway tunnel in dense sand	D	Surveying methods	Full-scale test section	Baker et al. (1981)
			D	Subsurface settlement points		
			D	Combined inclinometers and probe extensometers		
Massachusetts Bay Transportation Authority, Davis Square, to Porter Square, Somerville, MA, U.S.A.	Stability	Subway tunnel in clay, glacial till, and argillite	D	Surveying methods	Mixed face excavation	Beloff et al. (1981)
			D	Subsurface settlement points	Floating crown bars used for primary support	
			D	Combined inclinometers and probe extensometers		
			S	Vibrating wire strain gages		
Various	Various	Various	G	Piezometers	Emphasis given to purpose and benefits of monitoring programs	Dunnicliff et al. (1981)
			D	Various deformation gages		
			L	Load cells		
			S	Strain gages		

Table 23.3. Summary of Selected Case Histories of Underground Excavations (*Continued*)

Project	Principal Concerns	Type of Excavation	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Lexington Market Tunnels, Baltimore, MD, U.S.A.	Demonstrate suitability of precast concrete liners for soft ground tunnels	Twin subway tunnels in soft ground, one lined with precast concrete, one with steel	G	Open standpipe piezometers	Performance and cost of two systems compared	Wightman et al. (1980)
			D	Surveying methods		
			D	Convergence gages		
			D	Crack gages		
			D	Subsurface settlement points		
			D	Fixed borehole extensometers		
			D	Inclinometers		
			D	Tiltmeters		
			L	Pressure transducers in lines to shoving jacks		
			S	Embedment strain gages: sister bars		
Port Richmond Water Pollution Control Project, Staten Island, NY, U.S.A.	Justify reduction of liner thickness in future tunnels	10 ft (3 m) diameter tunnel in dense sand, organic silt, weathered serpentine, and schist	G	Pneumatic piezometers	Tunnel lined with 0.5 in. (13 mm) thick rolled steel liner plates	Beloff et al. (1979)
			D	Surveying methods	Six full-scale test sections	
			D	Subsurface settlement points		
			D	Probe extensometers		
			D	Convergence gages		
			L	Pressure transducers in lines to shoving jacks		
			S	Electrical resistance strain gages		
Sewer tunnel in Mississauga, Ontario, Canada	Improvement of design procedures	14 ft (4.3 m) diameter tunnel in very dense till	G	Piezometers		DeLory et al. (1979)
			D	Convergence gages		
			D	Subsurface settlement points		
			D	Inclinometers		
			S	Electrical resistance strain gages		
			TS	Contact earth pressure cells		
Genesee River Interceptor Southwest, Rochester, NY, U.S.A.	Effect of high horizontal stresses	18.5 ft (5.6 m) diameter tunnel in highly stressed dolostone	D	Convergence gages		Guertin and Flanagan (1979)
			D	Fixed borehole extensometers		

Table 23.3. Summary of Selected Case Histories of Underground Excavations (*Continued*)

Project	Principal Concerns	Type of Excavation	Measurement[a]	Instrumentation Discussed	Special Features	Reference
			D	Inclinometers		
			D	In-place inclinometers		
East 63rd Street Tunnel, New York City Transit Authority, New York, NY, U.S.A.	Improvement of design and construction procedures	Tunnel in schist, 45 ft (13.7 m) high and wide	D	Surveying methods		Guertin and Plotkin (1979)
			D	Fixed borehole extensometers		
			S	Vibrating wire strain gages		
Various projects in Germany	Stability and settlement	Subway tunnels	D	Surveying methods	NATM used, resulting in minor settlements and no building damage	Müller and Spaun (1977)
			D	Convergence gages		
			D	Fixed borehole extensometers		
			D	Multiple deflectometers		
			TS	Contact earth pressure cells		
London Underground, Regents Park, London, England	Improvement of design and construction procedures	Subway tunnel in soft ground, with expanded concrete lining	G	Pneumatic piezometers		Barratt and Tyler (1976)
			D	Surveying methods		
			D	Convergence gages		
			D	Combined inclinometers and probe extensometers		
			L	Load cells		
			TS	Contact earth pressure cells		
Various	Various	Full-scale test sections	G	Piezometers	Emphasis on benefits of full-scale test sections	Lane (1975)
			D	Various deformation gages		
			L	Load cells		
			S	Strain gages		
Washington Metro, Washington, DC, U.S.A.	Improvement of design and construction procedures for subway tunnels	Subway tunnels in rock	D	Fixed borehole extensometers	Summary of rock deformations and strains in shotcrete linings	Mahar et al. (1972)
			S	Embedment strain gages		
Potomac Interceptor Sewer, Washington, DC, U.S.A.	Determine cause of excessive settlement Improve design and construction procedures	11 × 13 ft (3.6 × 4 m) tunnel in organic and clayey silts	D	Surveying methods		Rebull (1972)
			D	Crack gages		
			D	Convergence gages		
			D	Fixed borehole extensometers		
			TS	Contact earth pressure cells		

Table 23.3. Summary of Selected Case Histories of Underground Excavations (*Continued*)

Project	Principal Concerns	Type of Excavation	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Anvil Points Oil Shale Research Center, Rifle, CO, U.S.A.	Improvement of design and construction procedures	Underground oil shale mine	D	Fixed borehole extensometers	Full-scale test section	Sellers et al. (1972)
			L	Load cells		
			SCR	Soft and rigid inclusion stress gages		
Morrow Point Power Plant, Colorado, U.S.A.	Stability	Underground powerhouse in high-quality metamorphic rock	D	Surveying methods	Monitoring revealed moving polyhedron of rock, requiring stabilization	Brown et al. (1971)
			D	Fixed borehole extensometers		
			D	Acoustic emission monitoring		
			L	Rockbolt load cells		
Various	Various	Soft ground tunnels	D	Surveying methods	State-of-the-art report, including observational data and some design recommendations	Peck (1969b)
			D	Convergence gages		
			D	Inclinometers		
			L	Load cells		
			S	Strain gages		
Garrison Dam, North Dakota, U.S.A.	Need to establish design criteria	36 ft (11 m) diameter test tunnel in clay-shale	G	Piezometers	Substantial cost savings for prototype tunnels	Burke (1957)
			D	Surveying methods		
			D	Convergence gages		
			S	Mechanical strain gages		
			TS	Contact earth pressure cells		

[a]D: deformation
G: groundwater pressure
L: load in support
S: strain in support
SCR: stress change in rock
T: temperature
TS: total stress between lining and soil.

CHAPTER 24

DRIVEN PILES*

24.1. GENERAL ROLE OF INSTRUMENTATION

The subsurface length of a driven pile cannot usually be inspected after driving; thus, its physical condition and alignment are unknown. Figure 24.1 shows a group of piles, driven to bedrock: the figure conveys the contrast between how the project will usually appear on engineering drawings and what may actually happen in the field. Primary mechanisms that control the behavior of driven piles are outlined in Chapter 2.

Subsurface geotechnical conditions are rarely known with certainty, and therefore the design of driven piles involves assumptions and uncertainties that are often addressed by conducting instrumented full-scale tests. Tests may examine the behavior of the pile under load applied to the pile butt (head) or under load caused by settlement of soil with respect to the pile.

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

*Written with the assistance of Douglas G. Gifford, Associate & Vice President, Haley & Aldrich, Inc., Cambridge, MA.

24.2. PRINCIPAL GEOTECHNICAL QUESTIONS

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

24.2.1. What Is the Load–Movement Relationship of the Pile? (Determination of Relationship by Static Testing)

The load–movement relationship of the pile and/or designed length of the pile may be uncertain during the design phase, and static load tests can be performed to resolve these uncertainties. Load tests may be conducted either during the design phase or at the beginning of the construction phase. A load test allows application of load to the pile butt so that the actual response to load can be verified experimentally. The loading mode may be axial compression or uplift, lateral, eccentric, moment, torsional, or a combination of these modes.

Static Load Tests Without Determination of Comprehensive Load Transfer Data: Loading in Axial Compression and Uplift

Procedures for static load tests in axial compression and uplift are described by ASTM (1981a) and ASTM (1983), respectively. Load at the pile butt should be measured using a load cell, equipped with a spherical bearing, because load determinations based on hydraulic jack pressures are often unreliable (Chapter 13). Vertical deformation at the pile

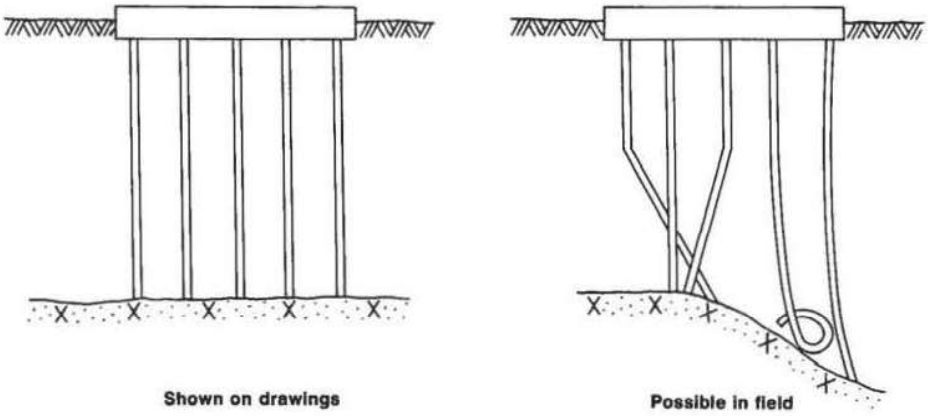

Figure 24.1. Pile group driven to bedrock.

butt is normally measured using dial indicators attached to a reference beam. A wire/mirror/scale arrangement is often used as backup. Similar measurements of horizontal deformation are recommended to indicate whether or not the applied load is axial and concentric. The reference beam should ideally be supported on ground that is not influenced by the test, but often this is not possible, and any movement of the beam should be measured throughout the test by surveying methods. Great care should be taken to minimize the effect of temperature changes on the reference beam.

Most conventional static load tests in axial compression include measurements of load and deformation at the pile butt only. However, such measurements are insufficient for evaluating load tests of relatively short duration on long end-bearing piles because much of the test load is carried by skin friction. They are also insufficient for concrete piles if they are tested while the concrete is "green," as is often the case when available time for testing is very limited. In general, therefore, tip movement should be measured, using a telltale and dial indicator, requiring an open hole along the length of the pile. Installation of telltales in closed-ended pipe piles is straightforward, but installation on other steel piles requires welding one or more pipes, channels, or plates to create an open hole along the length of the pile, through which telltales are later installed. Access for telltales in a concrete pile is created by casting a plastic pipe along the axis of the pile. The American Society for Testing and Materials (ASTM, 1981a) illustrates possible methods for installing telltales in or on steel H, pipe, and timber piles. A caution needs to be given about attachment of telltales (or any other instrumentation) to the outside of spiral-welded pipe piles: these piles tend to rotate as they are driven, and external instrumentation is not likely to survive. ASTM (1981a) also illustrates a *load transfer assembly* suitable for creating access to the tops of the telltales. When a load transfer assembly is used, its members should be welded together to ensure that no instability develops during load testing.

Evaluations of results of load tests in axial compression are severely hampered if tip load is not known, because total load cannot be separated into point resistance and skin friction. This statement leads to a recommendation for measurement of tip load and, although this is not standard procedure, measurements of tip load should be made if a financial payback can be identified. For steel piles, strain gages should be installed near the tip: some suggestions for gage type and methods for preventing damage are given in the following subsection. Multiple telltales can be used as backup but, as discussed in the following subsection, they are subject to several limitations. Measurements of tip load in concrete piles should be made by attaching a contact earth pressure cell to the pile tip: for the reasons given in the following subsection, strain measurements are not adequate. Figure 24.2 shows a possible layout of instrumentation during a static load test of a driven concrete pile, loaded in axial compression, for the case where comprehensive load transfer data are not required.

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

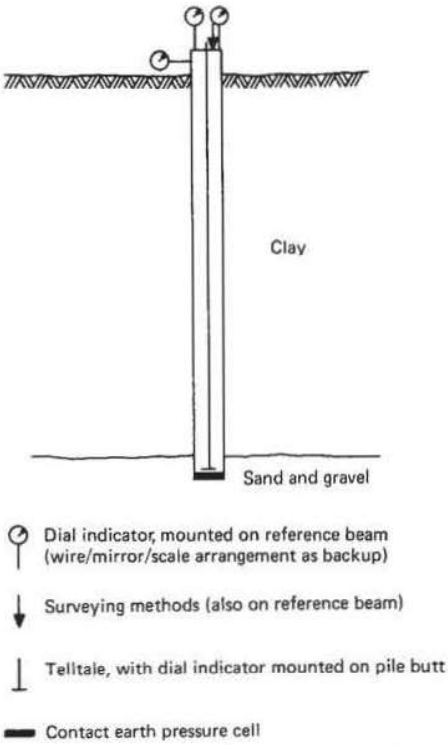

Figure 24.2. Possible layout of instrumentation during a static load test of a driven prestressed concrete pile, loaded in axial compression, for the case where comprehensive load transfer data are not required. Note: Loading arrangements, load transfer assembly, and reference beam are not shown. Also, a load cell is required. Use of surveying methods requires benchmark, remote from pile.

Normally, a waiting period of about a week is sufficient. However, if the test pile is selected from a group of piles, the excess pore water pressures may be significantly higher than for a single pile. Pore water pressures may also be significantly higher if the soil is excessively disturbed. In these cases, one or more piezometers should be installed so that effective stresses at the soil/pile contact can be evaluated.

Static Load Tests with Determination of Comprehensive Load Transfer Data: Loading in Axial Compression and Uplift

When comprehensive load transfer data are required along the lengths of driven piles during static load tests in axial compression or uplift, strain gages and/or multiple telltales are used.

Multiple telltales cannot be the primary measurement system because they can be installed only after pile driving and therefore cannot provide *residual stress* data. Residual stresses are stresses that remain in a pile after pile driving: without a knowledge of their magnitude, absolute load in a pile cannot be calculated from telltale data. As discussed in Chapter 13, multiple telltales are subject to additional limitations. First, load data are extremely sensitive to error in telltale data; therefore, duplicate telltales are preferred at each depth. The entire length of each telltale should be straight (the arrangement with an elbow, as shown in ASTM (1981a) for pipe piles, is suitable for monitoring tip settlement but introduces excessive errors when using multiple telltales for determination of strain), generally requiring use of a welded load transfer assembly as illustrated in ASTM (1981a). Second, the shape of the load transfer plot between telltale attachment points must be assumed; therefore, telltales at two depths are usually inadequate. Third, an extrapolation must be made for determination of tip load. When planning multiple telltale installations and interpreting data, one must pay careful attention to the guidelines given in Section 13.3.1.

For steel piles, strain gages provide the primary measurements, and multiple telltales can be used as backup. A possible layout is shown in Figure 24.3. Weldable vibrating wire gages are preferred on H-piles and also on pipe piles if they are installed on the outside. However, a method of installing the larger type of vibrating wire gage inside a closed-ended pipe pile is outlined in Chapter 13 and is an attractive option. Very great care must be taken to prevent damage to gages and cable during driving, and if there is likelihood of damage to the tip of the pile, gages should be located above the potentially damaged length. Gages on the outside of the pile should be protected with robust cover plates tapered toward the pile tip. Cables should usually be installed within steel angles or channels. An effective method involves welding channels to the pile, with 6 in. (150 mm) long windows for cable pull-through points. After pulling cables through, foam material (e.g., the two-part type used for home insulation) is injected through holes drilled in the channel at 2 ft (600 mm) intervals, to support the cable during driving. The windows are then closed with small lengths of channel, tack welded to the pile. It is often necessary to protect the cable, beyond the point of exit from the channel, with flexible metallic conduit. During driving, the cables and conduit can pass around a curved guide (a large-diameter pulley is suitable) mounted near the pile head, on the pile or hammer. A rope can be at-

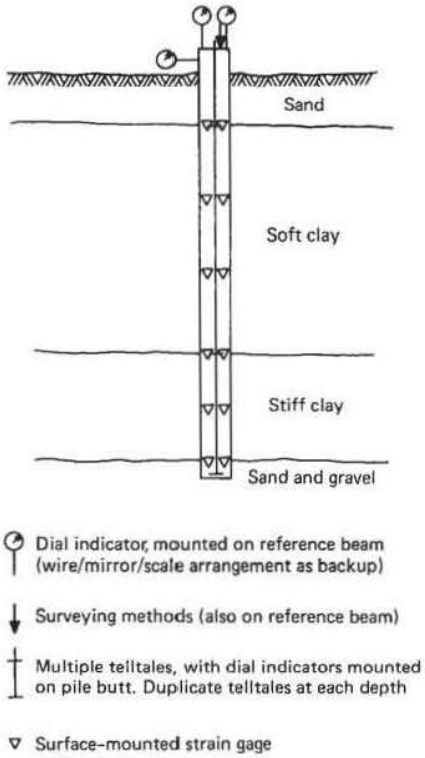

Figure 24.3. Possible layout of instrumentation during a static load test of a driven steel pile, loaded in axial compression, for the case where comprehensive load transfer data are required. Note: Loading arrangements, load transfer assembly, and reference beam are not shown. Also, a load cell is required. Use of surveying methods requires benchmark, remote from pile.

tached to the ends of the cables and held at the ground surface to prevent damage as the pile is driven. Special arrangements for cable protection must be made at any splices in the pile.

For concrete piles, strain data can be misleading, because they may be greatly influenced by cracks that develop in the concrete during driving. Ideally, several load cells should be installed along the length of the pile, but practical and economic considerations usually prevent this approach. When comprehensive load transfer data are required for concrete piles, both embedment strain gages and multiple telltales should be used, with full regard for their limitations. Conventional vibrating wire embedment strain gages are appropriate in small-diameter concrete piles, but in piles of 10 in. (250 mm) or greater diameter, the author favors use of the *sister bar* type of embedment strain gage, with a vibrating wire transducer. Very great care must be taken to prevent damage to gages and cables during driving, as discussed above for steel piles, and special care must also be taken in the casting yard. However, if the pile has a central hole for inspection or other purposes, or if a pipe can readily be cast on the axis of the pile, an alternative approach is possible for minimizing the damage potential. Weldable vibrating wire strain gages can be attached to a suitably sized steel pipe, angle, square tube, or plate and installed and grouted within the central hole after pile driving. A special effort must be made to ensure that the gages are subjected to the same strain as the concrete in the pile: the pipe used to form the central hole should be steel and not plastic, and the grout mix should be selected carefully. A mix of water and cement in a ratio of 1 : 0.3 by weight, with an expander additive, has been used successfully. This alternative approach does not allow determination of residual stresses but may nevertheless be the preferred approach in cases where there is a high potential for damaging cables and/or gages during pile driving. If a sliding micrometer is available, it can also be considered as an alternative (Carvalho and Kovari, 1983). Whenever possible, measurements in a concrete pile should include direct measurement of tip load, by installing a contact earth pressure cell at the tip.

As discussed in Chapter 13, special arrangements must be made for converting measured strain to stress in a prestressed concrete pile. Two methods are possible during an axial load test and are described below. Preferably, both methods should be used, to maximize confidence in the data. First, the top portion of the pile can be used as an *unconfined compression test specimen*. If possible, the length should be at least three times the diameter of the pile and should not include any length that may have been damaged during driving. Strain in the specimen is measured with strain gages, backed up with a pair of telltales if the specimen is long enough. Embedment strain gages can be used if the elevation to which the pile will be driven is known before driving; otherwise, surface-mounted gages can be used. The specimen can be entirely above the ground surface or within a sleeve just below the ground surface. Strain gages should be installed at the midheight of the specimen. If there is concern that gage readings may be influenced by damaged concrete or by end effects during loading, a second set of gages should be installed at or just below the lower end of the specimen. Because change in moisture content can cause significant strain, the specimen must be under the same moisture conditions as the length of the pile in which the other strain measurements are made. If that length is below the water table, the specimen should also be either below

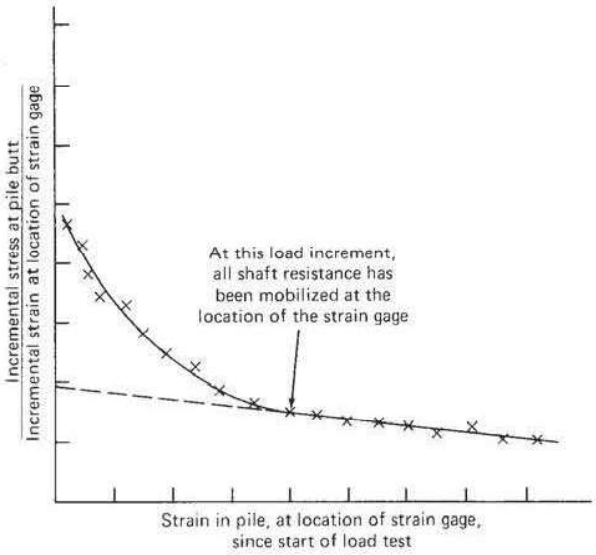

Figure 24.4. Typical plot for conversion of strain to stress in a prestressed concrete pile, using subsurface measurements of strain.

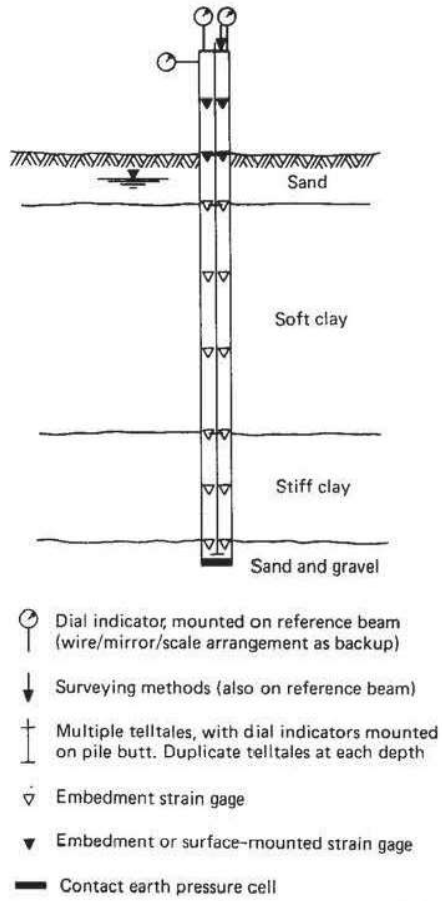

Figure 24.5. Possible layout of instrumentation during a static load test of a driven prestressed concrete pile, loaded in axial compression, for the case where comprehensive load transfer data are required. Note: Loading arrangements, load transfer assembly, and reference beam are not shown. Also, a load cell is required. Use of surveying methods requires benchmark, remote from pile. Top of pile should be kept wet.

the water table or kept wet by artificial means. The relationship between strain and stress is then determined during the load test, using strain data from the specimen and load data from the load cell at the pile butt. The method provides data for every load increment.

The second method for strain/stress conversion makes use of load cell data and measurements of subsurface strain, determined from strain gages or multiple telltales in the pile, plotted as shown in Figure 24.4. During the load test, when all shaft resistance has been mobilized at the location of the strain gage, the pile acts as a column in compression and the plot becomes straight. The straight line is likely to be sloping, because the modulus of the pile generally decreases as the pile is loaded. The slope of the straight line part of the plot, together with the value of the intercept on the vertical axis, are used to determine the relationship between strain and stress at the location of the strain gage. Fellenius (1987) describes the method in detail.

Figure 24.5 shows a possible layout of instrumentation during a static load test of a driven prestressed concrete pile, loaded in axial compression, for the case where comprehensive load transfer data are required.

Static Load Tests: Lateral Loading Mode

The procedure for static lateral load tests is described by ASTM (1981b). Load measurements at the pile butt require use of a load cell or cable tension meter. Deformation measurements at the pile butt require horizontal and vertical dial indicators and/or a wire/mirror/scale arrangement. Horizontal deformation measurements should be arranged to monitor whether or not the loading is causing torsion in the pile. Any movement of the reference beam should be measured throughout the test by surveying methods, and temperature effects should be minimized.

When bending data are required during a lateral load test, strain gages are installed on or near both faces of the pile in the plane of loading. Surface-mounted vibrating wire gages are normally preferred on steel piles, of the weldable type if installed on the outside and of the larger type with bolted end blocks if installed on the inside of a pipe pile. *Sister bars* with vibrating wire transducers are preferred

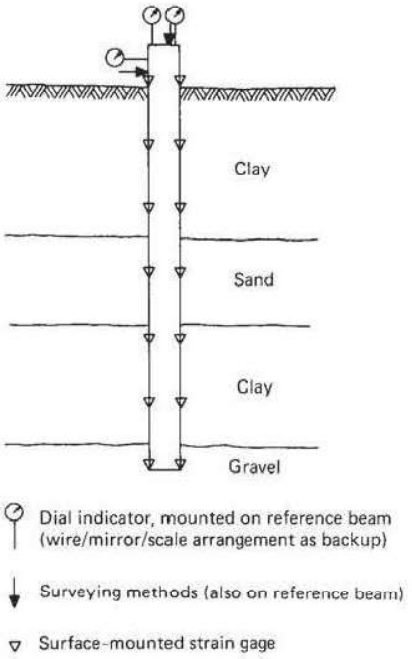

Figure 24.6. Possible layout of instrumentation during a static load test of a driven steel pile: lateral loading mode. Note: Loading arrangements and reference beam are not shown. Also, a load cell is required. Use of surveying methods requires benchmark, remote from pile.

in concrete piles of 10 in. (250 mm) or greater diameter. If the test is likely to develop significant lateral deformation at the butt (more than about 6 in., 150 mm), the opportunity exists to estimate bending stresses from inclinometer data as described in Chapter 12. However, this approach should only be used as backup. If it is used, inclinometer readings of maximum accuracy should be taken during each loading increment, and a tiltmeter should be attached to the butt so that more frequent readings of overall tilt can be made during the test. An inclinometer can also be used to determine the deformed shape of a pile at increments during a lateral load test.

When a lateral load test is conducted in cohesive soils such that deformation is affected by consolidation of soil on the compression side of the pile, piezometers in the soil will provide additional data for evaluation of the test.

Figure 24.6 shows a possible layout of instrumentation during a static lateral load test of a driven steel pile. This is only one example, and it must be emphasized that many other configurations are possible, as indicated by the case histories described later in this chapter.

Other Static Loading Modes

When moment, eccentric, torsional, or combined static loading modes are used, appropriate monitoring methods should be selected from those described for axial and lateral load tests. If loading is applied during moment or torsional tests by pulling on a steel cable, load will normally be monitored by using a cable tension meter. Torsional load tests may require the use of strain gages mounted horizontally to measure torsional strains.

Cyclic Loading Modes

Cyclic load tests require similar instrumentation to static tests, but the dynamic response of instrumentation must be sufficient to follow the loading sequence.

Remote Readings During Load Tests

During conventional load tests, instruments are normally read manually, but considerations of safety and/or convenience may lead to selection of instruments that can be read remotely and perhaps by an automatic data acquisition system. This option is particularly attractive when the number of instruments is large. DCDTs can be used instead of dial indicators to monitor deformation of the pile butt and telltales, and it is often worthwhile to use both DCDTs and dial indicators in parallel, to provide backup and to satisfy personnel who are concerned about trusting electrical devices.

24.2.2. What Is the Load–Movement Relationship of the Pile? (Determination of Relationship by Dynamic Testing)*

Static load tests are expensive and time consuming; therefore, the small number of piles that are tested may not be representative of the behavior of all piles at a site. As an alternative, the pile-driving system can be used as a loading system, or a load can be applied by use of a simple vibrator or hammer. These alternative methods allow dynamic load tests to be made on a large number of piles. Dynamic tests are much less expensive than static tests, primarily because a static loading system is not required. The recent development of on-site automatic data acquisition systems has caused a rapidly increasing application of analytic methods based on wave transmission.

*Written with the assistance of Allen G. Davis, Director, Testconsult CEBTP Ltd., Thelwall, England.

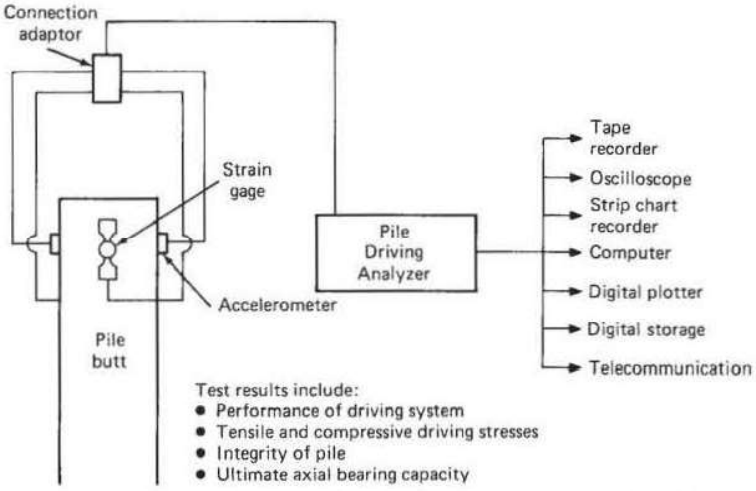

Figure 24.7. Schematic of Pile Driving Analyzer™ (courtesy of Pile Dynamics, Inc., Cleveland, OH).

Many successful correlations have been made between dynamic and static load tests. However, it is clear that much work still remains to be done to establish appropriate methods of dynamic testing and analysis, because pile performance is dependent on both strain rate and the magnitude of pore water pressure.

In the following paragraphs, three dynamic tests are described for determining axial load–movement relationships: *analysis of stress waves, steady-state vibration testing,* and *transient dynamic response testing*. The last two tests can also be used in the horizontal mode for determining lateral load–movement relationships.

Analysis of Stress Waves

The most commonly used and effective method for determining the axial load–movement relationships of driven piles by dynamic testing involves analysis of the stress wave during driving. The method, which has become known as the *Case Method,* after research programs conducted at Case Western Reserve University, requires measurement of strain and acceleration at the pile butt as the pile is driven, using a *Pile Driving Analyzer*™. By introducing these parameters into a pile–soil model based on a closed-form analysis of stress wave transmission through an elastic medium, an ultimate axial bearing capacity can be calculated for the pile under test.

Figure 24.7 shows a schematic of the test arrangement and indicates some of the test results.

The model requires a knowledge of the elastic properties of the pile, and a selection of soil damping constants must also be made, depending primarily on soil type. The recorded data may be processed by using an iterative analysis routine known as *CAPWAP*. This analysis, which is based on the wave equation theory as originally developed by Smith (1960), provides a refined prediction of pile behavior, including the load–movement relationships, and aids in evaluating the correctness of the soil damping constants. Rausche et al. (1985) describe the method and indicate correlations with static testing.

Axial Steady-State Vibration and Transient Dynamic Response Testing

Stain (1982) describes *steady-state vibration* and *transient dynamic response testing*.

Axial steady-state vibration testing involves attaching a vibrator to the pile butt to cause the butt to move up and down at the same frequency as the vibrator. A load cell is placed between the pile butt and the vibrator, and the velocity of movement is measured with an accelerometer, as shown in Figure 24.8. The measured axial stiffness of the pile butt allows calculation of the displacement of the butt under a given load (Davis and Dunn, 1974).

Axial steady-state vibration testing has largely been superseded by a development referred to as *transient dynamic response testing, TDR, shock testing,* or *impact testing*. As shown in Figure 24.9, the method involves placing a load cell centrally on the pile butt, striking the load cell with a hammer, and measuring velocity with a geophone.

Steady-state vibration and TDR tests were developed principally for drilled shafts, initially for concrete integrity determination. During their use in

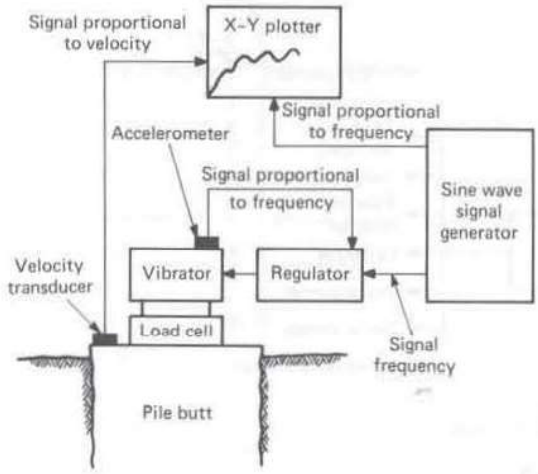

Figure 24.8. Schematic of axial steady-state vibration test.

the 1970s, it became apparent that the measurement of axial dynamic stiffness at the pile butt was a useful parameter in assessing performance under axial working load. Many correlations have been made between axial dynamic stiffness at the pile butt and the linear portion of the load–settlement curve as determined during static load testing in axial compression (Davis and Robertson, 1976). However, the tests cannot be used to estimate ultimate static axial capacity: to do so it is necessary to displace the tip of the pile, and neither method gives adequate input force.

Lateral Steady-State Vibration and Transient Dynamic Response Testing

S.A. Robertson (1979) describes a dynamic method for determining the lateral load–movement relationships of driven piles. The method is a development of the axial steady-state vibration test. It involves shaking the pile butt horizontally at controlled frequencies between 1 and 10 Hz and measuring the *lateral* velocity of the butt as a function of frequency. The lateral dynamic stiffness at the butt can be measured as in the axial steady-state vibration test, thus giving the displacement of the pile butt under a given load. If a vertical tube is built into the pile, a vertical profile of displacement for a given applied force can be obtained by measuring the pile velocity at different depths. The TDR test, used in the horizontal mode, has been demonstrated to give identical results to the lateral steady-state vibration test.

Comparison Among Dynamic Tests for Determining Load–Movement Relationships

When both the load–movement relationship and the ultimate axial bearing capacity of a driven pile are required, the only practical dynamic method to date is the Case method. The dependency on strain rate and magnitude of pore water pressure must be remembered when analyzing the results.

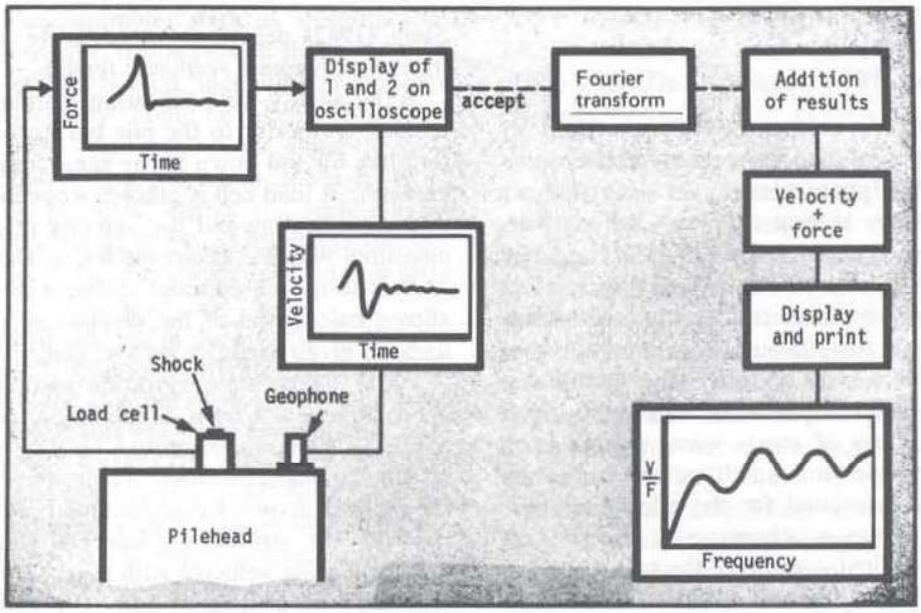

Figure 24.9. Schematic of axial transient dynamic response test (after Stain, 1982).

The Case method creates a higher strain level than the TDR test; therefore, it is generally preferred when only the axial load–movement relationships are required. However, the TDR test is a viable alternative, and it can be used rapidly and inexpensively to determine the axial or lateral load–movement relationships of all piles at a site. Although measurements are made at low strains only, results of the TDR test can be correlated with results of a static load test on each size and type of pile in a population. The TDR test has the great advantage of relatively low cost, and the test can be made without involving the pile-driving rig. The steady-state vibration test has largely been superseded by the TDR test.

24.2.3. Will There Be Significant Downdrag Loading?

When soil settles with respect to a pile, frictional forces between the soil and pile may cause a load increase in end bearing piles or increased settlement of friction piles. The loading is referred to as *downdrag* or *negative skin friction*. Possible causes of downdrag loading are described in Chapter 2.

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

For steel piles, adequate load data can be obtained by measuring strain, preferably by using vibrating wire strain gages, and very great care must be taken to prevent damage to gages and cables during driving. Despite the limitations discussed in Section 24.2.1, multiple telltales are recommended as backup. Figure 24.10 shows a possible layout of instrumentation during a full-scale test to determine downdrag loading on a steel pile. This is only one example, and it must be emphasized that many other configurations are possible: other configurations are indicated in the case histories described later in this chapter.

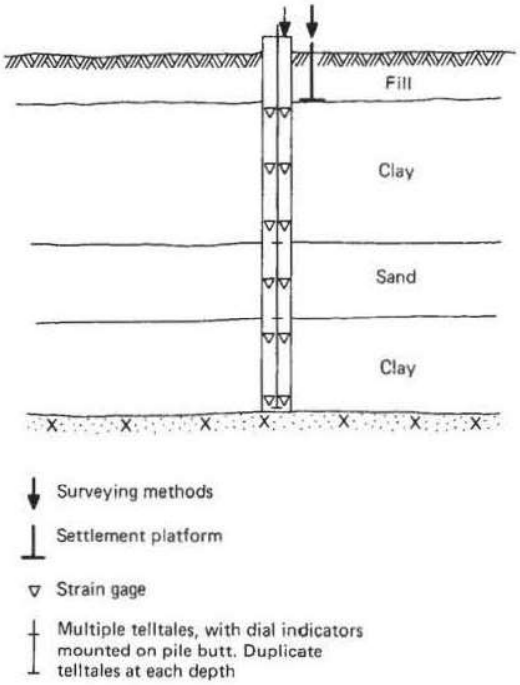

Figure 24.10. Possible layout of instrumentation during a full-scale test to determine downdrag loading on a steel pile. Note: Use of surveying methods requires benchmark, remote from pile.

For driven prestressed concrete piles, the methods described in Section 24.2.1 for converting strain to stress cannot be used, because no load is applied to the pile butt. Measurements of modulus made while load testing other similar piles also cannot be used, because a typical test to evaluate downdrag loading will be conducted for several weeks or months, during which time significant creep will usually occur. Load determinations in concrete piles must therefore include at least one **direct** measurement of load, by incorporating a load cell in a pile splice and/or by attaching a contact earth pressure cell to the pile tip. Because instruments for direct measurements of load are expensive, their number should be limited. If the soil profile is uniform, a single instrument deep down in the pile may be adequate, but in the general case measurements are preferable at two or three depths, to provide redundancy and to improve the accuracy of data evaluation. Load data should

be supplemented by installing embedment strain gages, and multiple telltales should be used as backup, duplicating telltales at each depth as described earlier. Conventional vibrating wire embedment strain gages are appropriate in small-diameter concrete piles, but in piles of 10 in. (250 mm) or greater diameter, the author favors use of the *sister bar* type of embedment strain gage, with a vibrating wire transducer. The relationship between load and strain is determined, at points where load is measured directly, by relating load to strain measured near those points. The same relationship is applied to strains measured elsewhere.

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

For a full understanding of soil–pile interaction, it is necessary also to determine pore water pressure in the soil and effective stress at the soil/pile interface. Specially manufactured pneumatic piezometers and contact earth pressure cells have been used successfully for determining effective stress at the soil/pile interface (e.g., Clemente, 1979).

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

Many building codes impose allowable limits on the curvature of piles, because excessive bending reduces capacity and increases deformation of piles under long-term loading. Capacity is also reduced if piles are dog-legged. The curvature of piles can be determined by using an inclinometer. When used in pipe piles, the inclinometer can be fitted with long arms and guide wheels that ride on the inside wall of the pipe and can be lowered down the pile on orientation rods. When used in other types of pile, a steel or plastic guide pipe is required, attached to or embedded in the pile. For an approximate determination of curvature, the shear probe (Chapter 12) can be used within a guide pipe.

The application of the shear probe procedure can be extended to become part of the design/construct engineering package. A steel or plastic guide pipe, usually referred to as an *inspection pipe,* can be specified in or on all piles, and information on the condition of the pile below the ground surface can be obtained by using the shear probe as part of the routine inspection procedure. The specification includes acceptance/rejection criteria, a major design uncertainty is resolved, and the designer can allow a higher load per pile and therefore fewer piles than would be allowed without the inspection pipes.

Table 24.1. Types of Integrity Test

Category	Test
Tests carried out at pile butt	Analysis of stress waves (Case method)
	Echo test
	Axial transient dynamic response test
	Axial steady-state vibration test
Tests carried out within pipes in pile	Sonic coring
	Nuclear density probe, single-hole
	Nuclear density probe, cross-hole

The condition of a driven pile can also be determined by static load tests, as described above. However, they are too expensive for use as routine quality control tests.

The condition of a driven pile can also be determined by *integrity testing*.* Integrity tests are listed in Table 24.1 and described in the following subsections. They are usually applied to concrete piles but some are also valid for wood and steel piles.

Analysis of Stress Waves

As indicated in Figure 24.7, the *Case method* can be used to determine integrity of driven piles.

Echo Test

The *echo test* (Stain, 1982), also referred to as a *surface reflection test* and *seismic test,* involves striking the pile butt with a hammer and measuring the time for the resulting compression wave to travel down the pile and back to the butt by reflection from the tip. The arrangement is shown in Figure 24.11. For a given quality of concrete, the approximate wave velocity is known; therefore, breaks or other irregularities will be indicated by a

*The text on integrity testing has been written with the assistance of Allen G. Davis, Director, Testconsult CEBTP Ltd., Thelwall, England.

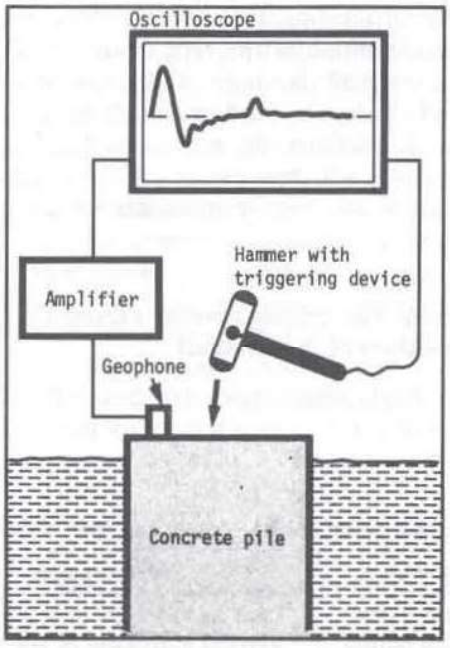

Figure 24.11. Schematic of echo test (after Stain, 1982).

short travel time. The method is rapid and inexpensive but cannot be used if there are splices in the pile. Damping of the signal causes a further limitation to the use of echo testing, particularly for long, slender piles or for piles driven into stiff soils. In stiff clays, for example, the pile tip can rarely be detected when the pile length/diameter ratio exceeds 30/1.

Axial TDR and Steady-State Vibration Tests

The *TDR* test has been used to test the integrity of driven piles of all types (Higgs and Robertson, 1979). Although having the same limitations as the echo test, measurements of axial dynamic stiffness can be used to compare piles within a population. The *axial steady-state vibration test* can also be used but has largely been superseded by the TDR test.

Sonic Coring

Sonic coring (Stain, 1982), also referred to as *sonic logging*, requires measuring the propagation time of a sonic signal between a transmitter and a receiver. Measurements can be made in a single hole on the axis of a pile, but the effective sampling area is small and significant defects may be missed. Reliable data can be obtained by cross-hole measurements between two metal pipes cast parallel to the

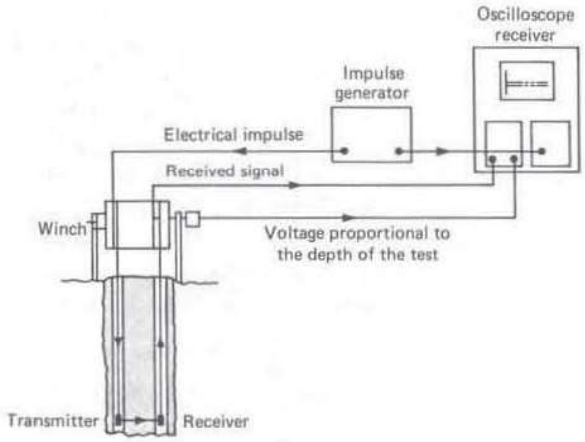

Figure 24.12. Schematic of sonic coring with cross-hole measurements (after Davis and Robertson, 1975).

axis of the pile during construction, as shown in Figure 24.12. Plastic pipes are not suitable, because the bond between concrete and plastic is sometimes imperfect, resulting in a loss of signal that can suggest that a fault is present in the pile. An emitter probe and receiver probe are lowered, one in each pipe, and measurements are made as the probes are raised in unison. Variations in sonic velocity indicate defects. Two, three, or four pipes are used, depending on the diameter of the pile. Sonic coring is not often used for testing driven piles because of the requirement for pipes in the piles: its primary application is for testing drilled shafts and slurry trench (diaphragm) walls.

Nuclear Density Probe

A nuclear density probe can be traversed along an embedded pipe to examine the integrity of concrete. Preiss and Caiserman (1975) describe use of a probe in a single pipe of plastic or metal, using the backscatter technique, which allows a relatively weak source to be used. Photons emitted from the source are partially absorbed by the material in the pile, partially scattered toward the detector, and counted. The count rate is indicative of the density of the material surrounding the probe. The arrangement is shown in Figure 24.13. The Preiss and Caiserman equipment allows a radial extent of inspection about the pipe of approximately 4 in. (100 mm), and measurements can be made within several individual pipes. Alternatively, cross-hole measurements can be made, using a transmission method, with the source and detector in different pipes. Cross-hole measurements require a stronger

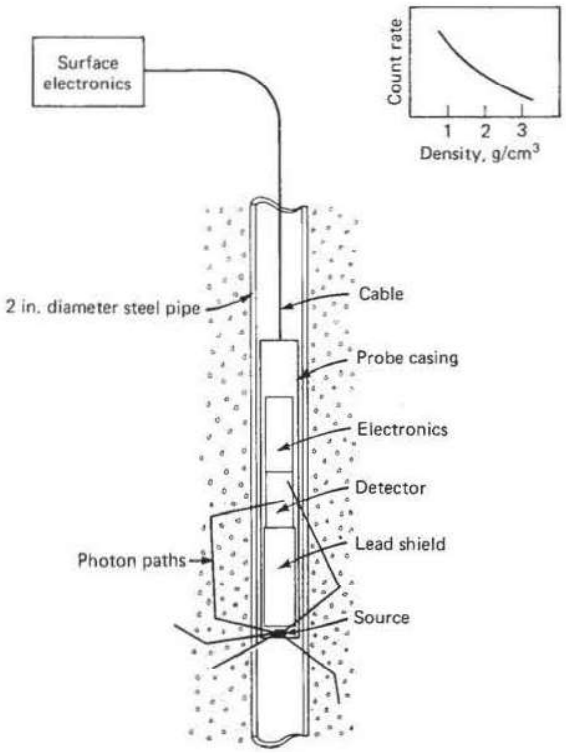

Figure 24.13. Schematic of nuclear density probe in single hole, using backscatter technique (after Preiss and Caiserman, 1975).

source than single-hole measurements and are limited to a cross-hole spacing of about 24 in. (600 mm).

Comparison Among Integrity Tests

The four tests carried out at the pile butt will be considered first. Driven precast concrete piles usually have a relatively high length/diameter ratio, and most integrity problems arise during handling and driving. These problems are best detected by monitoring during driving using the Case method. However, because of the need to instrument each pile butt during driving, this is a relatively expensive approach. As an alternative, integrity testing can be carried out at the pile butt after driving by using the echo or TDR method. As indicated earlier, the TDR method has largely superseded axial steady-state vibration testing. The TDR method gives more information than the echo method, including a value for axial dynamic stiffness and more detail of any defects within the top 6 ft (2 m).

In considering the three techniques carried out within pipes, these methods are more costly than those carried out at the pile butt. However, they are capable of locating **any** defect and giving a much greater definition of the type of defect. The sonic coring method is more able than the nuclear methods to locate small-scale defects such as fine cracks. In addition, the nuclear methods are sometimes restricted, because of problems inherent in the use of radioactive materials on construction sites.

24.2.5. Is Pile Driving Causing Excessive Lateral Deformation of the Ground?

When displacement piles are driven in soft clay, there will normally be a temporary increase of pore

Table 24.2 Overview of Routine and Special Monitoring

Application	Measurement
Routine monitoring during static axial load testing	Load and movement at pile butt Vertical movement of pile tip
Routine monitoring during static lateral load testing	Load and movement at pile butt
Special load testing applications	Load at pile tip during axial load testing Axial stress along length of pile during axial load testing Bending stress along length of pile during lateral load testing Moment, eccentric, torsional, or combined static loading modes: appropriate parameters as for static axial and lateral modes Load, velocity, strain, and/or acceleration as required for dynamic tests Deformation and stress during full-scale test to determine downdrag loading Excess pore water pressure
Other special applications	Integrity of pile As-driven alignment along length of pile Lateral deformation of ground and excess pore water pressure during pile driving Surface settlement and/or groundwater pressure for evaluation of downdrag loading

water pressure and a reduction of soil strength, perhaps leading to excessive lateral deformation and perhaps even a stability failure. Examples include pile driving on a slope that already has a low factor of safety and pile driving nearby an existing structure or nearby other piles. When this potential exists, an instrumentation program can be conducted to quantify displacements and pore water pressure changes, thus allowing adoption of a driving procedure or sequence that minimizes detrimental effects. Primary instruments used for this purpose include surveying methods, inclinometers or shear probes in the piles and/or soil, and piezometers.

24.3. OVERVIEW OF ROUTINE AND SPECIAL APPLICATIONS

For those readers who seek general guidelines on what is typically *routine* monitoring and what is monitoring for *special applications,* Table 24.2 presents a broad generalization.

In practice, the most frequent use of instrumentation for driven piles is to examine load–movement relationships during static axial load testing.

24.4. SELECTED CASE HISTORIES

Table 24.3 summarizes selected case histories of instrumented driven piles.

Table 24.3. Summary of Selected Case Histories of Driven Piles

Project	Principal Concerns	Type of Load Test	Type of Pile	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Newport News Shipyard, Tidewater, VA, U.S.A.	Load–movement relationships	Static axial compression	Prestressed concrete	L S G	Load cell Strain gages Vibrating wire piezometers		Martin et al. (1987)
I75 near Tacoma, WA, U.S.A.	Load–movement relationships	Static axial compression	Steel H	D D,S L S	LVDTs Multiple telltales Load cell Strain gages		Rieke and Crowser (1986, 1987)
Various	Load–movement relationships	Static axial compression, uplift, lateral	Steel H and pipe	D D S	Telltales Inclinometers Strain gages	29 tests, conducted 1954–1984	AISI (1985)
Mar Pequeno Highway Bridge, São Paulo, Brazil	Load–movement relationships	Static axial compression	Large-diameter steel (650 mm, 26 in.) in silty clay and fine sand	D D S	Surveying methods Dial indicators Sliding micrometer	Load transfer data determined using sliding micrometer	Carvalho and Kovari (1983)
Various	Load–movement relationships	Static axial compression, uplift	Various	D D L S TS G	LVDTs Telltales Load cells Strain gages Contact earth pressure cells Piezometers	130 tests	Kulhawy et al. (1983)
Terminal 46, Port of Seattle, WA, U.S.A.	Stability of sloping ground during pile driving	—	Prestressed concrete in fill and loose silt	D G	Inclinometers Pneumatic piezometers	Prediction and use of hazard warning levels	Fellenius et al. (1982)
Group load test, Houston, TX, U.S.A.	Behavior of pile group in overconsolidated clay	Static axial compression: research test	Steel pipe Single and group of nine in overconsolidated clay	D,S D D S TS	Multiple telltales Subsurface settlement points Inclinometers Strain gages Contact earth pressure cells	Very comprehensive instrumentation	O'Neill et al. (1982a,b)

Table 24.3. Summary of Selected Case Histories of Driven Piles (*Continued*)

Project	Principal Concerns	Type of Load Test	Type of Pile	Measurement[a]	Instrumentation Discussed	Special Features	Reference
				G	Pneumatic piezometers in soil and pile		
Highway bridges between Berthierville and Yamachiche, Quebec, Canada	Load–movement relationships Performance of bridge pier foundations	Static axial compression, single and group	Timber, precast concrete, and steel pipe in soft sensitive clay	D D G	Surveying methods Subsurface settlement points Pneumatic piezometers in soil		Blanchet et al. (1980)
Test site at Hendon, England	Mechanism of load transfer from piles in stiff clay Behavior of piles at close spacing in stiff clay	Measurements made while jacking piles into clay	Tubular steel in overconsolidated clay	D D D L	LVDTs Subsurface settlement points Horizontal inclinometers with electrolytic level transducers Load cells within piles and at pile butts	Very comprehensive instrumentation	Cooke et al. (1980, 1979)
Bridge at Fredericton, New Brunswick, Canada	Load–movement relationships	Static axial compression, uplift	Steel H and pipe in granular fill and clayey silt	D D,S L	LVDTs Multiple telltales Load cell		Bozozuk et al. (1979)
Keehi Interchange, Honolulu, HI, U.S.A.	Downdrag loading	Downdrag	Prestressed concrete in soft underconsolidated clay, sand, and silty clay	D D,S D D D D S TS G G	DCDTs Multiple telltales Settlement platforms Subsurface settlement points Probe extensometers Inclinometer Strain gages Contact earth pressure cells Pneumatic piezometers in soil and pile Observation wells	Tests on bitumen-coated and uncoated piles Very comprehensive instrumentation	Clemente (1979)
Metropolitan Sewage Treatment Plant, Syracuse, NY, U.S.A.	Load–movement relationships	Static axial compression	Prestressed concrete in sand and silty clay	D D,S D L S	Dial indicators Multiple telltales Inclinometer Load cell Strain gages		Lacy (1979)
Fuel Reprocessing Facility, West Valley, NY, U.S.A.	Load–movement relationships	Static axial compression, cyclic axial and lateral	Steel H in glacial till	D D L S	Tiltmeters Inclinometer Load cells Strain gages	Load applied by automated closed-loop system	Lu et al. (1979)

Table 24.3. Summary of Selected Case Histories of Driven Piles (*Continued*)

Project	Principal Concerns	Type of Load Test	Type of Pile	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Test Site at Brent, England	Load–movement relationships	Static and cyclic lateral	Steel pipe in weathered overconsolidated clay	D D S	Tiltmeters Electrical linear displacement transducers Strain gages		Price (1979)
Test Site at Hendon, England	Behavior of soil around friction piles	Static axial compression: research test	Steel pipe in overconsolidated clay	D D D L	Tiltmeters Subsurface settlement points Inclinometer Load cells	Measurements also made while jacking pile into clay	Cooke and Price (1978)
Test Site at Lewisburg, PA, U.S.A.	Development of improved design procedures for laterally loaded pile groups	Static lateral: research test	Steel H Single and group of six in clay	D S	Inclinometer Strain gages		Kim and Brungraber (1976)
Test Site at Melbourne, Australia	Downdrag loading	Downdrag: research test	Steel pipe in sand, clay, and gravel	D,S D S G T	Multiple telltales Settlement platforms Strain gages Piezometers Thermocouples	Tests on bitumen-coated and uncoated piles	Walker and Darvall (1973)
Quebec Autoroute, Berthierville, Quebec, Canada	Downdrag loading Load–movement relationships	Downdrag and static axial compression: research test	Steel pipe in marine clay	D,S D L S G	Multiple telltales Subsurface settlement points Load cell Strain gages Open standpipe piezometers	Pile filled with concrete and load tested after 10 years of downdrag measurements	Bozozuk (1972a, 1981)
Test Site at Gothenburg, Sweden	Downdrag loading	Downdrag	Precast concrete in soft clay, silt, and sand	D L G	Probe extensometers Load cells Piezometers	Vibrating wire load cells installed at splices in piles	Fellenius (1972) (Fellenius and Broms (1969)
Arkansas River Project, Arkansas, U.S.A.	Load–movement relationships	Static axial compression, uplift, lateral	Steel H and pipe, concrete, and timber in sand	D,S S	Multiple telltales Strain gages	Various pile lengths, diameters, and driving procedures	Mansur and Hunter (1970)
Sörenga and Heröya, Norway	Downdrag loading	Downdrag	Steel pipe driven to rock	S	Telltales	Tests with bentonite slurry, cathodic protection, and bitumen coating	Bjerrum et al. (1969)
Test site in Norway	Downdrag loading	Downdrag	Steel pipe driven to rock	D,S D D D G	Multiple telltales Settlement platforms Subsurface settlement points Inclinometers Piezometers		Johannessen and Bjerrum (1965)
Old River Control Project, Louisiana, U.S.A.	Load–movement relationships	Static axial compression and uplift	Steel H and pipe in sand, silt, and clay	S	Multiple telltales		Mansur and Kaufman (1956)

[a]D: deformation L: load in pile T: temperature
G: groundwater pressure S: strain in pile TS: total stress between pile and soil

CHAPTER 25

DRILLED SHAFTS*

25.1. GENERAL ROLE OF INSTRUMENTATION

Drilled shafts (also referred to as *drilled piers, caissons, drilled piles, bored piles, drilled caissons,* and *cast-in-place piles*) are constructed by augering a hole into the ground, placing a reinforcing cage, and backfilling the hole with concrete. A temporary outer casing may be required. Water or drilling mud may be used during the construction process, and in this case the water or drilling mud is displaced by tremie concrete as the casing is withdrawn. Construction procedures are described by Reese (1978).

Figure 25.1 shows drilled shafts bearing on bedrock: the figure conveys the contrast between how the project will appear on engineering drawings and what may actually happen in the field. Primary mechanisms that control the behavior of drilled shafts are outlined in Chapter 2.

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

*Written with the assistance of Fred H. Kulhawy, Professor, School of Civil and Environmental Engineering, Cornell University, Ithaca, NY.

25.2. PRINCIPAL GEOTECHNICAL QUESTIONS

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

25.2.1. What Is the Load–Movement Relationship of the Drilled Shaft?

The load–movement relationship of a drilled shaft will be uncertain during the design phase, and one or more load tests may be conducted either during the design phase or at the beginning of the construction phase. A load test allows application of load to the butt (head) of a shaft, so that the actual response to load can be verified experimentally. The loading mode may be axial compression or uplift, lateral. eccentric, moment, torsional, or a combination of these modes. Guidelines for conducting load tests are currently being prepared by Cornell University under the sponsorship of the Electric Power Research Institute, with an estimated publication date during 1988. This will be a comprehensive and important reference document.

Application of sufficient load to large drilled shafts can be expensive; therefore, the number of load tests is limited, and instrumentation programs

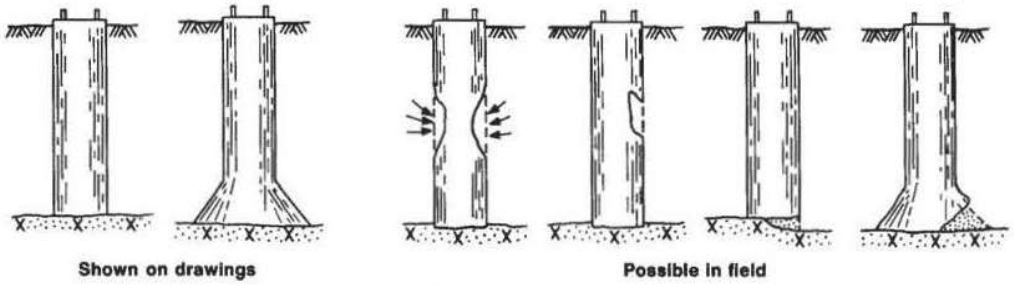

Figure 25.1. Drilled shafts (after DiBiagio and Myrvoll, 1981).

should be planned to provide comprehensive performance data. The cost of instrumentation is usually a minor part of the total cost.

Static Load Tests: Loading in Axial Compression and Uplift

Kulhawy et al. (1983) summarize 410 published and unpublished static load tests in axial compression or uplift, indicating that most such tests are conducted while measuring load and deformation at the shaft butt only. Such limited instrumentation does not provide comprehensive performance data, and subsurface measurements should also be made.

Measurement methods at the shaft butt are similar to those described for driven piles in Section 24.2.1. Vertical deformation is measured using dial indicators, often with a wire/mirror/scale arrangement as backup. Measurements of horizontal deformation, using similar methods, indicate whether or not the applied load is axial and concentric. As for driven piles, any movement of the reference beam should be measured throughout the test by surveying methods, and great care should be taken to minimize the effect of temperature changes on the reference beam. Load measurements require the use of a load cell.

Measurements of load in the shaft and at the tip should always be made when load testing large drilled shafts that have an anticipated capacity in excess of about 200 tons (1.8 MN). When conducting a load test to provide a basis for an efficient design, one cannot interpret the overall performance of the system completely unless load transfer data are available. When a load test is conducted during the construction period, as a proof test, the additional cost of subsurface instrumentation is minor. Load in the shaft should be measured using embedment strain gages, with a preference for the *sister bar* type with vibrating wire transducers. If a sliding micrometer is available, it can be used as an alternative to strain gages. Multiple telltales can be used as backup to strain gages, and the longest telltale provides data for tip settlement. Figure 12.106 shows one of a group of multiple telltales used during a recent static load test with which the author was involved. When planning multiple telltale installations and interpreting data, one must pay careful attention to the guidelines given in Section 13.3.1. Because the accuracy of loads that are calculated from telltale data is highly dependent on the quality of raw data, duplicate telltales should be installed at each depth. Data for conversion of strain to stress are provided as described in Chapter 24 for driven piles: by gaging an *unconfined compression test specimen* within a sleeve or above the ground surface, supplemented by use of strain data after all shaft resistance has been mobilized at a particular depth. However, for a drilled shaft it is not normally reasonable to leave a length of shaft above the ground surface, equal to three shaft diameters, and end effects in the *specimen* must be accounted for by averaging between several strain gages installed at one or more cross sections.

An attempt can be made to determine load at the tip from either strain gage or multiple telltale data. However, both are subject to inaccuracies in converting strain to stress, and use of multiple telltales involves an extrapolation. It is recommended that a hard data point be obtained by installing a contact earth pressure cell at the tip. Suitable cells are described in Chapter 10.

Figure 25.2 shows a possible layout of instrumentation during a static load test, loaded in axial compression. This is only one example, and it must be emphasized that many other configurations are possible, as indicated by the case histories described later in this chapter. A similar layout is applicable to a static load test, loaded in axial uplift, except that the contact earth pressure cell would be

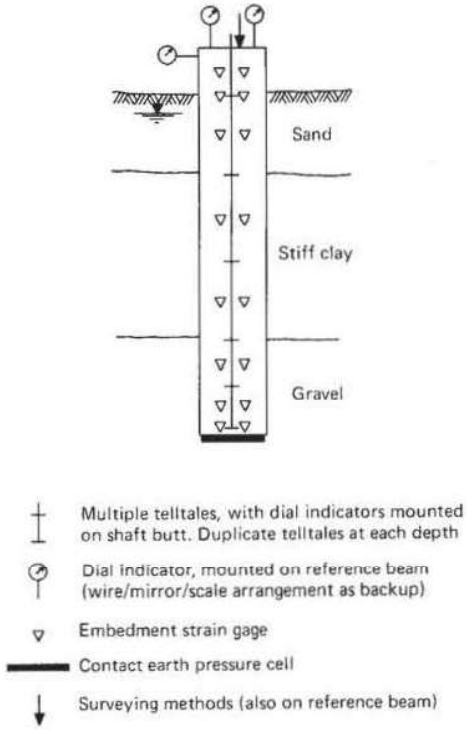

Figure 25.2. Possible layout of instrumentation during static load test: loading in axial compression. Note: Loading arrangements, load transfer assembly, and reference beam are not shown. Also, a load cell is required. Use of surveying methods requires benchmark, remote from shaft. Top of shaft should be kept wet.

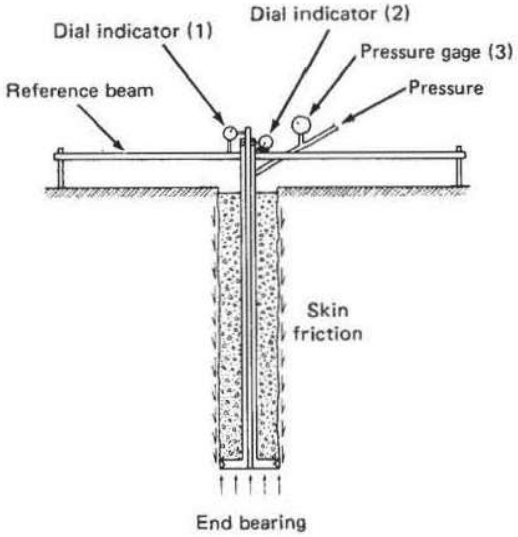

Figure 25.3. Arrangement for load testing a drilled shaft by applying load at the bottom (after Osterberg, 1984). Reprinted with the permission of the ADSC: an international association of foundation drilling contractors.

omitted. However, it should be recognized that determinations of stress from strain data during an uplift test are subject to significant errors if the concrete cracks. For this reason some engineers have installed surface-mounted electrical resistance strain gages on the reinforcing cage in preference to installing *sister bars* or other types of embedment strain gage. The author prefers use of *sister bars,* because well-designed sister bars will give the same data as gages on the reinforcing cage, and they are not subject to errors caused by installing gages in a field environment.

As an alternative to a hydraulic jack for load application, Horvath (1985) used three flatjacks, positioned in series with intervening bearing plates, thereby ensuring that the applied load was axial and concentric. The composite system was calibrated prior to use in the field. During a load test, one flatjack was inflated for load application and also used for load measurement. When the maximum aperture of the jack was reached, its valves were closed, the jack subsequently used as a passive load cell, and another flatjack activated for load application. Although this procedure is not accepted as a standard, it does present an interesting alternative.

Osterberg (1984) describes an innovative method for determining the load–movement relationship of a drilled shaft under static axial load by applying load at the bottom of the shaft. As shown in Figure 25.3, a special flatjack is placed at the bottom of the shaft before the concrete is poured. After the concrete has cured, the flatjack is pressurized internally, creating an upward force on the shaft and an equal but opposite downward force in end bearing. Attached to the bottom of the device is a steel pipe that extends upward within a steel sleeve. As pressure is applied, the inner pipe moves downward, and the movement is measured by using dial indicator (1). Dial indicator (2) is used to measure the upward movement of the shaft, and a pressure gage (3) is used to measure the internal pressure in the flatjack: this pressure is related to the total load during a separate calibration. Failure will occur either in skin friction or in end bearing, and the test is finished. Osterberg indicates that if the same failure load is applied as a downward load at the top of the shaft, the factor of safety against failure will be greater than 2 since, after allowing for the weight of the shaft, the skin friction and end bearing now both act upward to resist the downward load. When the test is completed, the flatjack is left in the hole.

Clearly, this test is considerably less expensive than a conventional load test, but additional comparative tests will be needed before its general applicability can be accepted. During the test, the major transfer of load occurs in the lower length of the shaft, the opposite of a foundation loaded in compression; thus, the load–movement pattern is different. This different pattern of load transfer distribution will be dependent on variation in soil type and on the state of in situ stress and requires further examination. Additional testing is planned.

Static Load Tests: Lateral Loading Mode

Measurement methods at the shaft butt include the use of a load cell or cable tension meter and horizontal and vertical dial indicators. A wire/mirror/scale arrangement is often used as backup. Horizontal deformation measurements should be arranged to monitor whether or not the loading is causing torsion in the shaft. Any movement of the reference beam should be measured throughout the test by surveying methods, and temperature effects should be minimized.

When subjected to lateral loads, the behavior of a drilled shaft is largely controlled by the flexural stiffness of the shaft in relation to the stiffness of the surrounding ground. When the shaft is rigid or short, it is often assumed to be infinitely stiff and to rotate as a rigid body, and failure usually occurs when the supporting capacity of the surrounding ground is reached. On the other hand, a flexible or long shaft exhibits elastic behavior; failure is usually structural and occurs when a plastic hinge forms at the point of maximum bending moment. Design of flexible or long shafts usually requires iterative techniques because the ground response depends on the magnitude of deformation. The designer of drilled shafts subjected to lateral loading must estimate the relative flexibility of the ground and shaft before deciding whether to use rigid or elastic design procedures. The information available to assist with this decision is very limited, and until a sufficient bank of data has been accumulated, load tests should include, wherever possible, determinations of bending stress in the shaft. Embedment strain gages can be used, again with a preference for the *sister bar* type with vibrating wire transducers, and again with limitations if concrete cracks. If a sliding micrometer is available, it can be used as an alternative to strain gages. A well-conducted test could develop up to 6 in. (150 mm) or more of lateral deformation at the butt; therefore, the opportunity exists to estimate bending stresses from inclinometer data as described in Chapter 12. However, this approach should only be used as backup. If it is used, inclinometer readings of maximum accuracy should be taken during each loading increment.

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

When a test with a lateral loading mode is conducted to determine how the soil reaction varies with depth, contact earth pressure cells can sometimes be installed at the interface between the shaft and the ground. However, this is possible only when constructing the shaft in an uncased hole. If casing is used, the only feasible approach is installation of an in situ testing device, such as a pressuremeter or dilatometer, in the ground alongside the shaft.

When a test is conducted in cohesive soils, such that deformation is affected by consolidation of soil on the compression side of the shaft, piezometers in the soil will provide additional data for evaluation of the test.

Figure 25.4 shows a possible layout of instrumentation during a static load test with a lateral loading mode.

Other Static Loading Modes

When moment, eccentric, torsional, or combined static loading modes are used, appropriate monitoring methods should be selected from those described for axial and lateral load tests. If loading is applied during moment or torsional tests by pulling on a steel cable, load will normally be monitored by using a cable tension meter. Torsional load tests may require use of embedment strain gages mounted horizontally to measure torsional strains.

Cyclic Loading Modes

Cyclic load tests require similar instrumentation to static tests, but the dynamic response of instrumentation must be sufficient to follow the loading sequence.

Remote Readings During Load Tests

During conventional load tests, instruments are normally read manually, but considerations of safety and/or convenience may lead to selection of

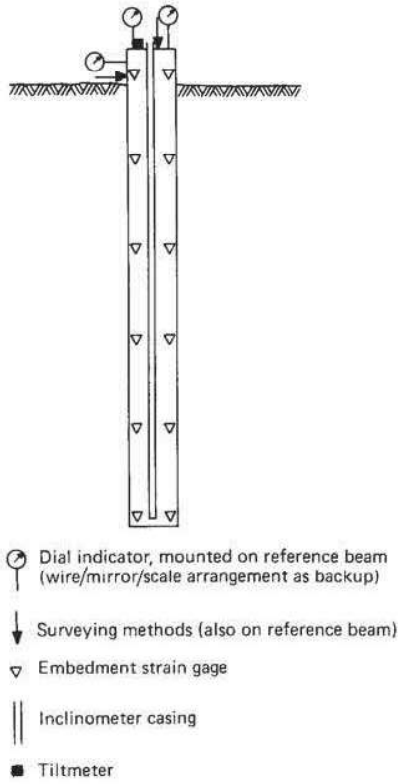

Figure 25.4. Possible layout of instrumentation during static load test: lateral loading mode. Note: Loading arrangements and reference beam are not shown. Also, a load cell or cable tension meter is required. Use of surveying methods requires benchmark, remote from shaft.

instruments that can be read remotely and perhaps by an automatic data acquisition system. This option is particularly attractive when the number of instruments is large. DCDTs can be used instead of dial indicators to monitor deformation of the shaft butt and telltales, and it is often worthwhile to use both DCDTs and dial indicators in parallel, to provide backup and to satisfy personnel who are concerned about trusting electrical devices.

*Dynamic Tests for Determining Load–Movement Relationships**

As discussed in Chapter 24 for driven piles, static load tests are expensive and time consuming, and less expensive dynamic tests are available. The *Case method* is not appropriate for drilled shafts, because no pile-driving hammer is used. *Transient dynamic response testing (TDR)* and *steady-state vibration testing,* described for driven piles in Chapter 24, are suitable for the determination of static axial and lateral load–movement relationships of drilled shafts. However, as for driven piles, the tests cannot be used to estimate ultimate static axial capacity, as neither method gives adequate input force.

An additional test, referred to as the *falling weight test,* has been developed specifically for determining the static axial load–movement relationships of drilled shafts and extends the application of stress wave methods to cases where no pile hammer is used. The test arrangement is identical to the arrangement for the *Case pile-driving analyzer,* shown in Figure 24.7: the pile hammer is replaced by a falling weight. Stress waves are generated by a mass of up to 4500 lb (2000 kg) dropped from varying heights on to a striker plate fixed to the butt of the shaft. The force transmitted to the butt is determined from strain measurements, using strain gages attached to the shaft near the butt. Simultaneously, the deformation of the butt is determined either by attaching an accelerometer to the butt or by use of a special laser theodolite. Static axial capacity is determined by using a similar analysis to the stress wave method. The major drawback of the falling weight method is the requirement to use strain measurements in the concrete near the butt of the shaft for a determination of force transmitted to the butt. An assumption must be made for the modulus of the concrete, and the concrete near the butt may not be representative of the total shaft. For determination of ultimate static axial capacity, it is necessary to displace the tip of the shaft during the test. This requirement imposes limitations in the case of large-diameter drilled shafts, because dynamic forces of about 4000 kips (18 MN) or more are required. Application of such large dynamic forces will often damage the concrete at the shaft butt, and it is usually preferable to lengthen the duration of the impact between the falling weight and the shaft butt by placing cushion material on the shaft.

In summary, the application of dynamic methods for the determination of load–movement relationships of drilled shafts has not progressed as rapidly as for precast concrete driven piles. Much of the experience at lower displacements (up to working loads) has been obtained on drilled shafts using TDR and vibration techniques. Extension of these methods, incorporating accelerometers in the body of the shaft, will undoubtedly increase their value.

*Written with the assistance of Allen G. Davis, Director, Testconsult CEBTP Ltd., Thelwall, England.

The falling weight test is an advance in the right direction. It has not reached the same status as the pile-driving analyzer for driven piles, principally because of the problem of using the concrete near the butt of the shaft for determination of force transmitted to the butt.

25.2.2. Will There Be Significant Downdrag Loading?

The causes and effects of downdrag loading on drilled shafts are described in Chapter 2 and the loading may be significant when shafts are constructed through soft cohesive soil and founded on firm soil. Settlement measurements, groundwater pressure measurements, and/or a full-scale test may be appropriate, as described for driven piles in Chapter 24.

25.2.3. What Is the Integrity of the Concrete?

Reese (1978) lists construction problems that can lead to a drilled shaft of inferior quality. These include improper excavation that allows collapse of the hole during drilling, inadequate cleaning at the base, poor concrete, improper placement of concrete, and improper pulling of casing. In all these cases, the integrity of the concrete may be questionable, and concerns are increased if the contractor is inexperienced or careless.

Continuous diamond core drilling is sometimes used to obtain samples for assessing the quality of the concrete and detecting flaws. However, this method is expensive and time consuming, and borehole direction surveys are often required to avoid misleading data in long shafts because of alignment problems.

An alternative procedure for detecting flaws is to survey the shaft by using one of the integrity testing procedures* described in Chapter 24 for driven piles. Arias (1977) and Weltman (1977) describe and compare the various procedures for integrity testing of drilled shafts by nondestructive methods. All methods that rely on measurements at the shaft butt gather maximum information from the upper part of the shaft and are therefore of limited value. In particular, for shafts cast in stiff soils, damping out of the response can result in very limited or no information on the integrity of concrete near the bottom of the shaft. If sensors are placed in the shaft during construction, additional information can be obtained, but at extra cost and inconvenience. The *echo test* cannot differentiate between bulbs formed by overbreak of the soil around the shaft and necking in the shaft.

As discussed for driven piles in Chapter 24, *sonic coring* and *single-hole* and *cross-hole nuclear density measurements* can be carried out within pipes. Sonic coring is the preferred method for a complete evaluation of concrete integrity in drilled shafts. The method requires three or four embedded metal pipes along the length of the shaft. When four pipes are used, measurements are made across the four sides and two diagonals. When one of the nuclear density methods is employed, at least four pipes should be used, but the methods are subject to the limitations given in Chapter 24.

*The text on integrity testing has been written with the assistance of Allen G. Davis, Director, Testconsult CEBTP Ltd., Thelwall, England.

Table 25.1. Overview of Routine and Special Monitoring

Application	Measurement
Routine monitoring during static axial load testing	Load and movement at shaft butt Load in shaft and at tip
Routine monitoring during static lateral load testing	Load and movement at shaft butt Bending stress in shaft
Special load testing applications	Earth pressure at contact with shaft during static lateral load testing Moment, eccentric, torsional, or combined static loading modes: appropriate parameters as for static axial and lateral modes Load, velocity, strain, and/or deformation, as required for dynamic tests Deformation and stress during full-scale test to determine downdrag loading
Other special applications	Groundwater pressure Integrity of concrete Surface settlement and/or groundwater pressure for evaluation of downdrag loading

25.3. OVERVIEW OF ROUTINE AND SPECIAL APPLICATIONS

For those readers who seek general guidelines on what is typically *routine* monitoring and what is monitoring for *special applications*, Table 25.1 presents a broad generalization.

In practice, the most frequent use of instrumentation for drilled shafts is to examine load–movement relationships during static axial load testing.

25.4. SELECTED CASE HISTORIES

Table 25.2 summarizes selected case histories of instrumented drilled shafts.

Table 25.2. Summary of Selected Case Histories of Drilled Shafts

Project	Principal Concerns	Type of Load Test	Ground Conditions	Measurement[a]	Instrumentation Discussed	Special Features	Reference
First International Plaza, San Antonio, TX, U.S.A.	Performance of foundation: comparison with predictions	—	Clay and clay shale	S	Embedment strain gages (Carlson type)		Briaud et al. (1985)
Test site at Burlington, Ontario, Canada	Load–movement relationships	Static axial compression	Weak mudstone	L S S	Flatjacks Multiple telltales Embedment strain gages (vibrating wire type)	Flatjacks used for load application	Horvath (1985)
Two sites in Denver, CO, U.S.A.	Integrity of concrete	—	Sand and clay-shale overlying bedrock Fill, sand, and gravel overlying bedrock Shafts socketed into bedrock at both sites	V	Modified version of sonic coring method with surface receivers		Olson and Thompson (1985)
Two sites in Texas, U.S.A.	Integrity of concrete	—	Various	V	Modified version of sonic coring method with surface and embedded receivers		Harrell and Stokoe (1984)
Power plant in South America	Integrity of concrete	—	Marine sediments, residual soil, weathered rock, overlying bedrock Shafts socketed into bedrock	V	Sonic coring with one and three embedded pipes		Kissenpfennig et al. (1984)
Various	Load–movement relationships	Static axial compression and uplift	Various	D D D D D L	Surveying methods Dial indicators Wire/mirror/scale LVDTs Telltales Load cells	410 tests	Kulhawy et al. (1983)
Two sites in southern Florida, U.S.A.	Load–movement relationships	Static axial compression	Fill and sands overlying weak porous limestone Shafts socketed into limestone	S S	Multiple telltales Embedment strain gages (Mustran type)		Gupton et al. (1982)

Table 25.2. Summary of Selected Case Histories of Drilled Shafts (*Continued*)

Project	Principal Concerns	Type of Load Test	Ground Conditions	Measurement[a]	Instrumentation Discussed	Special Features	Reference
Various	Integrity of concrete	—	Various	V	Steady-state vibration and transient dynamic response testing		Robertson (1982)
Test sites in Texas, U.S.A.	Load–movement relationships under lateral loading	Static lateral	Various	D L TS	Tiltmeters Cable tension meter Contact earth pressure cells		Bierschwale et al. (1981)
Sixteen-story building in North London, England	Magnitude of load taken by raft/clay interface and by shafts	—	London clay	D TS	Probe extensometers Contact earth pressure cells	Shafts capped with reinforced concrete raft	Cooke et al. (1981)
Eight-story building at Ponce, Puerto Rico	Load–movement relationships	Static axial compression	Sand and silty sand	S	Embedment strain gages (Mustran type)		Farr and Aurora (1981)
Sammis Plant, Steubenville, OH, U.S.A.	Load–movement relationships	Static axial compression, uplift, lateral	Sand and gravel overlying bedrock Shafts socketed into bedrock	S	Embedment strain gages (sister bars with weldable electrical resistance strain gage transducers)		Newman et al. (1981)
Power station in southeast Oklahoma, U.S.A.	Load–movement relationships	Static axial compression	Clay–shale	S	Embedment strain gages (Mustran type)		Goeke and Hustad (1979)
Various sites in Texas, U.S.A.	Load–movement relationships	Static axial compression	Various	S S CS TS	Multiple telltales Embedment strain gages (Mustran type) Concrete stress cells Contact earth pressure cells		Reese et al. (1976)
Test site in Houston, TX, U.S.A.	Load–movement relationships under lateral loading	Static lateral	Stiff and very stiff clay	L S	Load cell Strain gages	Strain gages bonded to steel pipe at axis of shaft	Reese and Welch (1975)
Westminster Bank Tower, London, England	Magnitude of load taken by raft/clay interface and by shafts	—	London clay	D L TS	Probe extensometers Photoelastic load cells Contact earth pressure cells	Shafts capped with reinforced concrete raft	Haws et al. (1974)
Test site in Wembley, London, England	Load–movement relationships	Static axial compression	Various	TS	Contact earth pressure cells		Whitaker and Cooke (1966)

[a]CS: stress in concrete
D: deformation
L: load in shaft
S: strain in shaft
TS: total stress between shaft and soil
V: velocity, load, strain, and/or deformation as required for dynamic tests.

Part 6

The Key to Success

Part 6 consists of Chapter 26. It is designed as a closing chapter to summarize the key issues for maximizing the success of geotechnical instrumentation for monitoring field performance.

CHAPTER 26

THE KEY TO SUCCESS: THE CHAIN WITH 25 LINKS

Full benefit can be achieved from geotechnical instrumentation programs only if every step in the planning and execution process is taken with great care. The analogy can be drawn to a chain with many potential weak links: this chain breaks down with greater facility and frequency than in most other geotechnical engineering endeavors. Shortcomings in instrumentation programs can usually be attributed to a weakness in one or more links. In this chapter, the major links are defined in chronological order, and guidance is given for maximizing the strength of each. There are 25 links.

The first 17 links in the chain are forged during the planning phase, and more detail is given in Part 2 of this book. The last eight links in the chain are forged during the execution phase, and more detail is given in Part 4.

1st Link. Predict Mechanisms that Control Behavior

One or more working hypotheses should be developed for mechanisms that are likely to control behavior.

2nd Link. Define the Geotechnical Questions that Need to be Answered

Every instrument on a project should be selected and placed to assist in answering a specific question: **if there is no question, there should be no instrumentation.** Before addressing measurement methods themselves, one should make a listing of geotechnical questions that are likely to arise during the design, construction, or operation phases.

3rd Link. Define the Purpose of the Instrumentation

Instrumentation can provide benefits during the design, construction, or operation phase of a project but should not be used unless there is a valid reason that can be defended.

4th Link. Select the Parameters to be Monitored

The question *which parameters are most significant?* should be answered. It is often found that deformation measurements are the simplest, most reliable, and least ambiguous.

5th Link. Predict Magnitudes of Change

Predictions should be made to establish the required range and accuracy of each instrument. Whenever measurements are made for construction control or safety purposes, a predetermination of *hazard warning levels* should be made.

6th Link. Devise Remedial Action

Inherent in the use of instrumentation for construction purposes is the absolute necessity for deciding, in advance, a positive means for solving any problem that may be disclosed by the results of the observations. If the observations should demonstrate that remedial action is needed, that action must be based on appropriate, previously anticipated plans. An open communication channel should be maintained between design and construction personnel, so that remedial action can be discussed at any time.

7th Link. Assign Tasks for Design, Construction, and Operation Phases

When tasks are assigned for monitoring, the party with the greatest vested interest in the data should be given direct line responsibility for producing it accurately. Reliability and patience, perseverance, a background in the fundamentals of geotechnical engineering, mechanical and electrical ability, attention to detail, and a high degree of motivation are the basic requirements for qualities needed in instrumentation personnel.

8th Link. Select Instruments

When instruments are selected, the overriding desirable feature is **reliability.** Inherent in reliability is maximum simplicity and, in general, transducers can be placed in the following order of decreasing simplicity and reliability: optical, mechanical, hydraulic, pneumatic, electrical. Lowest cost of an instrument should never be allowed to dominate the selection, and the least expensive instrument is not likely to result in minimum overall cost.

The state of the art in hardware design is far ahead of the state of the art in user technology. It is the responsibility of users to develop an adequate level of understanding of the instruments that they select. Users will often benefit from discussing their application with geotechnical engineers or geologists on the manufacturer's staff before selecting instruments, conveying as much as possible about the application and seeking out any limitations of the proposed instruments.

9th Link. Select Instrument Locations

The selection of instrument locations should reflect predicted behavior and should be compatible with the method of analysis that will later be used when interpreting the data.

A practical approach to selecting instrument locations involves three steps. First, zones of particular concern are identified, and appropriate instrumentation is located. Second, a selection is made of zones, normally cross sections, where predicted behavior is considered representative of behavior as a whole. These cross sections are then regarded as *primary instrumented sections,* and instruments are located to provide comprehensive performance data. Third, because the selection of representative zones may be incorrect, simple instrumentation should be installed at a number of *secondary instrumented sections* to serve as indices of comparative behavior.

When selecting locations, survivability of instruments should be considered, and additional quantities should be selected to replace instruments that may become inoperative.

10th Link. Plan Recording of Factors that May Influence Measured Data

Measurements by themselves are rarely sufficient to provide useful conclusions. The use of instrumentation normally involves relating measurements to causes, and therefore complete records and diaries must be maintained of all factors that might cause changes in the measured parameters.

11th Link. Establish Procedures for Ensuring Reading Correctness

Personnel responsible for instrumentation must be able to answer the question: *Is the instrument functioning correctly?* Input to the answer can often be provided by visual observations, by duplicate or backup instruments, or by studies of data consistency and repeatability.

12th Link. Prepare Budget

A budget should be prepared to ensure that sufficient funds are available for all monitoring tasks.

13th Link. Write Instrument Procurement Specifications

Procurement of other than the most simple geotechnical instruments should not be considered as a routine construction procurement item because, if valid measurements are to be made, ex-

treme attention must be paid to quality and details. The cost is usually minor.

The *low-bid* method should never be used unless regulations allow for no alternative, and one of the following two methods is recommended:

- The owner or design consultant procures the instruments directly, negotiating prices with suppliers.
- The owner enters an estimate of procurement cost in the construction contract bid schedule and subsequently selects appropriate instruments for procurement by the contractor. Price is negotiated between the owner and suppliers of instruments, and the construction contractor is reimbursed at actual cost plus a handling fee.

In cases where neither of these methods can be used and the *low-bid* method with an *or equal* provision is unavoidable, a clear, concise, complete, and correct specification must be written. The specification should cover all salient features to guard against supply of an undesirable substitution, following the guidelines given in Chapter 5.

While writing instrument procurement specifications, one should determine the requirements for *factory calibrations,* and *acceptance tests* should be planned to ensure correct functioning when instruments are first received by the user.

14th Link. Plan Installation

Installation procedures and schedules should be planned well in advance of scheduled installation dates. Written step-by-step procedures should be prepared, including a detailed listing of required materials and tools and an *installation record sheet,* on which as-built installation details will be documented.

Procedures should ensure that the presence of the instruments do not alter the very quantities that instruments are intended to measure. When the owner or design consultant is responsible for installing instruments, a special effort must be made during the planning stage to establish a cooperative working relationship with the construction contractor.

15th Link. Plan Regular Calibration and Maintenance

Regular calibration and maintenance of readout units are required during service life. Planning should include procedures and schedules for regular maintenance of field terminals and any embedded components for which access is available.

16th Link. Plan Data Collection, Processing, Presentation, Interpretation, Reporting, and Implementation

Many consulting engineering firms have files filled with large quantities of partially processed and undigested data because sufficient time or funds were not available for tasks after completion of instrument installation.

Written procedures for data collection, processing, presentation, and interpretation should be prepared before instrumentation work commences in the field. A verification should be made to ensure that personnel responsible for interpretation of instrumentation data have contractual authority to initiate remedial action, that communication channels between design and construction personnel are open, and that arrangements have been made to forewarn all parties of the planned remedial actions.

17th Link. Write Contractual Arrangements for Field Instrumentation Services

Field services include instrument installation, regular calibration and maintenance, and data collection, processing, presentation, interpretation, and reporting. Contractual arrangements for the selection of personnel to provide these services may govern success or failure of a monitoring program.

Geotechnical instrumentation field work should not be considered a routine construction item, because successful measurements require extreme dedication to detail throughout all phases of the work. The *low-bid* method should never be used unless regulations allow for no alternative, and one of the following two methods is recommended:

- The owner or design consultant performs field work that requires specialist skill, if necessary retaining the services of a consulting firm specializing in instrumentation. Supporting work is performed by the construction contractor.
- The owner enters an estimate of specialist field service costs in the construction contract bid schedule. Subsequently, the owner and construction contractor select an appropriate specialist consulting firm, which is retained as a subcontractor by the construction contractor

to perform field work that requires specialist skill. Charges for specialist work are negotiated between the owner and consulting firm, and the construction contractor is reimbursed at actual cost plus a handling fee. Supporting work is performed by the construction contractor.

One of these two methods is essential for data processing, presentation, and interpretation. In cases where regulations do not allow either of these methods for installation, calibration, maintenance, and data collection, and where the *low-bid* method is unavoidable, a clear, concise, complete, and correct specification should be written to maximize the quality of field services, following the guidelines given in Chapter 6.

18th Link. Procure Instruments

Instruments should be calibrated, inspected, and tested before shipment to the user. On receipt by the user, *acceptance tests* should be made to ensure correct functioning.

19th Link. Install Instruments

Before starting installation work, field personnel should study and understand the written step-by-step installation procedure. They should be aware that the instrument may not serve its purpose if a single essential but apparently minor requirement is overlooked during the installation.

When installing instruments in boreholes, one should respect the driller's advice. If problems arise while installing downhole components, and there is any possibility of borehole collapse or premature grout setting, act fast and discuss later: *if in doubt, pull it out.*

20th Link. Calibrate and Maintain Instruments on a Regular Schedule

Readout units should be calibrated and maintained on a regular schedule. Field terminals should be maintained, and maintenance should include any embedded instrument components for which access is available.

21st Link. Collect Data

Special care should be taken when making initial readings, because most data are referenced to these readings, and engineering judgments are based on changes rather than on absolute values.

Data collection personnel should take the first step in determining whether the instrument is functioning correctly, by comparing the latest readings with the previous readings. Any significant changes can then be identified immediately, and if *hazard warning levels* have been reached, supervisory personnel should be informed. Data collection personnel should record factors that may influence measured data and should be on the lookout for potential for damage, deterioration, or malfunction of instruments.

22nd Link. Process and Present Data

The first aim of data processing and presentation is to provide a rapid assessment of data in order to detect changes requiring immediate action, and this should usually be the responsibility of data collection personnel. The second aim is to summarize and present the data in order to show trends and to compare observed with predicted behavior for determination of the appropriate action to be taken. Specially prepared data forms will usually be required, and data should always be plotted, normally versus time. Plots of predicted behavior and causal data are often included on the same axes.

23rd Link. Interpret Data

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

Communication channels between design and field personnel should remain open, so that discussions can be held between design engineers who framed the questions that need to be answered and field engineers who provide the data.

Early data interpretation steps have already been taken: evaluation of data to determine reading correctness and also to detect changes requiring immediate action. The essence of subsequent data interpretation steps is to correlate the instrument readings with other factors (cause and effect relationships) and to study the deviation of the readings from the predicted behavior. When faced with data that on first sight do not appear to be reasonable, there is a temptation to reject the data as false.

However, such data may be real and may in fact carry an important message. A significant question to ask is: *Can I think of a hypothesis that is consistent with the data?* The resultant discussion, together with the use of appropriate procedures for ensuring reading correctness, will often lead to an assessment of data validity.

24th Link. Report Conclusions

After each set of data has been interpreted, conclusions should be reported in the form of an *interim monitoring report* and submitted to personnel responsible for implementation of data. The initial communication may be verbal but should be confirmed in writing. The report should include updated summary plots, a brief commentary that draws attention to all significant changes that have occurred in the measured parameters since the previous interim monitoring report, probable causes of these changes, and recommended action.

A *final report* is often prepared to document key aspects of the monitoring program and to support any remedial actions. The report also forms a valuable bank of experience and should be distributed to the owner and design consultant so that any lessons may be incorporated into subsequent designs.

If important knowledge gaps have been filled, the conclusions should be disseminated to the profession in a technical publication.

25th Link. Implement Data

When instrumentation is used to provide input to the initial design of a facility or for the design of remedial treatment, data will be used directly in design. When instrumentation is used during construction, remedial action should have been planned and implementation should follow these plans.

Summary to the Key to Success

Peck (1972) states, "We need to carry out a vast amount of observational work, but what we do should be done for a purpose and be done well."

Part 7

Appendixes

A. Checklist for Planning Steps
B. Checklist for Content of Specifications for Procurement of Instruments
C. Checklist for Content of Specifications for Field Instrumentation Services
D. Commercially Available Geotechnical Instruments
E. Details of Twin-Tube Hydraulic Piezometer System
F. Dimensions of Drill Rods, Flush-Joint Casing, Diamond Coring Bits, Hollow-Stem Augers, and U.S. Pipe
G. Example of Installation Procedure, with Materials and Equipment List
H. Conversion Factors