DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
441 G Street, NW
Washington, DC 20314-1000

*EM 1110-2-1908

CECW-EG

Manual
No. 1110-2-1908

30 November 2020

Engineering and Design
INSTRUMENTATION OF EMBANKMENT DAMS AND LEVEES

1. Purpose. This manual provides guidance to U.S. Army Corps of Engineers (USACE) personnel who are responsible for instrumentation, monitoring, and assessing the performance of embankment dams and levees.

2. Applicability. This manual applies to HQUSACE elements, Major Subordinate Commands (MSC), districts, laboratories, and field operating activities (FOA) involved with planning, design, construction, installation, data management and processing, monitoring, analysis, and maintenance of instrumentation systems. Project partnering agreements and associated operations and maintenance (O&M) manuals should include considerations for instrumentation consistent with this manual.

3. Distribution Statement. Approved for public release; distribution is unlimited.

FOR THE COMMANDER

LLOYD.JOHN.PHILLIP.1021070595 Digitally signed by LLOYD.JOHN.PHILLIP.1021070595 Date: 2021.02.02 16:21:50 -05'00'

JOHN P. LLOYD
COL, EN
Chief of Staff

---

*This manual supersedes Engineer Manual (EM) 1110-2-1908, dated 30 June 1995

Summary of Changes

1. Changes involving the entire manual or whole chapters and appendixes consisted of:

a. Included risk-informed decision-making concepts and guidance throughout the document.

b. Revised content to ensure appropriate instrumentation guidance pertaining to dams, levees, and floodwalls.

c. Deleted Appendix B: Drilling Methods. Added references to EM 1110-2-1804: Geotechnical Investigations and ER 1110-2-1807: Drilling in Earth Embankment Dams and Levees.

d. Split Chapter 2: Behavior of Embankments and Abutments into two separate chapters: Chapter 2: Geotechnical Concepts and Chapter 3: Embankment and Floodwall Behavior-

e. Split Chapter 7 "Data Management, Analysis, and Reporting" into two separate chapters: Chapter 8: Data Management and Chapter 9: Data Processing, Evaluation, and Reporting.

f. Merged Chapter 3: Instrumentation Concepts, Objectives, and System Design Considerations and Chapter 9: Continual Reassessment for Long-Term Monitoring into a single chapter—Chapter 4: Planning and Risk Informed Decision Making.

g. Added five appendices:

(1) Appendix B: Case Studies: F.E. Walter Dam, Wolf Creek Dam, and Upper Wood River Levee System.

(2) Appendix C: Open Standpipe PZ Falling Head Test and Flushing Procedures.

(3) Appendix D: Instrumentation Reporting.

(4) Appendix E: PZ Installation Log.

(5) Appendix F: Inclinometer Installation Log.

2. Notable changes in individual chapters and appendixes included:

a. Chapter 1: Introduction.

(6) Included content on potential failure mode and risk assessment.

(7) Added floodwalls as part of a levee or levee system.

h. Chapter 2: Geotechnical Concepts.

(1) Included content on effective stress.

(2) Modified content on time-lag and seepage patterns.

i. Chapter 3: Embankment and Floodwall Behavior:

(1) Included discussion of Potential Failure Modes (PFMs) for floodwalls.

(2) Updated content on embankment PFMs and conditions to be observed.

j. Chapter 4: Planning and Risk-Informed Decision Making.

(1) Discussed how instrumentation data informs risk assessments and risk management.

(2) Discussed how risk assessments can inform surveillance and monitoring planning and application.

(3) Included content on selection of instrumentation in relation to identified PFMs.

(4) Included content on the importance of the instrumentation redundancy.

(5) Included content on uncertainty.

(6) Included additional detail on instrumentation objectives.

(7) Modified content on personnel qualification and responsibilities.

k. Chapter 5: Measurement Methods.

(1) Modified instrumentation content to include additional available technology and provide more detail on various instruments, including grouted piezometers (PZs), in-place inclinometers, time domain reflectometry (TDR) cables, and fiber-optic cable options.

(2) Expanded time-lag discussions and adding tables to compare PZs of different types installed in different conditions.

(3) Updated table of advantages and limitations of various instruments.

(4) Reduced the content related to seismic measurements and referenced ER 1110-2-103.

(5) Modified seepage monitoring discussions to include thermal monitoring and dye tracing.

(6) Included content on monitoring the environment, including measuring precipitation, barometric pressure, and water levels.

l. Chapter 6: Automation.

(1) Included system design recommendations.

(2) Included automated data acquisition system communication and display options.

(3) Modified content on system installation and contracting concerns.

(4) Highlighted the importance of redundancy of data storage and manual data retrieval.

m. Chapter 7: Instrument Installation.

(1) Expanded content on instrument installation, including multi-level PZs and fully grouted PZs.

(2) Added references to existing EMs and ERs related to drilling.

n. Chapter 8: Data Management.

(1) Included guidance on data preservation, integrity, redundancy, and electronic data management.

(2) Added additional guidance on data collection frequency.

(3) Added additional guidance on establishing data threshold values.

o. Chapter 9: Data Processing, Evaluation, and Reporting.

(1) Expanded discussion of data processing and evaluation for various instrument types.

(2) Provided guidance on instrumentation reporting, including a suggested report outline in Appendix D.

p. Chapter 10: Instrument Maintenance.

(1) Included content on O&M manual submittals.

(2) Added content on the importance of a spare parts inventory.

(3) Expanded content on recalibration requirements.

Contents

LEFT BLANK INTENTIONALLY

Chapter 1
Introduction

1.1. Purpose. This manual provides guidance to USACE personnel who are responsible for instrumenting, monitoring, and assessing the performance of embankment dams, levees, and floodwalls.

1.2. Applicability. This manual applies to HQUSACE elements, MSCs, districts, laboratories, and FOA involved with planning, design, construction, installation, data management and processing, monitoring, analysis, and maintenance of instrumentation systems. Project partnering agreements and associated operations and maintenance (O&M) manuals should include considerations for instrumentation consistent with this manual.

1.3. Distribution Statement. Approved for public release; distribution is unlimited.

1.4. References. Required and related references are listed in Appendix A.

1.5. Records Management (Recordkeeping) Requirements. Records management requirements for all record numbers, associated forms, and reports required by this regulation are included in the Army's Records Retention Schedule—Army. Detailed information for all record numbers, forms, and reports associated with this regulation are located in the Army's Records Retention Schedule—Army at https://www.arims.army.mil/arims/default.aspx.

1.6. Discussion. This manual addresses aspects of instrumentation for embankment dams and levees including planning, risk-informed decision making, design, installation, maintenance, data management, analysis, and reporting.

1.6.1. This revision includes discussions of potential failure modes, visual monitoring techniques, automated data acquisition systems (ADAS) and data management, data processing, analysis, and presentation, appropriate analysis methods, correlation plots, and instrumentation report preparation.

1.6.2. Case studies have been included for dams and levees to illustrate sample surveillance and monitoring plans and resultant data. Interpretations were added to illustrate various graphical outputs and potential implications. The revised document also includes updates to references and other USACE EMs to reduce redundancies between manuals without compromising the quality of the discussions.

1.7. Approach.

1.7.1. This manual addresses the instrumentation of embankment dams and levees (including floodwalls). Monitoring before, during, and after construction is important to assess performance until the project is decommissioned.

1.7.2. This manual presents theoretical and practical concepts intended to help staff develop a successful instrumentation plan, referred to as the surveillance and monitoring plan in this document. Guidance regarding data retrieval, processing, evaluation, and instrumentation maintenance is provided to persons responsible for operating the system. Since dam and levee risk can change over time and because aging projects and associated instruments require increased attention, long-term evaluation is discussed.

1.7.3. Instrumentation and monitoring is unique to the needs of a particular project. Therefore, an engineering manual cannot completely address the specific needs of every project. Engineering judgment is required in all aspects of instrumentation and monitoring including planning, design, installation, management, processing, data integrity, analysis, reporting, and any associated recommended actions.

1.7.4. This manual stresses the importance of staff members. Instrumentation cannot substitute for persons who are skilled, diligent, and responsible.

1.7.5. Potential failure mode analyses, risk assessments, and appropriate risk reduction measures need to be understood and used as the basis for recognizing the need for action, assessment of appropriate action, and implementation of that action. Instrumentation data and evaluation can play a crucial role in risk-informed decision making and associated actions. Risk assessments can also be used to identify critical instruments or if instruments important to a failure mode do not exist and should be pursued.

1.7.6. Waterside and landside water levels for levees and pool/headwater and tailwater for dams are used interchangeably in this document. Examples may discuss pool/headwater and tailwater of a dam, but the concepts are typically the same for levees or vice versa.

1.8. Scope.

1.8.1. This manual addresses:

a. Geotechnical and structural concepts frequently useful for understanding project performance.

b. Surveillance and monitoring plan development and considerations.

c. The role of instrumentation in risk assessment and risk reduction.

d. Instrument types and installation.

e. Monitoring frequencies and thresholds.

f. Rehabilitation, replacement, and maintenance.

g. Automation.

h. Data interpretation and evaluation.

i. Data management.

j. Graphical presentation of data.

k. Formal reporting.

1.8.2. The information presented in this manual can be applied to project abutments and foundations and to reservoir rims.

1.8.3. Guidance and procedures for documentation of dam performance, with respect to instrumentation, are included in Sub-Appendix U-1 of Engineer Regulation (ER)

1110-2-1156: Safety of Dams: Policy and Procedures. Similar practices may be followed for levees if no other official guidance document exists.

1.8.4. This manual does not address the topics of:

a. The instrumentation of concrete and steel structures, which is addressed by EM 1110-2-4300: Instrumentation for Concrete Structures.

b. Instrumentation for research and investigative purposes.

1.9. Use. The manual has five primary uses:

1.9.1. Planning and design of a surveillance and monitoring plan.

1.9.2. Implementation and maintenance of an instrumentation system.

1.9.3. Management and processing of instrument data.

1.9.4. Analyzing, reporting, and presenting data to support assessment of the project and making recommendations for risk-informed actions.

1.9.5. Assessment of the instrumentation to support decisions to modify or replace an instrument system.

LEFT BLANK INTENTIONALLY

Chapter 2
Geotechnical Concepts

2.1. Introduction.

2.1.1. For embankments, a basic familiarity with geotechnical concepts is necessary to understand the use of instruments, make observations, and to analyze the data collected. A full understanding of the data also requires familiarity with embankment construction and behavior. The key geotechnical aspects of embankment dam and levee behavior described in this chapter are related primarily to porewater pressure and soil deformation.

2.1.2. For a detailed treatment of these geotechnical topics, the reader is referred to other EMs and geotechnical textbooks (Holtz et al., 2010); (Das, 2007); and (Terzaghi et al., 1996). For detailed information on design and construction considerations, refer to EM 1110-2-1913 and EM 1110-2-2300.

2.1.3. This chapter addresses geotechnical concepts and embankment behavior under the headings of:

a. Soil and rock structure.

b. Types of soils.

c. Stress and pressure.

d. Consolidation.

e. Hydraulic conductivity.

f. Groundwater level and porewater pressure.

2.2. Soil and Rock Structure.

2.2.1. Soil.

a. In general, soil is a matrix of solid, liquid, and gaseous matter. Whether natural soil or compacted earthfill, soil consists of granular solid particles separated by voids. Although soil may contain organic solids, this discussion of soil structure is limited to mineral solids.

(1) A void may be completely filled with either gas or water or filled with both gas and water. The gas present in voids is typically air. If voids are completely filled with water, the soil is referred to as saturated. However, if gas is present in the voids, the soil is referred to as unsaturated or partially saturated. Soil structure is described in the context of a soil profile and as an abstract phase diagram.

(2) A soil profile may consist of a vadose zone underlain by a water table, as shown in Figure 2.1. The soil below the water table or phreatic surface is saturated. A capillary fringe may exist in the soil immediately above the water table. Water in a capillary fringe is held in soil voids by capillary action against the force of gravity. The aerated vadose zone extends from the water table to the ground surface. The voids in the vadose zone are completely filled with gas or are filled with gas and water.

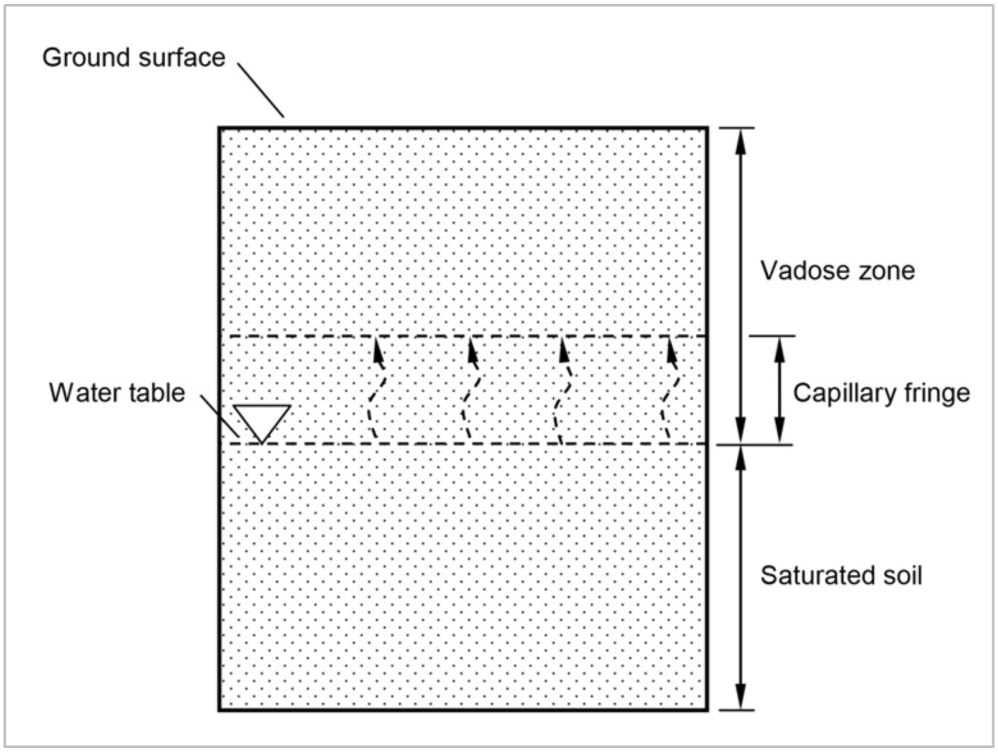


Figure 2.1. Moisture Zones in a Soil Profile

b. A phase diagram is an abstract depiction of soil structure, representing the fractions of solid, liquid water, and gas in a volume of soil. Phase diagrams of both saturated and unsaturated soil are shown in Figure 2.2. Figure 2.2 (a) of the saturated soil includes only solid and liquid. Figure 2.2 (b) of the unsaturated soil includes solid, liquid, and gas. Although the phase diagrams are simple, the illustrated volume relationships facilitate soil mechanics calculations required for geotechnical design.

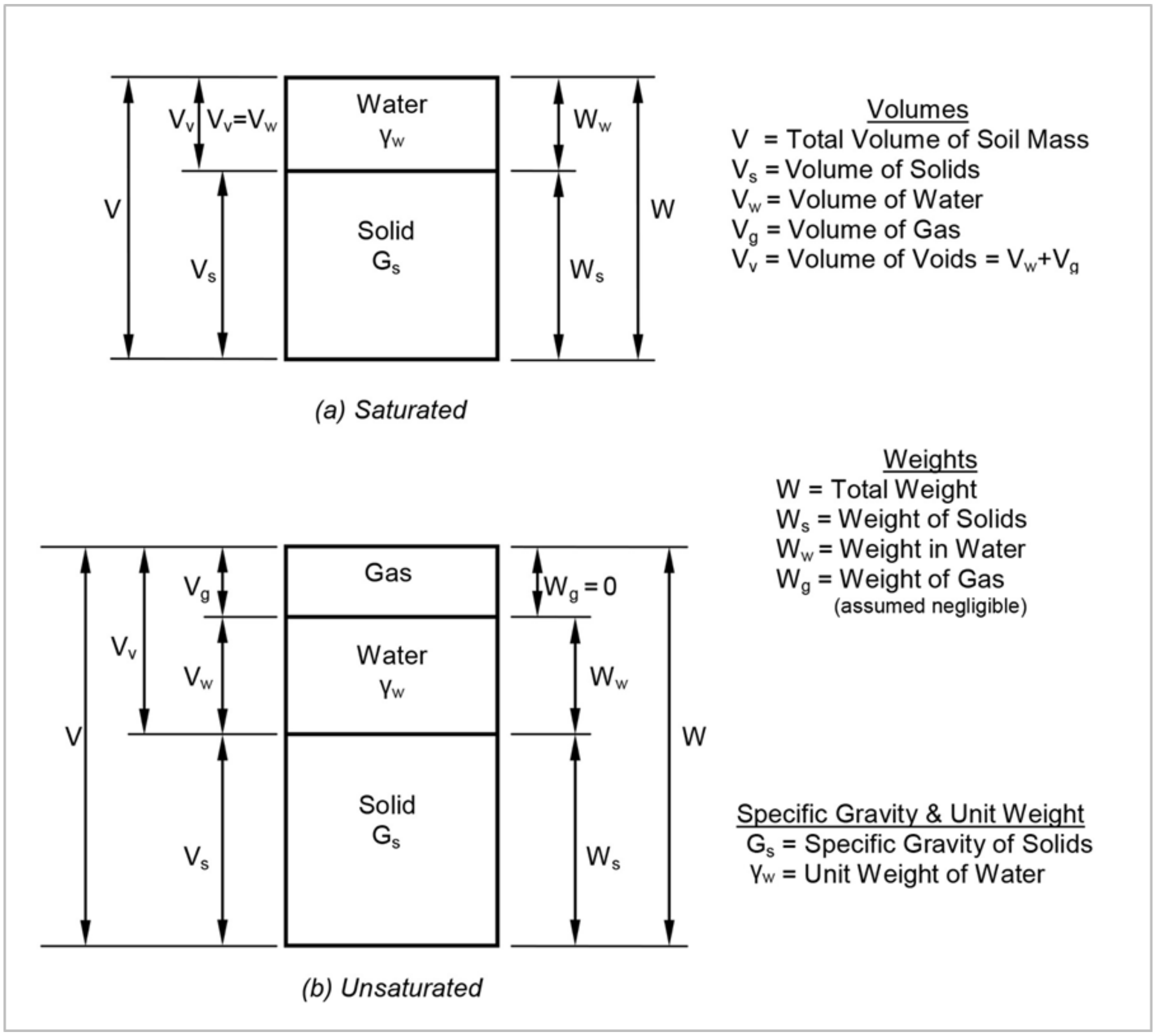


Figure 2.2. Phase Diagrams for Saturated and Unsaturated Soil

2.2.2. Rock. As a foundation material, rock shares many characteristics with soil. However, the mineral grains in rock are typically firmly bonded in a matrix, and the monitoring emphasis is placed upon the structural deficiencies presented by discontinuities across distances much larger than the mineral grains, referred to as secondary features. Secondary features include bedding planes, foliation, faults, fractures, joint patterns—and especially, open joints. Problems with strength and seepage in rock are typically related to secondary features.

2.3. Types of Soils.

2.3.1. Identification and characterization of soils facilitate the proper design and monitoring of an embankment or the foundation of a rigid structural element. Soils are categorized into two broad groups: cohesionless or cohesive, which may be liquifiable or dispersive, respectively. Soil types are discussed under the headings of:

a. Cohesionless.

b. Cohesive.

c. Liquefiable.

d. Dispersive.

2.3.2. Cohesionless. Cohesionless soils consist of particles of rocks or minerals. Based on the Unified Soil Classification System, fine-grained soils have 50% or more passing the No. 200 sieve by weight. Course grained soils have more than 50% retained on or above No. 200 sieve by weight. The coarsest cohesionless soils—sands and gravels—include grains distinguishable to the naked eye. Sands and gravels are granular and nonplastic. Sands and gravels have a low unconfined strength, and the individual grains do not cohere in the air-dry state.

a. Typically, cohesionless soil particles are not a product of chemical decomposition. Silt is composed of sub-rounded or blocky particles, similar to sand but finer grained. Silts exhibit lower plasticity than clays, which are composed of flat, elongated particles. Inorganic silts generally drain faster than clays, and the Mohr-Coulomb drained strength envelope should pass through zero, indicating little or no cohesion.

b. A shaking test of a pat of moist silt within the palm of the hand (as described by Terzaghi) can quickly serve to distinguish silt from clay.

2.3.3. Cohesive. Cohesive soils, known as clays, consist of microscopic and submicroscopic mineral particles which are products of the chemical decomposition of rocks. Cohesive soils exhibit strength in the unconfined and air-dry state. Water affects the interaction between the mineral grains and creates cohesion between the particles. Plastic soils retain shape after remolding, which is made possible by the cohesion of soil grains.

2.3.4. Liquefiable.

a. Liquefiable soils are cohesionless and fine-grained soils subject to significant strength loss during seismic loading. The occurrence of nearly complete loss of strength is referred to as liquefaction. Soils that contract during seismic shaking may experience an increase in porewater pressure great enough to result in significant yield under load.

b. A rigid structural element founded on a large volume of liquefiable soil may lose all foundation support during an earthquake. PZs have detected pressure increases indicative of liquefaction during earthquakes.

2.3.5. Dispersive. In a dispersive clay, individual clay particles lose cohesion when exposed to water and are taken into suspension. Therefore, dispersive clays are very susceptible to surface erosion and internal erosion. Information on dispersive clay characteristics and identification is available in Appendix XIII of the USACE publication EM 1110-2-1906: Laboratory Soils Testing.

2.4. Stress and Pressure.

2.4.1. Modern design of embankments is based on Terzaghi's principle of effective stress, which states that total stress in a soil mass is equal to the sum of the effective stress and the porewater pressure. In the case of a saturated soil, the pressure of liquid water in the pores of a soil is referred to as porewater pressure. The porewater pressure in a saturated soil is exerted equally in all directions. The effective stress is the portion of the total stress supported by the soil grains through intergranular forces at the contact points between soil grains.

2.4.2. The engineering properties of a soil mass, such as strength, compressibility, and hydraulic conductivity, are controlled by effective stress. Therefore, the geotechnical engineer should have a clear grasp of the concept of effective stress.

2.4.3. Figure 2.3 uses a spring analogy to illustrate the concept of soil effective stress by characterizing the behavior of a soil under load. The spring analogy illustrates how porewater pressure changes as load is gradually transferred from water to soil solids. Diagrams B–F illustrate the spring analogy for the soil shown in Diagram A.

a. Diagram A: saturated soil is placed within a cylinder of known cross-sectional area and covered with a porous piston; no load is applied, and the pressure gauge is set to zero-out the initial water pressure.

b. Diagram B: analogous to Diagram A; the ability of soil to resist compression under load is represented by an uncompressed spring; the soil hydraulic conductivity is represented by an impermeable piston equipped with a flow restricting valve; the valve is open and the pressure gauge is set to indicate zero water pressure.

c. Diagram C: the valve is closed; a load, P, is applied; the essentially incompressible water supports the entire load because water cannot drain through the closed valve; the pressure gauge indicates the consequent water pressure.

d. Diagram D: the valve is opened just enough to allow the water to drain slowly from the cylinder; pressure is partially relieved, the spring has compressed slightly, and the pressure gauge indicates less pressure than in Diagram C.

e. Diagram E: the valve is still slightly open; as water slowly drains, the applied load, P, is transferred from the water to the spring; the spring compresses and the piston descends; the compressing spring represents the soil consolidating under load; the load supported by the spring represents the effective stress; the water pressure gauge indicates less pressure than in Diagram D.

f. Diagram F: the valve is still open, but outflow of water has ceased and compression of the spring is complete; the spring bears the entire load, P, and represents the fully consolidated soil; the pressure gauge indicates zero, as it did in Diagram B.

2.4.4. The mechanism of the transmission of effective stress between soil particles depends on the type of soil. In cohesionless soils, effective stress is transmitted between particles through grain-to-grain contact. Alternatively, effective stress in fine-grained soils is thought to be transmitted through adsorbed water, also known as double-layer water captured by electrical forces at the surface of soil particles.

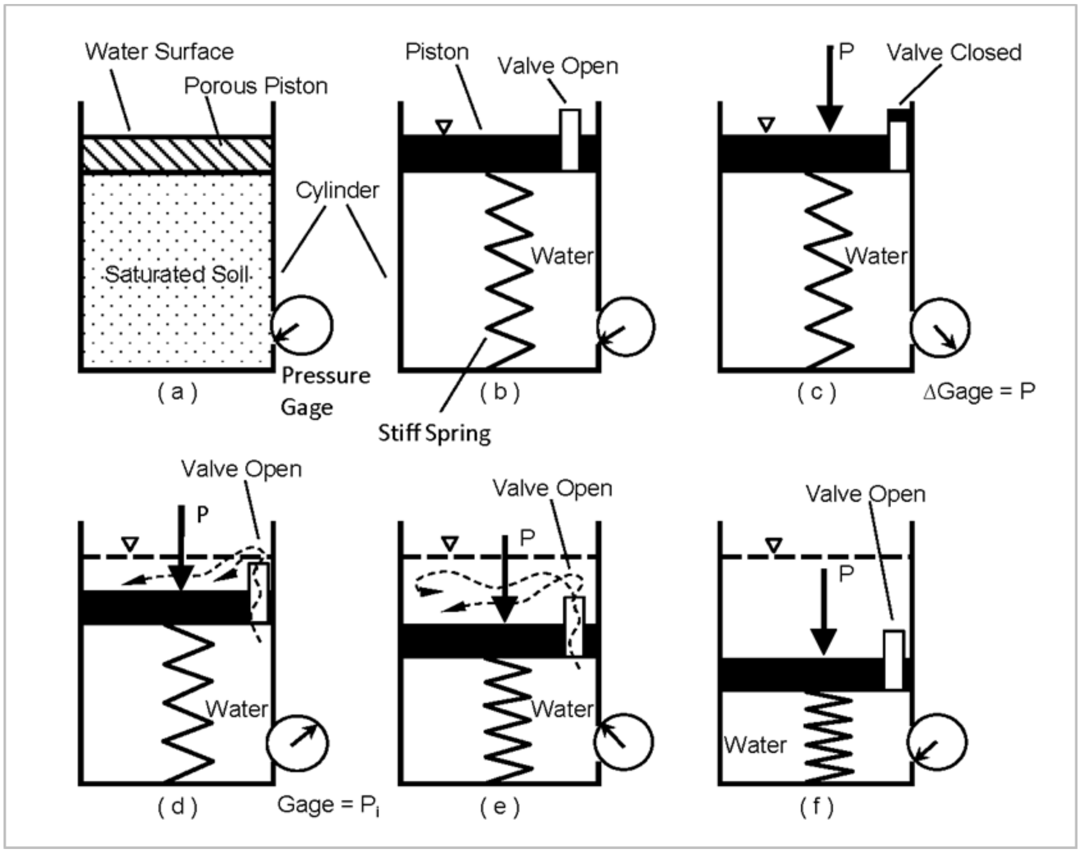


Figure 2.3. Spring Analogy for Soil Effective Stress
(Modified from Dunnicliff, 1993, Figure 2.3, p. 14)

2.4.5. In unsaturated soils, a component of the total stress is transmitted to the pore-gas portion of the voids and is called the pore-gas pressure. In such a case, the mathematical difference between pore-gas and porewater pressure is defined as matric suction. This suction, or capillary action, produces an upward flow of water from a saturated zone into an overlying unsaturated soil.

2.4.6. For cohesive soils, an additional suction, called osmotic suction, is generated by cations in the double-layer water of fine-grained soil particles and the surrounding free water. Typically, the behavior of cohesionless soils is influenced primarily by changes in matric suction, although the effects of osmotic suction can be significant in very fine-grained cohesionless soils as well.

2.5. Consolidation. Consolidation is a gradual process. For a rapidly loaded saturated soil, the increased load is initially supported by the porewater pressure. Over time, water drains from the soil, reducing the porewater pressure, and the loss of porewater pressure is offset by an increase in effective stress. The soil layer compresses in response to the increased effective stress and increases in density. The volume reduction associated with consolidation results in settlement of the soil profile and ground surface.

2.5.1. The increase in density typically increases the soil shear strength, which may increase resistance to sliding in an embankment, for example. Consolidation is much more pronounced in low-hydraulic conductivity, fine-grained soils than in coarse-grained soils.

2.5.2. The degree of consolidation in a soil profile is described by reference to the loading history of the soil. A normally consolidated soil is defined as a soil that has never been subjected to an effective stress greater than the existing overburden pressure. An over-consolidated soil is defined as a soil that has been subjected to an effective stress greater than the existing overburden pressure. For example, a soil that was subjected to a heavy glacial load and is now glacier-free is an over-consolidated soil.

2.5.3. Knowledge of the degree of consolidation facilitates an accurate prediction of the response of a foundation to a change in load. Monitoring of porewater pressure during consolidation provides an indication of shear strength in a soil profile. For example, pore pressure in a soft foundation may be monitored as an embankment is raised, and the data may be interpreted to determine the rate at which embankment fill may be placed safely.

2.6. Hydraulic Connectivity.

2.6.1. Soil hydraulic conductivity influences the rate at which porewater pressure changes in a soil mass. Hydraulic conductivity is a measure of the rate at which water can move through the soil and controls how rapidly settlement occurs. The flow of water through soil is described by Darcy's law, commonly expressed as:

$$Q = KiA \quad \text{(Equation 2.1)}$$

where:

Q = flow, cm^3/s
K = coefficient of permeability, cm/s
i = hydraulic gradient, (dimensionless)
A = cross-sectional area of flow, cm^2

2.6.2. For soils, the coefficient of permeability is related to the ratio of the volume of the voids to the volume of the mass. In rock, however, the secondary hydraulic conductivity, which may be due to openings such as fractures and dissolution channels, is typically more consequential than the hydraulic conductivity of the rock matrix.

2.6.3. Soil texture influences hydraulic conductivity. Figure 2.4 compares the hydraulic conductivity and drainage characteristics of various soil textures. Clean gravel and clean sand have good drainage characteristics and a high coefficient of permeability. Very fine sands, silts, and clays have poor drainage characteristics or are practically impervious and have a low coefficient of permeability. As shown in Figure 2.4, soil hydraulic conductivity spans many orders of magnitude.

<table>
<tr><th rowspan="2"></th><th colspan="12">Coefficient of Permeability, k (cm/s)</th></tr>
<tr><th>10^{2}</th><th>10^{1}</th><th>10^{0}</th><th>10^{-1}</th><th>10^{-2}</th><th>10^{-3}</th><th>10^{-4}</th><th>10^{-5}</th><th>10^{-6}</th><th>10^{-7}</th><th>10^{-8}</th><th>10^{-9}</th></tr>
<tr><td>Drainage</td><td colspan="6">Good</td><td colspan="2">Poor</td><td colspan="4">Practically Impervious</td></tr>
<tr><td rowspan="2">Soil Types</td><td colspan="2" rowspan="2">Clean gravel</td><td colspan="3">Clean sands, clean sand and gravel mixtures</td><td colspan="4">Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, and stratified clay deposits</td><td colspan="3" rowspan="2">“Impervious” soils (e.g., homogeneous clays below zone of weathering)</td></tr>
<tr><td colspan="2"></td><td colspan="5">“Impervious” soils modified by the effects of vegetation and weathering</td></tr>
</table>

Figure 2.4. Hydraulic Conductivity and Drainage Characteristics of Soils (Casagrande and Fadum, 1940)

2.6.4. The hydraulic conductivity of soil in the horizontal and vertical directions is typically not equal. For example, in compacted fill, the horizontal hydraulic conductivity is typically several times greater than the vertical hydraulic conductivity.

2.7. Groundwater Level and Porewater Pressure.

2.7.1. Groundwater levels and porewater pressures in a soil profile typically vary over time. Precipitation, evaporation, atmospheric pressure, ocean tides, and infiltration from rivers and reservoirs may cause significant variations in groundwater levels.

2.7.2. Groundwater level and porewater pressure are discussed under the headings of:

a. Hydrostatic groundwater condition,

b. Porewater pressure,

c. Perched and confined aquifers,

d. Piezometric level, and

e. Time lag in piezometer (PZ) measurements.

2.7.3. Hydrostatic Groundwater Condition.

a. Under the simplest groundwater conditions, the distribution of porewater pressure is hydrostatic beneath the level of a water table in an unconfined aquifer. An aquifer is a permeable water-bearing stratum of soil or rock.

b. An unconfined aquifer features a groundwater level (also known as the water table), which is free to rise. This level is the free water surface in the saturated portion of the unconfined aquifer. Water at the level of the water table is at atmospheric pressure. If the velocity of groundwater below the water table is zero, and the water is not subject to an acceleration or a localized change in pressure, then the porewater pressure increases linearly with depth below the groundwater level, or hydrostatically.

c. Hydrostatic pressure may be calculated by multiplying the unit weight of water by the vertical distance from the point of interest, such as at a well screen or sensor, to the groundwater surface.

d. Due to the complex structure of most aquifers and the changes in flow and pressure that may occur, the hydrostatic groundwater condition may be considered an ideal situation against which actual porewater pressures may be compared.

2.7.4. Porewater Pressure.

a. Porewater pressure in a soil may vary considerably, and discussion of porewater pressure should be made in the context of a datum. The two most common pressure datums are absolute pressure and atmospheric pressure. Consistent with sign convention for geology and geotechnical engineering, this manual uses compression as positive and tension as negative. Atmospheric pressure is commonly addressed in "atmospheres": a unit of pressure approximately equal to the atmospheric pressure at sea level.

b. A pressure based on the atmospheric pressure datum is referred to as a gauge pressure. The gauge pressure of atmospheric pressure is zero. Therefore, gauge pressures greater than –1 atmospheres indicate compression in the absolute sense. In geotechnical work, pressure is normally expressed as a gauge pressure.

c. Absolute porewater pressure may be negative. The capillary films in the pores of a partially saturated soil exhibit surface tension and pull individual soil grains together, causing compression within the soil mass and contributing to the effective stress present in the soil mass.

d. In an unconfined aquifer, the water table may be referred to as the level in a soil profile at which porewater gauge pressure equals zero. Above the zero-pressure level, porewater pressure in the capillary fringe is less than atmospheric pressure, and so has a negative porewater gauge pressure. Negative porewater pressure is referred to as soil suction.

e. Below the water table, porewater pressure is greater than atmospheric pressure and is referred to as positive porewater pressure.

f. Porewater pressure responds to the application of a load on a soil profile. Porewater pressure may be increased by applying a compressive force to the soil, or by applying a shearing force to a loosely packed soil that decreases in volume without dissipation of the porewater pressure.

g. Excess porewater pressure resulting from any type of stress change is referred to as induced porewater pressure. Negative porewater pressure may occur if a compressive load is removed, or if a densely packed soil increases in volume in response to shear, or if a soil dries and becomes unsaturated.

2.7.5. Perched and Confined Aquifers.

a. Perched water tables and confined aquifers under artesian pressure are two conditions: each resulting from discontinuities in vertical hydraulic conductivity, which cause a non-hydrostatic variation in the more general vertical porewater pressure distribution.

b. A perched aquifer is a minor unconfined aquifer overlying a relatively impermeable stratum which is comparatively isolated from the more general groundwater condition in the area. The impervious stratum may cause a discontinuity in porewater pressure between the base of the perched aquifer and any aquifer below.

c. Confined aquifers may be pressurized by a water table, as shown in Figure 2.5. Therefore, the porewater pressure at a given point in an aquifer confined between upper and lower impervious strata may not be related to water depth directly above the point.

d. Figure 2.5 illustrates an artesian aquifer in section. As shown in Figure 2.5, for a remote water table of sufficient height, a well drilled in the confined aquifer flows without pumping and is called a free-flowing or artesian well. The flow is driven by the head difference between the water table and the top of the well. The head is communicated as porewater pressure through the aquifer from the water table to the well intake.

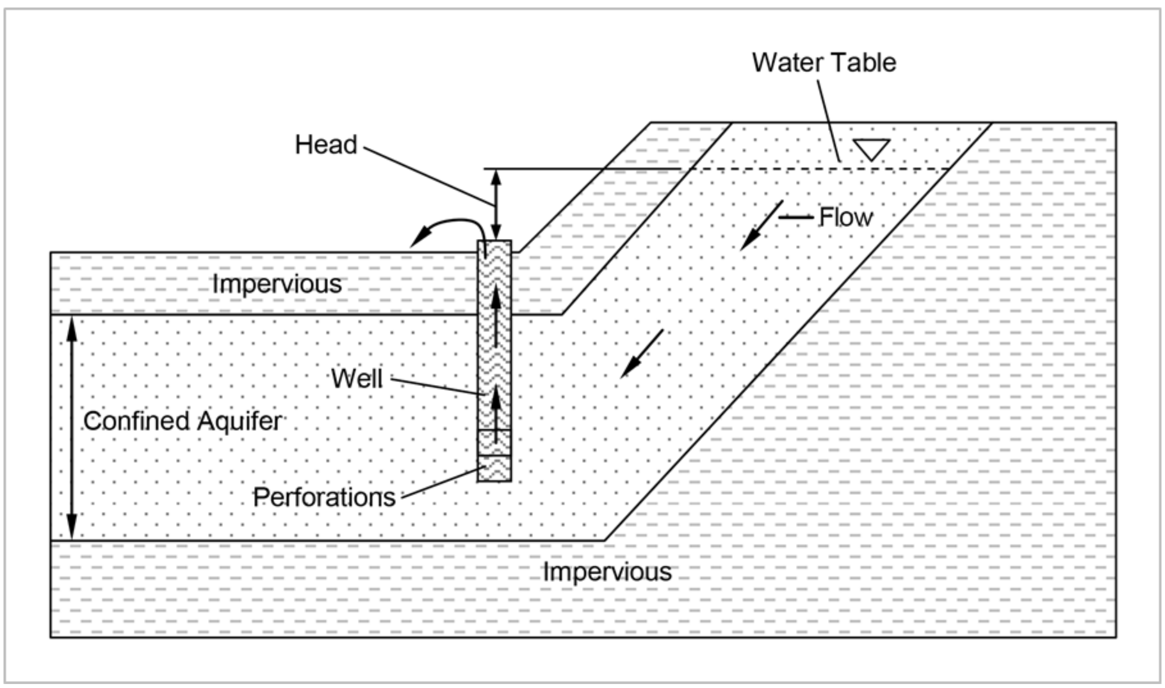


Figure 2.5. Artesian Aquifer

2.7.6. Piezometric Level.

a. The piezometric level is the elevation to which water would rise in a PZ standpipe open to the atmosphere. Figure 2.6 illustrates the piezometric levels soon after a layer of material is placed over existing soil layers, and before consolidation is complete. As a result of the placement of earthfill, excess porewater pressure exists in the clay. The four perforated pipes shown in Figure 2.6 are open to the atmosphere and are installed such that the soil is in intimate contact with the exterior of the pipes.

b. Pipe perforations are indicated by a pattern of horizontal line segments. The perforated section of pipe (a) is located in the sand layer and indicates the groundwater level. Pipe (b) is perforated throughout its length, but the other pipes are perforated only near the bottom. Pipe (b) is an observation well (i.e., a pipe with no subsurface seals to prevent a vertical connection between multiple strata). The response of Pipe (b) is intermediate between the upper and lower range of all the piezometric levels the perforations intercept.

c. Due to the high hydraulic conductivity of the sand, the excess porewater pressure of the clay immediately dissipates into the sand, as indicated by the dashed horizontal arrows. Therefore, the water level in Pipe (b) closely approximates the groundwater level in the sand, rather than the piezometric level in the underlying clay. Pipes (a), (c), and (d) are PZs, being only affected by the porewater pressure at the depths of the pipe perforations. Pipes (a), (c), and (d) do not measure groundwater pressure at other elevations.

d. Pipes (c) and (d) indicate two separate porewater pressures in the clay at the elevation of the pipe perforations. More excess porewater pressure has dissipated at Pipe (c) than at Pipe (d), because the drainage path for the relief of excess porewater pressure is shorter and the rate of dissipation is greater at Pipe (c) than at Pipe (d). The length of the drainage paths is indicated by the upward pointing dashed arrows beside Pipe (c) and Pipe (d).

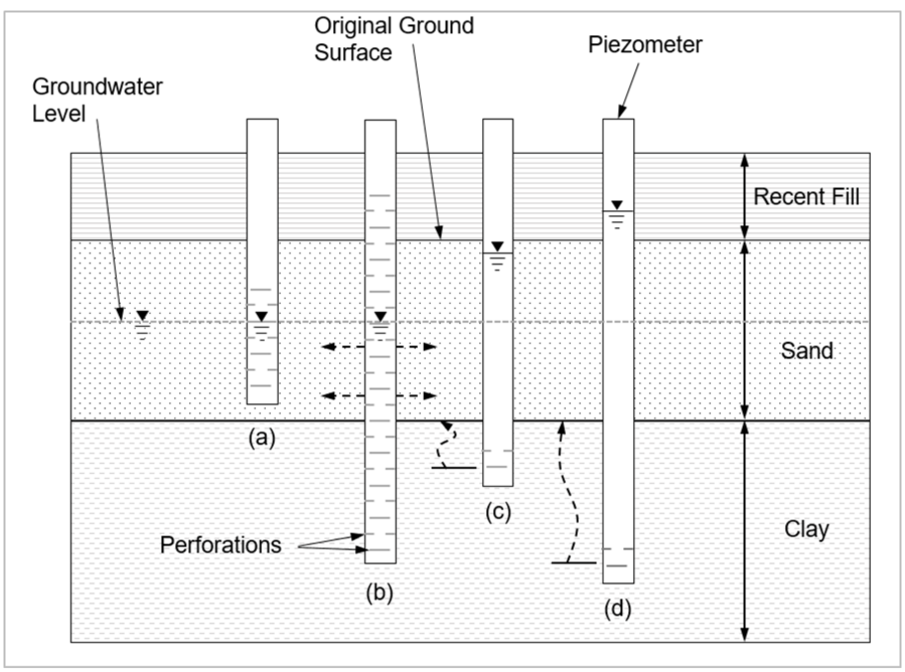


Figure 2.6. Groundwater Level and Porewater Pressure
(Dunnicliff, 1993, Figure 2.6, p. 17)

LEFT BLANK INTENTIONALLY

Chapter 3
Embankment and Floodwall Behavior

3.1. Introduction.

3.1.1. Understanding embankment and floodwall behavior requires knowledge of the index and other engineering properties of the materials, the embankment or wall design features, the foundation geology, and the pertinent potential failure modes. Typical potential failure modes are described in greater detail in Best Practices in Dam and Levee Safety Risk Analysis, the United States Bureau of Reclamation (BOR) and USACE (July 2019).

3.1.2. Embankment failure modes are further described in EM 1110-2-1913 and EM 1110-2-2300. Floodwall types, behavior, and typical failure modes are further described in EM 1110-2-2502. Further discussion of slope stability can be found in EM 1110-2-1902.

3.2. Earthfill Embankments.

3.2.1. Embankments and their foundations can undergo deformation during and after construction. These deformations are normal and unavoidable to some degree, but excessive deformations can lead to problems. Stress-deformation characteristics of cohesionless soils are influenced by rearrangement of the relative positions of grains under applied shear. Settlement of cohesionless soils typically occur quickly after loading.

3.2.2. For cohesive soils, the stress-deformation characteristics are governed by the time required for water to flow through the pores in the soil and for subsequent volume changes to occur. Settlement of cohesive soils is due to consolidation and plastic deformation.

3.2.3. The flow of water through pores (seepage) results in a drag force exerted on the soil particles in the direction of flow. Excessive velocity relative to the soil cohesion and confining forces results in erosion. All embankments are subject to seepage through the fill, foundation, and abutments.

3.2.4. Design of an embankment may provide for seepage control to prevent excessive downstream-toe uplift pressures, slope or foundation instability, internal erosion through the embankment or foundation, and the migration of embankment material into open joints in the foundation and abutments. Embankment seepage does not necessarily indicate a critical safety issue. Yet, monitoring the quantity and quality of any surfacing seepage flows is necessary to provide assurance that a potential failure mode is not developing in the embankment or foundation.

3.2.5. As the pool or river levels increase, a seepage pattern can develop through, under, or around embankments. Over time, this seepage can stress and damage embankments and/or foundations. Seepage issues can be attributed to materials, design, construction, or operation. Sudden changes may occur during cyclic loading, such as during floods and seismic events.

3.2.6. For embankments regularly impounding water, seepage flows and piezometric response should become predictable as piezometric equilibrium has been established. However, decades may elapse before piezometric equilibrium is attained for impervious embankment materials.

3.2.7. Decades may also elapse before construction-induced excess pore pressures dissipate in very large embankments and thick clay foundations. Consolidation settlement of embankments and foundations may be uniform or may feature a sharp settlement differential, either longitudinally or in section. An embankment constructed on a soft foundation may be subject to significant long-term general settlement. An embankment on a foundation having a sharp discontinuity in strength, shape, or preparation can exhibit differential settlement.

3.2.8. A poor understanding of foundation characteristics may lead to problems in the construction and functioning of an embankment. A good understanding of foundation characteristics is based on adequate site characterization data and the interpretation of that data.

3.2.9. Because design freeboard (the distance between the water surface and the top of the embankment) should be maintained as an embankment settles, monitoring of settlement facilitates planning to raise the embankment to restore design elevations. The decision to pursue restorative actions, however, should consider results of a risk assessment including expected overtopping frequency and associated consequences.

3.2.10. Measurement of piezometric levels within an embankment and foundation can be used to indicate the effectiveness of seepage barriers, filters, relief wells, drains, and downstream seepage blankets. The interior of a drainage blanket or chimney drain should maintain a low pressure, indicating non-pressurized flow and adequate capacity to convey drain flow.

3.2.11. The function of a downstream shell on a zoned embankment is to increase stability. Such a downstream shell should be free-draining. Piezometric levels on the downstream side of seepage barriers in an embankment should be significantly lower than upstream piezometric levels, and uplift at the downstream embankment toe should not be great enough to initiate internal erosion.

3.3. Rockfill Embankments.

3.3.1. Embankments can be constructed with significant zones of rockfill. Rockfill embankments have excellent strength and resistance to earthquakes, but appropriate internal zoning is critical for proper performance. If deformation of a rockfill embankment with a central impervious core results in disruption of the filters, then the core material can be washed into the rockfill.

3.3.2. Some rockfill embankment use a concrete facing or another type of material on or near the upstream face to manage through seepage, which can be disrupted, causing excessive leakage. Whether rockfill embankment feature internal impervious zones or impervious material on the upstream face of the embankment, excessive differential deformation of a rockfill embankment is undesirable. Thus, monitoring deformations of rockfill embankment—especially dumped rockfill—is critically important.

3.3.3. In a rockfill embankment, high stress is likely to exist at points of contact between individual rocks. The strength of the rock at the points of contact may be affected by cycles of rising and falling water levels against the dam.

3.3.4. As the level of impounded water rises, the wetting of the rock may decrease the strength of the rock at the contact points. That loss of strength is not likely to be a problem while the submerged rocks are partially supported by a buoyant force. However, when the water level against the embankment is lowered, the rocks are no longer supported by a buoyant force, and the weakened contact points may crush or particles may be rearranged.

3.3.5. The cumulative effect of crushing the contact points or rearrangement of particles is observed as settlement and deformation of the embankment. Long-term monitoring may make possible the correlation of rates of settlement with cycles of water impoundment.

3.3.6. Zoned rockfill embankments have a greater potential for unconformable internal zones than do earth embankments because of the compressibility of the finer grained material. Excessive deformation within the rockfill may crack impervious barriers and filter layers, allowing internal erosion.

3.3.7. Rockfill embankments equipped with an impervious material on the upstream face are also vulnerable to deformation, particularly if the material is steel or concrete. The perimeter joint between the material and the upstream embankment toe must be reasonably watertight. Because the fill is less stiff than the impervious material and any seepage velocities may be relatively high, perimeter monitoring may be necessary.

3.4. Floodwalls.

3.4.1. Floodwalls are sometimes founded on embankments, and behavior described in Section 3.2 for embankments also applies to floodwalls and their foundations. Floodwalls range from rigid, reinforced-concrete that may include deep pile foundations to flexible sheet-pile walls that in some instances require additional pile or anchor support.

3.4.2. Floodwalls include the failure modes and factors influencing behavior listed in Section 3.3 for embankments. Additionally, floodwalls have the potential for bearing capacity failure, overturning, or internal structural failure. Erosion and scour due to the freefall of water over the top of floodwalls is often more severe than that from flow over embankments. The interaction between floodwall and surrounding embankment or foundation soils can be complex, and landside scour could result in less soil passive resistance and wall instability.

3.4.3. The transition between walls and embankments constitute a discontinuity and in addition to concerns listed for appurtenant structures, such as differential settlement and contact erosion, these transitions concentrate surface erosion and increase scour when overtopped. Additional instrumentation should be considered for floodwalls to target settlement, translation, or rotation of the wall.

3.5. Appurtenant Structures.

3.5.1. Features constructed as part of, or adjacent to, dams and levees are known as appurtenant structures. Appurtenant structures can include intake and discharge structures associated with pump stations, gravity drains, outlet works, and spillways. They may be constructed of earthen fill, rock, or reinforced concrete. Discharge structures may include gates, conduits, chutes, spillways, and stilling basins. Intake structures may include trash racks, towers, bridges, gates, valves, and conduits.

3.5.2. Other structures such as levee and conveyance channels have walls and/or slabs that are subject to movement and uplift. Appurtenant structures made of reinforced concrete may include mechanical and electrical control mechanisms. Appurtenant structures may be damaged by the loads imposed by the embankment fill, by foundation deformation, and by structural deterioration.

3.5.3. Most appurtenant structures constitute a discontinuity with the adjacent dam and levee. Some appurtenant structures pass completely through an embankment. Other appurtenant structures, or a part of an appurtenant structure, may simply be located on, adjacent to, or at the toe of, an embankment. In many instances, these structures may be supported and loaded by earth or rock.

3.5.4. A large displacement of an appurtenant structure may compromise the safe operation of an embankment. For example, separation of a conduit joint in an embankment may direct flow along the outside of the conduit, resulting in internal erosion of the embankment or foundation. Another example would be settlement of earthfill adjacent to a structure may result in a crack and allow seepage and erosion along the contact.

3.5.5. Appurtenant structures are subject to the formation of holes, cracks, and corrosion. Such damage may increase the potential for embankment failure by overtopping, internal erosion, or instability.

3.5.6. Appurtenant structures are subject to hydraulic uplift and thus may require seepage and drainage control features incorporated to limit loads against the structures and to discharge collected seepage without causing erosion. Uplift pressure and drain flow rate responses to water impoundment may be monitored for consistency over time to help ensure that the foundation of the appurtenance remains stable.

3.5.7. Instrumentation to monitor performance of appurtenant structures or where embankments are contact with appurtenant structures are typically the same as used for embankments and floodwalls.

3.6. Potential Failure Modes. Typical dam and levee potential failure modes include overtopping, instability, internal erosion, and differential settlement as discussed below.

3.6.1. Overtopping.

a. The spilling of water over the lowest point on an embankment crest is referred to as overtopping. The cause of overtopping may be rising water, a reduction in embankment height, or both. In addition to a rise in the water level against the embankment, overtopping may be prompted by wave action, a sudden movement of soil or rock, or by a gradual process of consolidation.

b. Dams and levees with controllable outlet works and/or spillways may overtop due to an inaccurate forecast of pool or river stage, leading to a misoperation of outlet works, and a missed opportunity to lower reservoir levels in time to accommodate the volume of inflow. Alternatively, both dams and levees may overtop simply due to the extreme magnitude of a flood. A sliding failure at the crest of an embankment, possibly as a result of water rising against the embankment, may allow the water to spill through any resulting gap in the crest.

c. Gradual settlement of an entire embankment crest, or a portion of the crest, may lower the level of flood protection afforded, making overtopping more likely over time.

d. The consequence of overtopping may be severe damage to, or total failure of, an embankment. Once overtopping begins, options for gaining control of the situation are usually limited or nonexistent. Embankment monitoring associated with overtopping consists primarily of tracking crest elevation, slumping, and cracking.

3.6.2. Instability.

a. Embankment instability involves the downward sliding or rotation of a mass of earth within an embankment or foundation. The cause of instability is loss of equilibrium in the driving and resisting forces on a mass of soil in or beneath an embankment. The weights of the embankment and foundation materials are the primary loads, but additional loads may be applied by water impoundment, by the weight of masses placed on the embankment, and by seismic forces.

b. Potential sliding or slipping surfaces within an embankment or foundation coincide with weak material zones, joints, bedding planes, and with the contact surface between dissimilar materials such as earthfill, subsoil, or rock. If the applied shear stress in an embankment exceeds the available shearing resistance along a critical slip surface, instability exists.

c. Factors involved in embankment instability include porewater pressure, the shear strengths of the embankment and foundation, loads exerted against the embankment, and seismic forces. The critical conditions related to porewater pressure are:

(1) End of construction.

(2) Long-term steady-state seepage.

(3) Sudden drawdown.

d. The relative strengths of the embankment and foundation materials affect the characteristics of a mass failure. If the foundation is stronger than the embankment soil, then the mass movement is typically confined within the embankment. Alternatively, if the embankment overlies a soft foundation, the strength of the foundation material influences the proportion of mass movement—not only in the foundation but also in the embankment.

e. A foundation consisting of loose, uniform, or gap-graded cohesionless material may liquefy during an earthquake and result in general instability. Alternatively, smaller portions of a foundation consisting of such material may densify during an earthquake, resulting in localized vertical deformation of the embankment crest or appurtenant structure.

f. Instability may result in overtopping, internal erosion, soil piping, and malfunction of appurtenant structures. An embankment with water impounded against it may fail by overtopping if a slide or rotation is massive enough to lower the crest. If a slide or rotation shortens a seepage path, an increase in the hydraulic gradient across the embankment may lead to soil piping.

g. Another contributing factor for an instability failure mode is erosion of the water side of the levee or upstream side of the dam or foundation due to channel flow. This could occur during normal channel flow or during high channel discharge events. The mechanism could lead to erosion which decreases riverside resisting forces resulting in channel bank and/or levee instability.

h. Embankment monitoring associated with instability consists primarily of tracking surface and/or subsurface movement and measuring porewater pressure in the embankment and foundation.

3.6.3. Internal Erosion.

a. Seepage may cause internal erosion in an embankment, abutment, or foundation if the velocity of flowing water is sufficient to detach and transport soil particles. There are numerous potential flaws that could initiate internal erosion. These can be totally within the embankment, foundation, abutment, or at the contact with rigid structural elements.

b. Some of the flaws related to compacted earth and soil foundations include zones of more pervious or undesirable materials, improperly prepared surfaces or compacted zones, staged construction or seasonal shutdowns, filter incompatible embankment zones, and untreated foundation rock joints or seams. Poor compaction at the contact between soil and a steep rock abutment or rigid structural element may lead to cracking and a potential failure path.

c. Where the geometry of the exposed foundation does not favor the attainment of sufficient compaction, such as beneath an overhang or rock discontinuities, cracking of fill may result from hydraulic fracturing.

d. Higher hydrologic loading typically results in higher gradients and increased potential for internal erosion. Several types of internal erosion are discussed below that should be considered and monitored depending on the characteristics of the loading, embankment and foundation geometry and materials, and appurtenant rigid structural elements.

e. Backward Erosion Piping (BEP).

(1) BEP occurs when soil erosion (particle detachment) begins at a seepage exit point and erodes backward (upstream), supporting a “pipe” or “roof” along the way. As the erosion continues, the seepage path gets shorter and flow concentrates, leading to higher gradients, more flow, and increased potential for erosion. Four conditions must exist for BEP to occur:

(a) Flow path or source of water,

(b) Unprotected or unfiltered exit,

(c) Material susceptible to BEP within the flow path, and

(d) Continuous stable roof forms allowing pipe to form.

(2) BEP can occur in cohesionless soils or those with a low Plasticity Index (PI). BEP is particularly dangerous because it involves progression of a subsurface pipe to the reservoir which may be unobservable depending on the exit location. After developing slowly for an extended time, BEP may suddenly become obvious—at which point, without intervention,

failure of the embankment may follow in only hours or days.

f. Internal Migration (Stoping).

(1) Stoping occurs when the soil is not capable of sustaining a roof or pipe. Soil particles migrate downward primarily due to gravity, but may be aggravated by seepage or precipitation. A temporary void grows in the vicinity of the initiation location until a roof can no longer be supported, at which time the void collapses. This mechanism may be repeated progressively until the core is breached or the downstream slope is over-steepened to the point of instability.

(2) Since by definition roof support is lacking, this mechanism typically leads to a void that may stope to the surface as a sinkhole. Embankments constructed with broadly graded cohesionless soils (e.g., glacial till) and open defects in rock foundations or structures embedded in the embankments (such as leakage into conduits) are most susceptible conditions for stoping due to internal instability/suffusion.

g. Scour Erosion.

(1) USACE distinguishes two types of scour erosion: concentrated leak erosion (CLE) and contact erosion (CE). These occur when tractive seepage forces along a surface (e.g., a crack within the soil, adjacent to a wall or conduit, along the embankment-foundation contact) are sufficient to move soil particles into an unprotected area, or at the interface of a coarse and fine layer in the embankment or foundation.

(2) Once this begins, a process similar to BEP or internal migration could result. CLE or CE do not necessarily imply a backward (upstream) development of an erosion pathway. Enlargement of an existing defect may occur anywhere along the seepage pathway. Excessive seepage along the interface between soil and an abutment or other rigid element may lead to internal erosion and embankment failure or undermining of the rigid element.

(3) Cracking and displacement of a rigid structural element can reduce the hydraulic efficiency of an appurtenant structure and result in higher than expected foundation uplift pressure during hydrologic loading.

h. Internal Instability (Suffusion or Suffosion).

(1) Suffusion or suffusion are both internal erosion mechanisms that can occur with internally unstable soils. It is possible that these mechanisms and internal migration (stoping) can occur in complex glacial environments where tills, glacio-lacustrine, and outwash deposit co-exist.

(a) Suffusion involves selective erosion of finer particles from the matrix of coarser particles (that are in point-to-point contact) in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles. With suffusion there is typically little or no volume change, but seepage volume could greatly increase.

(b) Suffosion is a similar process but results in volume change (voids leading to sinkholes) because the coarser particles are not in point-to-point contact. Suffosion is less likely under the stress conditions and gradients typically found in embankments.

(2) Graded filters and drains can be used to prevent internal erosion for many cases in an embankment, by ensuring that soil particles cannot migrate along a seepage path. Internal erosion may cause severe damage or total failure of an embankment. Once internal erosion reaches the stage of obvious acceleration, options for gaining control of the situation are usually limited to rapid drawdown and the emergency placement of inverted filters at seepage exits. Therefore, the operational goal is to detect internal erosion before it progresses too far.

3.6.4. Differential Settlement. Differential settlement at an embankment or abutment is an abrupt difference in settlement in profile or section which has the potential to initiate cracking. Embankment weight results in foundation settlement and the compression and lateral displacement of the fill. Causes of differential settlement include discontinuities in load, soil compressibility, and foundation geometry. Differential settlement can result in lowering of the crest or cracking, which are discussed previously in the overtopping and internal erosion discussions.

Chapter 4
Instrumentation Program Planning

4.1. Introduction.

4.1.1. Field instruments are more important in geotechnical engineering than in most other branches of civil engineering in which designers have greater control over the materials used for construction. However, indiscriminate installation of instruments and data collection cannot substitute for keen insight and purposeful inquiry. Purposeful inquiry culminates in a specific geotechnical question that an instrument can help answer. In the words of Dr. Ralph Peck (Dunnicliff, 1993): “Every instrument on a project should be selected and placed to assist with answering a specific question; if there is no question, there should be no instrumentation.”

4.1.2. Instrumentation cannot guarantee good design, trouble-free construction, successful performance, or long-term maintenance-free operation. Selecting the wrong type of instrument or placing an instrument in an inappropriate location can provide information that may be confusing or divert attention away from other signs of potential distress. It is not appropriate to mandate instrumentation at every dam or levee with the expectation that some unknown defect will be revealed during monitoring and provide a warning of an impending failure.

4.1.3. Instruments cannot indicate signs of pending deterioration or failure unless they happen to be placed at the right location, maintained, read, and evaluated. Therefore, the use of instrumentation should be planned with the objective of providing the most useful information.

4.1.4. This chapter addresses instrumentation and monitoring planning for an embankment or floodwall. This includes the development and implementation of a surveillance and monitoring plan.

4.1.5. Role Instrumentation in the Risk Framework.

a. A dam or levee safety risk framework relies on the best available science and data in order to help assess, manage, and communicate risks and to make effective and appropriate decisions about associated risks. Instrumentation used to monitor project performance plays an integral role in all three aspects of the risk framework.

b. All Civil Works water control projects must have an adequate level of instrumentation to aid in understanding project performance and monitoring potential failure modes. Data obtained from instrumentation, combined with visual monitoring information, enables engineers to monitor and evaluate the performance of structures during construction and under all operating conditions (e.g., the full range of possible reservoir loads).

c. Where it is determined that instrumentation is a necessary monitoring component, instrumentation will be utilized to verify performance is within tolerable limits, aid in identifying any concerning trends, inform design and computer models, and aid in predicting future performance. Instrumentation evaluations should be used to update O&M manuals, water control manuals, and risk assessments when applicable.

d. The number of instruments, locations, types, and frequency of readings should be commensurate with the project/system risk and significant potential failure modes identified for each project. Installation, repair, and replacement of new devices must be evaluated throughout the life of the project subject to potential failure modes, understanding of the potential consequences, and other considerations that help inform the understanding of the project's risk.

e. Surveillance and monitoring plans should remain adaptive to real-time events but should also remain adaptive to changes in the understanding of the project risk. Planning for increased surveillance and monitoring may be required as loading conditions increase, due to Interim Risk Reduction Measures (IRRMs) or during critical/unusual events such as high/surcharge pool, or during high/record flooding as dictated by the current understanding of the project performance.

f. At the completion of a dam or levee related activity (e.g., inspection, risk assessment, periodic assessment, implementation of IRRMs, construction of an alteration, and development or update of an emergency plan), the final deliverable to the dam/levee owner, sponsor, or operations manager should include a summary of newly collected information, assessment of how the newly collected information in combination with other changed conditions may impact the risk characterization, and adapted risk management recommendations.

g. USACE will make risk-informed decisions, including the development of recommendations related to dams and levees with the goal of meeting Tolerable Risk Guidelines (TRGs). Refer to Engineer Regulation (ER) 1110-2-1156 Safety of Dam: Policy and Procedures and the latest Levee Safety Program policy and procedures guidance documents for background and rationale of TRGs for dams and levees.

h. Because risks are dynamic, meeting TRGs for dams and levees requires continuous assessment, management, and improvement. Instrumentation data collection and evaluation will play a key role in informing risk management recommendations with the goal of evaluating and achieving TRGs.

4.1.6. Uncertainty.

a. Monitoring the performance of dams and levees is necessary because the uncertainties in site investigation, design, construction, and operation cannot be fully resolved. The risk of failure, however remote, always exists. When planning instrumentation for a dam or levee, there will often be uncertainty related to whether a particular dam or levee feature should be monitored and, if so, to what level.

b. Risk assessments can be used during design and rehabilitation of dams and levees to identify PFMs. The results may readily indicate some conditions that should be monitored, but other conditions may have less estimated risk and it will be uncertain if instrumentation is justified.

c. When planning what project features to instrument, uncertainty should be discussed in terms of what may be certain, what is likely, but not certain, and what is possible, but not likely in evaluating which features that should be monitored for dam and levee safety.

4.2. General Planning Considerations. Instrumentation can provide data to assess a variety of parameters including groundwater pressure, deformations, total stress, temperature, seismic events, leakage/seepage, and water levels at various stages of a project's lifecycle. The following paragraphs discuss various situations in which instrumentation may be planned to monitor for these various parameters.

4.2.1. Inform Design/Modification Features.

a. Instrumentation data can play a critical role in developing design features and supporting modification decisions of dams and levees by reducing uncertainty of certain design parameters (e.g., hydraulic conductivity) and improve design efficiency.

b. Data collected throughout the life of dams and levees can be used to advance engineering design practices/manuals to support safer and more economical designs.

4.2.2. Verify Design/Modification Parameters. After construction of a dam or levee project or modification, instrument data can help engineers and geologists evaluate the performance of the as-built project and verify the design assumptions. Observations and instrument data combined with performance assessments help resolve unknowns.

4.2.3. Substantiate Construction Techniques. Instruments installed before or during construction may help determine the effectiveness of new design and construction techniques. After that, the same instruments may help engineers and geologists evaluate the effect of the new techniques on an embankment or floodwall during first filling or exceptional loadings.

4.2.4. Analyze Past Performance. Instruments installed before or after an event may provide useful data and help identify the causes of observed distress and/or satisfactory performance. This information could also identify new and/or developing potential failure modes.

4.2.5. Predict Future Performance. Observations and instrument data obtained during lesser floods may be projected to predict performance during high pool/flood levels. Past instrumentation data/observations indicating successful performance may not necessarily translate to future successful performance since conditions can change over time.

4.2.6. Providing Timely Warning.

a. Instrumentation may be designed and operated to provide a timely warning to operators of an impending failure. For instrumentation to be capable of providing timely warning to operators, the plan requires consideration of the relationship between measurements and observations as well as the evaluations required to ascertain the level of concern and needed actions.

b. The surveillance and monitoring plan should ensure appropriate instrumentation exists to inform emergency action plans and O&M plans so that timely actions can be taken when alarming conditions are detected.

4.2.7. Unintended Effects of an Instrument.

a. A dam, levee, floodwall, or foundation may be damaged during installation of an instrument, and an instrument may cause damage during operation of the project. For example, drilling to install an instrument may crack an embankment, floodwall, or foundation. During project operation, an instrument drill hole or cable trench may provide a pathway for seepage.

b. An instrument may obtain faulty measurements if installation or operation of the instrument has significantly altered the physical characteristics of the mass in the immediate vicinity of the instrument. Altered physical characteristics can include hydraulic conductivity, compressibility, and the state of stress.

c. Engineers should carefully balance the risks introduced by installing an instrument against the benefit the instrument is expected to yield. A strong, risk-informed case should be made before making the final decision to install instruments into the core of a dam or levee. Engineers should understand the installation requirements and techniques and avoid critically harming the embankment with instrument installations (refer to ER 1110-1-1807).

4.3. Instrumentation and Monitoring Program Team.

4.3.1. Successful instrumentation for an embankment requires a staff of competent engineers, geologists, and technicians. The quality and usefulness of instrumentation and monitoring data is directly related to the skill and training of the persons involved from design, installation, data collection, to evaluation. A lack of necessary skill and diligence among staff members may result in the collection of incomplete, incorrect, and misleading data.

4.3.2. A range of skills is necessary to collect good data, which may involve many persons performing various tasks. Instrumentation-related responsibilities may vary between organizations, depending on the size of the system.

4.3.3. The responsibilities of instrumentation and monitoring staff changes as individuals change disciplines, advance, transfer, or retire. Therefore, documented procedures, continual formal training and informal sharing of knowledge are needed to retain skills and site-specific knowledge among staff members.

4.3.4. All instrumentation and monitoring require the staff to perform eight general functions:

a. Instrumentation and monitoring lead,

b. Routine inspection,

c. Instrument installation,

d. Recalibration and maintenance,

e. Data collection,

f. Data entry,

g. Data management and plotting, and

h. Evaluation and reporting.

4.3.5. The eight functions are described separately, but an individual staff member may be assigned more than one function, depending on skill and other factors. For a small number of projects, one person may perform most, if not all, functions. In contrast, for a large number of projects, a staff member may be assigned a full-time, specialized function on one or more projects.

4.3.6. If instrumentation and monitoring at projects is large and complex, technical and budget roles may be kept separate while ensuring that the technical and budget staff members share information about multiple project needs and concerns.

4.3.7. Although communication is not listed as one of the eight general functions, communication is necessary to perform the general functions. For example, communication is required to ensure the monitoring objectives are thoroughly understood by the entire performance monitoring team and the specific responsibilities of each member is made clear. Open communication is needed between team members so that any abnormalities or concerns are adequately investigated and documented.

4.3.8. Instrumentation and Monitoring Lead.

a. To provide continuity and a point of contact for oversight, each USACE district project's instrumentation and monitoring program should be led by one full-time permanent employee. The lead function may be assigned to the Dam or Levee Safety Program Manager or to a Senior Engineering Geologist, Geotechnical, or Instrumentation Engineer. Responsibilities include:

(1) Ensuring project-specific surveillance and monitoring plans are up to date and implemented.

(2) Designing, selecting, procuring, and oversight of installation of instrumentation and automation equipment.

(3) Coordinating tasks for routine inspection, data collection, data management, and evaluation.

(4) Ensuring all instrumentation data collection and evaluation personnel are trained to recognize indicators of distress. For example, dam and levee safety training is available through:

(a) Project-specific dam or levee safety training.

(b) Federal Emergency Management Agency (FEMA), Training Aids for Dam Safety (TADS).

(c) Proponent-Sponsored Engineer Corps Training (PROSPECT) Dam and Levee Instrumentation and Performance Monitoring.

(d) PROSPECT Levee Safety Fundamentals.

(e) PROSPECT Dam Safety.

(5) Interpreting and analyzing data and providing technical assistance or guidance to other geotechnical engineers responsible for final data analysis.

(6) Coordinating instrumentation and monitoring conclusions, data, and associated actions between risk assessments, interim risk reduction measures, and potential failure mode analyses.

(7) Performing oversight to ensure overall success in monitoring the project.

(8) Communicating with the appropriate chain of command any project recommendations and results of evaluations.

b. A backup to the lead should also be established. This backup should be knowledgeable enough on the instrumentation and monitoring program that they can take on a lead role if a situation or event arises and the lead is unavailable.

4.3.9. Inspection. Inspection of project features for engineering evaluations is discussed in ER 1110-2-1156 and other applicable levee safety program guidance documents. All staff members setting foot on the project site are responsible for looking at the embankment and its appurtenances as they perform normal tasks and for reporting any abnormality discovered. The project lead and those involved in the evaluation should be involved in the inspections to be able to relate instrumentation analysis to visual observations.

a. Inspection data is valuable to instrumentation data evaluation and vice versa. Instrumentation is an extension of inspection, providing objective and quantitative measurements of subsurface conditions. Subsurface changes detected by instruments may be related to visible changes on the surface of the embankment. Instrument data should be compared with the visual appearance of the embankment to evaluate whether or not the data are consistent with surface conditions.

b. Where automation is installed, trained staff should avoid relying too much on instrument data and should perform an appropriate amount of visual monitoring also. Inspections should also note the physical condition of instruments to identify signs of disturbance or damage from mowers and vandalism that could explain instrumentation data abnormalities.

4.3.10. Instrument Installation.

a. Staff members engaged in installing instruments are responsible for ensuring the instrument is properly installed and the installation properly documented. Installation of field measurement devices and automation equipment should be coordinated by the instrumentation and monitoring lead, and where applicable, the appropriate district subject matter expert (SME).

b. All previous site investigation data should be carefully reviewed prior to drilling additional borings and/or installing instruments in a dam or levee. Review of existing data will provide the necessary information needed to target specific areas/zones to install instruments.

c. Changes in instrumentation layout and inventory should be checked for conformance to long-range plans for monitoring, considering interim risk reduction measures and potential failure modes. Drilling plans, details, and procedures must be reviewed for compliance with ER 1110-1-1807: Procedures for Drilling in Earth Embankments. A qualified geologist or geotechnical engineer familiar with device operation and requirements should be onsite to log and supervise installation.

d. Narrow impervious cores, inclined filters, and foundation cutoffs, including grout curtains can be damaged by drilling. In most cases, these features are best investigated by non-intrusive methods, such as surface geophysical methods, avoiding drill hole penetration, and carefully drilling holes at locations with a lower risk of inducing damage. Foundations, joints, bedding planes, shear zones, gravel beds, caverns, and faults that exist beneath a dam or levee may all be intercepted with a drill hole to place an instrument.

4.3.11. Recalibration and Maintenance. Staff members who maintain and recalibrate instruments are responsible for performing the work on schedule in the proper manner and for documenting the work. All work should be coordinated with the instrumentation and monitoring lead, as the work may impact the data results and be considered in evaluations.

a. The maintenance and recalibration of instrumentation, telemetry, and automated equipment is becoming increasingly more complex due to the partial or complete automation of many instrumentation systems. Some instruments and their ancillary components may require periodic recalibration by the manufacturers.

b. Maintenance and recalibration requirements should be reviewed and discussed as part of the planning process, ensuring that procedures and funds exist for these activities and that the personnel responsible for data collection and review know the requirements.

4.3.12. Data Collection.

a. Staff members who collect data are responsible for collecting and submitting the data as per schedule outlined in the surveillance and monitoring plan, reporting any signs of instrument malfunction, reporting any indication of distress or unusual reading. A good data collection staff is comprised of persons who continue to learn and train fellow staff members. Data collection may be a full-time or additional duty for a staff member. Persons assigned to a data collection staff may be:

(1) Members of the project operations staff employed by USACE or the sponsor,

(2) Central staff members of the engineering district, or

(3) Contract personnel.

b. Personnel who collect data should take measurements consistently, which will keep a person's measurement bias consistent. Personnel should be interested in the work and demonstrate appropriate attention to detail.

c. Emergency situations, flood fights, and high-water events have a critical need for monitoring and documentation of project performance. During these critical events, the amount of data to collect and interpret may be much greater than during normal operation. Therefore, contingency plans may be needed to supplement the project monitoring team.

d. Training.

(1) The training of temporary and contract employees to collect data requires diligence.

The length of time a temporary or contract employee may be involved in instrument monitoring can vary greatly, yet continuously high-quality data are needed over the life of the project. Therefore, temporary and contract employees should receive full training on the specific data collection tasks they are to perform and be informed of the importance of that work.

(2) The training of a person who collects data should include:

(a) General project information and history;

(b) Purpose of the instruments;

(c) Project performance and instrument response history;

(d) Steps to take if unusual data are obtained, and which field conditions are apt to produce unusual data;

(e) Instrument locations, setup efficiencies, and needed problem-solving methods;

(f) Proper data collection techniques to minimize data collection error; and

(g) Personnel safety requirements due to physical hazards of collecting data and maintaining the instruments.

4.3.13. Data Entry.

a. Staff members who enter data into digital records are responsible for timely, complete, and accurate entry. From the management perspective, accurate data entry depends on:

(1) Determining who enters data and assigning responsibility for accurate data entry to that person.

(2) Promoting good habits to minimize or detect errors.

(3) Ensuring that those entering data recognize abnormal or unsafe values.

(4) Making good use of technology to minimize opportunities for error.

(5) Checking data collected by automated instrument systems for obvious errors.

(6) Feedback to data entry personnel from data reviewers.

b. The person who collects data should be the person most familiar with the content of the data who is in the best position to ensure that the data are accurately recorded. Therefore, the person who collects data should also enter the data. If data are recorded in handwriting and later entered electronically, the person who recorded the data in handwriting should also enter the data electronically.

(1) Guidelines for minimizing errors are as follows:

(a) The number of persons involved in data collection and entry should be minimal.

(b) The number of steps required to collect and enter data should be minimal.

(c) Data should first be checked upon entry.

(2) To recognize abnormal or alarming data, persons entering data should have at hand the measurement values indicating both normal and unsafe operation. If abnormal or alarming data are observed, they should immediately notify personnel responsible for managing the instrumentation and monitoring.

c. The use of automated instrumentation does not relieve persons of the responsibility to check the data collected. Automated instrumentation uses system configurations and software packages to collect data and transfer the data directly to the project database. However, automated systems are susceptible to systemic errors, similar to the corruption of a spreadsheet by entering data out of sequence or under incorrect headings.

d. Although automated instrumentation can signal if data tolerances are exceeded, automatic or remote entry does not eliminate the need for a person to check for errors. Typically, automated data are checked by data managers rather than data entry personnel.

4.3.14. Data Management and Plotting.

a. Staff members who manage and plot data are responsible for preserving the data, noticing patterns that indicate faulty data, and organizing information in a useful manner. The duties of persons who manage and plot data include:

(1) Controlling data quality.

(2) Ensuring engineering units are correct.

(3) Using appropriate software skillfully.

(4) Coordinating with persons working in an information technology office to solve equipment and software problems.

(5) Transferring data from and to remote sites.

(6) Writing computer codes or scripts.

(7) Creating plotting routines.

b. Persons who manage data systems should not only be familiar with computers and software but should also understand geotechnical concepts sufficiently to work closely with the instrumentation and monitoring lead and the engineers and geologists who review and analyze data.

4.3.15. Evaluation and Reporting.

a. Staff members who evaluate and report data are responsible for analyzing and presenting information so that it can be used for decision making in daily operation, emergencies, and planning.

b. Evaluation and reporting should be performed by engineers or geologists experienced with dams and embankments who have the training and experience to perform the work according to USACE guidance requirements to meet the needs of the project.

c. The engineer or geologist should be familiar with the embankment, including the results of periodic inspections and risk assessments, identified potential failure modes, and the purpose of the instruments onsite, and performance history. Likewise, instrumentation of structural or mechanical features should be reviewed by engineers of the appropriate discipline.

d. Persons responsible for reporting the findings of an evaluation should prepare a report that supports the objectives of the surveillance and monitoring, the project's current and expected future performance, and the adequacy of the instrumentation and monitoring program.

e. Mathematical modeling using computer programs and calibrated with instrumentation data can improve the understanding of potential seepage quantity and pressure distribution, slope stability, foundation stability, and deformation. Modeling should be performed as appropriate to better understand instrumentation response as related to project performance.

f. Report content should focus on the proper topics, stress the implications and significance of the findings, and present the material in a suitable written and graphic format. Proper documentation of the instrumentation system, including instrument location plans and profiles and minimum reporting requirements, is discussed in ER 1110-2-1156, Sub-Appendix U-1.

g. Monitoring needs and objectives change and require regular re-evaluation to determine if the established surveillance and monitoring plan meets current needs of the project. Instruments and staffing patterns that do not satisfy current needs should be modified as necessary. During any re-evaluation or project modification, engineers should familiarize themselves with the success or failure of previous efforts at the project site and of relevant efforts undertaken at other sites.

h. The results of the evaluation should be communicated to those involved in the instrumentation and monitoring program, including the data collectors and project operators. Recommendations should be communicated to the appropriate chain of command and be implemented with the appropriate level of urgency. Conclusions may result in a need to modify the instrumentation and monitoring plan, emergency operations plan, or O&M plan.

4.4. Surveillance and Monitoring Plan.

4.4.1. This section is intended to outline the elements of an effective surveillance and monitoring plan. Ideally, this information would reside in a stand-alone comprehensive surveillance and monitoring plan. However, in an effort to reduce duplicative information, references to other relevant documents may be made. Related documents typically include risk assessments, O&M plans, program management plans, and emergency action plans.

4.4.2. The surveillance and monitoring plan should be considered a living document and should be modified over time as the data is evaluated, a better understanding of project performance is ascertained, and uncertainty is reduced. In some cases, additional instrumentation may be required, or monitoring can be increased or reduced as the evaluation dictates. The plan should be reviewed and modified as appropriate.

4.4.3. Planning for instrumentation is a team effort whether it is for new construction or an existing project. A team begins planning by collecting relevant up-to-date information and establishing a rationale for the instrumentation and monitoring needs of the project.

4.4.4. Relevant documentation may include risk assessments, maps and drawings that portray subsurface materials and material properties, drawings and descriptions of as-built conditions, and locations of existing instrumentation. For projects with existing instrumentation, the documentation should include plots of any existing instrument data and subsurface profiles and sections.

4.4.5. Having reviewed the documentation, the team can then establish a rationale for the surveillance and monitoring plan. That rationale is the dominant management concern that guides subsequent technical decisions and provides the basis for planning steps:

a. Define the objectives and parameters to be monitored.

b. Determine the specific locations for instrumentation.

c. Determine the monitoring frequency and thresholds.

d. Select the appropriate instrument.

e. Identify the data collection and management techniques.

f. Identify the staffing requirements.

g. Identify the budget requirements.

4.4.6. As the surveillance and monitoring plan develops, a review of the planning steps can determine if the plan remains true to the original rationale and if appropriate updates to the surveillance and monitoring plan are needed to improve risk-informed decision making.

4.4.7. Defining the Objectives and Parameters to Monitor.

a. In the context of dam and levee safety, the primary functions of instrumentation are to supplement visual monitoring, provide site condition information, assist in determining structural and foundation performance, and provide early indications of potential failure. The purpose of each instrument should be clearly documented in the surveillance and monitoring plan. Comprehensive performance monitoring considers, along with project risk, the special conditions and features of the site.

b. Each instrument used on, in, or near a project should be individually selected and placed to monitor for a specific concern which typically is related to a potential failure mode or to verify project performance. Before addressing measurement methods, a list should be made of concerns that could arise during the structure lifecycle. Relevant concerns that should be identified may include:

(1) Initial site conditions.

(2) Dam and levee performance during:

(a) Construction.

(b) First loading.

(c) Drawdown.

(d) High-water event.

(e) Seismic loading.

c. Site Characterization.

(1) A surveillance and monitoring plan should include the project-specific site characterization. The site characterization information includes:

(a) Describing the geologic setting, stratigraphy, and physical properties of the foundation.

(b) Describing groundwater and environmental conditions.

(c) Land use history.

(d) Seismic history.

(e) Performance history.

(f) Dam, levee, or floodwall type along with layout and construction details, and appurtenant structures.

(g) Construction methods.

(h) Data from and about existing or abandoned instruments.

(2) Collaboration with engineers specializing in geotechnical, hydraulic, and structural engineering and with geologists will further the understanding the project and embankment details. Where available, risk assessments should be reviewed and referenced appropriately.

(3) For some sites, knowing current and potential land uses near the site may be useful. Drawings combining performance data, subsurface information, and project sections help team members understand the project, support planning, and support an emergency assessment that may or may not result from the monitoring of the project.

d. Identifying Parameters to Monitor.

(1) Understanding the risk characterization of a dam or levee and how an instrumentation and monitoring plan can help reduce uncertainty, inform TRGs, and manage risk—keeping in mind that risk can change over time. The team should review potential dam and levee failure modes established from the most recent risk assessment and identify the related parameters to monitor. The typical parameters monitored include:

(a) Seepage quantity and quality.

(b) Groundwater and porewater pressure.

(c) Total stress.

(d) Compression.

(e) Horizontal and vertical deformation.

(f) Load and strain within structural elements.

(g) Seismic response.

(2) For example, a potential failure mode for a soft foundation may be related to a slope failure or settlement. Therefore, the key parameters to measure in that case will include porewater pressures and deformation. For a second example, if excessive seepage and internal erosion may occur in an abutment, the parameters to measure would include groundwater levels, seepage flow rate, and water quality.

(3) Additional parameters needed to support analysis and interpretation of data should also be monitored, which may include:

(a) Surface water levels;

(b) Ambient temperature, precipitation, and barometric pressure;

(c) Seismic load; and

(d) Ground and water temperatures.

(4) Plans should state the methods to use for measuring the additional parameters. For instance, analysis of PZ data may require barometric pressure corrections, and a means for reliably obtaining this correcting variable should be established.

e. Critical Instruments.

(1) The instruments selected for monitoring the most critical potential failure modes should be denoted as critical instruments, and a monitoring frequency should be established that is commensurate with the associated risk under various loads.

(2) Selection and reading frequency for critical instruments should be re-evaluated during instrumentation evaluations and updates to interim risk reduction measures plans, O&M manuals, water control manuals, and/or other risk management documents. Critical instruments should be documented within emergency action plans. Appendix B includes examples and discussion of selecting critical instruments for dam and levee projects.

4.4.8. Determine the Specific Locations for Instrumentation.

a. Locations for instruments should be determined based on the predicted behavior of the site, potential failure modes, and the methods of analysis used to interpret data. The number of locations that will require instrumentation will need to be determined by the engineer or geologist. Multiple instruments may be required to monitor a given parameter and/or to get enough data resolution. For example, to understand groundwater flow, multiple PZs may be required to evaluate gradients from one area of the project to another.

b. Monitoring locations should be chosen and identified in 3D space. For example, the location of PZ should be defined to include both the horizontal positioning (i.e., northing and easting/station and offset) and the vertical positioning (i.e., top and bottom elevation of monitored zone). It is critical to identify the intended monitoring interval in order to ensure the appropriate geologic unit or design feature is being monitored.

c. Drawings showing the instrumentation should include both plan and cross-section views. Geologic units, design features, and instrument location details should be shown on the drawings.

d. Site visits should be performed to verify the conditions surrounding the installation locations. The physical conditions in the area will likely impact the type of instrument that is selected. For example, if an instrument location is near a roadway, the installation method should consider traffic, runoff, and instrument access.

e. Instrument locations and layouts are best configured so that multiple instruments and different instrument types provide the data. This allows for the appropriate redundancy and ensures good data for comparison between monitoring intervals.

f. Survivability of an instrument during construction and long-term operation should also be considered. Harsh conditions not only affect the instrument readings but may also cause permanent instrument damage. Instruments may be damaged by electrical interference, by the elements, by pests, or by human actions.

(a) Damaging elements include lightning, precipitation, runoff, high and low temperatures, frost, wind, dust, and high humidity. Damaging pests include insects and rodents. Damage by human actions includes traffic collisions, vibrations caused by operating equipment, and vandalism.

(b) Damage to the instrument, riser, cable, and ancillary read-out structures during construction can be avoided with good design and temporary and permanent protection of exposed parts. Protection may include ballasts, waterproof housing, insulation, or desiccant packs.

4.4.9. Determine the Monitoring Frequency and Thresholds.

a. The data and evaluation for any instrument should be defined for both normal conditions and increased loading events. The selected frequency should consider the project risk and potential failure mode and monitored parameter. There should be enough data collected to understand the project response to various loading events. Critical instruments may require a higher data collection frequency.

b. Where higher data collection frequency is needed, or where the project site is remote, an ADAS should be considered. An ADAS can be operated remotely to provide accurate and reliable real-time data collection. For remote sites, sites difficult to access, and projects that require frequent monitoring, automated systems may be the only safe or viable option for data collection. Further discussion on ADAS is presented in Chapter 6. Additional details for determining monitoring frequencies are included in Chapter 8.

c. Parameter Thresholds.

(a) The team should identify a safe threshold value of a given measured parameter. This threshold should be set at a point to get attention, but well before the parameter is indicating unsafe operation. For example, deformation and porewater pressure thresholds may be established for raising an embankment on a soft foundation.

(b) If those thresholds are exceeded, this would trigger an evaluation of the results, which may result in the placement of fill being halted until deformation or higher porewater pressure than desired has dissipated. Additional details for developing thresholds are included in Chapter 8.

4.4.10. Select the Appropriate Instrument.

a. After careful review and identification of the project monitoring requirements and selection of the appropriate installation locations, the appropriate instrument type needs to be selected. Chapter 5 discusses in detail some of the various instruments that can be used to monitor each parameter and consideration. Additional considerations are included in this section.

b. Reliability is one of the most important features in a monitoring instrument, ensuring that data of adequate accuracy and sensitivity are obtained throughout the required monitoring period. Instrument reliability supports the management of long-term risk presented by embankments.

c. In some cases, engineers and geologists have mistakenly preferred other instrument characteristics over reliability. For example, designers have preferred instruments of unnecessarily high accuracy at the expense of resilience. The most reliable instruments are often the simplest devices, and the balance between reliability and needed accuracy should be assessed.

d. Material and component performance should be assessed by reading manufacturers' specifications and by discussions with project operators. A given parameter may be measured by different types and brands of instruments, each with different advantages and disadvantages that need to be considered before selection. In addition, the performance record of commercially available instruments should be considered.

e. The operating environment in which the performance of a given model of instrument was recorded should be compared to the operating environment of the proposed installation to determine the relevance of the performance record. Likewise, the performance and environmental limitations of the electronic components of an automation system should be assessed.

f. Understanding the operating principles and limitations of an instrument supports selecting the right instrument. Users of instruments benefit from discussing an application with a manufacturer's representative or with an instrumentation consultant before the selection of an instrument. During those discussions, all relevant information should be shared with the manufacturer's representative about the planned application and the objectives of the monitoring device.

g. Where instrumentation or an automated system are already in place, there should be a consideration of compatibility requirements as well as varying operational and maintenance requirements between instrument types. Further, the instrument longevity and ease of replacement should also be considered.

h. The accuracy required will be dictated by the amount of change that needs to be detected. For example, if there is a concern that a half an inch of movement will cause a concern, then the measurement tool should have an accuracy that is appropriate for that measurement.

i. Choosing an instrument that only has the accuracy of an inch will not provide the level of accuracy to adequately monitor the parameter. In addition, there should be consideration as to the range of measurement expected. For example, an instrument that can only detect up to an inch of movement would be insufficient where greater than an inch is expected to be measured and within tolerable limits.

4.4.11. Identify the Data Management and Quality Control Procedures.

a. The surveillance and monitoring plan should describe the data management procedures to ensure that data is collected, processed, and reviewed in a timely fashion and that appropriate personnel have timely access to the data. Details should include how data (both manual and automated) will be collected and transferred for processing and storage. Where automation is used, a schematic showing how data flows from the monitoring point to a database should be included.

b. The location, file structure, and appropriate access requirements should be outlined to facilitate the appropriate level of data organization and access. Appropriate procedures to verify data collection, review, and evaluation at the prescribed frequencies should be outlined. Data processing calculations required should be included. Additional discussions on data management are included in Chapter 8.

4.4.12. Identify Staffing Requirements.

a. Staffing is affected by the cycles of planning for instrumentation and monitoring. Most USACE embankments are decades old and the staffing arrangements are often stable. However, staffing is affected by changes in individual assignments, developing problems in the embankment, changes in safety criteria, technology, relationships with project sponsors, funding, and norms in hiring or contracting for labor. In addition, entirely new projects may arise, requiring an original comprehensive staffing plan for investigation, design, construction, and operation.

(a) When practical, individuals involved in the surveillance and monitoring plan activities should be the staff members who are most familiar with the dam project or levee system
(e.g., participated in inspections, involved in the risk assessments or periodic assessments,

reviewed/designed the most recent IRRMs or modifications, or evaluated past performance data).

(b) A staffing plan should provide for:

- Identifying different degrees of professional preparation,
- Identifying qualified persons,
- Continuing training,
- Defining routines,
- Assigning responsibility, and
- Preparing individuals for advancement.

(c) An efficient staff is a mixture of persons with different degrees of professional preparation, including technicians, engineers, geologists, and managers.

(d) Persons selected for staff membership should be responsible, diligent, task and detail oriented, able to recognize patterns, and possess the specialized technical skills needed. For example, persons responsible for monitoring instrumentation should determine if the instruments are functioning correctly and be able to quickly recognize anomalous and erroneous readings.

(e) Continued training is a staffing function and should be a budget component. To supplement internal site-specific training, participation in seminars and workshops helps staff members stay abreast of current technology and lessons learned from embankment projects. Instrumentation and inspection personnel need an appropriate background in the fundamentals of geotechnical engineering, particularly as applied to dam and levee safety.

(f) A staffing plan includes defining the routines that characterize each task. For example, procedures for collecting, storing, processing, presenting, interpreting, and reporting instrument data should be developed prior to installation of the system. During the planning and design phases, procedures and schedules should be developed for regular maintenance of the instruments, read-out units, and other accessible system components.

(g) Responsibilities for performing tasks should be made clear to each staff member. Depending on the size of the instrumentation and monitoring system and the staff, some responsibilities may be shared, and some team members may be tasked with several roles.

b. Opportunities should be provided for personal development and advancement. The knowledge and skill amassed by experienced staff members is valuable and should be preserved by rewarding excellence.

4.4.13. Identify Budget Requirements.

a. An embankment performance monitoring budget outlines the amount of money required to implement a complete instrumentation and monitoring system for the project. The budget should reflect the actual needs over an annual or longer period. The needs should be documented with respect to risk and likely costs. Therefore, if the funds available appear to be less than the budget estimate, the appropriate course is to present the case for obtaining the full amount of funding needed.

b. The duration of the monitoring period affects the budget. For example, the monitoring period may match the life of the structure or may only be a short period during construction.

c. Budget estimates should include the costs associated with both the procurement and installation as well as the instrumentation and monitoring program management costs. Costs associated with procurement and installation may include:

(a) Installation crew mobilization,

(b) Costs associated with drilling (subsurface instrumentation),

(c) Original and replacement instruments,

(d) Backfill materials,

(e) Field installation and ancillary components, and

(f) Alarms and emergency notification equipment.

d. Costs associated with the instrumentation and monitoring program management may include:

(a) Data collection,

(b) Data transmission and storage (e.g., cellular, satellite service, database),

(c) Data evaluation and reporting,

(d) Staff training,

(e) Instrument maintenance and calibration,

(f) Automation hardware and software upgrades and long-term protection,

(g) Instrument troubleshooting,

(h) Security and vandalism, and

(i) Decommissioning.

e. Once planning for instrumentation and monitoring is complete, the design should be reviewed and adjusted as necessary. After the design has been finalized, the preliminary budget should be updated and finalized.

4.5. Identify the Acquisition and Installation Strategy. Unlike the comprehensive planning described in Section 4.4, this section focuses on planning for the installation of an instrument after the type of instrument, instrument features, and location for installation have been determined.

4.5.1. Acquisition and Contracting.

a. Regarding the acquisition of material and equipment, direct government procurement of instruments, cable, tubing and conduit, collection equipment, database setup, and other items is recommended to the extent practical.

b. Regarding construction, setup, and testing, a given task may be performed by USACE staff or a contractor. If contractors are used, close oversight by government personnel is necessary.

c. The procurement method chosen for contracting may depend on the number of required instruments, the anticipated difficulty of installation, and legal requirements. In most cases, the procurement method adopted should permit evaluating the qualifications of the bidders and should not be based solely on the lowest bid. Typical contracting mechanisms include:

(a) Request for proposal (RFP),

(b) Indefinite delivery/indefinite quantity (IDIQ) task order, and

(c) Inclusion in a construction contract.

d. An RFP method may be used where low cost is not the sole consideration and the buyer is not required to use competitive low-cost bidding procedures. For installation services, maintenance and repair, hardware and software, and large projects, the RFP method may be preferable to other procurement methods because bids can be evaluated for the factors of:

(a) Technical expertise,

(b) Equipment and materials,

(c) Personnel qualifications,

(d) Financial and business strength,

(e) Safety and environmental concerns, and

(f) Price.

e. Each RFP procurement factor should be weighted for evaluation of proposals. Often, the RFP method results in a more favorable combination of quality and price than procurement by low-bid price alone.

f. Use of existing IDIQ contracts could be more timely than a RFP due to the reduction in contracting time, but may not offer the most competitive cost.

g. The inclusion of instrumentation systems in a construction contract may afford competitive pricing and reduce delay. However, the quality of the installation, resulting data, and maintenance during construction typically depends on a subcontractor and may not provide the best results.

4.5.2. Specifications and Scheduling.

a. Proper installation is critical for achieving reliable instrument performance and data acquisition. Planning for proper installation includes writing specifications and determining a schedule. Specifications may be either performance based or prescriptive.

(1) Performance specifications describe the desired result of the installation, leaving the means to the contractor. However, the contractor should submit detailed means and methods for review and approval by an instrumentation specialist.

(2) Prescriptive specifications describe in detail the means and methods of installation, although a contractor may submit a variance to the specification for review and approval.

b. Installation procedures should be planned well before the date scheduled for installation. If drilling is required, the guidelines presented in ER 1110-1-1807: Procedures for Drilling in Earth Embankments should be followed. Written step-by-step procedures should be prepared, making use of the manufacturer's instruction manual and the designer's knowledge of site-specific geologic conditions.

c. The written procedures should include a detailed listing of required materials and tools, not only for installation but also for protection and survivability. Buried cable locations and other subsurface locations should be clearly documented on as-built drawings to minimize the risk of future damage. In some areas, a bullet-proof housing or other tamper-resistant enclosure may be necessary.

d. Installation record sheets should be prepared for documenting factors that may influence measured data. Designers should establish standard nomenclature and labeling conventions to identify instruments, cables, and other components during installation, operation, and maintenance.

e. A preliminary schedule should reflect the time needed to procure instruments and to coordinate with other parties. If an instrumentation system is installed by contractors, government personnel should be onsite during installation to learn the procedures, to ensure quality, to obtain accurate as-built data such as installation depth, and to obtain initial instrument readings. In some cases, incorrect initial readings or as-built information can permanently limit the quality of future readings.

f. For embankments under construction, installation plans should be coordinated with the general construction contractor. Arrangements should be made for access, for safety, and for temporary and permanent protection of instruments and monuments. The final installation schedule should be compatible with the construction schedule.

g. For both new and existing embankments, survivability should guide decisions determining the precise location of instruments with respect to traffic patterns, operation of project equipment, and access to the site.

LEFT BLANK INTENTIONALLY

Chapter 5
Measurement Methods

5.1. Introduction.

5.1.1. This chapter describes measurement methods used to monitor embankment dams and levees and concrete appurtenant features. Methods of surveying and measuring parameters related to concrete dams, such as joint movement, uplift pressure, strain, stress, and leakage are outside the scope of this manual but are presented in the USACE publications:

a. EM 1110-1-1002: Survey Markers and Monumentation,

b. EM 1110-1-1003: NAVSTAR Global Positioning System Surveying,

c. EM 1110-2-1009: Structural Deformation Surveying, and

d. EM 1110-2-4300: Instrumentation for Concrete Structures.

5.1.2. Additional detailed information on measurement methods applicable to dams and levees is included in ASCE (2018), Bassett (2012), Dunnicliff (1993), Hanna (1985), and the BOR (1987).

5.1.3. Instruments can be read manually or automatically. Manually read instruments are often simpler and more cost-effective than automation in short-term applications that do not require high reading frequency and accuracy. Therefore, manual instrumentation may be preferable for projects with low risk, few instruments, and slowly changing parameter values.

a. In contrast, if many instruments are at a site or conditions are expected to change rapidly, such as PZ responses during grouting, electronic instruments connected to an ADAS have a considerable advantage over manual instruments.

b. Automation of electronic instrumentation generally permits the collection of much more data than is practical to retrieve manually or with portable read-out devices. Therefore, electronic instrumentation facilitates higher data resolution due to a higher frequency of data collection, and if readings can be transferred directly from the data-gathering device to the database, human transcription errors are eliminated.

5.1.4. For all instruments, the reliability of the measurements is limited by instrument accuracy, precision, and resolution; methods for transferring and uploading data; and human error in reading and recording data.

a. The differences between accuracy and precision are illustrated in Figure 5.1. The center of the target represents a true value, and the stars represent the measured values. Accurate measured values are near the true value, even if somewhat variable. Precise measurements provide closely similar results but may not be close to the true value. Instrumentation well manufactured, well installed, and properly operated should provide both accurate and precise measurements.

b. Instrument type selection and data collection methods should be mindful of the accuracy and precision required to adequately monitor the behavior of the structure. A less precise and/or less accurate measurement may be adequate for situations that seek a crude confirmation of structural stability. Other high-risk scenarios may warrant the expense that a highly precise and accurate monitoring system requires.

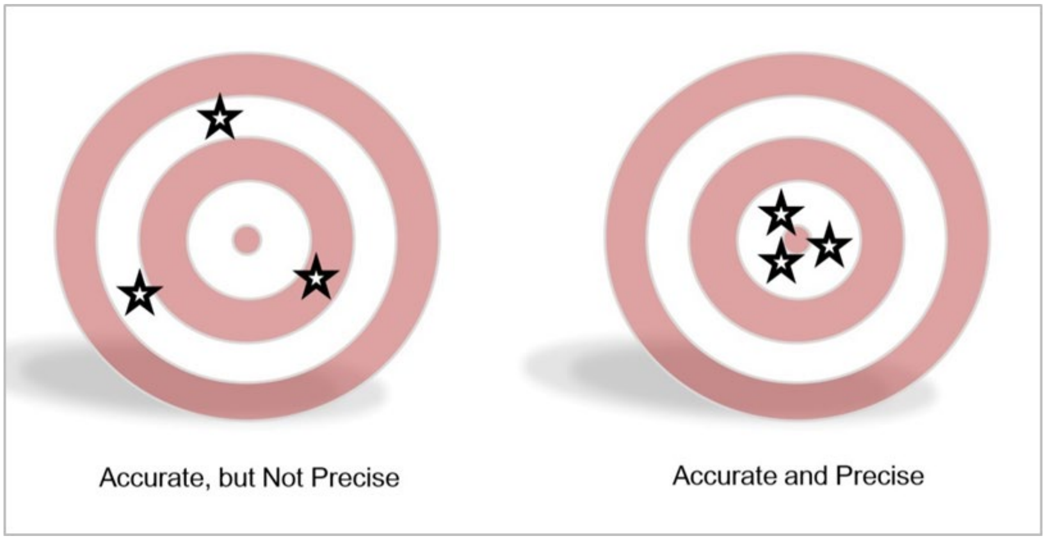


Figure 5.1. Accuracy vs. Precision

5.1.5. Electronic instruments consist of three main components: a sensor (or transducer), a data acquisition system, and a linkage between these two components. Data acquisition systems range from simple hand-carried read-out units to complex automatic systems that gather information from multiple instruments and transfer data directly to a database, which can then be transmitted or accessed offsite. Linkage is accomplished by signal cable or radio.

5.1.6. This manual focuses on selecting manual and electronic instruments. Thermal imaging, ground-based interferometric radar, and fiber-optic geotextiles have potential uses and benefits but are not discussed in this manual.

5.1.7. Geophysical Techniques.

a. Geophysical techniques such as microgravity (American Society for Testing and Materials [ASTM], 2005d; Rybakov et al., 2001); electrical resistivity (ASTM, 2005e, 2005f, 2005g); and ground-penetrating radar (Hoover, 2003) are considered investigation methods and not typically used for directly monitoring project performance. However, the data obtained can help define the location for instrumentation installation and inform the overall project performance evaluation.

b. Information on geophysical techniques is provided in EM 1110-1-1802: Geophysical Exploration for Engineering and Environmental Investigations (USACE, 1995); Advanced Non-Invasive Geophysical Monitoring Techniques (R. Snieder et al.,); and Applications of Geophysics in Geotechnical and Environmental Engineering (Environmental and Engineering Geophysical Society, 1998). Geophysics are not further discussed in this manual.

5.1.8. New types of devices should be tested extensively in the laboratory and field before being accepted for use at a project as the sole measurement device for a particular parameter. Instruments should be obtained from manufacturers that have obtained certification from a quality assurance organization such as the International Organization of Standardization (ISO).

5.2. Sensors. A variety of types of sensors (or transducers) are used to monitor various parameters, and some types have more than one application. For example, a vibrating-wire sensor can be used to measure piezometric level, strain, displacement, temperature, and soil total stress. The types of sensors frequently used in embankment measurement devices are described in the following paragraphs:

5.2.1. Pneumatic.

a. Pneumatic sensors are used in PZs, earth pressure cells, and liquid-level settlement gauges. Most modern devices are of the twin-tube type shown in Figure 5.2, which measures pressure at gas flow onset. Figure 5.2 (a) shows the PZ in the inactive state.

b. An increasing gas pressure is applied to the inlet tube, and while that gas pressure is less than the in situ pressure, gas merely accumulates in the inlet tube. However, if the gas pressure exceeds the in situ pressure, the diaphragm deflects as shown in Figure 5.2 (b), allowing gas to circulate behind the diaphragm into the outlet tube and be detected with a gas flow detector.

c. The gas flow is then stopped at the inlet valve and the diaphragm returns to its original position, as shown in Figure 5.2 (c). Accuracy can be increased by using a third tube to connect the gauge to the inside of the diaphragm instead of the gas supply.

d. The gas pressure at impending gas flow is read on a Bourdon tube or electrical pressure gauge. The Bourdon pressure gauge features a C-shaped, flattened, thin-walled, closed-end tube. The open end of the gauge tube is connected to the pneumatic tubing. As pressure is increased, the gauge tube straightens in proportion to the applied pressure. The closed end moves in an arc and transmits the movement through a mechanical linkage to a pointer shaft. Pressure is read on a circular scale behind the pointer.

e. Several issues need to be considered when selecting a pneumatic device, including:

(1) Sensitivity of the gauge to remote diaphragm displacement,

(2) The need to minimize gas flow,

(3) Sufficient time needed for a stabilized reading,

(4) Type of tubing,

(5) Tubing length and diameter,

(6) Tubing fittings,

(7) Gas type,

(8) Pressure gauge accuracy, and

(9) Operator skill.

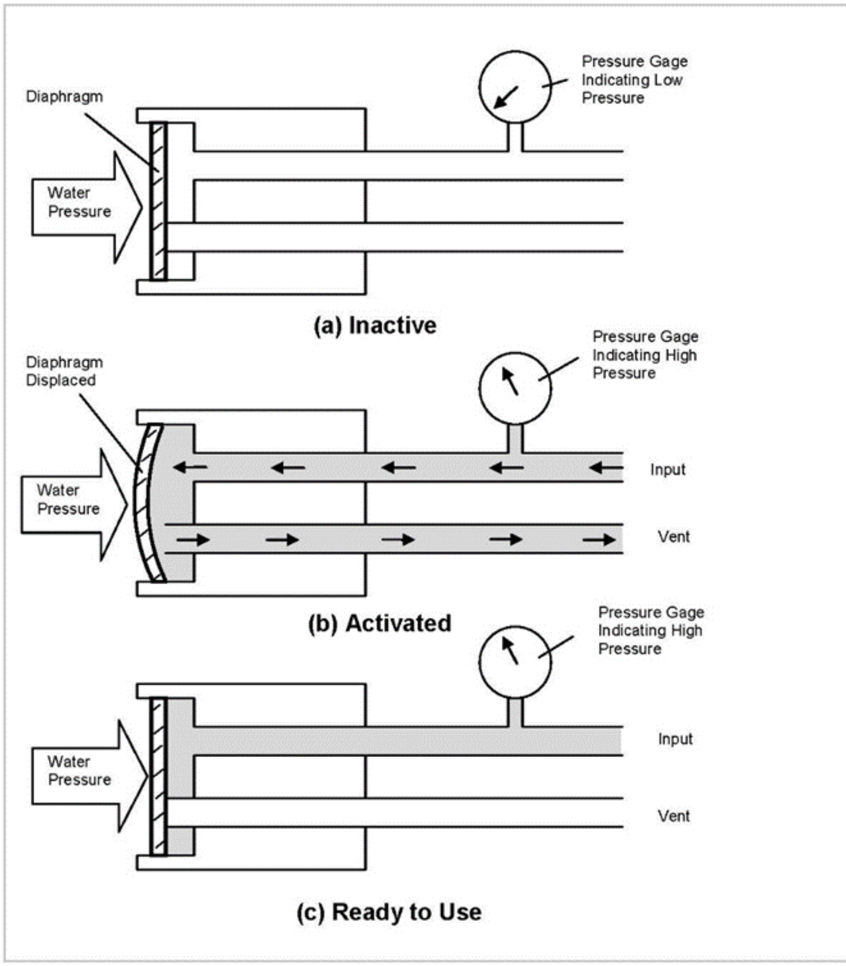


Figure 5.2. Pneumatic Device

5.2.2. Vibrating Wire.

a. Vibrating-wire devices are used as strain gauges and in pressure sensors for PZs, earth pressure cells, load cells, liquid-level settlement gauges, water-level monitoring in weirs and flumes, and in deformation gauges. A vibrating-wire device features a length of tensioned steel wire clamped at the ends and free to vibrate at its natural frequency. The wire is plucked magnetically near its midpoint by an electrical coil, and the frequency of vibration is measured by the same coil (or a second coil).

b. Like a guitar string, the frequency of the wire vibration varies with tension. The wire can be used as a pressure sensor, as shown in Figure 5.3, or can be adapted for measuring linear displacement by including a coil spring in series with the vibrating wire because the measured frequency of vibration is proportional to the strain in the wire.

c. With vibrating-wire transducers, the undesirable effects such as signal cable resistance, contact resistance, electrical signal seepage to ground, or length of signal cable associated with other electrical sensors are negligible.

d. Problems of "zero drift" have occurred with vibrating-wire instruments due to creep and slipping of the wire at the clamps, resulting in a reduction in frequency unrelated to diaphragm displacement. This problem has been associated with the manufacturing process and has been minimized by recent technological advances, such as aging the transducer before calibration. Nevertheless, monitoring is required to ensure that any zero drift is detected.

e. Vibrating-wire PZs subjected to moisture in the cable do not undergo a change in frequency, but may suffer a weakened signal. Electrical short circuits in the signal cable do change the reading frequency and should be avoided by minimizing sharp cable bends and the number of cable splices. A short circuit in a temperature sensor cable at a splice or where the cable is nicked frequently results in obviously inaccurate temperature readings.

f. Therefore, one of the first checks that should be performed on an anomalous vibrating-wire pressure reading where the transducer has a companion temperature sensor is to verify that the temperature readings are plausible. If cable splicing is required, splicing kits recommended by the manufacturer should be used.

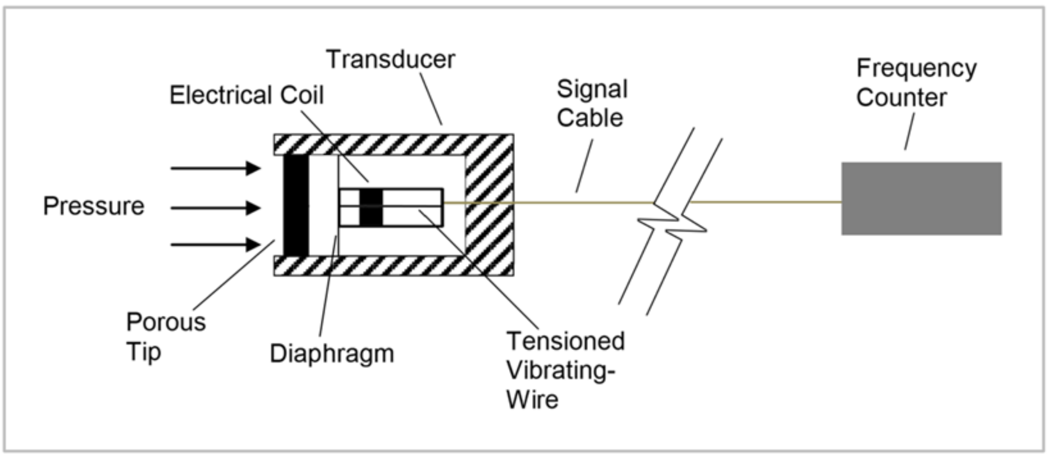


Figure 5.3. Vibrating-Wire Pressure Sensor

5.2.3. Electrical-Resistance Strain Gauge.

a. Electrical-resistance strain gauges have been used in extensometers to measure strain, in PZs to measure pressure, and as high accuracy water-depth indicators. A resistance strain gauge is a conductor for which electrical resistance changes in direct proportion to changes in length. The relationship between resistance change (ΔR) and length change (ΔL) is given by the gauge factor (GF), as shown in Equation 5.1:

$$(\Delta R/R) = (\Delta L/L)(GF) \qquad \text{(Equation 5.1)}$$

where:

R = resistance without strain
ΔR = change in resistance due to strain
L = length without strain

ΔL = change in length due to strain
GF = gauge factor

b. Output from a strain gauge is normally measured using a Wheatstone bridge circuit. Electrical-resistance strain gauges can be packaged as bonded wire, as shown in Figure 5.4, or as unbonded wire, bonded foil, or weldable gauges.

c. The measured resistance change can be strongly influenced by signal cable length, contact, moisture, temperature, and leakage to ground. A correction for these influences can be made by measuring the resistance of the various system components (e.g., cable, contact) and subtracting the resistance from the total resistance. Various companies manufacture low-current signal transducers (4 to 20 milliamp range) that minimize resistance problems.

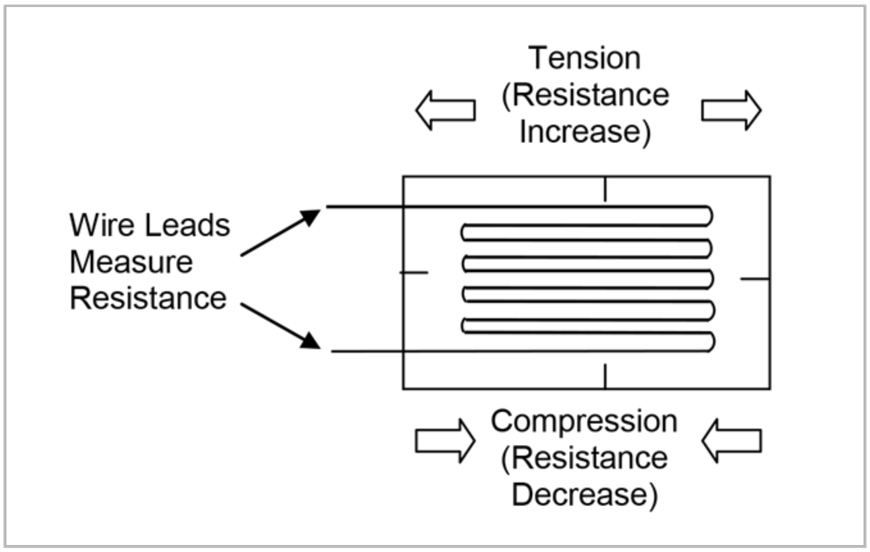


Figure 5.4. Bonded Wire Strain Gauge

5.2.4. Other Electrical Transducers.

a. Other electrical transducers are the:

(1) Linear variable differential transformer,

(2) Direct current differential transformer,

(3) Linear potentiometer,

(4) Force balance accelerometer,

(5) Micro electro-mechanical sensor,

(6) Magnet/reed switch,

(7) Induction coil transducer,

(8) Ultrasonic sensor, and

(9) Water-level indicator.

b. Linear Variable Differential Transformer (LVDT).

(1) A LVDT, shown in Figure 5.5, consists of a movable magnetic core passing through a magnetic shell containing one central primary and secondary coil in series on each end. Linear displacement of the magnetic core is proportional to voltage output. The assembly relies on magnetic coupling, has no internal friction, and provides high resolution in a dry transducer environment.

(2) An alternating current (AC) voltage is applied to the primary coil, inducing an AC voltage in each secondary coil. As the magnetic core moves from its neutral position in either direction, the AC voltage induced in the secondary coil is varied, with the magnitude of voltage depending on the position of the core. LVDT transducers are used for very fine resolution measurements but can be sensitive to moisture. Therefore, if the sensor environment is not dry, the readings may be faulty.

(3) A direct current (DC) differential transformer minimizes electrical noise from cables associated with LVDTs using DC voltage, which requires miniaturizing the electrical circuitry and placing components such as an oscillator in the transducer housing.

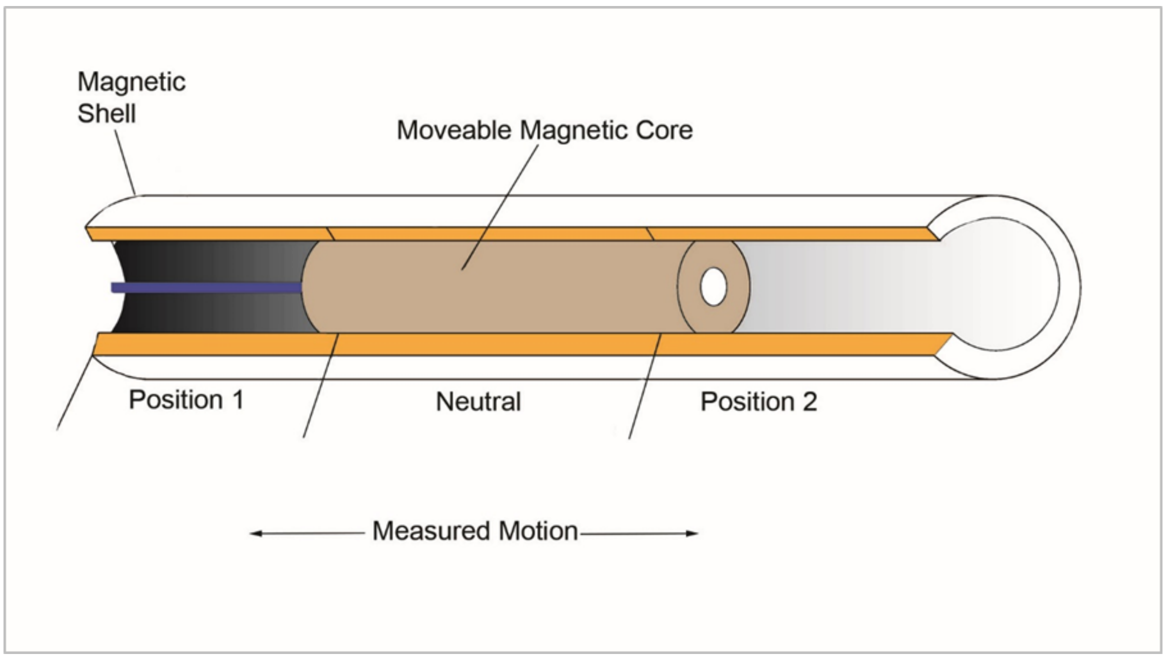


Figure 5.5. LVDT

c. Linear Potentiometer. A linear potentiometer is a device with a movable slider which makes electrical contact along a fixed resistance strip. A linear potentiometer is suitable for slow rates of displacement. As shown in Figure 5.6, if a regulated DC voltage is applied to the two ends of the resistance strip, the voltage between Points B and C is measured as the output signal. The voltage between B and C varies proportionally as the slider moves between Point A and Point B.

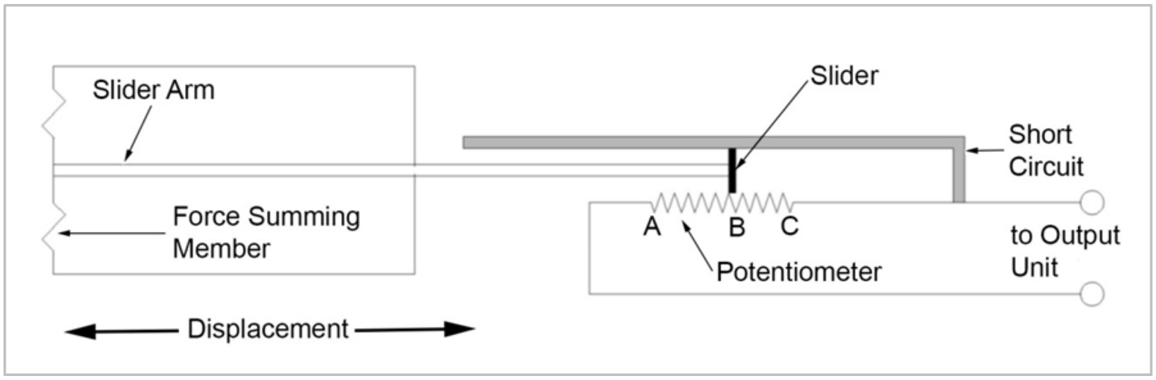


Figure 5.6. Linear Potentiometer

d. Force Balance Accelerometer.

(1) A force balance accelerometer is used as a tilt sensor in inclinometers and tiltmeters. The device consists of a mass suspended from a hinge in the magnetic field of a position detector, as shown in Figure 5.7. If the mass is subjected to a gravity force along the sensing axis, incremental movement occurs, and any motion induces a current change in the position detector.

(2) The current change is fed back through a servo-amplifier to a restoring coil, which imparts an electromagnetic force to the mass equal and opposite to the gravity force. The mass is thus held in balance and does not move. The current through the restoring coil is indicated by measuring the voltage across a precision resistor. Therefore, the voltage (shown as V) is directly proportional to the input force.

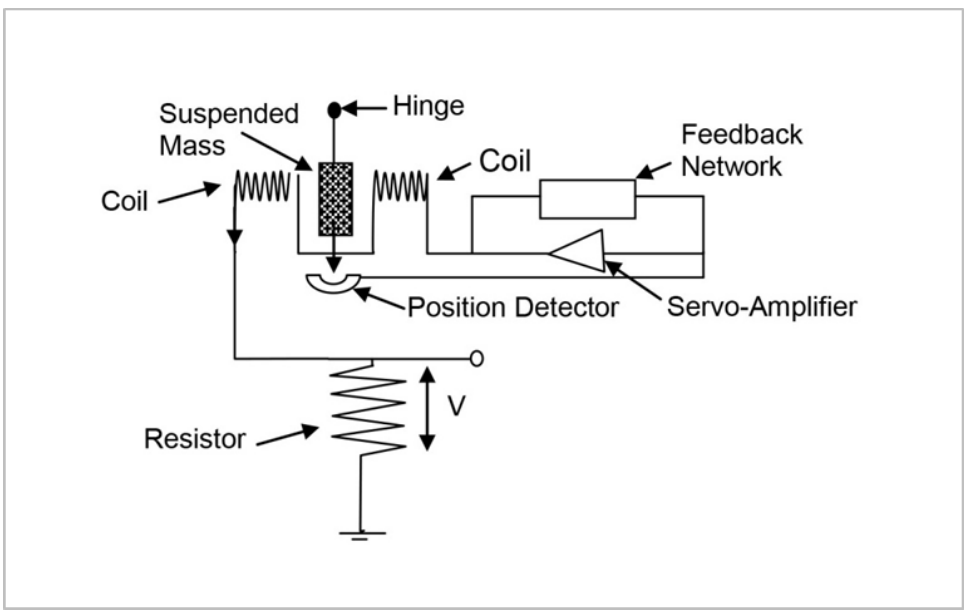


Figure 5.7. Force Balance Accelerometer

e. Micro Electro-Mechanical Sensor (MEMS).

(1) The MEMS (using nanotechnology) was introduced to the geotechnical field in 2005. The MEMS is a type of accelerometer typically used for horizontal and vertical monitoring applications in tilt sensors such as inclinometers, including shape accelerometer arrays. In such instruments, the gravitational force causes flexure of movable plates within the accelerometer if the instrument is at an angle to the vertical, changing the capacitance and output voltage with the output voltage proportional to the angle.

(2) Inclinometers equipped with MEMS consume little power and are relatively durable. Smaller sensors allow the use of smaller probes than are possible with a force servo-accelerometer. A MEMS can be designed to transmit data wirelessly.

(3) A MEMS can be installed at almost any angle to the vertical and has a low sensitivity to temperature. Data reduction and processing is typically performed with integrated electronics that can compensate for temperature. The MEMS and the simple electrolytic-level sensor have largely replaced force balance accelerometers for measuring tilt in embankments.

f. Magnet/Reed Switch.

(1) The magnet/reed switch is used in hollow extensometers read with portable probes. The switch is an on/off position detector arranged to indicate if the reed switch is in a certain position with respect to a ring magnet embedded in the embankment on telescoping or otherwise post-installation compressible/extensible casing. The switch contacts are normally open, and one of the reeds must be magnetically susceptible.

(2) If the switch enters a sufficiently strong magnetic field indicating ring proximity, the reed contacts snap closed and remain closed while remaining in the magnetic field of the ring. The closed contacts actuate a buzzer or indicator light in a portable read-out unit. Some magnetic settlement systems are compatible with inclinometer systems.

g. Induction Coil Transducer.

(1) An induction coil transducer, shown in Figure 5.8, is used in a probe extensometer. In the transducer, an electrical coil is energized to create a magnetic field around the coil. As the coil is placed inside a steel wire ring with no external electrical connection, a voltage is induced in the ring, which in turn alters the current in the coil because the inductance of the coil changes.

(2) The current in the coil is at a maximum if the coil is centered inside the ring. Therefore, by measuring the current in the coil, the transducer can be used as a proximity sensor. Although seldom used in embankments, the induction coil transducer is used where high accuracy is needed in rock slopes or tunnels.

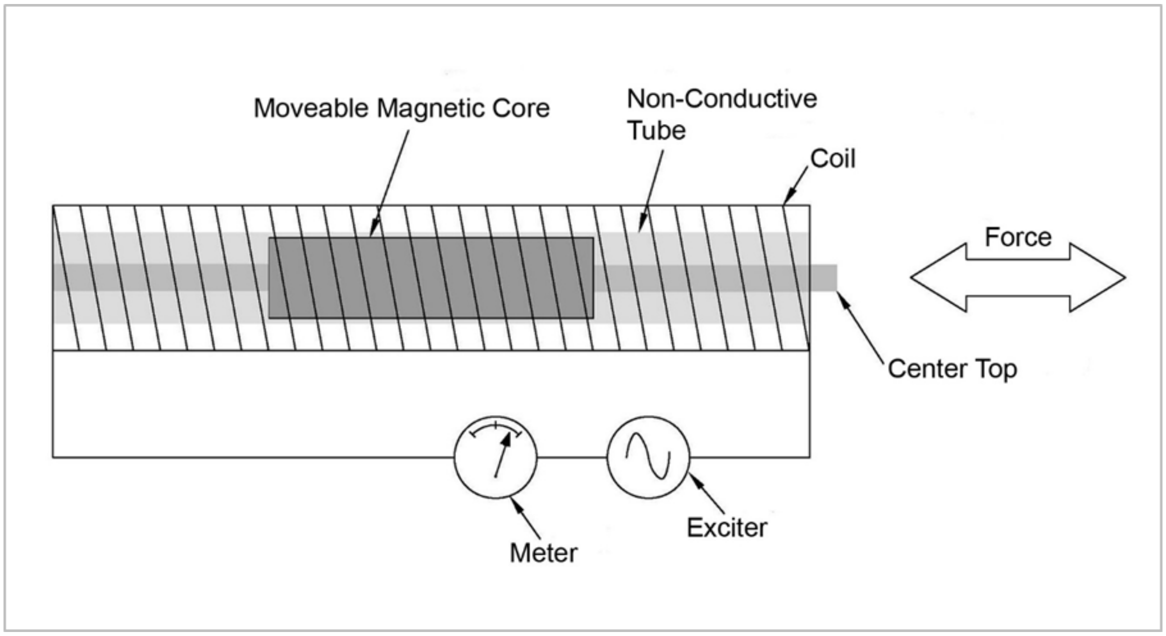


Figure 5.8. Induction Coil Transducer

h. Ultrasonic Sensor.

(1) An ultrasonic sensor can be used to monitor water level in an open-standpipe PZ and a weir stilling basin without direct water contact. In such a sensor, a transceiver is mounted above the water surface. Sound pulses travel to the water surface and are reflected to the transceiver. Distance to the water surface is determined from the measured pulse time and the known velocity of sound waves corrected for error induced by temperature change.

(2) These sensors may be appropriate to measure distance to a water surface with no floating debris or foam. The beam spread and range may be restrictive with regard to depth and width. A schematic of an ultrasonic transceiver is shown in Figure 5.9.

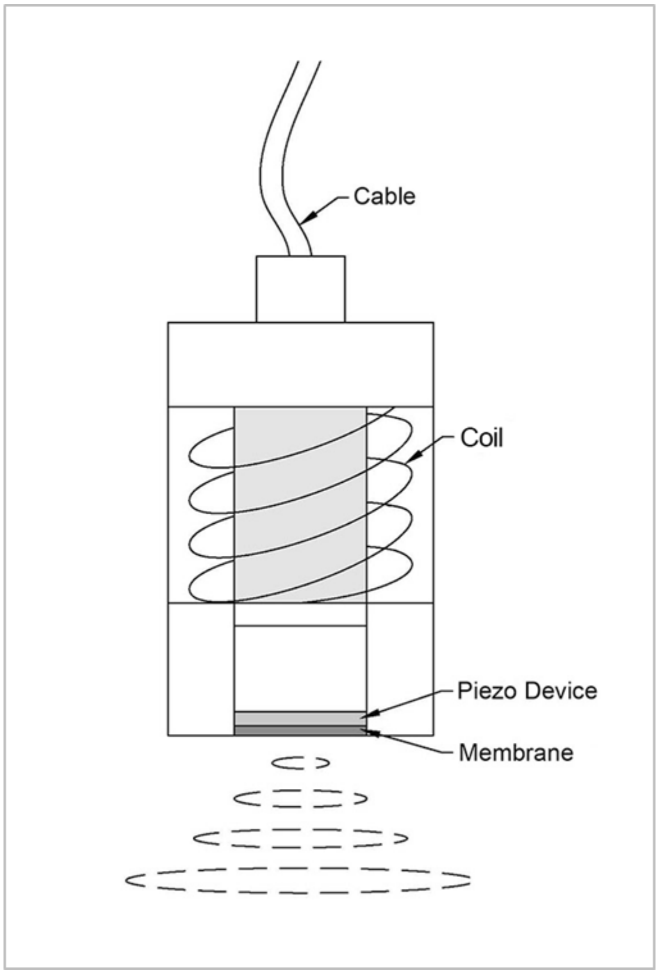


Figure 5.9. Ultrasonic Transceiver

i. Water-Level Indicator.

(1) A water-level indicator is a manual instrument used for measuring the depth to water in open standpipe PZs and observation wells. As shown in Figure 5.10, a typical meter consists of a submersible probe, graduated tape, and a hand-held reel with crank, buzzer, and indicator light. Most meters are powered by either a standard 9-volt or AA battery. To obtain readings, the portable probe is lowered into an observation well or open-standpipe PZ until contact is made with the water surface.

(2) The probe features an insulating gap between electrodes. If contact is made with the water, the circuit is completed, and the buzzer and indicator light are activated. The distance to the water is determined by reading the graduated tape at the top of the standpipe or well casing. Water-level meters are available in both U.S. customary and SI units. Various tape lengths, configurations, and probe diameters are available.

(3) Water-level indicators are susceptible to erroneous readings. For example, on the one hand, small diameter cables may stick to wet standpipe walls, or particularly with standpipes 0.5

inches (1.27 cm) in diameter or smaller, a probe tip may be too sensitive and react to moisture adhering to the wall of the pipe above the true water level. On the other hand, an electrical probe may not be sufficiently sensitive to indicate the water surface after crossing the air/water interface.

(4) A sensitivity control is typically mounted on the reel, which allows adjustments to the sensor to avoid false readings. Tubes or standpipes may be clogged or bent such that the tip of the probe cannot reach the water surface. In some cases, standpipes still transmit water levels past obstructions so that the device continues to function.

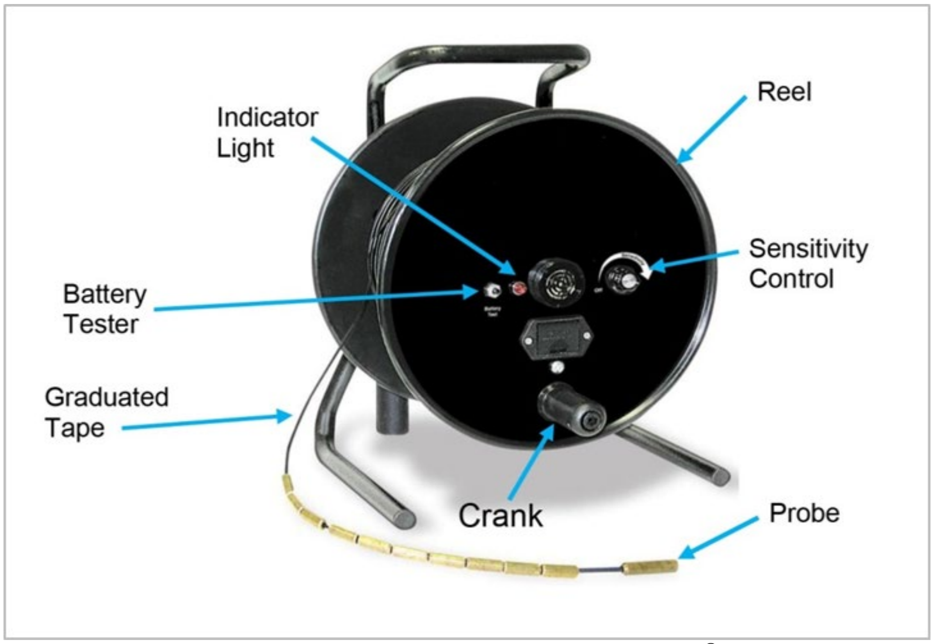


Figure 5.10. Water Level Indicator (Solinst,® 2013)

5.3. Measurement of Piezometric Level.

5.3.1. Definitions of groundwater level, porewater pressure, pore-gas pressure, piezometric level, and other terms associated with measurement of hydraulic pressure are presented in Chapter 2. The device for measuring these parameters in and near embankments is the PZ. Typical locations and purposes for measuring piezometric levels for embankments include:

a. Measuring dissipation of construction-induced excess pore pressure in foundation soils or in cohesive fills placed at wetter than optimum moisture content;

b. Checking for change in flows in or near internal drains;

c. Checking for leakage along conduits or interfaces with concrete structures;

d. Determining and monitoring saturation levels in the abutments;

e. Verifying downstream shells remain isolated from the pool by means of upstream drains and filters;

f. Foundation pressure downstream/landside of cutoff features;

g. Uplift pressure at the downstream/landside toe;

h. Checking conditions near suspected geologic anomalies, flaws, or deficiencies; and

i. Monitoring along potential seepage pathways.

5.3.2. Applications for PZs fall into two general categories—to monitor the pattern of water flow and to provide an index of soil strength.

a. Examples in the first category include determining groundwater conditions before construction, monitoring seepage pressure distribution, and determining the effectiveness of drains, relief wells, and seepage barriers.

b. In the second category, measuring porewater pressure permits an estimation of effective stress and soil strength. Examples include monitoring the dissipation of porewater pressure during consolidation of a foundation and embankment fill or following the rapid drawdown of a reservoir pool.

c. Table 5.3 summarizes the advantages and disadvantages of PZ types.

5.3.3. <u>PZ Type</u>. PZ types and associated measurement tools are described, along with recommendations for use.

a. Twin-Tube Hydraulic PZ.

(1) The twin-tube hydraulic PZ is often simply referred to as a hydraulic PZ and is a standpipe PZ having flexible tubing that does not rise above the piezometric elevation. A transducer, or more typically a Bourdon gauge, is attached to the observation end of the tubing, as shown in Figure 5.11, to determine piezometric elevation by summing the pressure gauge reading in units of head and the pressure gauge elevation.

(2) Two tubes are used to allow tubing circuit maintenance by flushing. Typically, each tube is connected to a gauge and the pressure at the PZ tip is considered the mean of the two gauge readings. Averaging is only performed if the gauges appear to be reacting to the same pressure with the readings differing within the specifications for the gauges. With appropriate installation and maintenance and provided the tubing is non-diffusing, hydraulic PZs have been operated successfully.

(3) If both plastic tubes are completely filled with liquid, identical pressure readings should be obtained. However, if a large amount of gas has entered the system through the filter, tubing, or fittings, an inaccurate pressure reading on one or both gauges results from the surface tension at the gas-liquid interface. Regular flushing and calibration checks are required to prevent error caused by gas in the tubes.

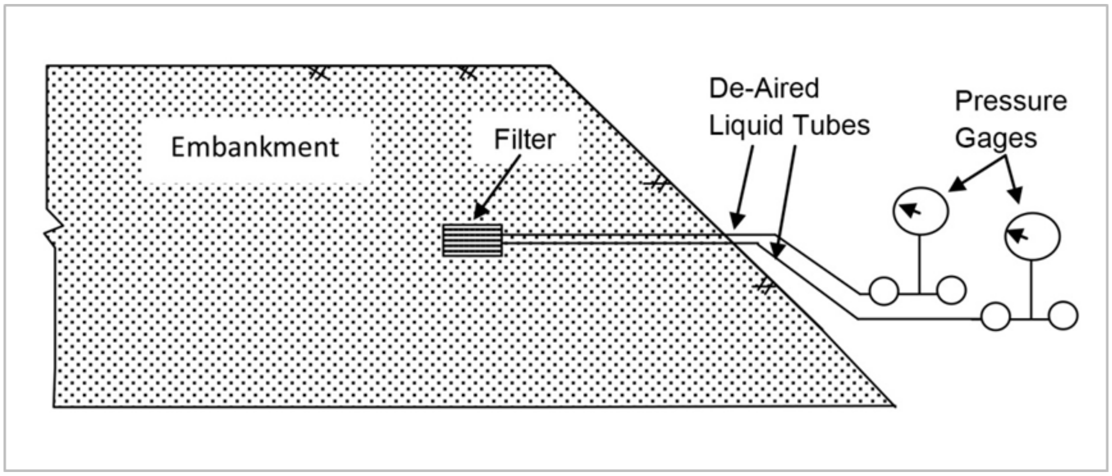


Figure 5.11. Twin-Tube Hydraulic PZ

(4) To reduce the entry of air into the PZ tubing, it is necessary to use high air entry tips which require special care to maintain saturation during installation. If the tips are not installed within saturated material or within material that becomes unsaturated at times, it may be difficult to obtain accurate measurements because the tips become unsaturated and allow air into the tubing.

(5) Unlike other closed PZs, hydraulic PZs generally do not need to be corrected for tip settlement. The gauge is located outside of the embankment and reads the pressure difference between the gauge and the upper end of the equipotential line intercepted by the PZ tip. The upper end of that equipotential line is the phreatic surface.

(6) If the equipotential line is approximately vertical, the head at the PZ tip is approximately constant even if the tip settles moderately. However, for other types of closed PZs, settlement results in a greater distance between the gauge and the phreatic or piezometric surface and may require adjustments to data reduction constants related to the installed tip elevation.

(7) Complex arrays of hydraulic PZ systems using flexible tubing are maintenance-intensive. The interpretation of readings of complex arrays of hydraulic PZs must take into account any gauges that have been replaced, hydraulic lines that have been regularly flushed and de-aired, and leaking hydraulic connections or tubes.

(8) Twin-tube hydraulic PZs are used primarily during the construction phase of a dam or levee and are still in use on some older USACE project sites. Twin-tube hydraulic PZs are no longer commonly installed at new projects.

b. Pneumatic PZ.

(1) A pneumatic PZ operates as described in Figure 5.2. A pneumatic PZ features a filter to separate the flexible diaphragm from the material in which the PZ is installed. Reading devices are available with a self-contained gas supply, pressure regulators, and gauges.

(2) A special type of pneumatic PZ, shown in Figure 5.12, is installed by pushing into

foundation material rather than by sealing in a borehole. The push-in instrument is applicable for monitoring consolidation porewater pressures in fine-grained levee foundations. For shallow installations, typically in soft clay, the PZ is pushed from the ground surface to the desired depth. For deep installation, a borehole will need to be drilled first, and then the PZ pushed into position. Care is required during installation to avoid damaging the sensor or cable connection.

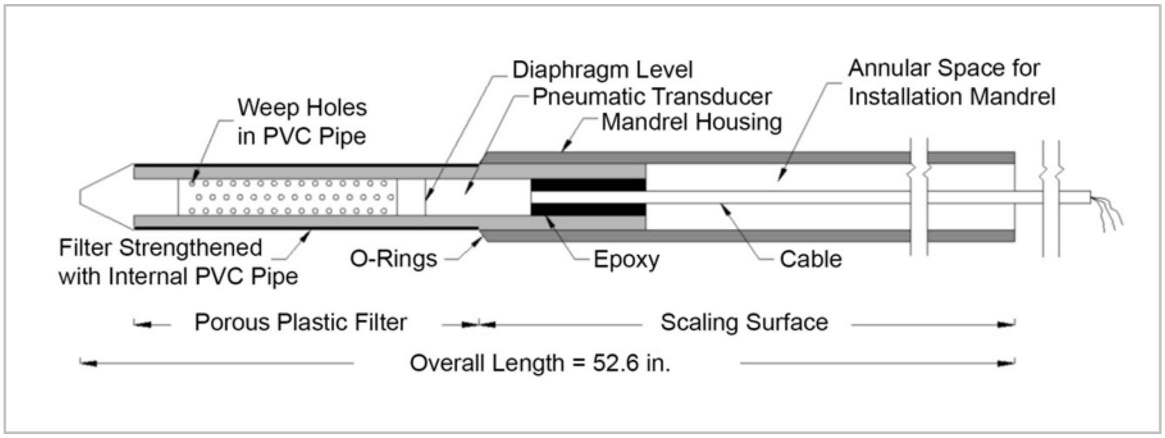


Figure 5.12. Push-in Pneumatic PZ
(Modified from Dunnicliff, 1993, Figure 9-31, p. 135)

(3) The pneumatic PZ is not used as commonly as the vibrating-wire PZ because the pneumatic PZ:

(a) Does not generate an electric signal amenable to automation;

(b) Is more tedious to read than electrical transducers;

(c) Can be difficult to diagnose if inoperable;

(d) Requires a compressed gas supply;

(e) Requires a consistent air flow to obtain consistent results;

(f) Forms a pressure gradient in the pneumatic line, possibly limiting the length of pneumatic line suitable for accurate measurements; and

(g) Is susceptible to over-pressurization by the operator.

(4) Interpretation of readings should also consider whether the gas pressure is measured at the PZ tip or at the pneumatic pressure source.

c. Observation Well.

(1) Observation well applications for geotechnical investigations are limited because the well does not isolate stratigraphy and are generally associated with unconfined aquifers or shallow interbedded stratigraphy where the pervious zones are assumed to represent the same groundwater level.

(2) The water level in the riser pipe can be measured by sounding with a manual water

level indicator or equipping the observation well with electrical or pneumatic transducers. Figure 5.13 shows an observation well. In the figure, the water level in the well equals that of the water table, and the well has a vented cap.

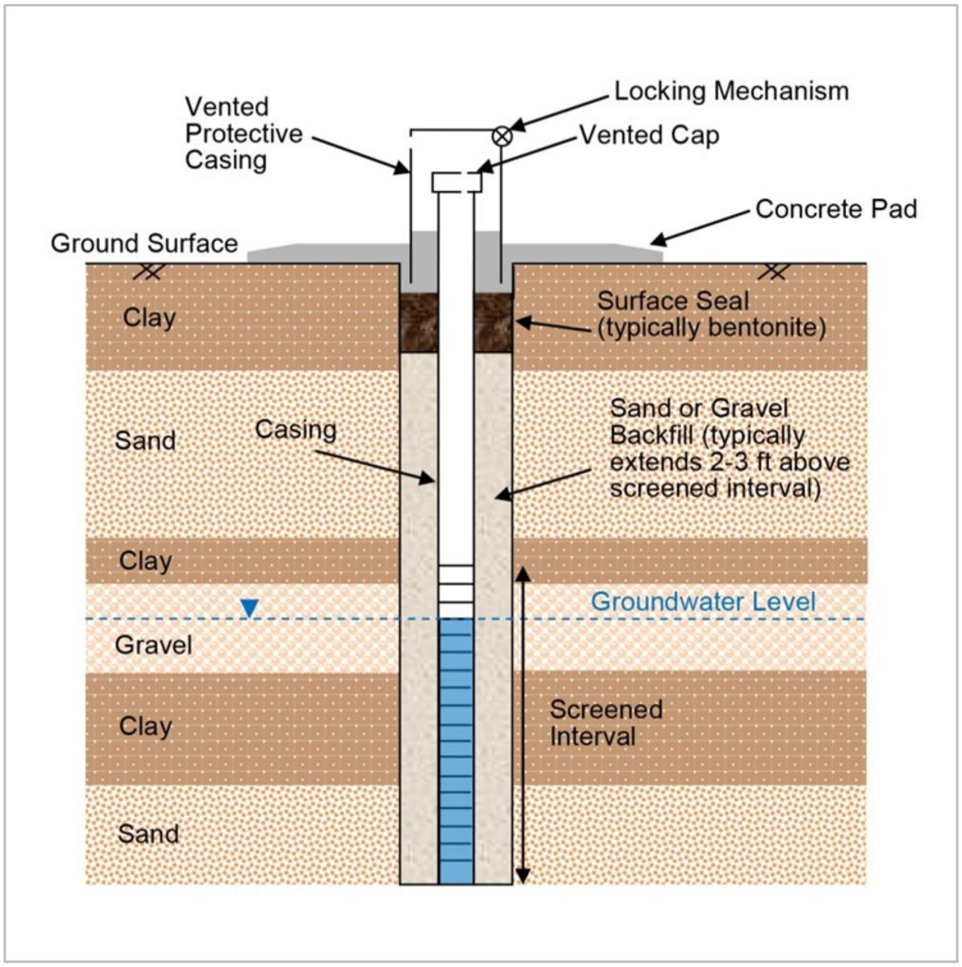


Figure 5.13. Observation Well

d. Open-Standpipe PZ.

(1) Figure 5.13 shows an open-standpipe PZ installed in a borehole. The components are identical to components in an observation well except subsurface seals isolate a particular stratum of interest. Screened intervals for PZs are typically much shorter than screened intervals for an observation well. As porewater pressure increases or decreases, the water level inside the standpipe rises or falls, respectively.

(2) Readings can be made by sounding with a water-level indicator, with a pressure transducer placed in the standpipe below the lowest piezometric level (Figure 5.13), or with an ultrasonic sensor. As for an observation well, water-level change depends on the flow of water into or out of the PZ. Thus, the time for a full response to a pressure change in a saturated, confined, low hydraulic conductivity stratum is considerable. The response for low hydraulic conductivity soils can be improved by using a transducer below a packer in the standpipe.

(3) If piezometric level is sufficient to cause water to overflow the top of the standpipe, the pipe may be sealed and pressure can be measured with a sealed-in transducer or using a pressure gauge mounted on the top of the standpipe.

(4) Single boreholes can be equipped to monitor the porewater pressure of multiple zones. Monitored intervals are backfilled with permeable filter media, and the intervals between monitored zones are sealed. Monitored zones must be sealed to prevent a connection to other aquifers. Potential seal leakage from improper installation and/or settlement makes one PZ per borehole more reliable. Pressure transducers can be installed at the monitored intervals to measure the water pressure of those zones.

(5) If a standpipe PZ does not appear to be functioning properly, a rising or falling head test may restore functionality and/or provide additional information to assess the instrument functionality. The response time depends on standpipe size and screened interval length and is highly dependent on the surrounding soil hydraulic conductivity.

(6) For pervious soils, a test can be performed by bailing or adding water and verifying that recovery occurs within a reasonable time. However, for low hydraulic conductivity soils, the response time to substantially recover initial piezometric level may be days or weeks. In such a case, measuring the water level, plotting the response versus time, and correlating the response with the theoretical response provides early indications of performance.

(7) Some PZs with an extended lag time can be incorrectly diagnosed as unresponsive. Exercise caution when introducing elevated hydraulic head to embankment materials to minimize the potential for hydraulic fracturing. Procedures for falling head tests are described in Appendix C. A rising head test is the opposite of a falling head test; in a rising head test, water is removed from the PZ and the time required for the water level to rise to the initial level is monitored.

(8) Cleaning and Rejuvenating.

(a) If an instrument is unresponsive to a rising or falling head test or exhibits increasing lag time as compared to prior tests, cleaning out accumulated sediment or rejuvenating connectivity with the soil may restore the PZ function. Clogging of PZs (especially the screens or filters near the tips) may be due to:

- Sediment accumulation in the standpipe;
- Insects or debris falling in the standpipes;
- A broken or leaking standpipe;
- Inundation of standpipe;
- Flooding of recessed standpipe;
- Mineralization;
- Bio-fouling; or
- Deterioration of the pipe or screen material such as excessive rusting, rotting wood

stave screens, or older plastics.

(b) Procedures for cleanout and rejuvenation are described in Appendix C.

(9) Rising or falling head tests may be performed regularly if experience shows that a PZ is prone to clogging and loss of responsiveness. Testing procedures should be carefully selected in consideration of the type of PZ tip, filter and in situ materials around the PZ tip, and pressures induced during testing.

(10) Clogging can often be recognized in time history plots as a smoothed or damped response. A schedule of maintenance should be part of the instrumentation and monitoring plan, and results should be included in data management records.

(11) Rising head tests may also be used to evaluate data. Measured values that plot in a pattern similar to the theoretical water-well equation drawdown lines provide evidence that the stratigraphy of the soil layers is understood. Test results with non-conforming response patterns may reflect sand seams, pervious veins or dikes, jointed rock, unhealed cracks (such as from tension stress or hydrofracturing), or leaking standpipes.

(12) Embankment settlement can crush Poly Vinyl Chloride (PVC) tubing and split the standpipe, allowing pressure to affect strata above the PZ tip.

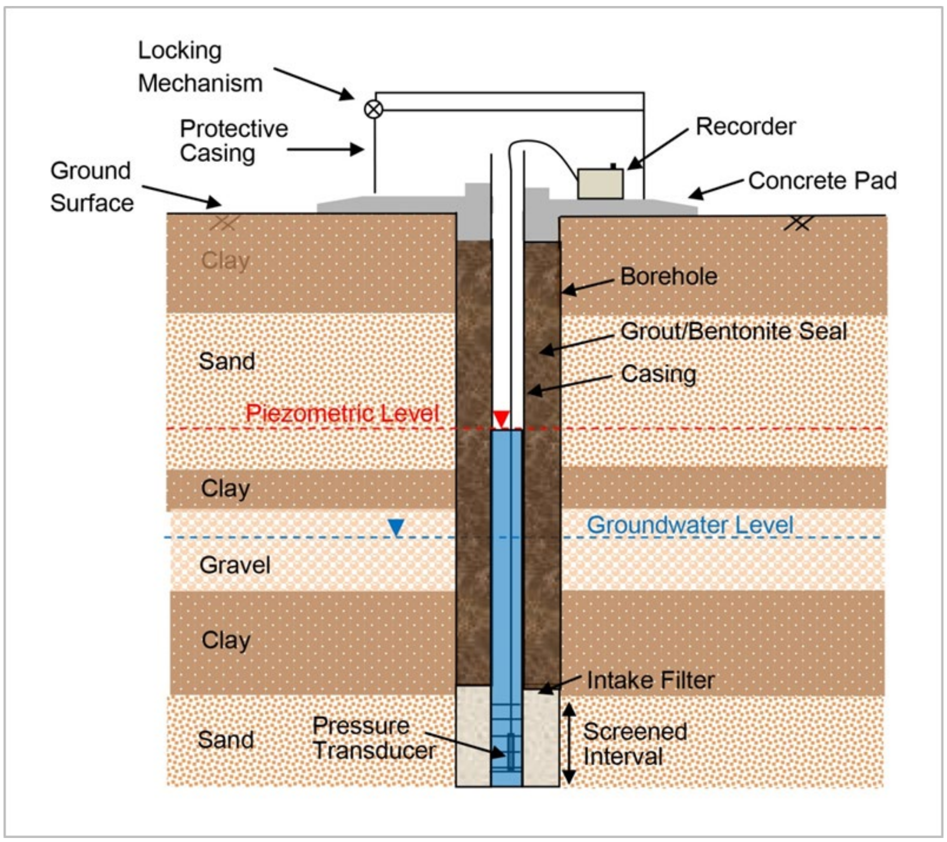


Figure 5.14. Open-Standpipe PZ with Pressure Transducer

e. Vibrating-Wire PZ.

(1) A vibrating-wire PZ sensor can be placed in an open-standpipe PZ riser or observation well for either short-term or long-term data collection. For installations embedded in compacted fill, heavy-walled instruments are available to ensure an instrument responds only to change in porewater pressure and not to total stress acting on the housing.

(2) The vibrating-wire PZ, shown in Figures 5.3 and 5.15, can feature a built-in thermistor to measure temperature for calibration and correction. Temperature correction factors are required for high accuracy if temperature fluctuation is significant. Temperature measurement can also be used to estimate how long the seepage has been in the ground. Push-in instruments are also available, similar to the push-in pneumatic PZ.

Figure 5.15. Vibrating-Wire PZs

(3) Vibrating-wire PZ sensors can also be grouted in place. Installation of fully grouted vibrating-wire PZs is faster and simpler than conventional installations because the borehole annulus is completely filled with non-shrinking, low hydraulic conductivity cement-bentonite grout.

(4) Low-flow, closed PZ transducers such as the vibrating-wire type work in fully grouted applications because pressure is effectively transmitted through saturated grout across the short radius of the borehole. Transmission along the much-longer grouted borehole length is usually negligible, although for this reason it is advisable to vertically separate fully grouted PZs at least a few borehole diameters distance from each other and from distinct strata.

(5) In fully grouted applications, multiple transducers can be installed along the length of the borehole and the borehole completely backfilled with grout. Figure 5.16 shows a multi-level fully grouted vibrating-wire PZ installation.

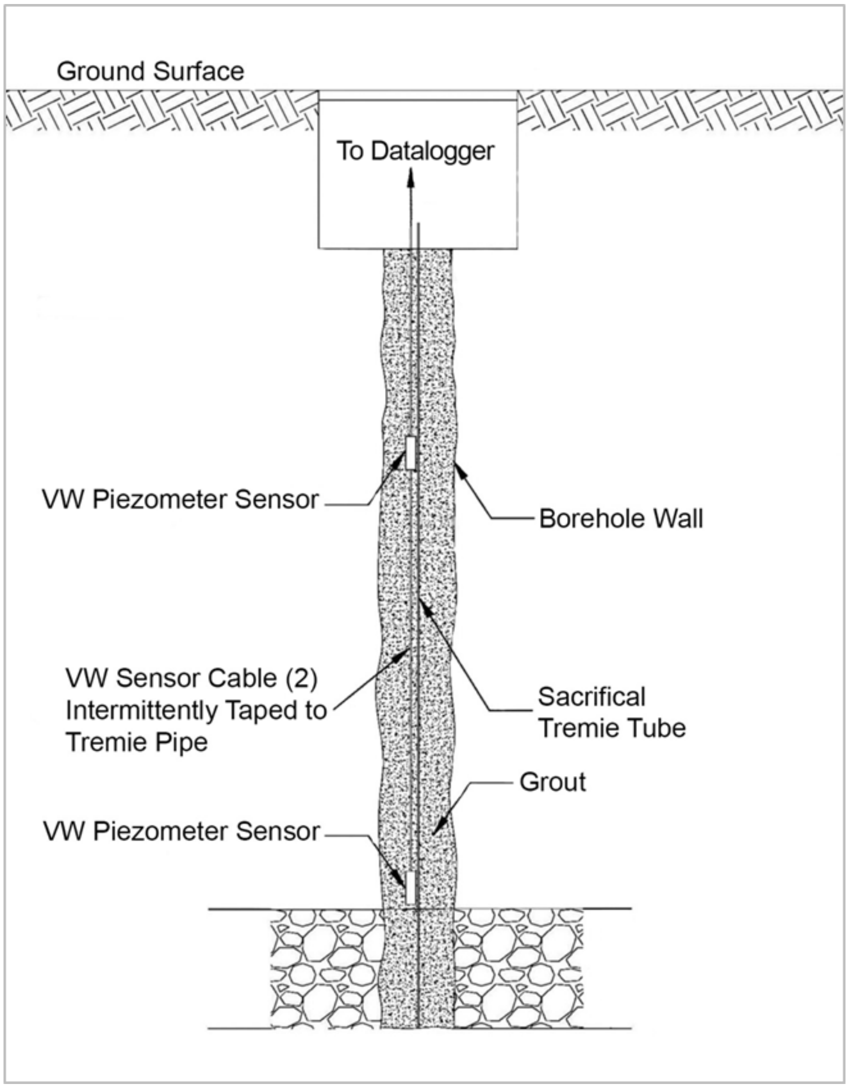


Figure 5.16. Multi-Level Fully Grouted Vibrating-Wire PZ Installation

(6) Vibrating-wire pressure transducers are either vented or unvented. Changes in barometric pressure can alter the indicated water level. If the sensor does not include vent tubes, then the need for barometric correction should be evaluated. Unvented transducers are hermetically sealed, and the zero-pressure reading at atmospheric pressure is taken at the time of installation before the PZ is subject to water pressure.

(7) Barometric effects on a sealed well may be minimal or attenuated with respect to surface conditions. In such a case, applying a barometric correction factor may introduce errors. Barometric pressure changes should be correlated with observed pressure changes to determine any needed correction factor.

(8) Vented transducers are equipped with a tube that conveys atmospheric pressure to the inside of the transducer, thus balancing the component of external pressure which is atmospheric

and eliminating the need to apply barometric pressure corrections. However, vented installations require a desiccant chamber to collect moisture, preventing moisture accumulation in the transducer.

(9) The desiccant typically changes color as moisture is collected, indicating the need for replacement. Desiccant chambers that are not maintained can lead to false readings or damage the PZ.

(10) Vibrating-wire PZs must have the zero-offset recorded correctly. Zero offset is the frequency reading at zero pressure used in the reduction equation. If the zero offset is incorrect, so are the reduced readings. Although not particularly sensitive to the surrounding temperature, vibrating-wire PZs are sensitive to differential temperatures in the body of the transducer.

(11) If the zero-reading recorded at the time of installation is taken while the transducer is briefly in contact with a heat source or sink, the zero reading may be significantly affected. The zero reading should not be recorded until the PZ reaches temperature equilibrium. For grouted applications, a zero reading and temperature are recorded with the sensor oriented in the same direction as for the subsequent installation (typically with the sensor in the upright position).

(12) For a standpipe installation, a read-out is connected to the PZ, and the PZ is allowed to demonstrate reading stability, after which zero reading and temperature are recorded.

(13) In situ zero readings should be obtained if possible. However, using the factory zero to calculate a site zero reading corrected for site barometric pressure and in situ temperature is appropriate at times. If readings are inconsistent with the site conditions, an incorrect zero reading determined in the field during installation may be the cause.

(14) Some early generation vibrating-wire PZs have been affected by zero drift caused by the wire element slipping in the housing attachment and losing tension. The unintended zero pressure offset change results in an apparent pressure increase. Vibrating-wire PZs using current technology are much less sensitive to this effect. Also, the proximity to power cables of sufficient voltage can influence the vibration signal and cause signal instability.

(15) If unusual pressure fluctuations are measured, a check for possible voltage sources, such as cables laid on the ground to provide temporary power, should be made along the entire length of signal cable.

(16) Temperature variations can also indicate a potential electrical fault in a vibrating-wire PZ equipped with a temperature sensor. If an anomalous reading of a vibrating-wire PZ is noted, one of the first checks should be to verify the temperature readings of the instrument are within a normal range. If temperature readings are well above or below expected levels, a short circuit in the signal cable is likely. Poor splices and water infiltration into in-line surge protectors and data gathering components can all produce similar errors.

f. Electrical-Resistance PZ. The electrical-resistance PZ is based on the electrical-resistance strain gauge and has been used at embankment dams and levees for fast response, accuracy, and high resolution, primarily for measuring water level within seepage flow devices.

However, the electrical-resistance PZ is not recommended for long-term monitoring or for installation in an inaccessible location because of susceptibility to errors from moisture, corrosion, and poor electrical connections.

g. Fiber-Optic PZ (FOP).

(1) A FOP responds to pressure by means of a deflecting diaphragm just as resistance and vibrating-wire PZs do. However, the FOP has a cavity between the diaphragm and a fiber-optic cable. The cavity is monitored by an optical signal. As the diaphragm position changes, the length of the cavity between the fiber-optic cable and the diaphragm is measured using interferometry.

(2) The read-out device also differs from that of a vibrating-wire or strain-gauge PZ in that the signal is a white light polarization interferogram rather than an electrical signal.

(3) FOPs offer advantages over other types of PZ. The accuracy, resolution, and size of FOPs are similar to those for vibrating-wire and strain-gauge PZs. FOPs are immune to electromagnetic interference and lightning. The longevity of FOPs is unproven, but a potential for long life appears reasonable due to the low number of moving components.

(4) FOPs have some disadvantages compared to other types of PZ. Portable read-out devices for FOPs are costly and may require weather-proof enclosures. Cable repair costs for FOPs are also typically greater than for vibrating-wire and strain-gauge transducers due to the special equipment and skill required. Most ADAS systems do not accommodate fiber optics without a special interface.

(5) Some fiber-optic PZs are about an order of magnitude more sensitive to temperature than vibrating-wire PZs. As with vibrating-wire PZs, the site zero pressure reading is a key constant for the reduction of readings. The fiber-optic PZ temperature sensitivity requires that the zero reading be determined for the expected in situ temperature.

(6) Various types of PZs provide in situ temperatures in drill holes. However, if these PZs are placed in or beneath fill, the in situ temperature must be estimated rather than measured.

(7) A year or more may elapse before the temperature reaches equilibrium at a PZ location beneath a significant depth of newly placed overburden. Therefore, the effect of changing temperature should be considered in the data interpretation until the temperature reaches a steady state, as indicated by temperature sensors in or near the fiber-optic PZ.

h. Self-Contained Water-Level Data Logger.

(1) A self-contained water-level data logger is a battery-powered water-level logger designed to record water depth and temperature over long periods. The sensor, data logger, and battery are all housed in one rugged, metal cylinder suspended in wells or PZs.

(2) During planning, the well or PZ riser diameter should be checked to determine whether these instrument types are viable. The sensors are typically piezo-resistive silicon units that measure deformation of a diaphragm, causing a change in resistance in a metal conductor surrounding the diaphragm. The change in voltage in the conductor is directly proportional to the pressure change.

(3) Changes in barometric pressure can alter the indicated water level. If the sensor does not include vent tubes, then barometric correction should be applied. As shown in Figure 5.17, barometer units are manufactured that can be installed above the water-level sensor in the riser pipe.

(4) Most manufacturers provide units with long battery life, such as 10 years dependent on reading frequency. The data is typically downloaded manually on site, but some manufacturers offer telemetry systems to support automation. Manually read units can be retrieved from the riser and connected to a PC for data collection. Some manufacturers offer direct-read cables that allow data collection without retrieving the unit from the riser pipe.

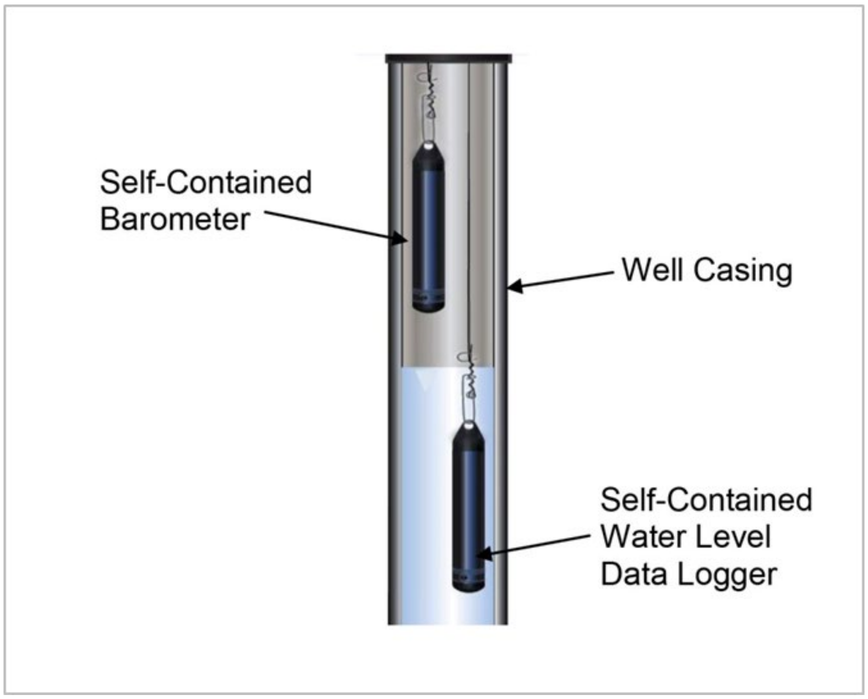


Figure 5.17. Self-Contained Water-Level Data Logger and Barometer Installation (Solinst,® 2013)

i. Portable-Probe PZ.

(1) Monitoring with a portable probe not only allows measuring piezometric level but also hydraulic conductivity and the sampling of selected layers for other parameters, such as chemical characteristics. Systems designed for long-term monitoring are available. Benefits of a portable-probe PZ are as follows:

(a) Borehole purging is not required before chemical analysis sampling.

(b) A single probe can be used at multiple boreholes.

(c) Portability of the probe facilitates maintenance and calibration.

(d) Operability at great depths.

(e) Avoids the need for multiple boreholes.

(2) Casing systems can be configured for different situations. Casing material may be plastic or stainless-steel. A seal placed in the borehole annulus separating the different piezometric realms may consist of bentonite or grout, as used in conventional backfilled boreholes or hydraulically inflated packers. Packers or backfill seals can also be placed in open boreholes. Packers can also be placed through temporary casing or in permanent casing. Casing ports at selected elevations allow piezometric level measurement and water sampling.

(3) Portable wireline-operated probes, lowered into the casing on a cable, dock with the ports. Various categories of probe can be used to collect in situ samples and in situ pressure and measure hydraulic parameters. Samples are retrieved for laboratory analysis. The probe can be used to index the relative hydraulic conductivity difference between ports and to introduce pressure so that interconnectivity with other monitored locations may be assessed. Tracer tests may also be conducted by injecting a substance into the monitored stratum.

(4) For automated multi-level pressure measurements, multiple probes can be distributed within a borehole with each probe located at a different port.

5.3.4. PZ Considerations. Considerations for designing a PZ installation are discussed in the following paragraphs.

a. Filter Types.

(1) Filters to prevent migration and subsequent clogging of diaphragms are recommended. These may be in the form of a sand pack surrounding the sensor, a built-in filter tip, or a combination of both. Diaphragm PZs feature an intake filter which separates the sensing diaphragm from soil particles but admits pore fluid.

(2) Twin-tube hydraulic PZs feature a similar filter. The filter must be sufficiently strong to avoid damage during installation and to resist effective soil stress without harmful deformation. Filters are classified as high air entry and low air entry types. Air entry refers to the bubbling pressure of the filter, which is the minimum pressure difference at which blow-through of gas occurs. For example, high air entry value filters have fine pores and require a high-pressure difference for blow-through to occur.

(3) Low air entry filters have coarse pores that readily allow passage of both gas and water and should be used for all PZ types that are installed in saturated soil or rock. Typical low air entry filters have a pore diameter of 20–80 microns and air entry values ranging from 3–30 kPa (0.4 to 4.0 pounds per square inch [psi]).

(4) For PZs that are not fully grouted, a low air entry filter theoretically does not need to be saturated when installed because the filter readily saturates and unsaturates with the

surrounding soil. Low air entry filters are suitable for fully grouted PZs, but need to be saturated when installed. Nevertheless, the filters are commonly saturated, and the transducers are mounted face-up with the filters to avoid air bubbles in the filter, which can delay the response time in low hydraulic conductivity soils.

(5) Saturated high air entry filters are used to separate the measurement of porewater pressure from pore-gas pressure and to keep gas out of the measuring system. High air entry filters are applicable for unsaturated soil, for which the porewater pressure is less than the pore-air pressure due to matric and osmotic suction. Such filters typically have a pore diameter of 1 micron and an air entry value of at least 100kPa (15 psi).

(6) Filters keep fluids in equilibrium by balancing the pressure differential with surface tension forces at the gas/water interface. The smaller the radius of curvature of the menisci is at the interface, the larger the pressure difference can be between water and gas. Because the minimum radius of curvature of the menisci is dictated by the pore diameter in the filter, the finer the pores are, the greater the pressure differential can be.

(7) If a high air entry filter must be saturated, removal of the filter from the PZ is required, followed by placing the dry filter in a container and applying a vacuum or boiling the filter. The filter should then be allowed to flood gradually with warm de-aired water. Once saturated, high air entry filters should be stored in a de-aired water bath until installed in the PZ. Filters are then placed on the transducer while both are submerged.

b. Intake Zone Length. Intake zones for PZs are normally minimal in length and placed to measure the pore pressure of a specific stratum. The intake zone may be made longer to increase the likelihood of intercepting strata affected by changes in transmissivity.

c. Estimating Time Lag.

(1) The significance of hydrostatic time lag should be examined in a project-specific context, as influenced by the purpose of the monitoring and anticipated porewater pressure fluctuations. Because the time required for 100% instrument pressure equalization is mathematically infinite, practical assessments have been expressed as a recovery ratio. For example, the values in Table 5.1 are based on a 90% instrument response as a benchmark for comparison of estimated time lags for various soil and PZ types.

(2) The estimated time lag is based on Hvorslev’s 1951 calculations for a confined, saturated soil. The values presented assume constant groundwater pressure and intake shape factor, isotropic soil, absence of gas pressure, and negligible stress adjustment time lag.

(3) Time lag in a confined strata is highly influenced by the type and dimensions of the PZ installed. For an open-standpipe PZ, the required flow and subsequent time lag may be considerable, depending on the surrounding material hydraulic conductivity. For most standpipe PZs in unconfined strata, the rate of water level rise within the standpipe is similar to the rate of the surrounding piezometric or phreatic surface rise.

(4) For semi-pervious soils, providing a larger tip (or screen) intake area or a sand pack increases the inflow rate associated with pressure equalization as water level changes. Also, reducing the riser-pipe diameter reduces the flow volume required for pressure equalization,

thereby reducing equalization time. Though reduced diameter risers increase the rate of equalization, standpipes less than 0.75 inches (1.91 cm) in diameter may be susceptible to bubbles that lift a column of water, and small standpipes are difficult to inspect and maintain.

(5) For a fully grouted PZ, the permeability of the grout mixture in comparison to the surrounding soil and the grout mix chemistry determines the effectiveness. The grout permeability should be similar to that of the surrounding soil. Table 5.2 compares the estimated time lag for various PZ installations, where L = length of intake zone and D = inside diameter of the casing. Mikkelsen and Green (2003) and McKenna (1995) provide additional details about fully grouted PZ installation and grout mixture design.

Table 5.1
Estimated Hydrostatic Time Lag of Various PZ Installations (Hvorslev, 1951)

<table>
<tr><th colspan="2">INSTALLATION TYPE</th><th colspan="10">FOR 90% EQUALIZATION = T_{90}</th></tr>
<tr><th colspan="2">Approximate Soil Type:</th><th colspan="3">SAND</th><th colspan="3">SILT</th><th colspan="4">CLAY</th></tr>
<tr><th colspan="2">Coefficient of Permeability (cm/s):</th><th>10^{-1}</th><th>10^{-2}</th><th>10^{-3}</th><th>10^{-4}</th><th>10^{-5}</th><th>10^{-6}</th><th>10^{-7}</th><th>10^{-8}</th><th>10^{-9}</th><th>10^{-10}</th></tr>
<tr><td>1</td><td>2" Casing: Soil in Casing, L = 3D = 6"</td><td>6m</td><td>1h</td><td>10h</td><td>4.2d</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>2</td><td>2" Casing: Soil Flush Bottom Casing</td><td>0.6m</td><td>6m</td><td>1h</td><td>10h</td><td>4.2d</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>3</td><td>2" Casing: Hole Extended, L = 3D = 6"</td><td></td><td>1.5m</td><td>15m</td><td>2.5h</td><td>25h</td><td>10d</td><td></td><td></td><td></td><td></td></tr>
<tr><td>4</td><td>2" Casing: Hole Extended, L = 12D = 24"</td><td></td><td></td><td>6m</td><td>1h</td><td>10h</td><td>4.2d</td><td>42d</td><td></td><td></td><td></td></tr>
<tr><td>5</td><td>⅜" PZ w/ Well Point
Diameter 1½," Length 18"</td><td></td><td></td><td></td><td>3m</td><td>30m</td><td>5h</td><td>50h</td><td>21d</td><td></td><td></td></tr>
<tr><td>6</td><td>⅜" PZ with Well Point and Sand Filter, D = 6,"
L = 36"</td><td></td><td></td><td></td><td></td><td>12m</td><td>2h</td><td>20h</td><td>8.3d</td><td>83d</td><td></td></tr>
<tr><td>7</td><td>1/16" Mercury Manometer, Single Tube with Porous Cup Point, D = 1¼," L = 2½"</td><td></td><td colspan="3" rowspan="2">One-half of Values for 1/16" Mercury U-tube Manometer or 4½" Bourdon Gauge</td><td></td><td>2m</td><td>20m</td><td>3.3h</td><td>33h</td><td>14d</td></tr>
<tr><td>8</td><td>1/16" Mercury Manometer, Single Tube with Well Point, D = 1½," L = 18"</td><td></td><td></td><td></td><td>6m</td><td>1h</td><td>10h</td><td>4.2d</td></tr>
<tr><td>9</td><td>3" W. E. S. Hydrostatic Pressure Cell in Direct Contact with Soil</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>16m</td><td>2.6h</td><td>26h</td></tr>
<tr><td>10</td><td>3" W. E. S. Hydrostatic Pressure Cell in Sand Filter, D = 6," L = 18"</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>16m</td><td>2.6h</td></tr>
</table>

Table 5.2
Estimated Time Lag for Various Conventional and Grouted PZ Installations (McKenna, 1995, Table 1, p. 358)

PZ Type	Installation Method	Typical Sand Intake Length (cm)	Typical Sand Intake Diameter (cm)	Approx. Equalization Volume per 100 cm Water Head (cm^3)	Approx. 99% Pressure Equalization Time
Pneumatic	Conventional	30	17	10^{-4}	< 1 s
		10	7.5	10^{-4}	1 s
	Grouted	20	5	10^{-4}	10 s
		10	5	10^{-4}	10 s
Vibrating-wire	Conventional	30	17	10^{-5}	<< 1 s
	Grouted	20	5	10^{-5}	1 s
2.5-cm diameter standpipe	Conventional	60	17	500	10 days
	Grouted	60	2.5	500	10 months

NOTE: Based on in situ material hydraulic conductivity of 10^{-7} cm/s and grout permeability of 10^{-8} cm/s. The 99% pressure equalization times are theoretical, and small bubbles near the PZ can make a large difference.

(6) The time lag for instrument response to pressure change must be distinguished from the time lag associated with an unconfined phreatic surface that follows a change in the hydraulic loading level on the embankment. Equations to estimate response time for confined piezometric level have also been developed by Penman (1960) and Hvorslev (1951).

5.3.5. Recommended Instruments for Measuring Piezometric Level in Saturated Soil.

a. Advantages and limitations of instruments for measuring piezometric level are summarized in Table 5.3.

b. Reliability and durability are often of greater importance than sensitivity and high accuracy for long-term installations. However, for a dam or levee safety modification or other short-term construction activity, the opposite may be true.

c. The reasons for monitoring, project risk, and the variability of the factors which will affect the instrument are crucial to the selection of the type of instrument and monitoring intervals needed. PZ installations with transducers may require corrections for temperature or barometric pressure to achieve high accuracy.

d. Open standpipes are reliable. However, readings obtained with manual water-level indicators have a practical limitation on reading frequency and are susceptible to human error. PZ filters can become clogged by various sources, which in extreme cases increase response time. Clogging can often be corrected by removing sediment and by other practices to dislodge debris in the screen.

e. Fully grouted or drop-in electrical and fiber-optic PZs facilitate a much higher frequency of readings and reduce human error. Fully grouted installations are simpler, provided the appropriate grout mixture has been designed, are quicker to install, and in confined aquifers have a relatively short time lag. However, a fully grouted PZ cannot be retrieved after installation, so replacement requires drilling another borehole.

f. Because of the high cost of mobilization for installation, redundancy can be obtained at critical locations by initially installing a pair of PZs near the same location. The redundant instrument serves as both a data quality control device and a replacement for the primary instrument.

g. For short-term PZ applications, defined as applications requiring high frequency readings and reliable data for a few years at most (during a typical construction period, for example), the choice is generally vibrating-wire PZs. The choice depends on the factors listed in Table 5.3 and an estimate of the contribution to the cost of the total surveillance and monitoring plan and the need for post-construction monitoring.

h. If computer monitoring is required, a vibrating-wire transducer installed in an existing open-standpipe PZ, a new sealed borehole, or a fully grouted installation is generally preferable.

i. For a long-term PZ application intended for infrequent reading, an open- standpipe PZ is an attractive option for simplicity and reliability. If vibrating-wire PZs should be used, vented cables are not recommended for long-term use due to the continual desiccant maintenance required and the susceptibility of the vent tube to becoming clogged.

j. For monitoring consolidation porewater pressure below a levee on soft soil, the push-in pneumatic or vibrating-wire PZ is a good choice for ease of installation. However, a correction for the elevation change of the physical instrument is required as the instrument settles. In such cases, a hydraulic PZ can be a good option due to insensitivity to settlement. However, the use of companion settlement gauges is a more likely solution.

k. As the economy of alternative PZs is being evaluated, the total cost should be estimated, including instrument procurement, calibration, installation, maintenance, monitoring, and data processing. The cost of the instrument itself is rarely the governing factor. In contrast, labor cost is significant for projects that are monitored frequently.

5.3.6. Measurement of Matric Suction.

a. Although matric suction is generally transient, and therefore does not contribute significantly to long-term soil strength, it can significantly contribute to strength in the short-term. Matric suction is a consideration in the proper interpretation of certain field-testing results and the explanation of certain failures.

b. In fine-grained soils, the pressure difference between water and air can be substantial, and special techniques are required to separate the measurement of porewater pressure from that of pore-gas pressure. Closed PZs are needed for the separation techniques that rely on saturated high-air entry filters in direct contact with the unsaturated soil. The installation details for matric suction measurements are critical.

c. Experience has led to refinements in the filter geometry to maintain intimate contact between the PZ and interstitial water. Such refinements include filters on the side of, or on a conical end of, the PZ. PZ response to negative gauge pressure is generally brief because gas in the ground will eventually break the saturation in the filters. The longevity of filter saturation is uncertain because gas may enter the filter by diffusion.

d. PZs of various types can be used to monitor matric suction. PZ selection criteria for measurement of matric suction require a low displacement diaphragm to minimize the water movement through the filter.

e. Summary of Advantages and Limitations of PZ Types. Table 5.3 is provided as an aid in summarizing the advantages and limitation of nine different types of PZs.

Table 5.3
Advantages and Limitations of PZ Types

Instrument	Advantages	Limitations
Observation Well (Readings Obtained with Water Level Indicator)	• Electrical sensors can be added for automated readings • Longevity • Allows water sampling	• Provides vertical connection between strata • Overflows under artesian conditions • Potential for misleading readings due to surface moisture accumulation inside • Surface riser vulnerable to damage
Open-Standpipe PZ (Readings Obtained with Water Level Indicator)	• Targets specific strata • Longevity • Seal integrity can be verified post-installation • Transducer can be added for faster or automated reading • Allows sampling • Falling/rising head tests can be used to assess instrument function	• Long lag time in confined low hydraulic conductivity material • Surface riser vulnerable to damage • Standpipe extension through embankment interrupts construction and may lead to poor compaction • Porous filters can plug due to cyclic inflow/outflow • Potential for misleading readings due to moisture accumulation inside standpipe
Self-Contained Water-Level Data Loggers	• Easy to program and deploy • Rugged and reliable • Long battery life	• Requires larger open standpipe or observation well • Data are typically downloaded manually
Twin-Tube Hydraulic PZ	• Long successful performance history • Integrity can be verified post-installation • Can be used to determine hydraulic conductivity • Short time lag in confined saturated stratum	• Large terminal with drainage and climate control needed • Gauge tubing must not be more than 32 ft (9.8 m) above minimum piezometric elevation • High air entry tips required to prevent air entry • Periodic flushing required • Possible confined space entry problems
Pneumatic PZ (Embedded)	• Short time lag • Calibrated part of system accessible • Resists freeze/thaw • Terminal areas are small • Immune to electromagnetic interference and lightning	• Not easily automated • Requires a compressed gas supply • Long tube lengths reduce accuracy • Skillful operator required • Reading time is relatively long
Transducer PZ (Drop-In/Retrofit to Open-Standpipe	• Easy to read • Shorter-term and long-term monitoring suitability	• May need lightning protection • Requires larger open standpipe or observation well

Instrument	Advantages	Limitations
PZ or Observation Well)	• New drilling not required to install • Easy to manually verify reading • Minimum construction interference • Resilient to freeze/thaw • Easy to replace	• Long lag time in confined low hydraulic conductivity material • Vibrating-wire not recommended where open standpipe or observation well is expected to dry out for extended periods • Potential for transducer and/or wire damage • Barometric pressure correction considerations for unvented PZs • Desiccant maintenance required for vented cables • Cable extensions to remote terminals results in clutter and susceptibility to lightning
Vibrating-Wire PZ (Fully Grouted, Embedded, or in Backfilled Borehole)	• Easy to read • Short time lag • Embedded PZs can read negative porewater pressures if high entry air filter is used • No freezing problems	• Need for lightning protection should be evaluated • Damaged PZs cannot be easily replaced • Manual verification using water level indicator or sounding not possible • Signal affected by nearby high voltage
Electrical-Resistance (Semiconductor) PZ (Fully Grouted, Embedded, or in Backfill Borehole)	• Rapid repeatability • Short time lag • Read-out equipment is more universal • Suitable for dynamic measurements • Can be used to read negative porewater pressures if high entry air filter is used • No freezing problems	• Low electrical output • Lead wire effects • Potential errors due to moisture, temperature, and electrical interference • Need for lightning protection should be evaluated
Fiber-Optic PZ	• Short time lag • Potential for rugged longevity • Immune to electromagnetic interference, vibration, and lightning • Low susceptibility to drift • Suitable for dynamic measurements	• Cost and fragility of read-out equipment • Temperature sensitivity • Cable cuts expensive to repair

5.4. Measurement of Deformation. Instruments for measuring deformation can be grouped in the categories listed in Table 5.4. The selection of a type of instrument depends on site-specific considerations such as the ability to install subsurface instrumentation and monitoring the direction of deformation. Advantages and disadvantages of the types of instruments to measure deformation are listed in Table 5.5.

Table 5.4
Categories of Instruments for Measuring the Location or Magnitude of Deformation

Category	Typical Deformation Measurement			
	Horizontal	Vertical	Surface	Subsurface
Geodetic Surveying Methods	X	X	X	
Tape Extensometer	X		X	
Crackmeters	X	X	X	
Tiltmeters*	X	X	X	
Extensometers	X	X	X	X
Embedded Settlement Monitoring Devices		X		X
Shear Indicating (TDR) Cables**	X	X		X
Inclinometers	X	X		X
Liquid Level Gauges		X		X

*Tiltmeters used in in-place inclinometers measure subsurface deformations.
**Instruments provide deformation location, but not a measurement of the amount.

5.4.1. Surface Monitoring. Surface deformations generally result from internal deformation. However, the feasibility of internal monitoring is limited by the installation or inclusion of intrusive devices and the cost of instrumentation. The normal progression of monitoring for unexpected deformation begins with visual identification, followed by surface monitoring to quantify the surface deformation and delineate areas of maximum movement, and proceeding to internal devices if needed. Surface monitoring of embankments and appurtenant structures is often used to indicate where internal monitoring is needed.

a. Structural Deformation Surveying.

(1) Geodetic surveys should be performed by specialists to obtain reliable measurements of high precision. Routine surveys to confirm as-built conditions are typically performed no more frequently than at 5- to 10-year intervals or following unusual events due to the expense. However, for high-risk features or during construction, the surveys may be performed frequently. Indeed, daily construction surveys are not uncommon during certain events.

(2) The survey data are used to determine the magnitude and rate of horizontal and vertical displacements of surface monuments located on and around embankment dams and

levees. If subsurface deformation measuring instruments are installed and determining absolute displacement from relative displacement is necessary, the surveys are used to relate the relative measurements to a reference datum.

(3) Surveying methods include:

(a) Optical leveling,

(b) Taping,

(c) Traverse lines,

(d) Measuring offsets from a baseline,

(e) Triangulation,

(f) Electronic distance measurement (EDM),

(g) Total station instruments combining EDM and angular measurements,

(h) Trigonometric leveling,

(i) Photogrammetry,

(j) Laser scanning (LiDAR), and

(k) Global Positioning System (GPS).

(4) A brief introduction on the use of surveying methods to monitor surface deformation of embankments is in Dunnicliff (1993). More detailed discussions on implementation of these various surveying methods are included in EM 1110-1-1000: Photogrammetric Mapping; EM 1110-1-1002: Survey Markers and Monumentation; EM 1110-1-1003: NAVSTAR Global Positioning System Surveying; EM 1110-1-1005: Control and Topographic Surveying; and EM 1110-2-1009: Structural Deformation Surveying.

(5) Regardless of the selected survey method, stable reference datums should be established in an immovable stratum and below the frost depth. A benchmark is needed for vertical deformation measurements, and a control station is needed for horizontal deformation measurements.

(6) Surface movement can be determined by comparing the position of these reference points to survey target points established on the monitored structure or feature. Benchmarks, control stations, and target points should be sufficiently stable and durable to survive for the life of the project. Each embankment needs a customized measurement scheme that provides the needed accuracy and confidence limits for the level of project risk.

b. Crackmeter.

(1) A crackmeter is a device used to monitor cracks and joints on embankment appurtenances. Crackmeters may be non-electronic or electronic.

(a) Non-Electronic Crackmeters.

- A non-electronic crackmeter may simply consist of a pair of monitoring points on either side of a crack or joint. Monitoring points can consist of pins, nails, or markings.

- The essential characteristics of this type of crackmeter are that the distance measuring equipment is not part of the crackmeter itself and that the measurement must be performed manually. The distance between the two points is measured with a tape, ruler, or caliper but may also be determined with GPS devices if the distance between pins is great. A third point can be installed to form a triangular configuration that provides for perpendicular movement calculations.

- Non-electronic crackmeters are prone to reference error. Problems and confusion frequently arise concerning the orientation of the crackmeter, the orientation of the crackmeter relative to the structure, and the orientation of the observer's line of sight while collecting readings. The person reading the instrument needs to understand the instrument orientation to ensure that the true direction of any displacement is recognized.

- Written instructions that clearly specify a precise viewing location help ensure consistency in data quality. Depending on the location of the instrument and the degree of accuracy required, magnification may be necessary to permit consistent readings.

- Another type of non-electronic crackmeter includes measuring scales as part of the meter and can be installed on small cracks. Although a person must read the scales by eye, having the scales in place ready to be read is a considerable advantage. One type of visually read crackmeter consists of two overlapping acrylic plates: one with a millimeter grid and the other with cross-hairs as shown in Figure 5.18.

Figure 5.18. Visually Read Crackmeter with Cross-Hairs

- Another type of visually read crackmeter consists of overlapping rulers that permit measurement in two directions, as shown in Figure 5.19. One ruler slips through a slit in the other ruler. Three-dimensional crackmeters are available that can be read visually.

Figure 5.19. Visually Read Crackmeter with Slit Ruler

(b) Electronic Crackmeters.

- Electronic crackmeters typically use vibrating-wire or LVDT technology to measure strain displacements. Figure 5.20 shows a vibrating-wire crackmeter installed to monitor crack width on a concrete surface. The red canister contains the plucking and sensing coil. Recording the initial reading reference at the time of installation and identifying a sign convention for movement is necessary.

- Electronic crackmeters are sensitive to temperature changes, requiring correction factors. A pair of electronic crackmeters can be installed on a crack such that one crackmeter measures the width of the crack and the second crackmeter, mounted perpendicular to the first, measures displacement parallel to the crack. Alternatively, electronic three-dimensional crackmeters are available.

Figure 5.20. Vibrating-Wire Crackmeter

c. Tiltmeter.

(1) Tiltmeters measure inclination and rotation about a point and can be manual or electronic. Manual devices typically incorporate plumb lines or bubble levels and are used on concrete walls and slabs to identify and determine the rate of displacement. In some cases where relative movement has been detected, tiltmeters can help identify which structural elements are moving.

(2) Electronic devices may incorporate vibrating-wire, servo-accelerometer, electrolytic-level, or MEMS technology. A vibrating-wire tiltmeter is shown in Figure 5.21. Tiltmeters may be fixed or portable, and both types are very sensitive.

(3) For portable tiltmeters, verifying that the probe positioning is consistent between readings is necessary. A stable reference measurement facilitates a check of both the calibration and operator skill at each reading session. In all cases, to convert the rotation into a displacement, the center of rotation must be known.

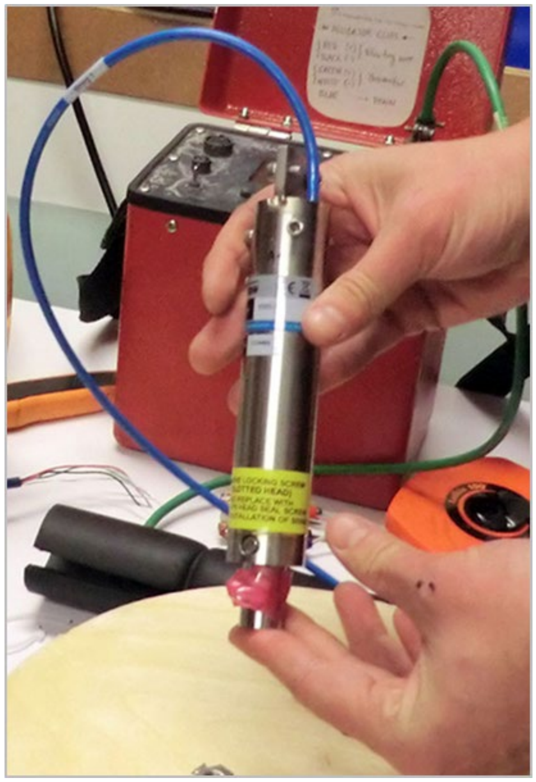


Figure 5.21. Vibrating-Wire Tiltmeter

d. Tape Extensometer.

(1) Installation of a tape extensometer (Figure 5.22) requires the secure installation of two or more eyebolt pins and the measurement of the distance between adjacent pins. During installation, the body of the tape extensometer device is attached to one of the eyebolt pins while a graduated invar tape is extended to an adjacent pin. The tape is then tensioned. Electronic or spring devices indicate if the appropriate tension is applied.

(2) Once appropriate tensioning has been reached, a measurement is taken and compared with initial readings to determine if the distance between the two pins has changed. A tape extensometer is typically used to determine changes in the width of interior surfaces, such as tunnels and other subsurface openings, and to measure changes in distance along embankment surfaces, such as the crest. One end of the extensometer must be fixed or surveyed for use as the reference for calculating distance changes to each point.

(3) Reference points should be stable and sensitivity to temperature change should be addressed with temperature correction factors or measurements from a stable control span.

Figure 5.22. Tape Extensometer (Geokon, 2019)

5.4.2. Subsurface Extensometer. Various types of subsurface extensometers are available, including the probe extensometer, embedded extensometer, and fixed borehole extensometer.

a. Probe Extensometer.

(1) Probe extensometers are used primarily for monitoring the vertical compression of the fill or the foundation of an embankment. A probe extensometer monitors the changing distance between two or more points along an axis by passing through a telescoping or longitudinally flexible pipe. Measurement points along the pipe are identified mechanically or electrically by the probe, and the distance between points is determined by measurements of probe position relative to the pipe entry.

(2) The deepest anchor should typically be set in or on a stable stratum to ensure reliable interpretation. Using the bottom end of the extensometer as a reference point makes necessary the subtraction of the displacement of the deepest anchor from each overlying anchor displacement to determine the true change relative to the lower end.

(3) Probe extensometers in unconsolidated material are usually installed vertically, measuring compression or heave, and anchor displacement measurements and elevation calculations are simple. However, horizontal installations measure lateral deformation in the downstream shells of embankments, for example, and elevation calculations must account for slope distance. This arrangement should be used with caution because the horizontal pipe can become a seepage conduit. The four types of probe extensometers are the:

(a) Crossarm.

• The crossarm (internal vertical movement, or IVM) extensometer was developed by the BOR for installation during the construction of embankment dams. Similar to the conceptual sketch shown in Figure 5.23 (a), the device consists of a series of telescoping pipe sections with alternating short inner sections of pipe anchored to the embankment by horizontal steel channel crossarms, typically at 10-foot (3.05 m) intervals. The crossarms ensure that the pipes move a distance equal to the settlement of the encompassing fill.

- Although able to measure extension, the IVM is primarily intended to measure compression, most of which occurs during and shortly after construction. Depths to the measuring point at the lower end of each interior pipe are sounded by a probe lowered on a graduated steel tape. As shown in Figure 5.23 (b), the probe is equipped with spring-loaded sensing pawls. The probe is lowered just below each interior pipe and is raised until the pawls latch against the lower end of the telescoping steel pipe.

- On impacting the bottom of the pipes, the pawls retract and lock inside the probe. Because the instrument assembly requires the crossarms to be placed on an exposed surface, the IVM cannot be installed in boreholes in the foundation and is limited to installation during the placement of embankment fill.

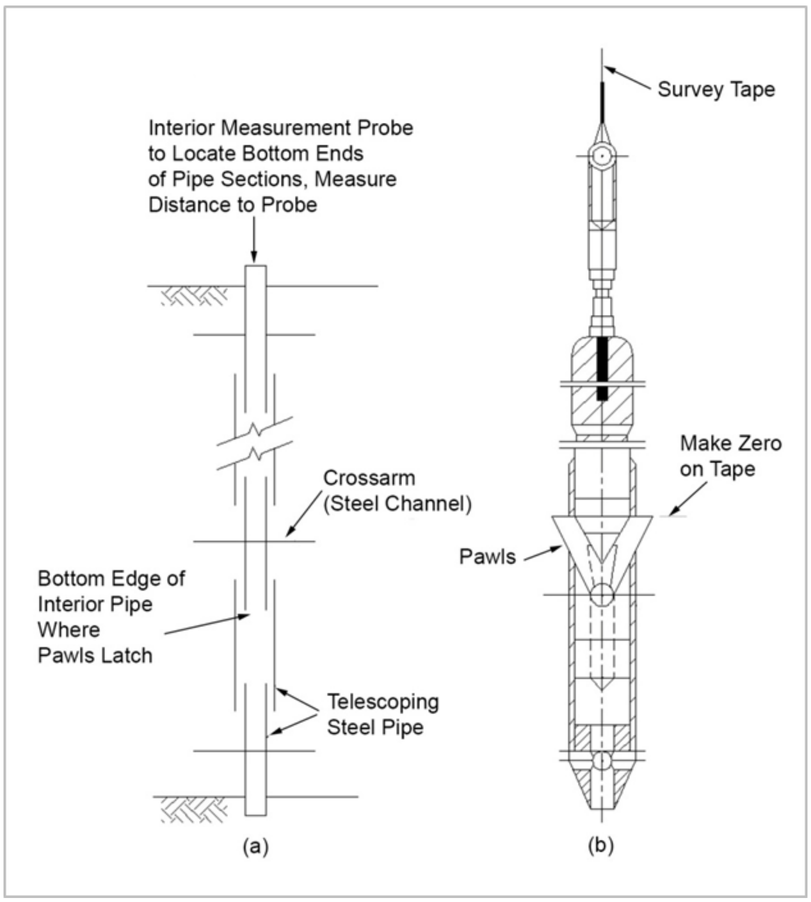


Figure 5.23. Telescoping Casing Arrangement and Measurement Probe
(Modified from Dunnicliff, 1993, Figure 12-29, p. 221)

(b) Mechanical.

- A mechanical probe, similar to the measurement probe shown in Figure 5.23 (b), can also be lowered within a telescoping inclinometer casing to determine the depths to the bottom ends of the longer inner-casing sections.

• Settlement collars cannot be installed for borehole installations. For telescoping foundation casings with no borehole anchors, settlement measurements do not necessarily conform to the actual soil compression.

(c) Magnetic.

• A magnetic extensometer, shown in Figure 5.24, features spider magnets, which are positioned into a borehole at specified depths with anchors (or legs) compressed. The anchors are released to extend prior to grouting of the borehole. As a magnetic/reed switch gauge is lowered into the borehole, the position of the spider magnets is detected.

• Magnetic extensometers are typically used in embankments for monitoring relatively large vertical compression. Having an accuracy within +/- 0.1 feet (3.05 cm), extensometers measured by a magnetic field are less accurate than mechanical probe extensometers.

• Magnetic settlement flanges, facilitating the use of a magnetic sensing probe, may be attached to a telescoping inclinometer.

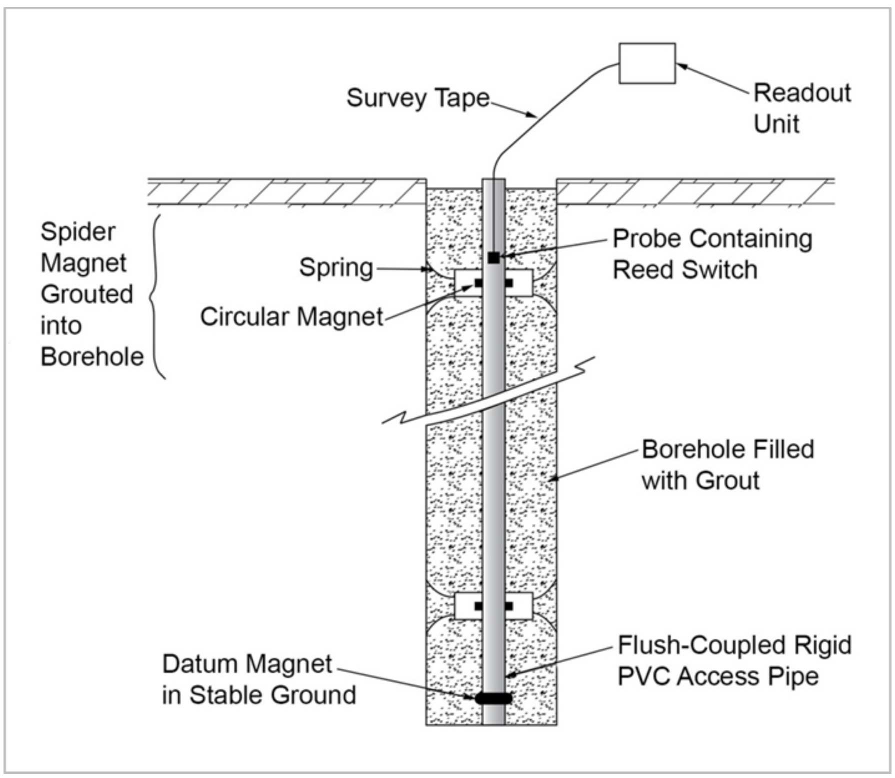


Figure 5.24. Magnetic Extensometer with Magnetic/Reed Switch Probe (Modified from Dunnicliff, 1993, Figure 12-38, p. 229)

(d) Induction Coil.

• An induction coil extensometer consists of an embedded telescoping pipe surrounded by steel rings or plates at the required measuring points, as shown in Figure 5.25.

- Although commonly used for rock tunnels, the induction coil extensometer has a potential use at embankments as a component of an inclinometer system. The reading device consists of a primary coil housed within a probe and an attached signal cable connected to a current indicator. Readings are made by traversing the probe along the pipe and noting the tape graduation at which output current is a maximum. Induction between the probe and sensing rings generates a voltage proportional to the sensing ring separation.

- If the casing cannot be anchored at appropriate intervals, the borehole grout must be carefully selected to ensure that the measurement system deforms axially in exact conformance with the soil. Such grout conformity may not be possible for variable strata or if vertical compression is large, such as in a levee foundation.

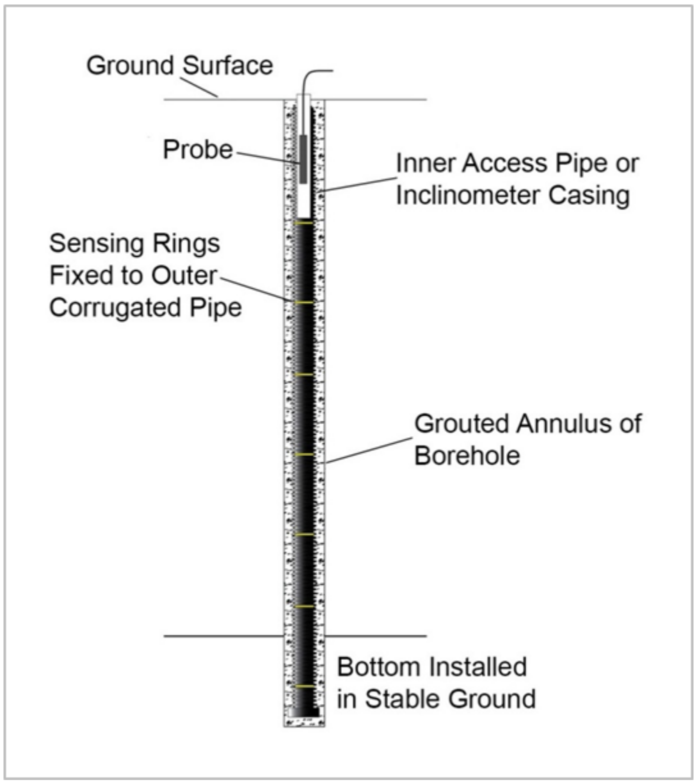


Figure 5.25. Induction Coil Probe Extensometer

b. Embedded Extensometer.

(1) An embedded extensometer, shown in Figure 5.26, is installed in a horizontal trench and is used to measure extension or contraction, typically parallel to the embankment axis. Compression of fill beneath the installation causes a change in anchor interval length. The purpose of the embedded extensometer is to reveal fill sections that can have enough differential settlement to induce cracking. This typically associated with an abrupt change in foundation slope or compressibility.

(2) Embedded extensometers are typically accurate over a relatively long extension range

of 3 inches (7.6 cm) but may be temperature sensitive. For a newly completed embankment located in the mid-latitudes, the temperature 10 feet (3.05 m) or more below the ground surface can be expected to stabilize in approximately 2 years. However, measurements obtained from installations less than 10 feet (3.05 m) below ground surface may require correction for seasonal temperature changes.

(3) Various types of sensors are available, but for each type, an initial reading is required. Although the position of the initial reading may not be accurately known, changes from that initial position are accurately indicated by subsequent measurements. Care is necessary to ensure the correct association between the sign convention of change and the identification of extension or contraction and to maintain an accurate representation of anchor intervals.

Figure 5.26. Embedded Extensometer Device

c. Fixed Borehole Extensometer.

(1) Fixed borehole extensometers are typically installed in boreholes in soil or rock to monitor the changing distance between two or more points along the axis of a borehole without use of a manual probe. The location of the fixed or anchored measurement point is referenced to the head (sensing end at the top of the borehole) if it is stable or otherwise to a bottom-most anchor within a stable stratum. The operating principle is shown in Figure 5.27.

(2) A typical application is the monitoring of deformation behind the face of an excavated slope. In rock, a change in the distance between anchors occurs if discontinuities such as joints, fractures, and shears slip, open, or close. The thermal stability of the reference head must be considered to obtain accurate results. The distance from the head of the rod anchor is measured using either a mechanical gauge or an electrical transducer. Having both capabilities is desirable so that manual verification is possible.

(3) The device shown in Figure 5.27 is a single-point borehole extensometer (SPBX), but

several down-hole anchors can be in a single borehole, each with an attached rod extending from a down-hole anchor to the collar to create a multiple-point borehole extensometer (MPBX), as shown in Figure 5.28. MPBXs are typically used to monitor the deformation or strain pattern along the axis of an appropriately oriented borehole such that multiple potential failure zones can be monitored and both deep-seated and shallow movements can be identified.

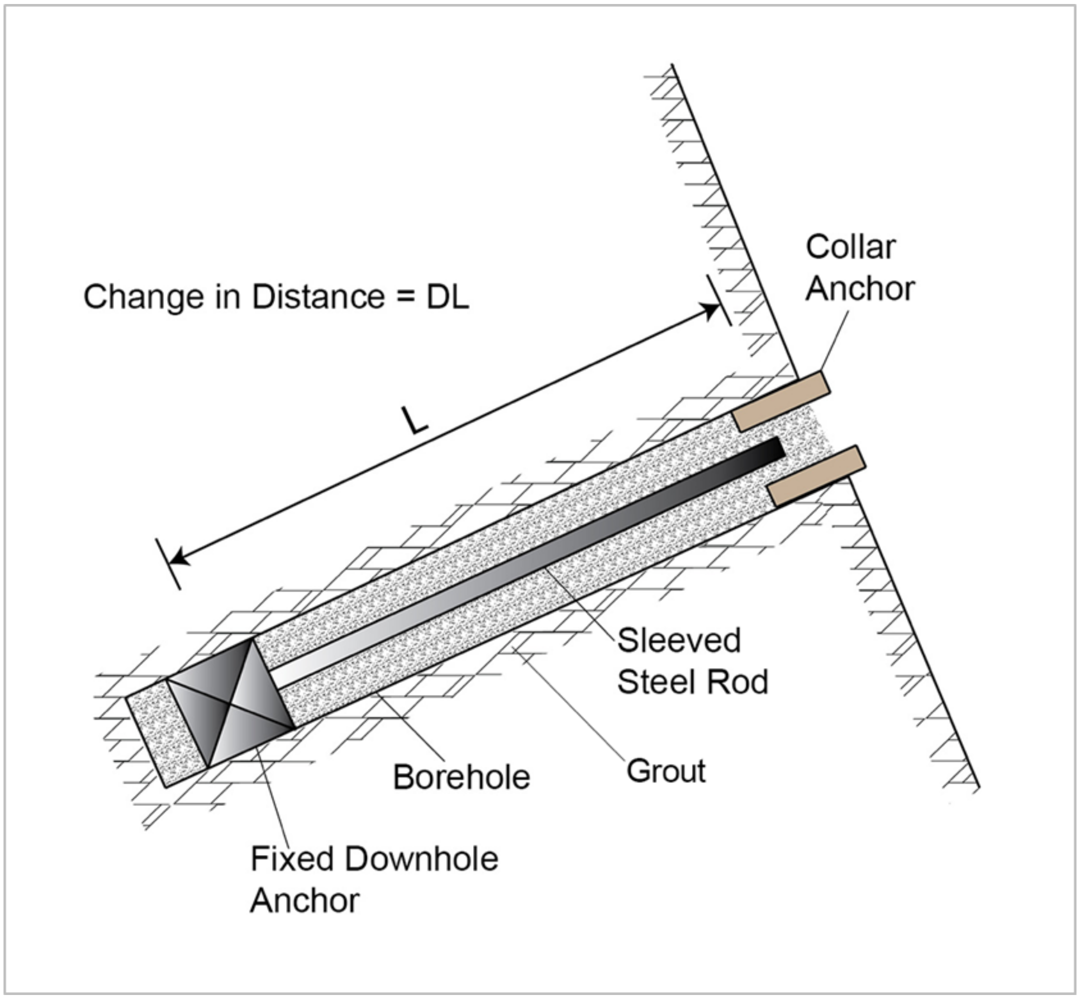


Figure 5.27. Fixed SPBX
(Modified from Dunnicliff, 1993, Figure 12.50, p. 237)

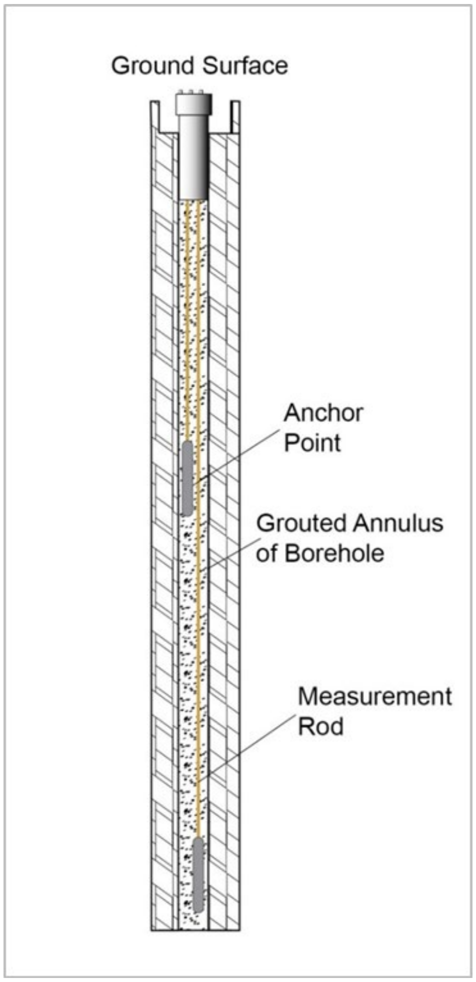


Figure 5.28. Fixed Embankment MPBX with Grouted Anchors

(4) Many types of fixed borehole extensometers are available, with the primary variables being the type of anchor (e.g., Borros type for soil and snap-ring, hydraulic, and groutable for rock), number of anchors, type of transducer, and extensometer head configuration.

(5) The selection of an extensometer assembly depends on the depth and diameter of the borehole, the length of monitoring intervals, the subsurface material, the groundwater quality, the monitoring frequency, and the degree of accuracy required. Many sensors used in rock can reliably measure displacements as small as 0.01 inches (0.25 mm). Fiberglass rods may be preferable to metal rods at an air/water interface where metal may corrode.

5.4.3. Embedded Survey Targets. Embedded survey targets are devices placed in embankment fill or on the foundation surface as placement of fill proceeds and are used for monitoring the changing distance between two points along a common vertical axis without the use of a movable probe. The initial installation elevation and the reference locations must be determined to ensure measurement accuracy.

a. Settlement Platform.

(1) A settlement platform can be used for monitoring the settlement of an embankment foundation by placing a square plate of steel, wood, or concrete at a known elevation on the original ground surface. A riser pipe is firmly attached to the plate, as shown in Figure 5.29. Although the pipe may be assembled in sections as the fill rises, the length of the pipe is constant once the installation is complete.

(2) The elevation of the top of the uppermost pipe can be determined by surveying, allowing the plate elevation to be calculated during construction and post-construction. Although a simple and frequently used device, the riser pipe interferes with fill placement and can be damaged if not protected. Staged installation of the riser pipe as the embankment construction progresses is not recommended, as it may interfere with achieving adequate compaction of the fill.

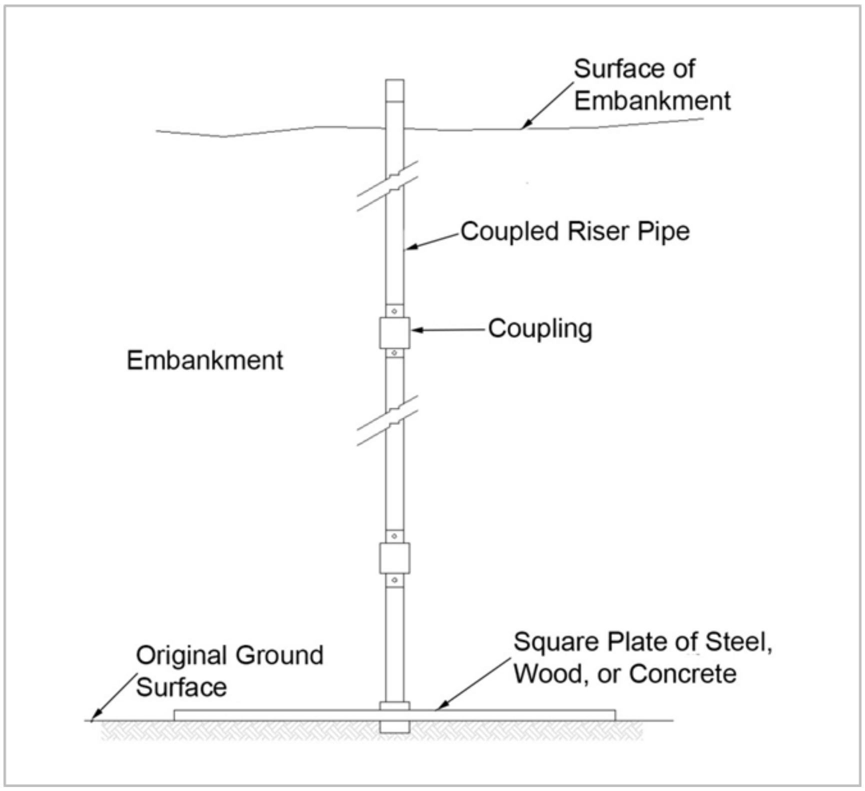


Figure 5.29. Settlement Platform (Modified from Dunnicliff, 1993, Figure 12.43, p. 234)

b. Subsurface Settlement Point.

(1) A subsurface settlement point is used to monitor consolidation settlement in an embankment or foundation. The device consists of a riser pipe anchored at the bottom of a vertical borehole and an outer casing to isolate the riser pipe from down-drag forces caused by settlement of the soil above the anchor.

(2) Settlement of the anchor is determined by surveying the elevation of the top of the riser pipe or using a hand-held scale to measure the distance between ground surface and the top of the riser. A common Boros anchor configuration used to monitor fill or foundation settlement

is shown in Figure 5.30.

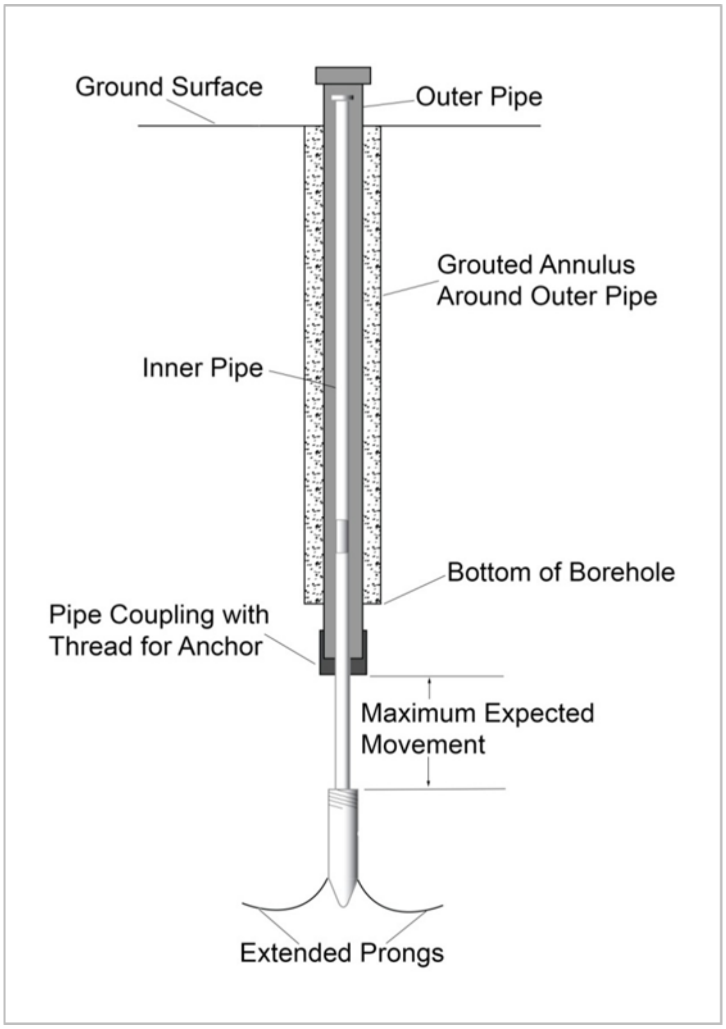


Figure 5.30. Subsurface Settlement Point
(Modified from Slope Indicator,® 2015)

5.4.4. Shear Indicators. A shear indicator detects a subsurface shear zone.

a. Time Domain Reflectometry (TDR).

(1) TDR is used to locate sharp cable bends caused by shear. TDR cables can be installed vertically in a borehole or horizontally in a trench. In TDR, a well-defined electrical pulse is sent along the length of a coaxial cable, and relative reflectance is monitored with a read-out device.

(2) TDR was originally developed by the telecommunications industry to locate and repair damaged coaxial cables. TDR measures the time required for the reflectance to return and converts the time into a distance along the cable. If the cable is sheared or displaced, the material properties of the cable at the location of deformation are altered as is the relative

reflectance at that location. Therefore, the location of shear or displacement can be accurately determined by monitoring changes in the signature pulse over time.

(3) For calibration, crimps can be made at predetermined intervals along the length of the cable to cause reflectance. However, the first deformation encountered may reduce the energy returned by more distant deformations along the cable length. As a result, multiple deformation locations or larger displacements may reduce read-out accuracy or render the instrument of no use. Therefore, defects intentionally affected for calibration should be detectable yet only moderately affect the electrical signal.

(4) A TDR cable may not always indicate an actual deformation of an embankment (false negative) and is subject to false indications of movement (false positive). For example, a mild deformation in a cable may not be detected if the deformation happens to be on the neutral axis of bending, because slight bending of the cable, without stretching or deformation, does not return a signal.

(5) Also, grout used during installation can cause false deformation readings if the grout significantly compresses the cable during curing. Water infiltration changes the electrical properties of the cable and may make the signatures difficult to interpret.

(6) Although a TDR signal locates an embankment deformation, the signal does not indicate the amount or direction of movement. However, the magnitude of movement may be estimated by the magnitude of reflectance change. TDR cables are commonly installed near or adjacent to other instruments, such as inclinometers and extensometers, to verify detected movement and to more precisely identify the location where movement has occurred.

(7) TDR cables are relatively inexpensive, and long distances can be monitored effectively. However, the associated read-out devices can be costly, and if automated, may require specialized conversion electronics and communication software. Significant advantages of TDR cables over more conventional inclinometers are that the TDR cables can be:

(a) Flexible;

(b) Installed in any orientation, including curves;

(c) Monitored continuously along the length of the cable; and

(d) Monitored continuously through time, if automated.

b. Optical Time Domain Reflectometry (OTDR).

(1) OTDR is similar to TDR, but OTDR uses light pulses transmitted through fiber-optic cables rather than electrical pulses transmitted through coaxial cables. If the fiber-optic cable has a fault or suffers deformation, light is reflected from the location of deformation. However, the use of a single cable to monitor multiple intervals requires the cable to have multiple dissimilar interference patterns etched into the cable.

(2) A fiber-optic cable can also be used to detect temperature distributions and changes. OTDR cables can be placed along the length of an embankment to detect differential settlement via strain distribution in the cable and to identify potential seepage locations via temperature

variations.

(3) A fiber-optic cable can be placed during or after construction and can be repaired using fiber-optic splicing equipment. However, splicing is typically expensive and requires a specialist. OTDR cables cost more than TDR cables, and specialized read-out equipment is required.

5.4.5. Inclinometer.

a. Inclinometers measure inclination of a casing at regularly spaced intervals. The movement of the casing can be tracked by integrating the incremental readings up the casing. The probes commonly use a biaxial sensor oriented in two orthogonal planes. Casing deflections from the vertical axis are calculated using the interval length and inclination. Inclinometers can be used to monitor both magnitude and rate of deformation. In addition, both incremental and cumulative displacement along the length of the casing can be determined.

b. The primary plane in which measurements are taken provides better precision than the secondary plane because of spring tension on the probe wheels and is oriented to parallel the plane of expected displacement closely, typically perpendicular to the embankment axis and toward the steepest slope.

c. Displacement along a shear zone causes deflections in the casing. Typically, the lowest point of the casing is anchored in a stable medium and is used as a reference point for calculating casing displacement. If the lower end of the casing cannot be considered anchored, the casing top must be located by surveying each time data are collected. Displacements of such casings are then calculated relative to the top of the casing.

d. Inclinometer casings can also be installed horizontally in the downstream shells of embankment dams or attached to the roofs of grouting galleries to determine vertical deformation.

e. Portable Inclinometer.

(1) Portable inclinometers are used for monitoring subsurface deformation by passing a probe manually through a casing installed, usually vertically, in soil or rock.

(2) In the section view, the casing has four longitudinal grooves spaced at intervals of 90°, permitting the probe to be guided along the length of the casing in two planes: typically one parallel and one perpendicular to the expected direction of movement. The probe contains two gravity-sensing transducers, often a force balance accelerometer designed to measure inclination with respect to the vertical.

(3) For deep installations, a spiral probe is used at the time of installation to determine the alignment of the casing groves. Spiral corrections are recommended for installations exceeding 100 feet (30.5 m) in depth, although the decision to implement the recommendation is influenced by the desired accuracy.

(4) The resulting data describing the spiral is used to correct all subsequent data for any twisting of the casing that may have occurred during installation. Alternatively, the probe may contain a digital compass that provides correction data for each reading session.

(5) Readings are obtained by raising the probe from the bottom of the casing and recording the inclination reading at specified depth intervals (typically 2 feet) and continuing to the top of the casing. Electrical cables typically have depth gradations marked along the cable. An initial set of readings along the length of the casing establish the initial deflection from the vertical. Subsequent reading sets are compared to the initial reading set to determine movement over time.

(6) It is critical to maintain consistent orientation of the probe between reading sets. The best practice is to use the same probe for every set of readings. Significant deviations from the vertical can cause a bias error in the data, which must be identified and corrected. Where the base of the inclinometer is not fixed, corrections can be made by surveying the top of the casing.

(7) As shown in Figure 5.31, a portable inclinometer system consists of four components: a guide casing, a portable probe, a portable read-out unit, and a graduated electrical cable.

(8) In the figure, the casing is shown to the left, directional grooves are shown at the top right, and an angled configuration is shown at the bottom right. Guide casings, made of acrylonitrile/butadiene/styrene, are provided by the inclinometer manufacturer and are available in various sizes. Diameter selection is dependent on the amount of casing deformation expected. Large diameters last longer during active deformation, but smaller diameter casings yield greater sensitivity, facilitating the early detection of displacement.

(9) Length selection depends on the number of casing joints desired and depth necessary to provide adequate anchoring of the inclinometer (zero displacement). Telescoping coupling casing is available to accommodate significant vertical compression. Aluminum alloy casing has been used, but instances of compromising corrosion within a few months after installation have been reported, even for epoxy-coated casing.

(10) If an inclinometer casing is installed in sections as an embankment fill is raised, construction is significantly affected, and poor compaction around the inclinometer is likely. Therefore, an inclinometer casing should not be installed in the core of an embankment as the dam or levee is raised. However, installation in the downstream shell or landside natural ground is commonly acceptable.

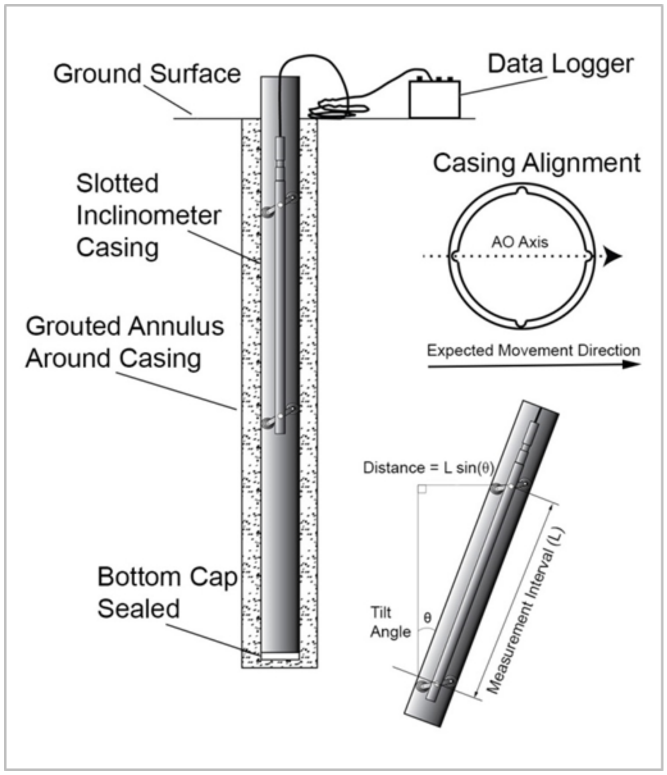


Figure 5.31. Inclinometer

(11) Inclinometer probes should be permitted to acclimate to the temperature at the bottom of the casing before any measurements are taken. Failure to do so can result in poor repeatability between readings. Care should also be given to centering the cable with a locking pulley system instead of pulling the cable against the edge of the top of the casing, which may damage the cable. The locking mechanism holds the cable and probe steady for consistent readings and extends the life of the cable.

(12) The quality of data obtained from an inclinometer depends on consistent data collection processes, probe calibration, and casing diameter. Consistency is enhanced by assigning one technician, probe, and data cable to an inclinometer. Such consistency reduces but cannot eliminate reading error. The typical precision a technician can achieve with a portable probe is approximately 0.10–0.25 inches (2.5–6.4 mm), which is adequate for most natural slopes and constructed fills.

(13) Inclinometer probe manufacturers for both servo-accelerator and MEMS inclinometer probes typically claim an error rate of 0.1 inches per 100 vertical feet (2.5 mm per 30.5 m) of casing. Precision may increase as casing diameter decreases, but small diameter casing is obstructed sooner than large diameter casing as deformation progresses.

f. In-Place Inclinometer (IPI).

(1) Although costly, an IPI is useful if a specific zone needs to be targeted. An IPI operates in the same guide casing as a portable-probe inclinometer but with a series of tiltmeters

held stationary in the casing, as shown in Figure 5.32.

(2) MEMS accelerometers, rather than force servo-accelerometers, are typically used with ADAS installations primarily due to cost, easier ADAS programming, and lower power consumption. The string of tilt sensors in the in-place inclinometers can be either uniaxial or biaxial. Uniaxial sensors should be installed parallel to the expected failure direction.

(3) IPIs traditionally consist of individual tilt sensors attached by rods at predetermined intervals. In such a traditional configuration, a cable extends from each individual sensor to the read-out location. Shape accelerometer arrays are a different type of configuration. This configuration consists of a string of tilt sensors connected directly to each other, without rods, with one cable connection to all tilt sensors, making installation simpler than traditional IPI configurations.

(4) Sensor spacing can be varied to allow for measurement at specific predetermined depths to focus monitoring on known or potential shear zones. Instruments can be removed to allow use of a portable probe for profiling the entire casing or to be used in another location. However, removing, relocating, or replacing an instrument disrupts the continuity of data.

(5) The advantages of using IPIs include reduced human errors associated with manual readings, reduced labor costs, continuous readings, and the option for connection to a console for transmission of data to remote locations or for triggering an alarm if deformation exceeds a predetermined amount. In addition, if appropriate casing and hardware is used, the instrument can be oriented to detect vertical displacement.

(6) Disadvantages include greater initial expense of the hardware if multiple sensors are employed, more difficult error detection, and possible difficulty retrieving the instrument if a shear deforms the casing.

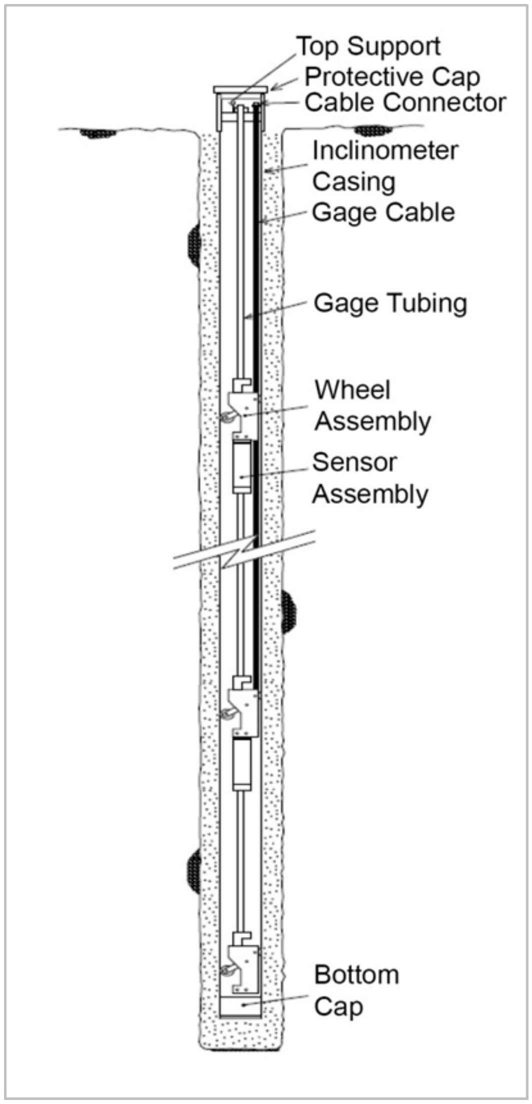


Figure 5.32. In-Place Inclinometer (Modified from Geokon®)

5.4.6. Liquid Settlement Cells.

a. A liquid settlement cell uses a liquid-filled tube to reveal differential vertical deformation. Relative elevation is determined from the pressure transmitted by that liquid to a transducer (Dunnicliff, 1993).

b. Figure 5.33 illustrates the liquid settlement cell principle with an overflow cell. The cell is read by adding liquid to the liquid-filled tube at the read-out station, causing overflow in the cell such that the visible level at the read-out station stabilizes at the same elevation as the overflow point. The vent tube is essential to maintain equal pressure on both surfaces of liquid, and the drain tube is needed to allow overflow to drain out of the cell.

c. Liquid settlement cells are an alternative to buried plates, settlement platforms, and subsurface settlement points in situations where the number of monitoring locations is few.

Because there is no vertical riser, installation during construction of the embankment interferes less with placing and compacting fill.

d. Liquid settlement cells are sensitive to liquid density changes caused by temperature variation, to surface tension effects in the tubing, and to any discontinuity of liquid in the liquid-filled tube. Errors due to these effects should be minimized. The use of liquid settlement cells requires more vigilance than do most of the instruments discussed in this manual. Liquid settlement cells outfitted with electrical transducers can be reliably automated.

e. Single-Point Liquid Settlement Cell.

(1) Single-point liquid level gauges (Figure 5.33) can be configured with the read-out unit higher or lower than the cell or at the same elevation as the cell.

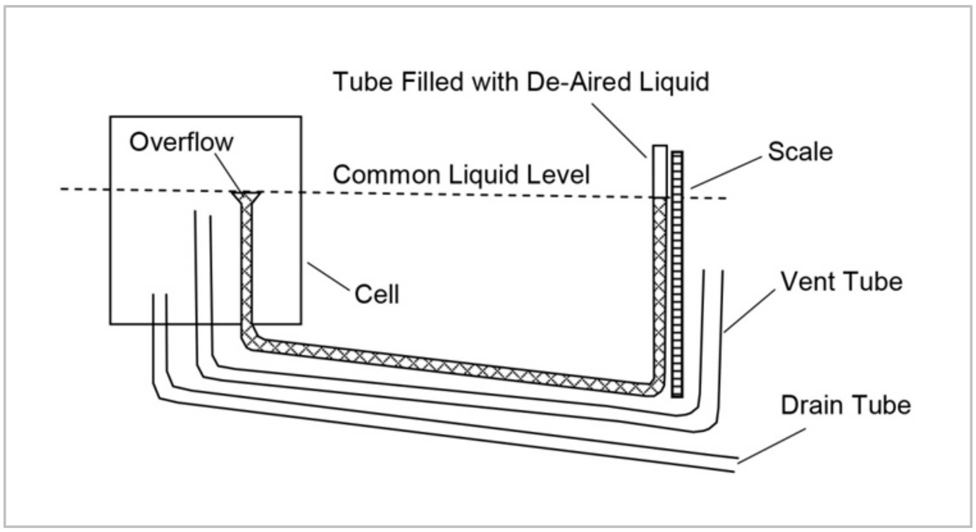


Figure 5.33. Single-Point Overflow Liquid Settlement Cell with Both Ends at the Same Elevation (Modified from Dunnicliff, 1993, Figure 12.91, p. 280)

(2) Figure 5.34 shows a read-out unit higher than the cell. The elevation difference, H, between the transducer and reservoir surface can be determined from the pressure measurement, P, and the liquid unit weight, γ. Despite using de-aired liquid in these instruments, continuity of liquid can rarely be maintained, causing reading errors.

(3) Improved versions are available in which the entire liquid-filled part is back pressured with gas. By recording the measured pressure and back pressure as the back pressure is increased, the pressure at which any gas pockets are forced into solution can be determined, eliminating that source of error.

(4) The single-point liquid level gauge with both the read-out unit and the cell at the same elevation can be converted to the configuration with the read-out unit at a higher elevation by applying a measured suction to the read-out end or by applying a measured back pressure to the vent tube. However, this method is limited to an elevation difference of 15 feet (4.6 m) due to the liquid becoming discontinuous at greater elevation differences.

(5) The overflow liquid level cell with both ends at the same elevation can be converted to the configuration with the read-out unit lower than the cell by applying a measured pressure to the read-out end of the liquid filled tube. However, this is not necessary using modern closed transducers.

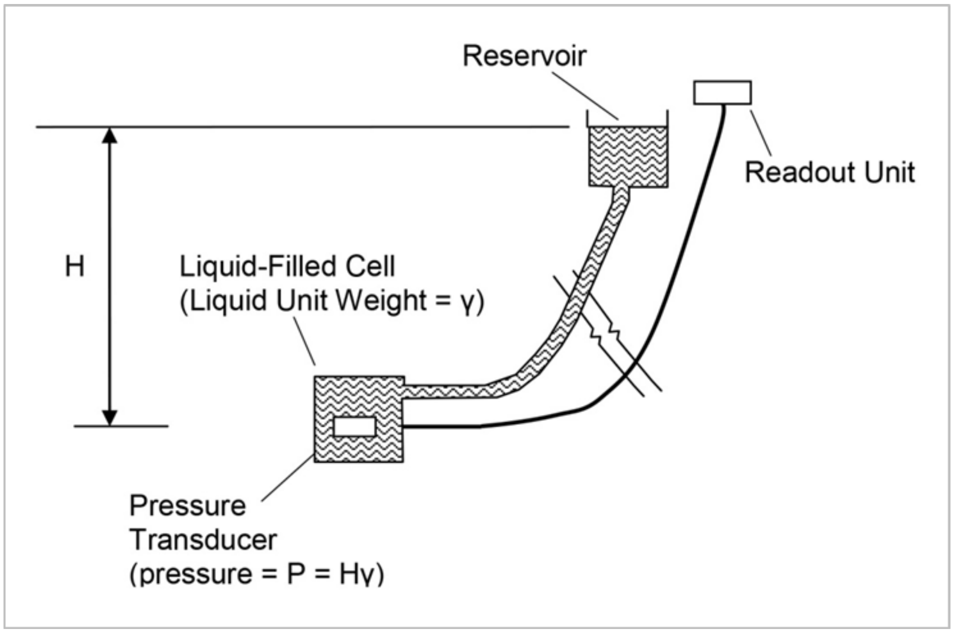


Figure 5.34. Liquid Level Gauge with Read-Out Unit Higher than the Cell (Modified from Dunnicliff, 1993, Figure 12.93, p. 282)

f. Multiple-Point Liquid Settlement Cell System. A multiple-point liquid settlement cell system consists of a group of single-point liquid settlement cells all connected to the same reservoir. In such a system, one transducer may be placed in a stable medium and used as a control point that can be measured relative to the other transducers. If unequal barometric pressures are a concern, a vent line can be connected to the system.

g. Full Profile Settlement Probe.

(1) A full profile settlement probe consists of a nearly horizontal plastic pipe and a probe that can be pulled along the pipe interior. Typically, readings are made at selected points, but the entire horizontal profile can be determined. Like the single-point liquid settlement cell, the probe contains a transducer that records the fluid pressure resulting from the elevation difference between the fluid reservoir at the read-out location and the point of the settlement measurement.

(2) The primary difference is that the fluid-tube length of the probe can be varied. Changes in profile over time provide data on vertical deformation for a general elevation within the embankment. Although intended for embankments, the devices generally should not be oriented transverse to the axis of embankments that impound water.

h. Comparison of Deformation Measurement Devices. Table 5.5 lists the advantages and disadvantages of deformation measurement devices.

Table 5.5
Advantages and Limitations of Types of Deformation Measurement Devices

Instrument	Advantages	Limitations
Tape Extensometers	• Relatively easy to install and use • Relatively high accuracy	• Tape wear • Must be properly tensioned • Non-stainless steel tapes susceptible to corrosion • Electronic readouts difficult to read in bright light • Calibration frame needed • Difficult to read long lengths in wind
Mechanical Crackmeters	• Low cost • Relatively easy to install and use	• Susceptible to temperature fluctuations if exposed • Protrudes from monitored surface • Displacement magnitude limited • Must protect and maintain portable read-out
Visual Crackmeters	• Low cost • Relatively easy to install and use	• Susceptible to temperature fluctuations • Orientation of observer when reading must be consistent • Epoxy can debond and result in failure to identify movement
Electronic Crackmeters	• Relatively high accuracy • Variable ranges available • Relatively inexpensive • Can be automated • Can be installed to monitor crack expansion/contraction or structural sliding	• Extremely susceptible to temperature variations • Sensitive to over-voltage and lighting • Installation with anchors can result in initial alignment errors
Tiltmeters	• High resolution and relatively high accuracy	• Sensitive to disturbances, vibration, and moisture

Instrument	Advantages	Limitations
	• Uniaxial and biaxial units available • Can be automated • Low temperature sensitivity	
Probe Extensometers	• Can be used with inclinometer casing to supplement inclinometer data • Relatively easy to install and use	• Magnetic installations have low accuracy • For depths greater than 30 feet (9.1 m), needs a locking assembly at the top of casing to support the tape reel—not readily available
Embedded Survey Targets	• Low cost components	• Installation affects fill placement and compaction • With risers, local survey control required for accurate readings
Fixed Borehole Extensometers	• Widely available • Long history of use • Can be automated • Multiple anchors can be installed in the same borehole to monitor different zones of potential movement • Can be installed at different angles	• Cost of drilling • Some units are susceptible to temperature variations • Corrosion of metal rods at the air/water interface
Shear Indicators	• Relatively high resolution • Relatively easy to install • Continuous readings along the entire length	• Can be subject to corrosion and water infiltration that can increase circuit resistance
TDR	• Monitoring device cables are inexpensive • Continuous readings along the entire length • Suited to identifying location of deformation • Can be installed with other instruments in same borehole or trench • Easy to install	• Do not identify magnitude or direction of movement • Water infiltration affects results • Movement must be great enough to deform cable • Read-out devices costly • Data interpretation requires specialized software and training

Instrument	Advantages	Limitations
	• Can be automated	
OTDR	• Can detect temperature changes; used to identify seepage pathways • Can measure strain • Can be automated	• Cost of read-out device • Cost of cable if multiple intervals to be monitored • Data interpretation and repair requires special skill
Portable Inclinometers	• One probe can be used for multiple casings • Time-proven performance • Relatively high accuracy • Can detect horizontal or vertical deformation with appropriate hardware and casing installation • Can be installed with other instruments	• Cost of drilling • Susceptible to human error • Data collection is labor-intensive for deep installations • Complete systems are expensive • Require temperature acclimation prior to taking readings
In-Place Inclinometers	• Relatively high accuracy • Can detect horizontal or vertical deformation with appropriate hardware and casing orientation • Can be installed with other instruments • Can be automated	• Cost of drilling • Number of monitored levels is small fraction of the levels monitored by portable inclinometers
Liquid Settlement Cells	• Installation has limited effect on fill placement and compaction • Sensitive detection • Read off structure	• Susceptible to temperature variations • Installation may create a seepage pathway • Requires a gallery, instrument hut, or other location for pipe/tube termination and data collection

5.5. Measurement of Total Stress.

5.5.1. Total stress in soil is measured with an earth pressure cell (EPC). EPCs are of two types: embedded cells and contact cells. An EPC may be completely embedded within a soil mass or placed at a contact interface between the soil and the face of a structural element, such as a footing, retaining wall, slurry wall, or culvert.

5.5.2. Modern EPCs typically consist of two circular stainless-steel plates welded together. A narrow cavity between the two plates is filled with de-aired fluid. As the plate exterior is subjected to pressure changes, a corresponding pressure change occurs in the fluid-filled cavity. Transducers convert the change in pressure into a signal transmitted to a read-out location. Transducers may be electrical, vibrating-wire, fiber-optic, or pneumatic. A contact EPC has one rigid side to be set against a stiff structure such as a concrete wall.

5.5.3. EPC measurements can be used to confirm design assumptions and to provide information for the improvement of future designs. However, an EPC is typically unsuitable for construction control.

5.5.4. Principal Stress.

a. Unlike a pressurized fluid, earth pressure is not isotropic. The distribution, magnitude, and direction of principal stress within an embankment or at a structural contact surface can be complex and variable. EPCs only measure the total stress acting normal to the cell, so an EPC should be placed at a point and in an orientation such that the measurements can be compared to predictable pressures.

b. Each orientation of a pressure cell in fill is subjected to a particular stress, as indicated by a Mohr circle diagram. Therefore, the complete state of stress can be determined by using multiple EPCs to obtain measurements of stress on three or more planes.

c. An example of the benefit of ascertaining the total state of stress is the estimation of shear stress on a potential failure surface. In such cases, rosettes of embedded earth pressure cells are typically oriented normal to, and at 45° to, the assumed principal stress axes. The three points of stress allow an estimate of the stress ellipse whereby the long axis of the ellipse represents the principal stress.

5.5.5. Measurement Accuracy.

a. Numerous factors affect measurements made with EPCs, including the ratio of cell thickness to diameter (aspect ratio), the ratio of soil stiffness to cell stiffness, cell size, stress anomalies from non-isotropic soil properties, and field placement effects.

b. Attempts to measure total stress within a soil mass are plagued by errors because the presence of the cell and the installation method change the field stress. Stiff cells cause stress concentrations, and soft cells allow stress arching around the cell.

c. Stiff diaphragm cells with large aspect ratios have been shown by elastic theory and experimentation to cause the greatest stress alteration if embedded in a soil mass. Even with favorable aspect ratios, matching cell stiffness to the soil modulus is at best difficult to achieve.

Although a cell is embedded at a specific orientation initially, the cell will likely become reoriented to an unknown degree by construction, compaction, and fill deformation.

d. For direct embedment applications, current practice and available products from geotechnical instrumentation suppliers have favored hydraulic cells having relatively flexible membranes and fluid with a high compressibility modulus.

e. To ensure that data reflects the actual earth load on a concrete structure, the cell should be placed within a block-out such that the cell-sensing surface is flush with the concrete surface. Even then, special compaction can result in the soil being too soft, which results in under-registering soil stress, or too stiff, which results in over-registering stress. The presence of gravel in the specially compacted soil can also result in high cell stress if rock points contact the cell.

5.5.6. Installation Guidelines.

a. The manufacturer's literature typically includes recommended installation practices for common EPC configurations. Installation procedures are intended to achieve two goals:

(a) Minimizing stress concentrations and changes in soil stiffness surrounding the EPC.

(b) Protecting the cell from damage during installation.

b. EPCs embedded in a soil mass are typically installed in a shallow trapezoidal trench. Even under ideal conditions, the probability is high that a cell and the trench become surrounded by a zone of stress concentration or relaxation because of the need to specially compact the encompassing backfill.

c. EPCs are temperature sensitive and should be used where temperature is stable. Contact cells may be subjected to significant temperature changes as the concrete in the structural element cures, which can result in inaccurate readings.

5.6. Measurement of Temperature. Temperatures in and beneath an embankment may be monitored to apply temperature correction factors for transducers sensitive to temperature change, to detect seepage, and to monitor changes in seepage patterns.

5.6.1. Instrument Types. For remote measurement, the most common types of instruments are typically used are thermistors and thermocouples.

a. Thermistors.

(1) The name thermistor is derived from thermally sensitive resistor. A thermistor features a small bead or disk of semiconductor material that changes markedly in electrical resistance as temperature changes.

(2) Of the two types of sensors discussed in Section 5.6, the thermistor is more precise over the temperature ranges commonly encountered at embankments and the type used by most instrument manufacturers. These sensors are typically included as part of other instrument types, such as the vibrating-wire pressure transducer.

b. Thermocouples.

(1) A thermocouple is composed of two wires of dissimilar metals, with the wires joined at one end to form a sensing element. A small voltage proportional to the temperature difference between the sensing and read-out ends is generated and measured between the wires at the near end. Thermocouples require no external power. Thermocouple accuracy is generally 1 degree Celsius (°C).

(2) Thermocouples are appropriate where quick, coarse, or inexpensive temperature measurements are appropriate.

5.6.2. Recommended Instruments for Measuring Temperature.

a. Temperature sensors are currently incorporated in many types of instruments suitable for remote measurement. The selection of the type of temperature sensor depends on the application. The two most common types of temperature sensors used for remote applications are listed in Table 5.6. Each type has a wide measurement range and a rapid response to changing temperature.

b. For a large area of an embankment, multiple remote measurements distributed across the area can be used to create a temperature profile. Interpolating between measurement points along that profile is generally adequate for estimating temperature at intermediate locations. If more continuous measurements along the length of a borehole, thermistor strings can be used.

Table 5.6
Instruments Used to Measure Temperature Remotely
(Modified from Dunnicliff, 1993, Table 14.2, p. 336)

Feature	**Thermistor**	**Thermocouple**
Read-out	Digital Ohmmeter or Datalogger	Thermocouple Reader or Datalogger
Sensitivity	Very High	Low
Accuracy	High	Moderate
Stability	Excellent	Good
Lead Wire Type	Two-Conductor	Special (Bi-Metal)
Lead Wire Repair	Simple	Complex
Response	Rapid	Rapid
Temperature- Correction	Preferred	Possible
Automation Suitability	Excellent	Excellent

5.7. Measurement of Seismic Events. Seismic instrumentation is primarily maintained to monitor for seismic induced ground motions that would trigger an assessment of project performance. ER 1110-2-103 provides requirements and guidance for installation and servicing of strong-motion instruments for recording earthquake motions on USACE dams.

5.8. Measurement of Seepage and Drainage.

5.8.1. Exiting seepage passing through, around, and under a water-impounding embankment should be monitored. Measuring seepage and drainage during construction, first filling, modification, and throughout the life of the project is essential for assessing the behavior of the structure. The first indication of an embankment problem is often a change in rate or color of flow. Correlations of seepage rate with precipitation, river or reservoir stage, and piezometric level can be used to assess the effectiveness of drains, relief wells, and cutoff features.

5.8.2. The source of embankment seepage can be the water held in a reservoir or river channel or groundwater flowing from the abutments or foundation.

5.8.3. Seepage that emerges at the downstream or landward side of an embankment is typically collected and measured at relief wells, drainage outlets, and open channels. The hydrology and geology of the site should be considered during the evaluation of potential locations for seepage monitoring.

5.8.4. Relevant literature includes manuals produced commercially and by the federal government. For example, guidance is provided in the Teledyne Isco Open Channel Flow Measurement Manual (2006). Flow measurement is described in detail in the BOR Interagency Water Measurement Manual (1987). Relief well guidance is presented in EM 1110-2-1914 and ER 1110-2-1942.

5.8.5. Measurement Method Considerations.

a. Methods of sensing water level may apply to more than one type or configuration of open channel flow measuring instrument. Open channel instruments for flow measurement—weirs and flumes—should be equipped with a properly mounted, easily observable, and readily accessible graduated staff gauge even if another type of sensor normally measures the stage used to calculate flow. Depending on the configuration of the device, the staff gauges may be vertical or inclined and should be graduated to permit readings accurate to 0.01 foot (3.05 mm).

b. Submerged pressure transducers need to be very sensitive over a small range. Vibrating-wire transducers generally do not have the sensitivity needed. However, a vibrating-wire transducer set to measure the buoyant force exerted by a partially submerged weight is an effective water-level indicator.

c. Ultrasonic sensors must be placed a minimum height above the water surface to avoid false return signals.

d. Measurement of water level by electrical capacitance uses an insulated electrode as one plate of capacitor and a reference electrode as the other plate. The capacitance varies with the liquid level between the two electrodes. A low-water level has a lower capacitance than a high-water level.

e. Capacitance measurement devices have the advantages of being quick to install and having no moving parts. However, changes in water temperature or chemical composition change the dielectric properties. Therefore, capacitance sensors should not be used where such changes may be significant.

5.8.6. Sharp-Crested Weir.

a. Sharp-crested weirs are simple open channel structures well suited to measuring the range of seepage and drainage discharge from most embankments. The discharge is a function of head, determined by measuring the water level in a stilling basin on the upstream side of the weir notch. The head is equal to the difference between the water surface elevation at that stilling basin and the lowest point along the weir crest. The notch is a standardized shape cut into a metal plate or other material capable of sustaining a sharp edge.

b. Selection of a notch shape for a weir is a trade-off between the capacity and accuracy afforded by V-, rectangular-, and trapezoidal-notch weirs.

c. V-notch weirs are typically used for low flow, are very accurate for flows less than 1 cubic foot per second (cfs) (0.0283 cubic meters per second) and are reasonably accurate for flows as great as 10 cfs (0.283 cubic meters per second). Rectangular-notch weirs can accurately measure greater flows than a V-notch weir, and a minimum crest width of one foot (30.5 cm) is recommended. Trapezoidal-notch weirs are used for greater flows but have a lower accuracy than V or rectangular notches.

d. Water-level sensing components of a weir or flume include:

(1) Staff gauge.

(2) Float with shaft encoder

(3) Gas purge device (bubbler).

(4) Submerged pressure transducer.

(5) Ultrasonic sensor.

(6) Buoyant-force transducer.

(7) Electrical capacitance device.

e. Figure 5.35 is a photograph of a V-notch weir equipped with a staff gauge and an automated vibrating-wire buoyant force transducer enclosed in a casing. A small flow is passing through the notch, and the direction of flow is from the background to the foreground of the photograph, as shown by the heavy red arrow. The weir is set in a rectangular concrete channel.

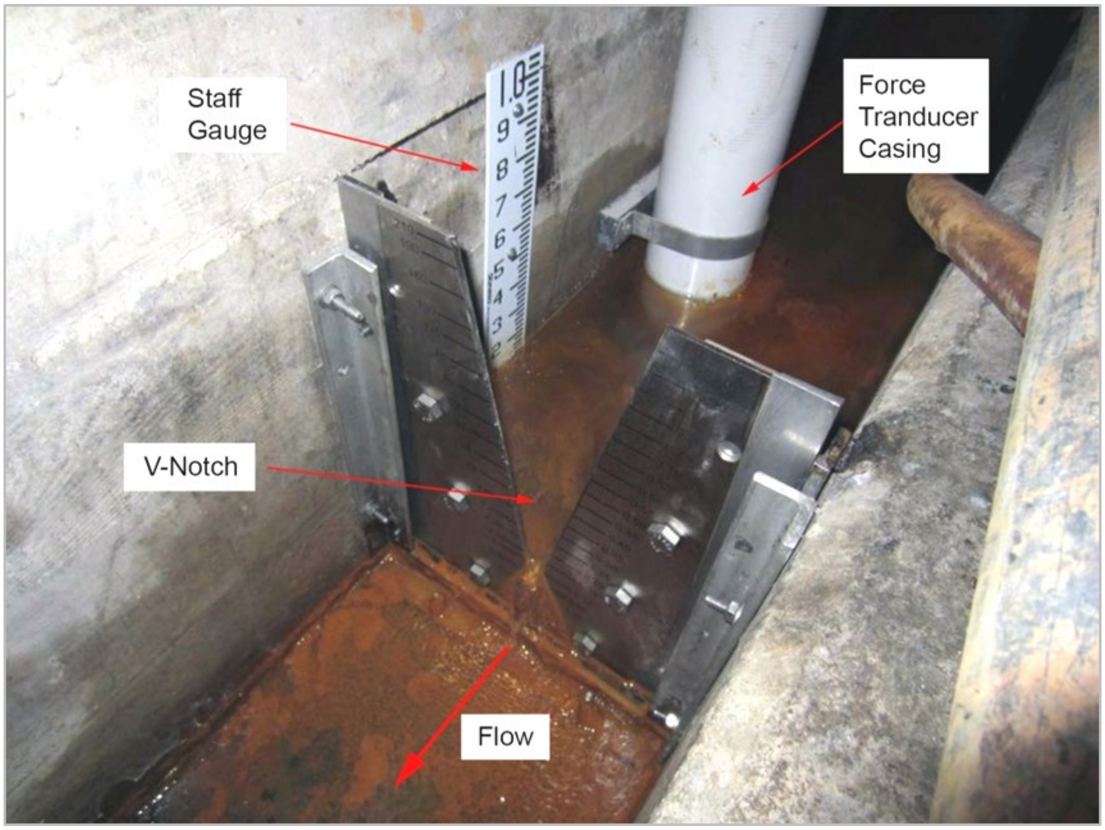


Figure 5.35. V-Notch Weir

f. Figure 5.36 illustrates flow measurement using a weir equipped with twin vibrating-wire force transducers. Water inlet pipes direct flow into the structure. A wave filter helps keep the surface of the water calm to permit accurate water level measurement. Water exits the structure through a weir with a 90° notch. Flow through the weir is a function of the water level in the stilling basin.

g. The water level is measured by vibrating-wire force transducers attached to partially submerged buoyancy cylinders in the stilling basin. (Refer to Figure 5.36 for a detailed illustration of a commercially available monitoring cylinder for weirs.) Because the buoyant weight of the cylinders is a function of the water level, the force measured by a transducer can be used to determine the water level, and thus, the flow. The signal produced by the transducer is sent to a read-out unit.

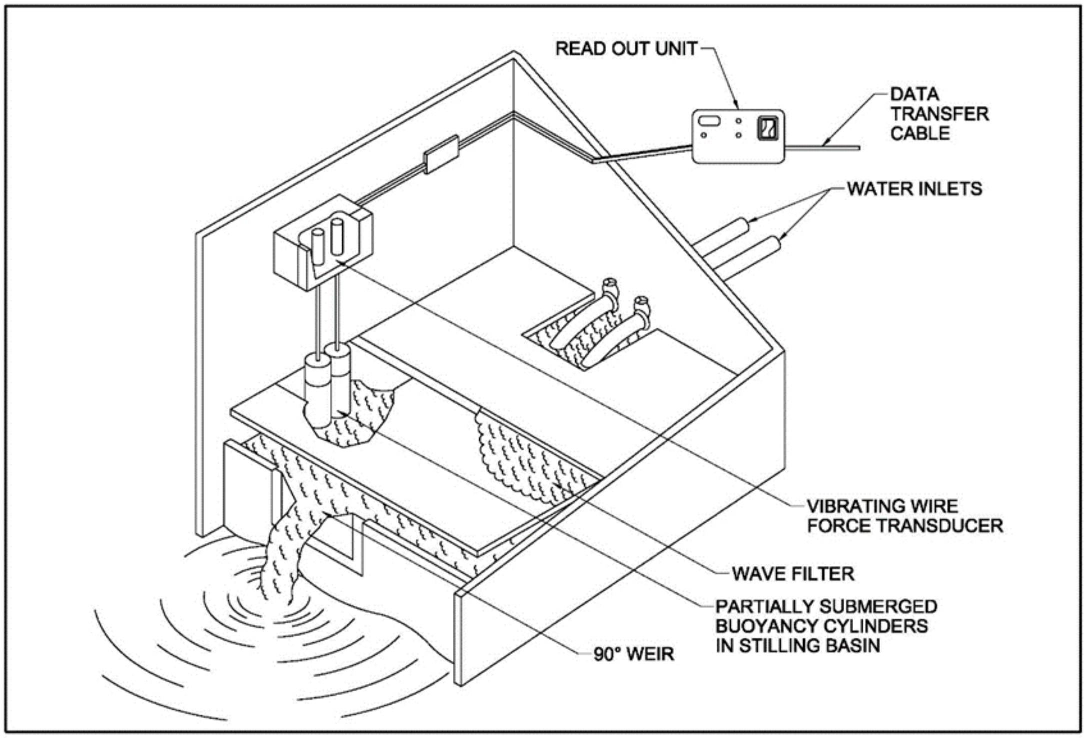


Figure 5.36. Weir Equipped with a Vibrating-Wire Force Transducer

h. Figure 5.37 illustrates a cylindrical weight enclosed in a casing and partially submerged in water. The weight is designed to be suspended from a force transducer to monitor water level in the stilling basin of a flow measurement weir.

Figure 5.37. Cylindrical Weight for a Vibrating-Wire Force Transducer (Geokon,® 2015)

i. Design considerations for weir stilling basins include determining suitable dimensions and preventing operational problems. Stilling basins need sufficient length and width to still water away from the notch to minimize the influence of the drawdown near the weir approach.

j. The channel bed profile should be sloped to prevent weir submergence. The weir should be protected from blockage by debris and from freezing to prevent misleading readings. A typical weir is easily inspected and cleaned. The zero reading on the staff gauge is set at the elevation of the bottom of the notch.

k. Weirs should be selected, sited, installed, and maintained correctly. Otherwise, problems could include:

(1) Incorrect size or shape for the flow range,

(2) Approach flow velocity too high,

(3) Submergence,

(4) Insufficient nappe ventilation,

(5) Improper orientation (not level),

(6) Weir plate reversed,

(7) Pool level gauge too close to the crest, and

(8) Leakage.

l. Due to the difficulty of predicting seepage rate, weirs sometimes prove too small or too large for the flows encountered. However, weirs should first be sized for the expected range of flow and then modified if ill-suited to the encountered flows. Poorly sized weirs are inaccurate due to undesirable nappe formation, faulty head measurement, or submergence.

m. Flow does not spring clear of the notch of an over-sized weir and the flow may be significantly greater than indicated by the head measurement. In contrast, flow approaching an undersized weir is likely to be too fast and turbulent, interfering with head measurement in the stilling pool. Also, an undersized weir may suffer submergence. The amount of flow reduction for a given percentage submergence is a function of the shape of the weir notch.

n. A weir plate must be oriented properly to provide correct data. A weir plate should stand in a vertical plane and the centerline of the notch should be vertical. The plate should be perpendicular to the flow. If the edge of the notch is beveled to obtain a sharp edge, the bevel should be on the downstream side of the plate.

o. The orientation of the weir plate should be checked during installation and should be checked afterward. If a mal-aligned weir plate cannot be made plumb, data reviewers should be made aware of the problem to devise a numerical correction.

p. Head should be measured upstream of the water surface drawdown that passes over the weir crest. Ordinarily, the gauge should be placed upstream a distance at least four times the maximum expected head. A gauge located too close to the weir crest indicates a flow less than actual.

q. Weirs used to measure seepage are normally intended to function as unsubmerged weirs. However, submergence may occur if the weir notch is poorly sized or if the channel bed profile does not permit a free overfall through the notch. Submergence can be minimized by maintaining the downstream channel free of obstructions.

r. In some cases, the site characteristics do not permit installation of an ideally functioning weir, which may require installation of a flow-measuring instrument of another type. If a less than ideal weir is installed, the persons designing the weir should determine the configuration and water-level measuring technique that obtains the best performance possible and also prepare a rating table that takes the characteristics of the weir and channel into account.

s. Leakage diverts flow that should pass through the notch, leading to misleading water-level readings. A typical weir has a plate attached to a bulkhead which is attached to the sidewalls and bed of the open channel. Therefore, seepage can occur through two contacts. Although leakage between the bulkhead and the channel is not uncommon, leakage between the weir plate and the bulkhead is more common. The use of gaskets and caulk typically suffice to repair leaks.

t. Weirs are preferred to flumes for measuring flow in small open channels at embankments because a weir traps sediment and debris possibly produced by internal erosion, increasing the likelihood the sediment is noticed. However, the open channel conveying the seepage must have enough vertical drop for a weir. If the channel grade does not allow the weir to function as an unsubmerged weir, a flume may be necessary instead.

u. For design and installation details on flumes and weirs, refer to the Department of Interior, BOR, Water Measurement Manual (BOR, 1997).

5.8.7. Flume.

a. A flume, such as a Parshall, cut-throat, or trapezoidal flume, is a constriction installed in an open channel to measure flow. The constriction is not abrupt but instead has smooth transitions upstream and downstream. The constriction is referred to as the throat and produces a drop-in water surface elevation compared to the elevation a short distance upstream. Flow is a function of the difference in the two water surface elevations.

b. The BOR Water Measurement Manual provides a detailed discussion of the limitations, reduction formulas, and dimensional requirements for accurate flow measurements.

c. Advantages of a flume compared to a weir are:

(1) The flume expends only 10% to 25% of the head a weir does.

(2) Sediment and debris tend to pass through a flume rather than accumulate, as happens on the upstream side of a weir plate.

d. Disadvantages of a flume compared to a weir are:

(1) A flume is typically more costly than a weir.

(2) Because sediment and debris tend to pass through a flume, a flume does not serve as a sediment trap if sediment monitoring is desired to assess internal erosion, which could therefore require installation of a sediment trap.

e. Considerations for the design of a flume include providing adequate length and good alignment and maintaining the proper orientation of plane surfaces in the transitions and constriction.

f. Although other types of flume are becoming more common, the Parshall flume is the most common type of flume installed and has the following characteristics:

(1) Accuracy,

(2) Low head loss,

(3) Available in a wide variety of sizes and materials,

(4) Difficult to fit into certain channels due to the required recess in the invert below the channel bed grade,

(5) Not affected by submergence unless the submergence head is 50% or more of the measuring head, and

(6) Measurement of the submergence head downstream of the flume constriction permits correction for various degrees of submergence depending on the type of flume.

g. Cut-throat flumes have a complex hydraulic behavior and are not recommended for measurement.

h. Compared to Parshall flumes, trapezoidal flumes:

(1) Can function in flatter channels.

(2) Conform better in shape with the cross-section of some channels.

(3) Are less sensitive to submergence.

(4) Can be proportioned to accurately measure flows as small as 1 gallon per minute (0.063 liter per second).

i. Palmer-Bowlus flumes have circular bottoms suitable for U-shaped channels attached to a pipe section. Calibrated Palmer-Bowlus flumes of the size typically needed for embankment monitoring are commercially available. A Palmer-Bowlus flume should only be installed in a channel as big as the flume or larger.

j. A smaller approach or exit channel causes inlet turbulence or submergence, respectively, preventing the flume from accurately measuring full-channel flow. A well-proportioned and properly installed flume can indicate flow accurately at submergences as great as 85%.

5.8.8. Calibrated Catch Container.

a. If occasional measurement of low seepage flow is required, the flow can be diverted into a container of known volume and the filling time measured to permit calculation of a flow rate. Clocking the filling time of a known volume is straightforward but use of a container too small with respect to the flow results in a timing error. The timing error is due to the sensitivity in starting and stopping the stopwatch.

b. The bucket and stopwatch method may also facilitate accurate measurement of a small flow in a setting where an installed weir is sized for larger flows. The bucket and stopwatch method is best suited for flows less than 10 gallons per minute (0.63 liter per second).

c. Very small flow rates can be determined by using measuring equipment that functions automatically over an extended time. For example, flow can be collected in a sump pit or basin of known volume. A sump pump equipped with a float switch empties the sump pit from time to time. A pump cycle counter on the sump pump allows operators to determine the volume of seepage collected over time. As a second example, a tipping rain gauge can be used to measure drips that might occur in a gallery or conduit.

5.8.9. Velocity Meter.

a. Measuring flow at collection system outlet points such as toe drains or relief well collectors is a typical application of a velocity meter. A velocity meter can also be used to measure flow for an individual relief well. Monitoring individual wells can provide data for seepage models and determine maintenance requirements. For example, a decreased flow rate at a particular head may indicate a filter problem in an embankment or screen fouling on a well casing.

b. Velocity measurement is typically accomplished by counting movements within a defined period but can also be accomplished by means of a change in voltage or a Doppler shift of light or sound. Types of velocity meter include:

(1) Pitot tube,

(2) Propeller,

(3) Ultrasonic, and

(4) Electromagnetic.

c. Velocity measurement is normally performed on closed conduits flowing full or at points where the cross-sectional area of flow may be readily determined as a function of flow depth. The point of installation and the orientation of velocity measuring devices are determined by the configuration of the attachment to the conduit.

d. Pitot tubes require full, fast flowing conduits and consist of a pair of tubes. One tube indicates the static head in the conduit, and the other tube indicates the total head. The difference between static and total head is the velocity head.

e. Full conduit flow seldom occurs at seepage outlet points. Therefore, a unit determining both flow depth and velocity is required in such cases, such as the electromagnetic area and velocity sensor.

f. An electromagnetic area and velocity sensor has a pressure sensor to determine the depth of flow in the conduit and an electromagnetic sensor to determine velocity. The pressure sensor is located on the conduit invert and the electrodes forming the sensor are located on either side. Flowing water passes through a magnetic field and creates a voltage across the electrodes. The voltage is proportional to velocity.

g. Fouling and freezing influence the choice of velocity meter type. If seepage is clean and freezing does not occur, a mechanical meter such as a turbine or paddle wheel can provide sufficient accuracy at low cost. For example, a relief well may be suited for a mechanical meter. If fouling or freezing is expected, instruments without moving parts, such as electromagnetic or non-intrusive ultrasonic sensors, are more suitable. In Figure 5.38, an ultrasonic velocity meter is shown attached to the exterior of a relief well riser pipe.

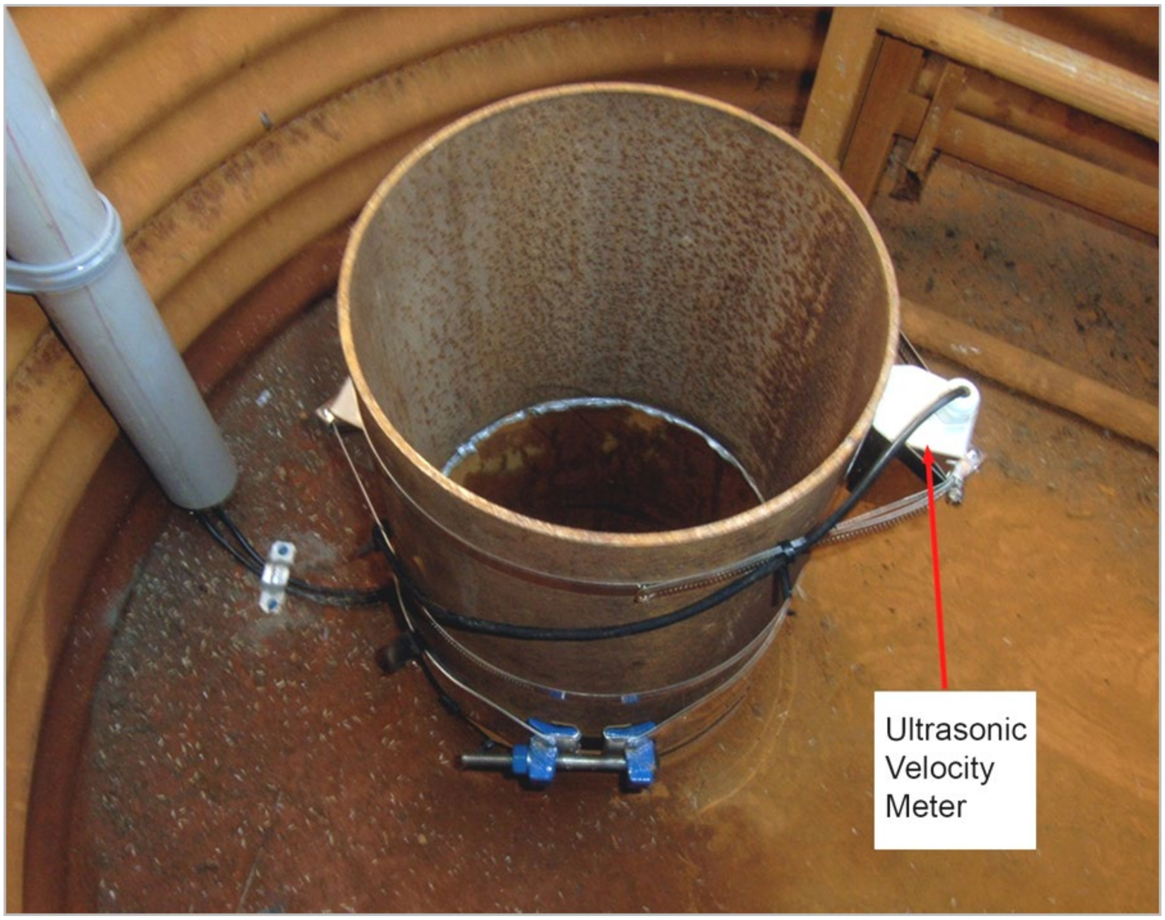


Figure 5.38. Ultrasonic Sensor on Relief Well Riser

h. In contrast to fixed velocity meters, a portable meter can be used at different locations provided the dimensions of the unit are compatible with the diameter of the conduits discharging seepage. A portable relief well velocity meter is comprised of an electromagnetic flow meter and a stainless-steel cage as shown in Figure 5.39. The sensor is attached to the bottom of the cage. The entire unit is lowered into the relief well riser. The cage ensures the sensor remains stable as data are collected, facilitating accurate readings.

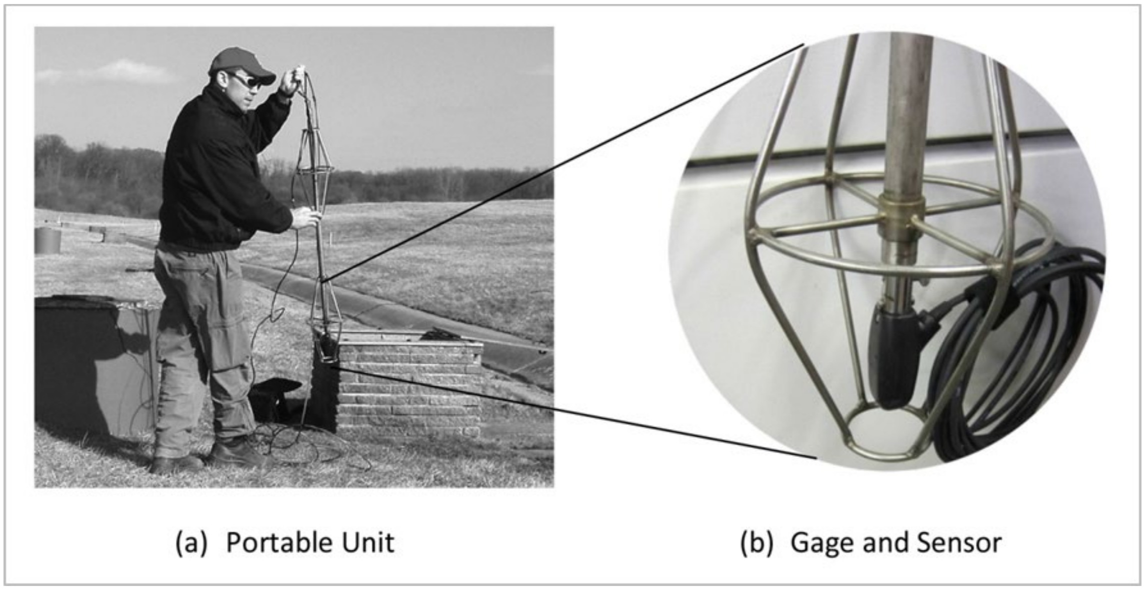


Figure 5.39. Portable Electromagnetic Flow Meter

i. Environmental conditions are a factor in selecting the proper type of velocity meter. Ice can immobilize or break parts. Biological or mineral fouling can immobilize moving parts in mechanical velocity meters.

j. Flow meters using electromagnetic fields to measure flow do not typically have mechanical limitations but can be damaged by corrosion or by induced electrical currents from unexpected sources and electrical conductors. If feasible for installation, ultrasonic velocity flow meters may be less vulnerable to damage than mechanical and electrical devices.

5.9. Investigation of Seepage Pathways. Knowing the approximate location of significant seepage paths in an embankment can be valuable for assessing performance.

5.9.1. Temperature.

a. The temperature pattern in an embankment may be influenced by water movement. Therefore, mapping embankment temperature distribution may be used to identify flow paths in the embankment.

b. For example, seasonal temperature variations at a depth where temperature is expected to be constant may indicate zones or paths of high hydraulic conductivity in an embankment or higher velocity flow in fractured rock in the foundation. Temperature differentials are useful for identifying seepage pathways (Birman, 1986; Welch, 1997; Johansson, S. and Sjodahl, P., 2004).

c. Some types of instruments not installed specifically for temperature measurement have temperature sensors. The temperature sensors can be used to obtain indications of seepage pathways. For example, some vibrating-wire transducers have built-in temperature sensing devices to aid in temperature calibration of the instrument. The temperature data recorded by these instruments can be used to identify temperature changes at the instrument locations.

d. Where multiple vibrating-wire transducers are installed, plotting temperature contours can aid in identifying temperature differentials and potential seepage pathways. As a second example, temperatures may be determined along a fiber-optic cable and analyzed to determine how temperature varies along the cable length.

5.9.2. Dye Tracing.

a. Dye tracing is not typically included in a monitoring program for detecting changes over time or monitoring project performance. It is considered an investigation method but can provide a lot of useful data pertaining to project performance.

b. The data obtained from dye tracing can be used to investigate seepage paths in the embankment, abutments, or foundation. Dye testing demonstrates connectivity between points and indicates velocity. Dye tracing may be very useful if fractured rock or karst formations are present in the abutments or foundation or if seepage rates are high.

c. The method of dye tracing consists of injection, detection, and timing. Dyes are injected into boreholes or observation wells or are released at specific locations in the reservoir, such as a vortex. The emergence of dye at the downstream end of the seepage path may be detected by eye or by collecting water samples or using granulated activated carbon units.

d. Dyes such as rhodamine may be detected at low concentrations using a field fluorimeter. Noting the time required for the dye to travel from the injection point to the downstream sampling point facilitates calculation of the average seepage velocity along the seepage path.

e. Standard dye tracing procedures have been established by ASTM (ASTM 2003a), the U.S. Environmental Protection Agency (Quinlin, 1992), and the USGS (Kilpatrick and Wilson, 1989). Several studies performed in karst geology have been published (Mull et al., 1988; Van Dike, 1985).

5.9.3. Moisture Content.

a. TDR cables, commonly used for deformation monitoring, can also be used for monitoring moisture content along the cable length. The speed of the electrical signal traveling the length of the coaxial cable is influenced by the moisture in the surrounding earth, decreasing with increasing moisture. Therefore, moisture content is estimated by measuring the propagation time along the length of the embedded cable.

b. Differences in moisture content along the cable can be monitored to identify potential seepage pathways. Changes in moisture content along a given interval of cable can help locate changes in seepage volume. A special interface is required to collect this data, and experienced technicians are required to interpret the data.

5.10. Measurement of Water Quality.

5.10.1. Suspended or dissolved solids in seepage should be measured if internal erosion of the embankment is suspected. Reasons to suspect internal erosion include:

a. Turbidity;

b. Increased seepage rate;

c. Known or suspected foundation or embankment flaws; and

d. Lack of seepage protection, such as a filter.

5.10.2. Measurement of water quality is discussed under the following paragraphs.

5.10.3. Suspended Solids.

a. An estimate of the rate of internal erosion occurring in an embankment can be based on direct weighing of deposited sediment or measuring the seepage rate and the concentration of sediment in the seepage.

b. Direct weighing of sediment can be facilitated by installing a sediment trap along a seepage flow path to collect sediment. Sediment unintentionally collected in the depression formed by an obstacle, such as a weir, can also be collected and weighed.

c. Volume of settled coarse sediment can be used as a substitute for weighing. For small quantities of coarse sediments, the depth of settled sediment in a series of sample jars filled at one sampling point can indicate a trend in erosion rate.

d. Optical methods can be used to indicate concentration of suspended sediment.

e. For fine sediment, the amount of light able to pass through a series of sample jars, as judged by the eye, can indicate a trend in erosion. However, the eye is not as accurate as optical instruments such as the turbidimeter or laser diffraction sensor.

f. A turbidimeter can obtain a much more accurate optical indication of suspended sediment concentration than the eye. However, a turbidimeter must be calibrated to obtain accurate results, especially if sediment grain size is not constant.

g. A turbidimeter can indicate suspended sediment concentration at an error as low as 1% if calibrated using a properly prepared suspension of formazin particles. In practice, the variability of sediment particle size and shape limit the accuracy attainable, especially if internal erosion has progressed to the point of imminent embankment failure.

h. A laser diffraction sensor requires no calibration for particle size and shape and is an advanced technology. The sensor can provide a particle size analysis for the range of particle sizes typically associated with embankment piping. If necessary, a sediment trap may be used to reduce water velocity, allowing the sediment to settle for analysis.

i. Laboratory mechanical sieve analyses can be performed on collected sediments. Results may indicate the source of the sediment by comparing with the embankment, filter, and foundation gradations.

5.10.4. Dissolved Solids. The concentration of dissolved solids and the seepage rate can be used to estimate the rate of solids removal. Laboratory testing is the primary means to determine the concentration and chemical composition of constituent dissolved solids in seepage water. A careful technique is required to obtain truly representative samples. However, various conductivity probes are available that can indicate the total dissolved solids (TDS) concentration onsite.

5.11. Measurement of Reponses to Meteorological and Hydrological Events. Measurement of meteorologic and hydrologic conditions that could be particularly useful for monitoring an embankment is discussed under the following paragraphs.

5.11.1. Precipitation.

a. Precipitation can affect the functioning of instruments. Therefore, precipitation data can help explain anomalies in other types of collected data.

b. Precipitation may affect instruments by damaging sensitive components, changing readings by adding water volume, and interfering with operation by introducing debris. For example, instruments mounted flush with the surface of the ground or a roadway may be susceptible to inflow from precipitation directly, to runoff, and to debris transported by runoff. Such flush-mounted instruments may include inclinometers and extensometers.

c. Precipitation or runoff entering an open-standpipe PZ may change the measured level and introduce debris. Rainfall runoff passing through a weir or flume intended to measure seepage may briefly increase flow to an unusual rate and may clog the instrument with debris. Precipitation gauges are available commercially and can be equipped to record and transmit data.

5.11.2. Barometric Pressure.

a. Barometric pressure has both nominal and physical aspects. Nominally, pressure may be expressed as an absolute pressure or a pressure with respect to atmospheric pressure. Physically, changes in atmospheric pressure affect absolute pressure at a point of pressure measurement in an embankment.

b. Persons reviewing piezometric data should be mindful of the importance of barometric pressure effects in principle and should be aware of the actual weather conditions that have influenced PZ data collected for the project. Review of piezometric data may include applying a barometric correction. For example, if a hydraulic gradient is low, then barometric variation may have a significant effect on the gradient.

c. Barometric pressure may affect PZ readings in an embankment. For example, compensation for change in barometric pressure may be required to obtain highly accurate information from an unvented PZ. Such compensation is essential at locations where barometric fluctuation is great.

d. Most piezometric measurements obtained at embankments reflect unconfined aquifer conditions. The water level in an unconfined aquifer is essentially unaffected by barometric fluctuation. However, an unvented PZ in an unconfined aquifer is affected by variations in atmospheric pressure. A lower atmospheric pressure results in less diaphragm deflection and a lower piezometric level reading. Higher atmospheric pressure results in a higher piezometric level reading.

e. In a confined aquifer, as might exist in the foundation beneath an embankment, the water level in an open-standpipe PZ rises as the atmospheric pressure drops and vice versa. A vented, closed PZ in a confined aquifer can be expected to function similarly.

f. If barometric pressure readings are obtained at a frequency sufficient to reflect peaks and troughs in barometric pressure, the data are useful for assessing piezometric measurements. Barometric pressure changes vary with the passage of weather fronts, during intense storms, and diurnally.

g. Barometric changes due to the passage of mild fronts are about 1 foot (30.5 cm) of head or 0.43 psi (2.96 kPa). Intense storms such as hurricanes and tornadoes cause larger barometric changes. Mild diurnal changes in barometric pressure are caused by the warming and cooling of the atmosphere.

h. Barometric fluctuation can be used to help determine if an open-standpipe PZ array is functioning well by comparing the response of the PZ to the known change in atmospheric pressure.

i. Barometric data adjusted to the pressure at sea level is available for most locations from the U.S. National Weather Service. Because barometric pressure patterns typically vary gradually across hundreds of miles, barometric readings obtained within a few miles of the embankment may be satisfactory.

5.11.3. Surface Water Level.

a. Seepage is driven by the difference between the water surface elevations on the upstream and downstream sides of an embankment dam or the riverside and landside of a levee.

b. Water surface elevations should be measured and compared with data collected from embankment instruments to evaluate embankment performance. Various instruments to measure water surface elevations are commercially available. Instrument to measure water level include staff gauge, float and encoder, pressure transducer, ultrasonic, and radar. Methods of recording data include chart and digital recorders. Simple staff gauges should be periodically compared as a check on automated systems.

This page intentionally left blank

Chapter 6
Automation

6.1. Introduction.

6.1.1. The use of automation should be considered when selecting instrumentation for a dam/levee, especially for monitoring significant failure modes to be installed for high hazard potential structures, new construction, major rehabilitations, structural modifications, or any major modification of the foundation.

6.1.2. An ADAS is composed of components that can be programmed to retrieve and store data from instruments at specified frequencies. Topics of interest include system complexity, frequency of obtaining readings, cost effectiveness, and continued developments in technology.

6.1.3. Data acquisition systems vary in complexity. An ADAS can be as simple as a single transducer with internal data logging capabilities. Alternatively, an ADAS can be a comprehensive series of instruments connected wirelessly to a data logger network that can transmit to a remote server and automatically display readings graphically and issue notifications.

6.1.4. Data retrieval, processing, and analysis options vary with system design. Tasks can be performed locally or remotely. The use of telemetry by modem, satellite, or radio facilitates data transfer to a central station and ultimately to a local area network (LAN) or the internet.

6.1.5. The frequency of automated instrument reading can be tailored to the needs of the project. Timing of readings can be set to monitor the rate of change in a parameter during critical periods such as construction or the exceeding of a threshold value.

6.1.6. An ADAS can be a cost-effective method of risk reduction. Although the installation cost of an ADAS is greater than that of a manual system, the long-term cost can be competitive in many cases.

6.1.7. Automation continues to develop. For example, the inclusion of video cameras in a monitoring system can supplement visual inspection for remote sites or difficult access points. As a second example, single channel data loggers, which reduce the use of signal cable but increase the use of communication elements, are improving rapidly and are being used more often.

6.1.8. Figure 6.1 is a schematic of an ADAS and associated telemetry. The three locations shown are the project reservoir, project office, and district office. Instrument locations are shown, and the parameters measured include reservoir level, tailwater level, toe drain flow, and piezometric level.

6.1.9. Communication pathways include line-of-sight radio, cellular modem, satellite, and wire. Automation components include a local monitoring unit (LMU) and remote monitoring units (RMUs). The figure indicates the complexity and interconnectedness possible with an ADAS.

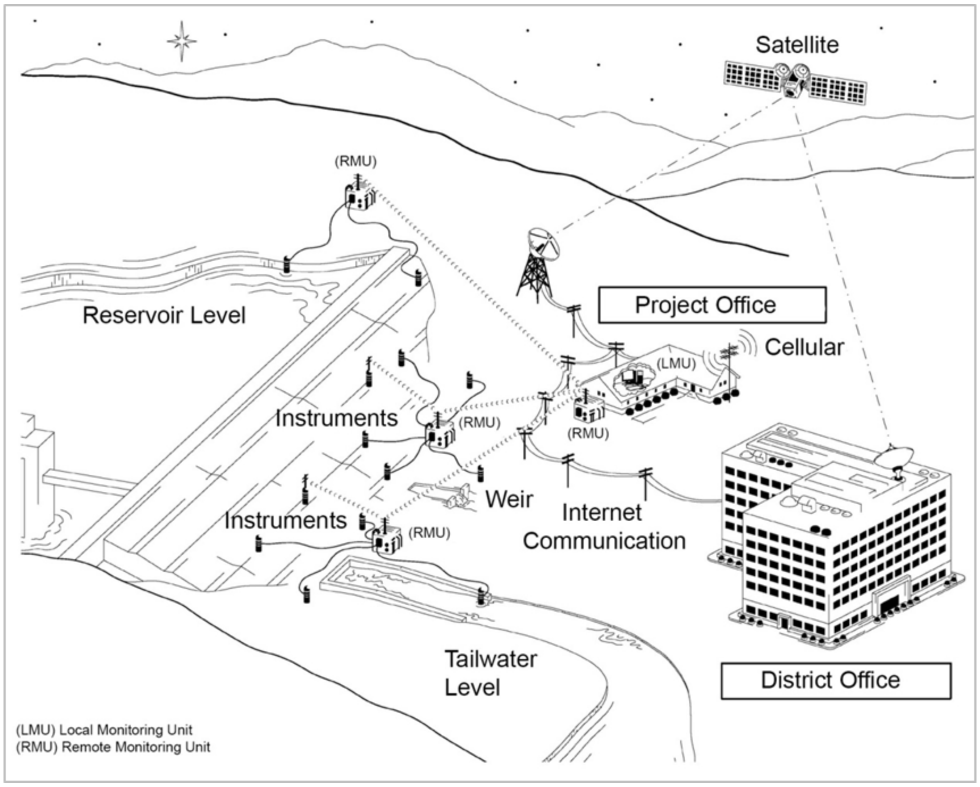


Figure 6.1. ADAS

6.2. Suitability.

6.2.1. Although not ideal for all projects, automation is suitable for many dam and levee projects, especially those requiring frequent monitoring. Many existing instrumentation types can be retrofitted to accommodate an ADAS. Combining an ADAS with a Supervisory Control and Data Acquisition (SCADA) system should be reviewed carefully because SCADA systems may have security considerations and large power requirements.

6.2.2. An ADAS is suited for frequent instrument reading, numerous instruments, or a site remote or difficult to access. However, an ADAS is not a substitute for inspection. An ADAS may be suitable for projects that need:

a. Frequent readings to establish trends and determine possible correlations with events.

b. Warning for a potential failure mode that can progress rapidly.

c. Frequent measurement because a formal study, risk assessment, risk reduction measure, or investigation has identified a specific or a potential problem area.

d. Monitoring during construction to:

(1) Ensure safe working conditions.

(2) Assess stability of the embankment and nearby structures.

(3) Determine deep foundation conditions to prevent hydraulic fracturing or slurry loss, which could harm the embankment.

e. To monitor pressures that consistently exceed established thresholds.

f. To monitor a rapid drawdown of water level against the embankment.

g. To monitor floods if personal access is difficult or unsafe or readings cannot be taken frequently enough to support decision making.

h. Data from remote sites where telemetry can yield data quickly and reduce travel.

6.3. Advantages and Disadvantages.

6.3.1. An automated system has advantages and disadvantages as summarized in Table 6.1. The advantages of automation over manual data collection are reduced labor for instrument reading and data entry and reduced incidences of human errors during data entry. Automation facilitates a frequency of data collection and processing and remote management of instruments otherwise unattainable manually. ADAS can be programmed to activate alarms and notify personnel of threshold exceedance.

6.3.2. Disadvantages of automation include initial installation cost and less visual observation of the structure. The cost of automation is significant and includes the long-term maintenance of equipment and software and specialized training for technicians. However, the cost of automation may be outweighed by the long-term reduced labor cost.

6.3.3. Visual observation is reduced because with its use, workers typically have fewer occasions to set foot on the project, physically handle instruments, and notice changes. Automated systems are not error-free. Inaccurate initial calibration or misuse of correction factors can cause error. Electrical noise can also cause error. Therefore, skilled technicians and data managers are required to install and maintain the automated system to ensure accurate data is being collected.

Table 6.1
Advantages and Disadvantages of Automation

Item	Advantage	Disadvantage
Reduced labor and human error	✓	
Increased frequency of data collection	✓	
Near-real-time access to data	✓	
Remote data acquisition	✓	
Remote diagnostics troubleshooting and editing	✓	
Notification of alarms for exceeding threshold	✓	
High cost of installation and maintenance		✓
Reduced visual observation during data collection		✓
Special training required		✓

6.4. Description.

6.4.1. Automation of embankment instrumentation is the integration of sensors, communication equipment, and data loggers. Components are configured in arrangements suited to the needs of a project. Components include:

a. Local monitoring units (LMUs),

b. Remote monitoring units (RMUs),

c. Central processing units (CPUs),

d. Central network monitors (CNMs), and

e. Remote input/output devices (RIOs).

6.4.2. LMUs and RMUs are programmable, stand-alone data logger systems. LMUs are wired directly to the CPU, but RMUs transmit data wirelessly to the CPU. LMUs and RMUs are the points in the ADAS where data are acquired, stored, and transmitted. Both LMUs and RMUs include all the parts needed to collect data from one or more sensors and can transfer data to a CPU.

6.4.3. A CPU functions as a central data collection point for the entire ADAS. Many options are available to the user once data are collected by the CPU, including reducing and storing data locally or remotely, displaying data via a local output device or webpage, and sharing data via a secure file transfer. Although LMUs and RMUs are able to store data, a permanent storage location should be maintained for longer-term applications and for data redundancy. The CPU typically stores data temporarily.

6.4.4. CNMs permanently store data for engineering use. A CNM may coordinate data collection across multiple sites.

6.4.5. A RIO device collects data from a sensor and transmits the data to an RMU or an LMU or directly to the CPU. A RIO combines a sensor with a communication device, either hardwired or wireless, but has no data processing or logging capability. A RIO can substitute for an RMU if internal data storage is not necessary.

6.4.6. Instruments exist that feature a combined sensor, data logger, and battery, such as a self-contained water level data logger. Although such instruments are typically collected manually, some manufacturers offer a telemetry feature. The compatibility of combined instruments with an ADAS should be checked.

6.4.7. Automation is described under the headings of:

a. Overall system configuration,

b. Communications, and

c. Power supply.

6.4.8. Overall System Configuration.

a. An ADAS configuration for data acquisition, storage, and transmittal depends on instrument types and locations and project requirements. Five basic configurations are:

(1) LMU,

(2) RMU,

(3) RIO,

(4) Multi-logger mesh network (MMN), and

(5) Distributed intelligence.

b. LMUs and RMUs.

(1) Components of both the LMU and RMU vary based on number and types of instruments being monitored, but a typical installation includes the following:

(a) Sensor. Sensors typically consist of an analog or digital transducer that reacts to the surrounding environment.

(b) Cabling. The type of cabling should be selected based on length of run, frequency and magnitude of signal, and environmental conditions, including temperature, proximity to electrical noise, and susceptibility to damage. Splices should be avoided only be made using manufacturer-recommended splice kits.

(c) Multiplexer. If multiple sensors are connected at a single RMU or LMU, a multiplexer is required. Several multiplexer options are available, depending on the number of sensors to be monitored. One data logger can be connected to more than one multiplexer. Depending on the sensor type, an interface may be required between the multiplexer and the data logger.

(d) Data logger. Suitable data loggers contain on-board, non-volatile memory used to store data from one or more sensors and use a real-time clock with an independent power supply.

- Data loggers typically require a computer connection for activation and configuration. One data logger can be configured to collect data from other data loggers via wired or wireless connection.

- A computer can also be periodically connected to the data logger to retrieve data. ADAS software provides for the configuration of the data logger, associated network, communications, alarms, and graphical displays.

(e) Radio. Spread-spectrum radios are used to transmit data between RIOs and RMUs or LMUs.

- Spread-spectrum radios spread the normally narrowband information signal over a relatively wide band of frequencies, making communication less vulnerable to noise and interference from radio frequency sources such as pagers, cellular phones, and multipath wave propagation.

- Spread-spectrum radios do not require an individual license nor frequency coordination through a regulatory agency. Some radio transmissions can be encrypted if necessary.

- While common radios don't require licenses or coordination through a regulatory agency, some 1-watt models with directional antenna can exceed the FCC limit of –36 dbm unless the transmit power is limited.

(f) Power Supply. A regulated power supply is required to power the data logger and sensors. The power supply configuration is determined by the instruments used, the communication link requirements, and the temperature of the operating environment. Typically, a battery is installed that can be charged from an alternating current supply or a solar or wind power source.

- The ability of solar panels to supply adequate power at high latitudes during winter should be checked. A regulator is often required to control the amount of charge transmitted to the battery.

- Cellular modems have high power requirements and need power management features such as a shutdown mode.

- Often, a data logger has a physically separate battery component. Where power requirements are low, non-rechargeable lithium batteries are suitable, especially in cold environments.

(g) Lightning Protection and Grounding. Lightning protection and grounding should be considered for each remote monitoring unit to protect the instrumentation from permanent damage and loss of data.

- Dry ground can prevent effective electrical grounding, and operation of over-voltage protection devices should be verified.

- Correct grounding techniques should be used to avoid ground loops that can add or subtract current or voltage from process signals and upset the proper functioning of instruments.

(h) Enclosure. All components in either an RMU or an LMU need to be enclosed in a housing to protect against the elements.

- Heaters may be helpful to reduce condensation and prevent freezing. Desiccants may be required.

- Locks are typically required to deter vandalism. Housings should be set at a height enough to protect equipment from flooding and be convenient for the worker. Enclosures should be large enough to contain all equipment needed initially and may be oversized to accommodate equipment needed afterward.

(2) Figure 6.2 is an example of an RMU mounted on a pole in the field. Figure 6.3 is an example of data acquisition components housed in the protective casing of the RMU.

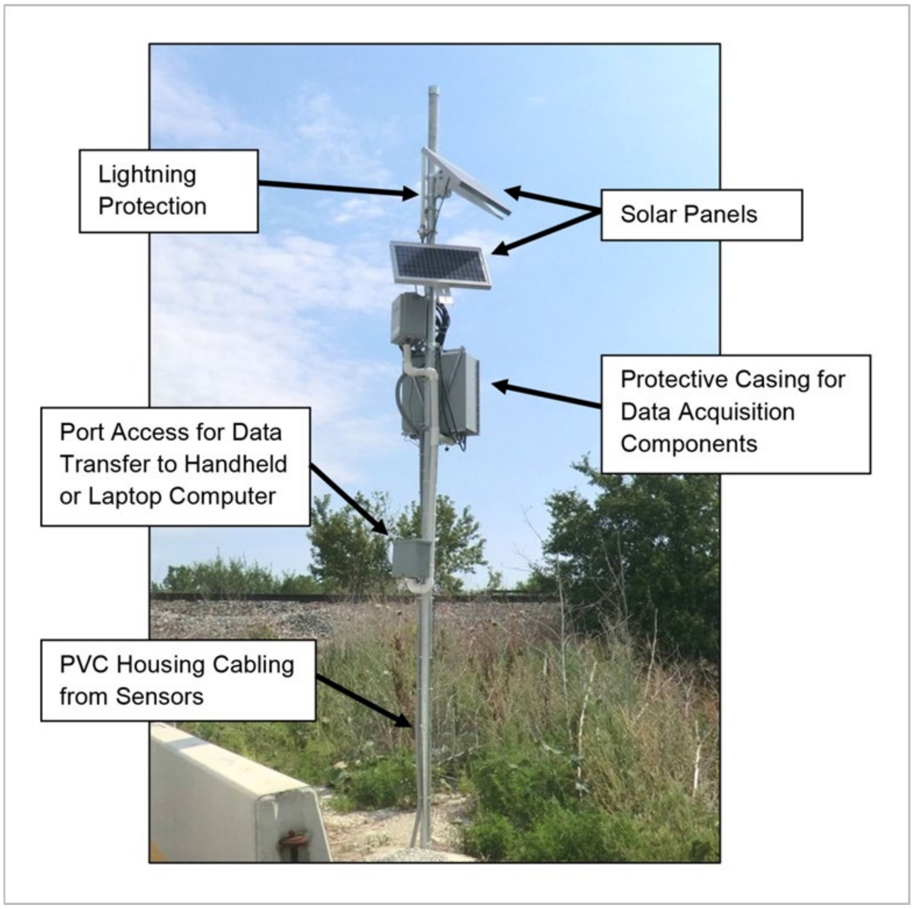


Figure 6.2. RMU

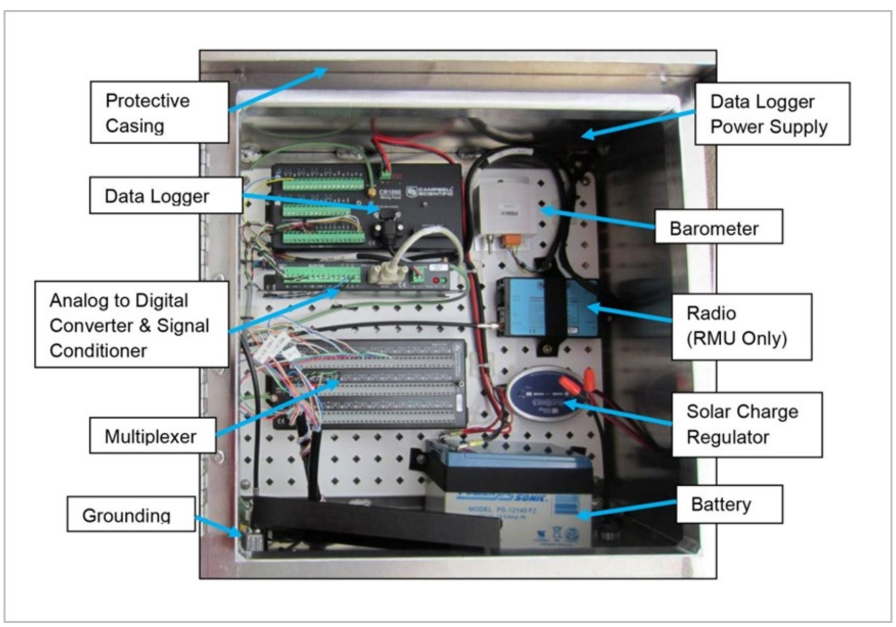


Figure 6.3. RMU Data Acquisition Components

c. A Remote Input/Output Device (RIO). A RIO usually comprises a sensor, analog to digital converter, radio, and power source within an enclosure. A RIO is less costly than an RMU or LMU because a RIO does not include a data logger. Without the data logger, the unit cannot store data and is only meant to transmit data. As a result, power requirements for a RIO are less than RMU or LMU installations, allowing smaller batteries and solar panels to be specified. As with an RMU, a RIO can be configured to read several instruments by using a multiplexer.

d. MMN.

(1) An MMN, shown in Figure 6.4, is composed of a CPU that controls two RMUs containing programmable data loggers. RMUs poll individual sensors, acquire and store the data, and then transmit the information to the CPU using wireless communications. Data reduction may be performed at either the RMU or CPU.

(2) The RMU can store data until communication with the CPU is established. The CPU is the system intelligence and is programmed to obtain data from the RMUs at defined intervals of time. Alternatively, an LMU can be used with a wired connection. Human intervention is not required for the system to operate. However, skilled personnel still need to review and analyze the collected data on a regular basis. Relying on an automated review of data is not desirable.

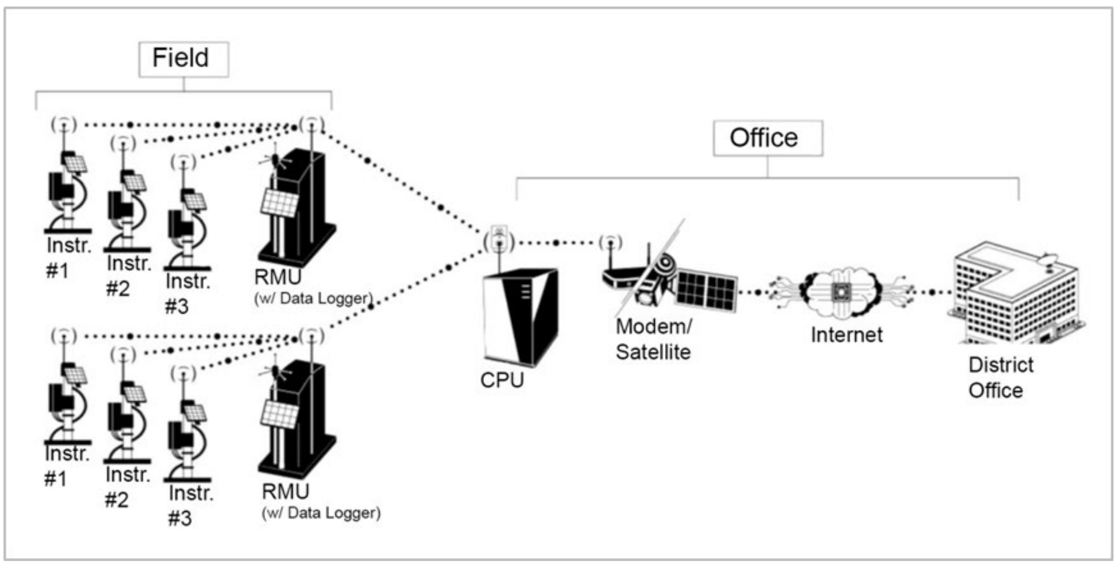


Figure 6.4. MMN

e. Distributed Intelligence. In a distributed intelligence configuration, shown in Figure 6.5, data loggers located at individual RMUs in the network are linked to a CPU. With proper software, this configuration allows the internal sharing of system hardware and data. A CNM (not shown) can establish communications with the CPU and with each RMU. The RMUs control data acquisition frequency and initiate communications. Raw data can be reduced and processed at these remote points without human or central computer intervention.

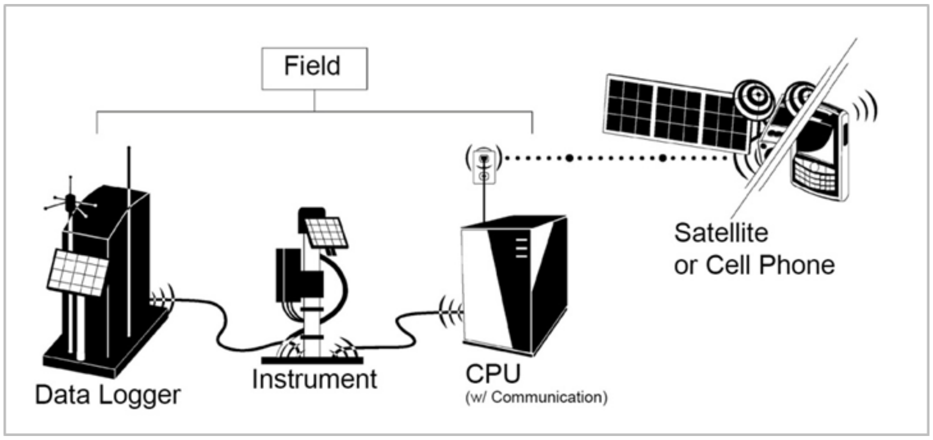


Figure 6.5. Distributed Intelligence Configuration

6.4.9. Communications.

(1) Basic communication links exist at three different points within an ADAS:

(a) Sensor to RIO to RMU (or LMU) or Sensor to RMU (or LMU). Data loggers within an RMU or an LMU can store data from multiple instruments but may require multiplexers.

(b) RMU (or LMU) to CPU. The CPU can be a handheld computer, laptop, remote or networked server, or desktop connected either through cable (LMU) or wirelessly (RMU) to the monitoring units. The CPU serves as the central data collection point for all automated instruments at the project site.

(c) CPU to CNM. Data can be transferred from the CPU to the CNM either through a wired or wireless connection. Alternatively, data can be transferred directly from the RMU (or LMU) to the CNM. Project personnel can access data from the CNM via a network connection or through remote means such as an internet website, file transfer protocol (ftp) site, or direct wireless transfer.

(2) Modes of communication vary. For example, the most common mode of communication between a sensor and an RMU (or LMU) is a signal cable. However, a common mode of communication between an RMU (or LMU) and CPU is by radio. Fiber-optic cables are also an option and can bef used for data transfer between the CPU and the CNM. Fiber-optic cables are less susceptible to lightning damage than electrical cables, but are more costly due to the need for transceivers to convert signals between optical and electrical.

(3) Remote communication from the project site to a database server or remote office is commonly by digital subscriber line (DSL), cellular modem, radio, or satellite constellation. However, a CPU may simply be transported to the remote office due to an inability to transfer data remotely.

(4) Alarms can be communicated to project personnel via sirens and lights, voicemail, email, and text messages. However, a given method of communicating an alarm may not be suited to a given instance. Whatever the means of communication, an automation system set to

automatically issue an alarm has the potential to issue a false alarm. If possible, the system should be designed to appropriately filter anomalous data to avoid issuing a false alarm.

(5) A computerized prompt for a manual data check before the issuance of an alarm may be appropriate. Emergency evacuation should include a human verification and not rely solely on ADAS instruments.

(6) The reliability of ADAS communications is affected by the operating environment and information security precautions. Communication components can be made more reliable by managing environmental factors, such as lightning and moisture, and maintaining clear lines of sight for radio. The ADAS must be compatible with information security precautions, such as firewalls and other data communication restrictions.

6.4.10. Manual Data Reading Option.

a. Regardless of the ADAS configuration, a person should be able to obtain data manually onsite from each instrument. For example, a person using a digital read-out device or portable computer can collect the data through a signal wye connection.

b. Manual data collection can be critical during an emergency if bad weather has interrupted power and communication networks. Because bad weather can affect more than one project at the same time, enough read-out devices to equip multiple projects should be available—preferably, one read-out device per project with a backup.

6.4.11. Power Supply. The power supply to an ADAS is determined by the type of available current and the source. Automated systems can be energized by alternating or direct current. The source of power can be supplied by a utility or generated onsite from sunlight or wind. The energy supply can be made more reliable by furnishing an uninterruptible power supply or an emergency generator. Those planning an ADAS should determine electrical demand and the primary and backup power source.

6.5. Planning.

6.5.1. Persons planning an ADAS need:

a. Specific information about ADAS, including lessons learned by others.

b. General knowledge of project management and procurement.

6.5.2. Chapter 4 describes planning an instrumentation system generally, including evaluating the need for an ADAS. Planning an ADAS is discussed in this section under the following paragraphs.

6.5.3. Identifying Sources of Expertise.

a. The persons who design, operate, and maintain automation for a USACE project may be USACE or contractor personnel. Manufacturer personnel may advise on the best use of ADAS components.

b. Contractor support for ADAS may be justified for new automation developments and complex applications. Reliance on contractors should be balanced against the need for USACE personnel to develop the knowledge needed to maintain and operate an ADAS. A contractor may design and install an ADAS, troubleshoot malfunctions, and make corrections.

c. Contracting ADAS operational expertise may be advisable for large construction projects where the volume of instrument readings and records exceeds the capability of the USACE staff. If a contractor is to participate in setting up an ADAS, USACE should:

(1) Specify test criteria for the ADAS.

(2) Require that an automated instrument can be read manually while simultaneously furnishing data through the ADAS to allow for testing of the ADAS during installation and any time afterward.

(3) Include the cost of contractor assistance in the instrumentation budget because the cost of operating an ADAS increases as dependence on a contractor increases.

d. Total dependence on an ADAS contractor can be a severe disadvantage, particularly during an emergency. An alternative to total dependence is to hire a contractor to install part of an ADAS system and provide training to USACE personnel on installation, management, and maintenance of the system.

6.5.4. Determining Which Instruments to Automate.

a. Factors to consider when determining which instruments to automate is influenced by:

(1) Significant failure modes,

(2) Instrument precision,

(3) Monitoring frequency and duration,

(4) Instrument location and access,

(5) Existing instrument technology, and

(6) Synchronizing instrument readings.

b. Automation of instruments that monitor significant PFMs should be considered when a high frequency reading data is needed for more precise correlation with pool or rainfall, or for displacement or other movement.

c. The precision afforded by different types of instruments may influence which instruments are selected for automation. Electrical and optical instruments normally afford more consistent precision than pneumatic, hydraulic, and mechanical instruments. The more precise instrument is usually chosen for automation because of concerns for data quality. However, increased reading frequencies can be used to enhance the accuracy of imprecise readings.

d. Automation should be considered for instruments that are designed to be monitored for extended periods of time at relatively frequent intervals. Long-term monitoring labor cost associated with manual acquisition system may be offset or reduced by automation and inclusion into an ADAS. Some data collection frequencies are unattainable with manual methods; thus, automation may be required.

e. The location of an instrument affects the feasibility of automation. All data connections should be feasible via a cable or radio signal. Nearby power sources may cause signal interference, precluding automation.

f. Instruments may need to be upgraded to be compatible with automation. For example, an open standpipe PZ may need to be retrofitted with an electrical transducer to cable the instrument to an ADAS.

g. Data evaluation requiring correlation of various readings and events often requires the ability to synchronize data collection. Automation facilitates the option to synchronize data collection. Ideally, the frequency of data collection in a system should be controllable, either automatically by programming or by remote access.

6.6. Timing the Purchase.

6.6.1. An ADAS has a high initial cost and a long-term maintenance cost. However, if funds are limited, an ADAS can be installed in stages over a period of years. An ADAS installation in stages should be based on prioritizing instruments, automating the most critical and least accessible instruments first. For new embankments, delaying the purchase of ADAS equipment until construction is underway may be prudent.

6.6.2. ADAS equipment is evolving rapidly, and components selected too early may be discontinued by the manufacturer at the time planned for installation. However, if baseline data collection before construction is necessary and an ADAS has been identified as the data collection and data transfer method, implementation of the ADAS should not be delayed.

6.7. Data Management. Automated systems can yield large quantities of data, requiring efficient data management. Excessive data volume can strain storage capacity, increasing the time required to perform calculations, and hampers analysis and reporting. Data management applicable to automation is discussed in Chapters 8 and 9.

Chapter 7
Instrument Installation

7.1. Introduction.

7.1.1. The installation of an instrument is not a routine task and requires attention to detail. Poor installation of instruments can lead to erroneous data or even adversely affect the embankment. Methods of installation depend on the parameter to be monitored, site conditions, and the type of instrument. Successful installation of an instrument requires the labor of qualified personnel.

7.1.2. This chapter presents general guidelines and recommendations rather than detailed procedures. Technical assistance for specific situations may be obtained from USACE instrumentation specialists, manufacturers, and suppliers. Dunnicliff (1993, Chapter 17) provides a good overview of installation procedures for geotechnical instruments, most of which have applications for embankments. The USBR Water Measurement Manual discusses requirements for various water flow measurement devices.

7.2. Personnel.

7.2.1. Project personnel should be involved in the installation of an instrument not only to ensure a good result, but also to obtain the greatest possible understanding of how the instrument functions. As discussed in Chapter 4, the district instrumentation specialist may provide oversight of the design and installation of an instrumentation and monitoring system.

7.2.2. The instrumentation specialist should be present during installation to determine if the instrument is being installed as designed and to be aware of circumstances that may affect the operation of the instrument. If possible, persons who will operate and maintain the instrument should be present during installation to obtain the greatest possible understanding of the instrument.

7.3. Contracting. USACE technical and contracting specialists should work together to determine the proper contracting method to select a qualified contractor. If a contractor must be responsible for the installation of any of the instrumentation system elements, the contract specifications should include the following items:

7.3.1. Purpose of each individual instrument.

7.3.2. Responsibilities of the contractor.

7.3.3. Instrumentation system performance criteria.

7.3.4. Qualifications of the contractor’s instrumentation personnel.

7.3.5. Quality control and assurance.

7.3.6. Submittals.

7.3.7. Scheduling of work.

7.3.8. Storage of instruments.

7.3.9. Materials (provide a detailed description of all types of instruments included in the contract, including spare parts commonly stocked).

7.3.10. Factory calibration requirements.

7.3.11. Pre-installation acceptance tests.

7.3.12. Verification of instrument function (including raw and reduced data collection retrieval and sample output submittal).

7.3.13. Installation instructions (providing a detailed step-by-step procedure to install each type of instrument).

7.3.14. Temperature and barometric pressure adjustment requirements.

7.3.15. Instructions for changed site conditions.

7.3.16. Field calibration and maintenance requirements.

7.3.17. Protection of installed and pre-existing instruments.

7.3.18. Terminal construction and connections.

7.3.19. Backfill and grouting requirements.

7.4. Instrumentation in Projects Under Construction.

7.4.1. The installation of instruments in projects under construction requires:

a. Planning the horizontal and vertical alignment of casings, tubes, and cables.

b. Coordinating with the contractor building the project.

c. Choosing cost-effective alternatives for various system components.

7.4.2. Installed casings, tubes, and cables should not significantly alter the mechanical properties of the embankment or provide seepage pathways. Moreover, casings, tubes, and cables should not be crushed or stretched by the surrounding material. Therefore, guidelines related to horizontal and vertical alignment of casings, tubes, and cables are as follows:

a. Horizontal runs of tubes and cables must not extend a significant distance across the core and should typically exit on the downstream face or toe of an embankment.

b. Vertical runs of casing, tubes, and cables through an embankment core are generally not acceptable.

c. Short non-inter-zone vertical or horizontal runs through the downstream shell to the crest or abutment of an embankment are typically acceptable.

d. Extending a casing vertically through the embankment core material during construction is generally not advisable, even if the casing may be installed in sections as embankment fill is placed.

7.4.3. Guidelines related to coordinating installation with the contractor responsible for building the project include:

a. Recognizing that the inclusion of instrumentation during construction almost inevitably causes delays and requires a general contractor to protect instruments.

b. Scheduling of cable installation must be coordinated with other activities to avoid damaging the cables with construction equipment.

c. Protecting the instruments and associated tubing and cabling, with locations prominently marked.

d. Avoiding poor compaction around the casing as fill is placed.

e. Pulling the casing while simultaneously backfilling the void below the lower end of the casing.

7.4.4. Regarding cost, in certain situations, installing a greater number of instruments requiring tubes or electrical cables without a casing may be more cost-effective than installing one instrument with a casing.

7.5. Instrumentation in Existing Embankments. USACE document ER 1110-2-1807: Procedures for Drilling in Earth Embankments provides the policy, requirements, and guidance for drilling in embankments and foundations. Some considerations related to the installation of an embedded instrument by drilling are discussed below:

7.5.1. Follow USACE guidelines in selecting the drilling method to be used.

7.5.2. The embankment should be protected from fracturing due to the pressure of air or water in the borehole during drilling.

7.5.3. The installation of the instrument should not erode, contaminate filters, or otherwise damage the embankment.

7.6. General Installation Procedures.

7.6.1. Although general procedures are described for installing an instrument, a project instrumentation engineer has the responsibility of specifying installation procedures suitable for each instrument. If significant embankment or foundation deformation is expected, installation of an instrument should prevent damage to elements such as pipes, tubes, and cables. Guidelines for installing an instrument in a borehole include the following:

a. Ensure compatibility between the diameter of the borehole and any minimum and maximum diameter requirements of the instrument.

b. Specify the required diameter, depth, alignment, drilling method, and sampling requirements.

c. Establish conventions for instrument naming, field labeling, and borehole logging.

7.6.2. General installation procedures are described under the following paragraph.

7.6.3. Scheduling. Preparing a schedule for the installation of an instrument requires:

a. Determining lead-time for obtaining instrument components.

b. Listing materials.

c. Listing tools and equipment.

d. Assessing site hazards to installers.

e. Estimating delays due to other construction or project operation.

f. Coordinating with other parties, such as utilities and transportation officials.

g. Providing for contracting, if necessary.

7.6.4. Signal Transmission Alternatives. Signals from instruments may be transmitted through cables or by radio. Guidelines for installing elements to transmit signals from an instrument include:

a. Providing extra cable length for:

(1) Unanticipated routing deviations.

(2) Future embankment deformation.

(3) Connections inside a junction box.

b. Burying electrical cable at an appropriate depth to reduce the risk of lightning or other damage.

c. Avoiding or minimizing the splicing of cable.

d. Considering the use of a radio link or cellular communication instead of a cable for cost reduction.

7.6.5. Pre-Installation Care. Instrument components should be protected from the elements prior to installation. Guidelines for protecting components before installation are as follows:

a. Handle components gently.

b. Follow the manufacturer's recommendations for protecting components.

c. Keep components free from dirt and dust.

d. If necessary, keep components and cables out of direct sunlight.

e. If necessary, shield components from electric or magnetic fields.

f. Protect components from freezing, extreme temperatures, high humidity, water, shock, and corrosive chemicals.

g. Protect cables and tubes from abrasion, nicking, kinking, and bending beyond specified limits.

7.6.6. Pre-Installation Acceptance Test. Instrument components and read-out units received at the project site may not be in suitable condition for installation for reasons such as damage in transit, incorrect calibration, or improper documentation. Therefore, pre-installation acceptance tests should be performed at the project site before an instrument is installed. A pre-installation acceptance test should include:

a. Checking components for damage.

b. Checking procurement documents to confirm that the model, dimensions, materials, product performance criteria, and quantities are correct.

c. Examining factory calibration documentation.

d. Examining the manufacturer's quality assurance inspection checklist.

e. Checking the cable length.

f. Checking tag numbers on the instrument and cable.

g. Bending the cable back and forth, within specified limits, at the point of connection to the instrument while reading the instrument to verify connection integrity.

h. Checking water and humidity barrier components to identify leaks.

i. Verifying that the instrument reading meets factory specifications and calibration expectations.

j. Performing electrical-resistance and insulation testing according to criteria provided by the instrument manufacturer.

k. Verifying that all components fit together in the correct configuration.

l. Documenting the results of the test, including the:

(1) Project name.

(2) Instrument model, identity, and serial numbers.

(3) Calibration and site zero offset factors.

(4) Identification of any testing or read-out equipment used during testing.

(5) Outline of field-testing procedures.

(6) Names of personnel responsible for field testing.

(7) Date and time of testing.

(8) Measurement and observations made during testing as listed above.

(9) Test results: pass or fail.

7.6.7. Installation Documentation. A record of installation for all new and modified instruments should include the following items, which are adapted from Dunnicliff (1993):

a. Project name.

b. Contract name and number.

c. Instrument type, model, location, and serial number, including the read-out unit.

d. Planned location, orientation, depth, length, and backfill volume.

e. Names of persons responsible for installation.

f. Equipment used, including the diameter and depth of drill casings or augers.

g. Method of trenching and backfilling.

h. Date and time of start and completion.

i. Measurements or readings required during installation to ensure that all previous steps have been followed correctly (follow the manufacturer's procedures and document any deviations made).

j. A log of subsurface data, including the elevations of stratum changes encountered in the boreholes and changes in backfill material. (Reference Appendix E for sample installation logs for PZs and inclinometers.)

k. A determination of whether the stratum at the transducer location agrees with that anticipated in the surveillance and monitoring plan.

l. Type of backfill used.

m. Recording initial instrument reading and any site specific zero offset or calibration change.

n. As-built sketch, including location orientation, depth, length, and backfill volume data, with instrumentation details labeled.

o. Plan and section schematics of utility runs to the instrument, locating all junction boxes (buried and with surface access).

p. Results of post-installation acceptance test.

q. Weather conditions at the time of installation.

r. Description of any problems encountered, delays, unusual features of the installation, and details of any events that may have a bearing on instrument function.

7.6.8. Post-Installation Acceptance. Instruments are tested after installation. The test should be performed after the end of any construction or other activity that may damage the instrument. Instruments should not be accepted unless a post-installation test has a favorable result. Guidelines for post-installation testing are discussed below:

a. Follow the details of the testing to be performed, which vary, depending on the type of instrument.

b. All quality checks or tests described in the literature of the manufacturers should be performed.

c. Enough time should elapse between installation of the instrument and the post-installation test to ensure that the parameter to be measured has stabilized.

d. At least three readings should be made to determine if the reading is repeatable.

e. Test details should be included with the installation report.

7.7. Installation Procedures for PZs in Boreholes.

7.7.1. Each PZ installed in a borehole is a unique instrument and should be planned and installed as such. Common installation configurations include:

a. Standpipe PZs sealed by placement of bentonite chips or pellets through a vertical interval of a few feet above the sand filter.

b. Standpipe PZs backfilled with a cement-bentonite grout from the sand filter to the ground surface.

c. Fully grouted diaphragm PZs requiring no filter sand or bentonitic seal. A specially prepared cement-bentonite grout from the bottom to the top of the borehole fully encapsulates the transducer.

7.7.2. Installation procedures for PZs in boreholes are described under the headings of:

(1) Installation planning issues.

(2) Selection and placement of sand filters.

(3) Selection and placement of seals.

(4) Multi-level PZs.

7.7.3. Installation Considerations. When planning for boreholes necessary for instrumentation installation, reference EM 1110-2-1804: Geotechnical Investigations and ER 1110-2-1807: Drilling in Earth Embankment Dams and Levees, when appropriate. Issues to be resolved before selecting a method for installing a PZ in a borehole in soil include, in no particular order:

a. What are the soil types, and are soil samples required?

b. Are borehole locations near an existing structure such as utilities?

c. What is the appropriate riser height to prevent inundation and allow access?

d. Is hydraulic fracturing a concern?

e. What drilling methods can be used without harming the embankment?

f. Will artesian flow occur?

g. If artesian flow occurs, will ice form near the ground surface?

h. Which method of installation is most appropriate: a standpipe, a transducer embedded within backfill, or a transducer fully grouted in place?

i. What borehole diameter will be used, and how can the borehole be supported?

j. How will the casing or augers be prepared, cleaned, backfilled, and pulled?

k. Is the planned PZ standpipe diameter adequate for the instrument sensor?

l. Are multiple instruments to be installed in one borehole?

m. Will cable or tubing centralizers be required in the boreholes?

n. How much vertical compression will occur in the soil above the instrument?

o. What transducer pressure ratings and cable lengths are required?

p. What response time is required?

q. Will the drilling method allow for placing bentonite-cement grout as the seal above the filter zone?

r. What seal lengths are required?

s. What type of bentonite will be used, and how much time is required for complete swelling?

t. Is a swelling retardant needed? If needed, what retardant will be used?

u. Is a sounding line (a weighted tape to check the elevation of backfill) required?

v. Would cable leading to read-out locations create an internal erosion potential?

w. Is there enough lead-time for procuring special PZs?

x. How much time is available between completion of installation and the need to establish baseline readings?

y. What are the skills and experience of the persons installing the instrument?

7.7.4. Selection and Placement of Sand Filters.

a. Guidelines for the gradation of filter material include:

(1) The proper gradation of filter material to place around an open standpipe PZ should be based on the screen size and the gradation of the receiving soil layer. (Reference EM 1110-2-1914: Design, Construction, and Maintenance of Relief Wells for a discussion of filter gradation.)

(2) Filter material for PZs, including diaphragm PZs, should be free of fines and should not delay the response of the instrument.

b. Guidelines for filter placement by pouring filter from the top of the borehole include:

(1) Filter material may be placed by pouring the material in the top of the boring for dry holes less than 25 feet (7.6 m) deep

(2) Filter material should be poured slowly to avoid bridging. Place sand no more than 2 feet thick in the borehole casing/auger, then raise the casing/auger 2 feet. Limiting the volume of sand will also help prevent the casing/auger from getting stuck.

(3) A sounding line should be used to verify placement depths.

c. Guidelines for filter placement by tremie include:

(1) Filter material should be placed by a tremie to avoid grain size separation for dry holes at least 25 feet (7.6 m) deep or water-filled holes.

(2) Water or compressed air should be used to flush the material through the tremie pipe.

(3) Care must be taken with flow rates as too high a flow rate can result in material grain size separation as the material exit.

(4) Place a tremie pipe at the elevation of the base of the sand filter and gradually raise the pipe to the top of the filter interval as the material accumulates in the hole.

(5) The tremie pipe may be used to measure depth.

7.7.5. Selection and Placement of Seals.

a. In recent installations, the vertical interval of the borehole between the filter sand and the ground surface has typically been backfilled with cement-bentonite grout rather than installing a seal by placing bentonite. Nevertheless, bentonite seals still are installed to isolate filter zones, ensuring that readings are collected from a single stratum only. Issues related to placement of bentonite include:

(1) The size of the bentonite particles placed.

(2) The tendency of bentonite to fill the borehole non-uniformly.

b. Guidelines for installing bentonite seals include:

(1) The pneumatic injection of bentonite gravel or chips can facilitate the installation of accurately located short seals and is appropriate for longer seals as well in dry boreholes.

(2) The bentonite pellet method may be used for shallow installations if fluids are not needed—during the winter, for example, when grout mixing can be difficult—or if local well codes require pellets.

(3) In water-filled boreholes, only dry manufactured pellets treated to slow the rate of hydration should be used because bentonite pellets bridge readily across the casing if poured too quickly through water.

(4) Generally pellets, because of less unit surface area and a more streamlined shape, are more likely to reach the target elevation before hydration becomes a problem.

(5) If a soil stratum must be sealed below a PZ, the bentonite should not be allowed to contaminate the sidewalls within the sand filter interval; a casing may be used to protect the sidewall.

7.7.6. Multi-Level PZs.

a. More than one PZ may be installed in a single borehole. Such PZs are installed at different elevations to monitor separate strata. The zones in the borehole are separated by a seal of bentonite, a cement-bentonite grout, or fully grouted. Constructing reliable seals between multiple standpipes may be problematic.

b. For layered stratigraphy with limited confining layer thickness, pilot holes or adaptive design for the PZ layout during drilling are required with carefully logged holes to determine adequate layer thickness where seals can be successfully placed (and kept in place considering potential settlement of hole backfill). Spacing between standpipes and spacing between the standpipes and borehole walls of 1 to 2 inches (2.5 to 5.1 cm) needs to be maintained to prevent short-circuiting the seals.

c. Spiders and spacers to keep the standpipes properly positioned in the hole are normally required. To meet the spacing requirements through seals, no more than two or three standpipes are allowable in boreholes up to 8 inches (20.3 cm) in diameter. If more intervals are needed, then pressure transducers should be installed instead of standpipes. Guidelines related to installation of multi-level PZs include:

(1) The manufacturer's recommendations.

(2) Any seal must prevent vertical flow of water in the borehole.

(3) Achieving satisfactory seals between more than two standpipe PZs in a single borehole of 6-inch (15.2 cm) diameter or less is problematic.

(4) If the situation permits, fully grouted PZs may be easier to install successfully than PZs separated by bentonite seals because each PZ may be attached at a point along a PVC tremie pipe which remains in the borehole; the borehole is then fully grouted with a suitable cement-bentonite grout.

(5) In a fully grouted borehole, a properly installed cement-bentonite grout backfill adequately isolates the PZ from zones above and below; the isolation is possible because the radial pressure gradient in the grout is normally one or more orders of magnitude greater than the vertical pressure gradient.

(6) In a fully grouted borehole, the orientation of the vibrating-wire transducer should be facing upward to help maintain saturation of the filter tip.

(7) A cement-bentonite grout with a permeability of 10–6 cm/sec should form a satisfactory seal between monitoring zones within a fully grouted borehole while still allowing low-flow transducers, such as vibrating-wire PZs, to adequately measure in situ water pressure changes. Cement-bentonite should not be viewed as a tight swelling seal.

7.8. Installation Procedures for Instruments Other than PZs.

7.8.1. Much of the same guidance in Section 7.7 applies to developing installation procedures for instruments other than PZs. Manufacturers of inclinometers and borehole extensometers supply specific instructions for the installation of the downhole components. However, site conditions and soil characteristics may dictate the installation procedure selected.

7.8.2. In addition to knowing the location of placement within the borehole or trench, using appropriate grout mixtures and procedures along with measures to control instrument buoyancy and any artesian conditions are key to an effective installation. Multiple methods are available for overcoming buoyancy until grout sets and for ensuring that the casing remains as straight as possible in the borehole.

7.8.3. An adequate bottom weight can be attached to the bottom cap. Second, the weight of the grout pipe can be used, temporarily weighting the bottom of the inclinometer casing while the grout pipe is raised to level with water. Another method is preferable for deep holes, for which the magnitude of the bottom weight would become excessive. The first method must be used if the inclinometer casing has telescoping couplings.

7.8.4. Alternatively, water may be used to counter buoyancy by filling the casing with water above the elevation of the groundwater. Deep PZs may require multiple placements of grout. The first grout placement should extend to a depth sufficient to anchor the casing. The second placement can be performed without buoyancy issues. Additional information may be found in Dunnicliff (1993).

7.9. Backfilling Boreholes. Grout is often used to backfill instrument boreholes. Cement-bentonite grout is not the only mixture that has been used as a backfill material for boreholes. Rather, manufactured high solids grout, bentonite chips, native soils, and various granular fills have been used successfully as backfill material for boreholes.

7.9.1. Preferred Grout Mixture Components. Cement-bentonite grout is the preferred backfill material for boreholes, including standpipe PZs and fully grouted PZs. Both technical guidance and regulations should be considered in such grouting. Grouting guidance is available in EM 1110-2-3506: Grouting Technology, and in Driscoll (1986). State and local authorities also enforce regulations for backfilling boreholes.

7.9.2. Grout Mechanical Properties.

a. Important mechanical properties of the grout used to backfill boreholes include the ability to fill voids in the borehole and the strength and permeability of the grout after curing. The mechanical properties depend on the grout mixture. Grout may be composed of water, cement, bentonite, and sand as a filler. However, sand is rarely included in a mixture used to install instruments in an embankment due to possible segregation and wear on pumping equipment.

b. The required characteristics of grout depend on the situation. For example, grout-in-place vibrating-wire instrument installations typically require a grout mixture based on the consistency and hydraulic conductivity of the native soil.

c. Grout should be fluid enough to fill voids in the borehole but not so fluid as to plug a screen or soil pores. Assuming that sand is not included in the mixture, grout mixtures should be tested to determine the ratios of water, cement, and bentonite that will ensure intimate contact between the instrument and the surrounding soil or rock.

d. A grout mixture should be fluid enough to envelop a cable or tube located in the grouted zone of the borehole. Cables or tubes in parallel should be held apart by separators, and the grout should be fluid enough to envelope the separated cables or tubes. A grout mixture should not be used if the mixture can bleed into the surrounding soil or clog a PZ screen. Pure bentonite grout is not recommended due to an excessive change in volume as moisture content changes.

e. The strength of grout after curing is a function of the mixture. In some situations, the ratio of grout strength to soil strength may determine the success of the instrument installation. Therefore, multiple grout mixtures may need to be tested for strength to identify a mixture with the strength needed.

f. Although a mixture of only cement and water develops strength, the addition of bentonite reduces the tendency of the cement to settle out of suspension, resulting in more uniform grout properties throughout the borehole. The strength of the grout mix for inclinometer installations should closely match the strength of in situ materials so that the grout deforms at the same rate as the structure and foundation.

g. Grout set times will vary depending on the percentage of grout in the design mix. The set time should be considered when planning the initial, baseline survey of the inclinometer. It is important to allow sufficient time for grout curing to establish a baseline that is not influenced by partially cured grout. Multiple surveys should be performed until the data evaluator is confident that no movement indications can be attributed to grout set-up.

h. Grout should be dense enough to counteract artesian conditions. Some grout mixtures may require additives having a high specific gravity, such as barite, but the resultant pressure should not be allowed to cause hydraulic fracturing in the embankment.

i. The permeability of the grout after set-up may determine the success of the grouting. Permeability is determined by the water-cement ratio of the grout; a higher water-cement ratio has a greater permeability. Various mixtures of water and cement should be tested to determine a mixture with the desired permeability. The addition of a small amount of bentonite reduces the tendency of the cement to settle out of suspension and has little effect on permeability.

7.9.3. Example Mixtures for Fully Grouted Vibrating-Wire Installations.

a. References for cement-bentonite grout design include Contreras, Grosser, and VerStrate (2007), Mckenna (1995), and Mikkelsen (2002 and 2003). Two representative cement-bentonite grout mixtures for fully grouted vibrating-wire installations noted in Mikkelsen (2003) include a mixture for medium to hard soils and a mixture for soft soils.

b. A typical mixture ratio for hard soils is:

(1) 30 gallons (113.6 L) of potable water,

(2) One 94-pound (42.6 kg) bag of Portland cement, and

(3) 25 pounds (11.3 kg) of bentonite.

c. A typical mixture ratio for soft soils is:

(1) 75 gallons (283.9 L) of potable water,

(2) One 94-pound (42.6 kg) bag of Portland cement, and

(3) 30 pounds (13.6 kg) of bentonite.

d. The cement and water should be mixed first and the bentonite then added to obtain a fluid, creamy consistency. The grout mixture should be tremied to the bottom of the borehole. Sample mixtures, such as the mixtures listed in this section, should be tested before use.

7.10. Protective Housings.

7.10.1. Each instrument read-out terminus should be contained within a secure, enclosed protective housing. If a building cannot be used to house collected terminals, then each individual terminal should have a smaller lockable and vented protective housing. Small protective housings should be grouted or cemented into place for stability.

7.10.2. Some instrument installations feature a casing that extends a short distance above the ground surface, referred to as a stick-up casing. A protective housing mounted atop a stick-up casing is easy to find and access, even if snow is on the ground. Also, the elevated housing is protected from runoff, ponded water, and mud. If brightly painted, the stick-up casing is easy for a driver to see and avoid, but is still vulnerable to damage from a vehicle if a protective post is not installed.

7.10.3. For some instruments, the protective housing is a gate box mounted flush with the ground surface, similar to the boxes used by utility companies. A flush mounted box is protected from vehicle damage but may be difficult to find in snow or may be covered by ponded water or mud. Proper drainage should be ensured so that surface runoff does not infiltrate or otherwise influence the instrument.

7.10.4. Protective caps or plugs should be tethered or hinged to the housing to prevent loss.

7.10.5. The design of the protective housing may need to provide for the immediate or eventual routing of buried cables to automation equipment.

7.10.6. A protective housing should protect an instrument from weather, vehicles, and vandals. Damage from weather may be minimized by:

a. Grouting or cementing the housing into place to prevent the intrusion of surface water.

b. Preventing frost heave using a locally proven design.

c. Extending the concrete or grout seal below the frost depth and using frost sleeves or other means to maintain an annulus between the standpipe casing and the protective housing.

d. Separating a surrounding concrete pad from the grouted housing with drained or waterproof material to prevent the pad from heaving the housing in freezing weather.

7.10.7. Damage by vehicles may be minimized by:

a. Locating protective housings a safe distance from roads.

b. Installing protective posts or barricades near the protective housing.

c. Making the protective housing highly visible.

d. Flush-mounting instruments in roadways.

7.10.8. Damage by vandals may be minimized by:

a. Making the housing lockable.

b. Making the housing difficult to move or break.

7.11. Protection from Transient High Voltage.

7.11.1. Transient high voltages are unintentional, short duration, and high magnitude spikes in voltage and current that can cause damage to equipment as well as injury or death. Sources of high voltage include:

a. Direct lightning strikes.

b. Induced electric fields caused by lightning.

c. Switchyards and other high voltage equipment.

7.11.2. Lightning is not only damaging but frequent. In the contiguous United States, lightning strikes per square mile (2.59 sq km) per year vary from approximately 100 in the southeast to approximately 5 to 10 in the northwest. The International Electrical Commission publishes relevant recommendations for lightning protection in the Series 61312 and 62305 standards.

7.11.3. Methods of protecting life and electrical equipment from lightning include:

a. Surge protectors and associated ground cable/rods.

b. Air terminals and associated ground cable/rods.

c. Locating electrical instrumentation at a sufficient distance from potential lightning attractors and electrical conduits.

d. Limiting the horizontal area within which instrument components are connected by signal cables.

7.11.4. To maximize safety from electrical system faults and lighting, grounding systems should have a low impedance path for fault and lighting-induced currents to enter the earth. Dry fill in an embankment is not an ideal soil type for a low impedance electrical grounding system. While it is not impossible to install an acceptable grounding system, it is more difficult.

7.11.5. Surge protectors are devices that are designed to protect electrical equipment from transient voltages. Surge protectors must be properly grounded and located within a close range to the electrical equipment to be effective. Moreover, even with a good electrical ground, a surge protector should be tested or otherwise certified.

7.11.6. Electrical instrumentation should be located a sufficient distance from potential lightning attractors and unintended electrical conduits. For example, lightning striking the embankment surface is mostly dissipated in the horizontal direction, so installing electrical instrumentation a horizontal distance of at least 30 feet (9.1 m) from potential lightning attractors affords some protection. A comparable horizontal distance should be maintained from potential electrical conduits, such as metal railings and pipe buried near the ground surface.

7.11.7. Limiting the horizontal area within which instrument components are connected by signal cables limits the probability of damage to the group of components due to a direct or nearby lighting strike.

7.11.8. Providing grounded transient voltage protection for individual instruments and the ADAS, limiting the length of signal cable connections between components, and transferring data by radio over greater distances all afford protection from lightning. A good system is likely to have all four protective features.

7.11.9. The methods of protecting instrumentation from switchyards and other high voltage equipment are similar to the methods used for protection from lighting.

7.12. Final Documentation.

7.12.1. After completion of the instrument installation, a report should be prepared. The report should contain the information listed in Section 7.6.5. In addition, the report should contain the following items, which are adapted from Dunnicliff (1993):

a. Purpose and description of instruments.

b. Read-out unit and automation equipment data sheets.

c. Plan and section drawings showing all instrument locations, trench backfill details, and routing of cables.

d. Narrative of instrument installation lessons.

e. Manufacturer's instrumentation and automation documentation, including calibration data and warranty information.

f. Pre-installation acceptance test results.

g. A list of spare parts needed for repair or maintenance.

h. Procedures for data collection and processing.

i. Names and contact information of maintenance and repair points of contact.

7.12.2. A printed copy of the documentation should be provided to the:

a. Onsite project office.

b. District dam or levee safety office.

LEFT BLANK INTENTIONALLY

Chapter 8
Data Management

8.1. Introduction.

8.1.1. Data management is a vital component of a surveillance and monitoring plan. Basic data management principles, including backups, data access, and data integrity need to be considered for each surveillance and monitoring plan. Data management also involves setting appropriate collection frequencies and reviewing schedules such that geologists and engineers have timely and accurate data with which to perform analyses. Failure to analyze data in a timely fashion can compromise the effectiveness of monitoring.

8.1.2. This chapter discusses the variety of considerations required to ensure data are collected and preserved in a timely fashion while maintaining complete records. Discussions of alarms and thresholds, including recommended values, are presented. In addition, the challenges and benefits associated with large data sets and considerations for management of data during construction projects are presented.

8.2. Fundamental Data Management Principles.

8.2.1. Data that cannot be easily retrieved, readily verified, and associated with a location within the project do little to help a geologist or engineer gain an understanding of a site. This applies to both raw and reduced data, and both forms should be stored in a database. Poor data management can be a dam/levee safety issue. Even for low-risk situations, the period between data gathering and evaluation should be brief to help facilitate timely risk management decisions.

8.2.2. Reports generated from the management system should include readings, associated
field conditions, and expected ranges. Readings may be sorted by either date/instrument or instrument/date. In addition, nearby instrument data plots, installation logs, reading schedules, and reduction setup (reduction constants and equations) should be readily available.

8.2.3. Correlation data plots can also help evaluate relationships between an instrument and field conditions such as waterside and/or landside (pool and/or tailwater). Contour plots aid understanding of relationship of multiple instruments along a cross-section. Time history plots are efficient for evaluating multiple instruments over various time periods.

8.2.4. Two extremes of data management are a:

a. Small manual data entry and processing operation or

b. Large automated electronic database.

8.2.5. At one extreme, little labor is required to handle, process, and store data for an embankment equipped with only a few, infrequently read instruments. In such a case, the manual entry of data is not arduous, and plots can be generated quickly.

8.2.6. All instrumentation data should be stored electronically. Paper works well as a backup, but as a long-term storage medium is prone to being lost or damaged. At the other extreme, the volume of data produced by numerous automated instruments can make the use of a spreadsheet impractical. In those situations, a more robust database should be utilized.

8.2.7. Despite the adoption of electronic databases, some data management programs still rely partly on paper records stored in file rooms or off-site. Printing and storage of paper records can be expensive and time-consuming and retrieval from off-site facilities may involve significant time, resulting in unacceptable delays when a sudden need for data evaluation occurs.

8.2.8. Regardless of project size, amount of paper, or digital storage volume, all database programs benefit from good data management practices. Data sets stored in a spreadsheet program may at first be relatively small and manageable but become useless over time if not consistently organized, regularly backed up, properly identified, and stored. In all systems, performance data should be reviewed for accuracy, stored for ease of access, and backed up regularly.

8.2.9. The goal of data management is to ensure that information is easily accessible and available to current and future engineers for analysis, interpretation, and evaluation. The data management system should be implemented to allow those responsible for monitoring to verify that data transmission is timely, complete, and correct. To facilitate the achievement of those goals, a database should be organized by project features, such as particular embankments. The system should help project staff:

a. Identify readings that are abnormal or exceed thresholds.

b. Identify missing data.

c. Note evidence for trends requiring additional investigation by field personnel.

d. Understand how reservoir operation, river fluctuation, and other loadings may affect instrument readings.

e. Log comments for records or data displays (annotated data fields should clearly indicate to future reviewers why unexpected changes occurred).

f. Determine the need to adjust instrument calculation constants or formulas.

g. Determine if instrument behavior conforms with expected structural and geotechnical behavior.

h. Communicate findings with other personnel or management, including field personnel, the Dam Safety Officer (DSO), and Levee Safety Officer (LSO).

8.2.10. Fundamental data management principles are further discussed in subsequent paragraphs.

8.2.11. Data Preservation. Performance data should be treated as a project asset and available for review throughout the life of the project. All instrument records, boring logs, installation logs, and database entries should be stored in systems ensuring preservation and ready access. For example, if an inclinometer indicates a developing shear plane, the original boring and installation logs for the inclinometer are crucial information to assess the structural stability.

a. Redundancy.

(1) Data storage should be redundant. Backups are crucial to ensure lost data can be replaced. Data should never be stored on only one hard drive, only on paper, or only on a server that is rarely backed up. Nor should data be stored onsite on a hard drive or server if no backup is stored offsite.

(2) Computer records that do not have off-site backups share the same problems as paper records—only one flood, one fire, or in some cases merely the activation of a sprinkler system can destroy years of important data. Therefore, redundant and frequent backups are essential. Backup should also be tested at least annually to ensure reliability of the backup.

b. Future Compatibility.

(1) Electronic data should be stored in, or migrated to, formats expected to be compatible with future computer software and hardware. To the extent possible, data should be kept in commonly used, non-proprietary formats. For example, data stored as ASCII text 25 years ago is still readable by computer systems today.

(2) As a second example, Structured Query Language (SQL), originally developed in the 1970s, has proved an enduring format. SQL databases have been a standard format since the late 1980s, when SQL was adopted by the American National Standards Institute in 1986 and by ISO in 1987. Using formats such as these that facilitate continued compatibility promote long-term access of data.

c. Small Projects.

(1) For small projects, for which no electronic database has been implemented, a minimum standard of recordkeeping should include scanning paper records into a portable document format (PDF), which is then organized and backed up regularly.

(2) Although a small information system can be maintained solely though paper records and PDF backups, the benefits of a database, even for small instrumentation and monitoring systems, quickly outweigh the costs of implementation. Records consistently entered into a small database can later be migrated into a more robust system if needed.

(3) A spreadsheet program can serve as the database, provided that users carefully protect and back up data. If spreadsheets are used, a standard requiring the column name to include units (or always including units as part of the chart) helps keep the data meaningful. Changing the format a Microsoft Excel spreadsheet that is normally uploaded to a database can cause incorrect data to be uploaded, and the errors may not be immediately apparent.

d. Historic Paper Records.

(1) Paper records, such as data gathering sheets and data evaluation, should be preserved in PDF format. The PDF files should be stored on a regularly backed-up server accessible by project engineers and geologists.

(2) The original data record is vital to checking for data transcription, transmission, or corruption errors. Such errors frequently occur if those responsible for data collection and entry are not the same as those responsible for database maintenance, and this kind of problem is much easier to diagnose and troubleshoot if the original records are preserved.

8.2.12. Accessibility.

a. Data should be easily accessible to all charged with ensuring safe project operation. Insufficient or delayed access can lead to inefficiency and possibly to poor decision making. Access to data can be hampered by the:

(1) Data storage medium.

(2) Software restrictions associated with proprietary file formats, license purchasing or availability, and version compatibility user permissions.

(3) User skill in operating the database.

b. The data storage medium can physically limit accessibility. Paper files are an example of a data storage medium that can only be read in one place at one time. Paper files stored in an offsite records-holding facility are not always readily accessible, which can be a critical problem during an emergency. Similarly, paper records stored at a project office may not be accessible to engineers and geologists working at other locations.

c. An insufficient number of users with permission to access the database needlessly limits access to data. For example, a database should not be set up to allow access to only one qualified person in an office. Redundancy in staffing expertise are crucial for emergency situations. All engineers and geologists assigned to a project should know how to obtain the data stored in the database.

8.2.13. Data Handling. Good practices in handling data are as follows:

a. Data should be handled as few times as possible.

b. The number of people with permission to alter data should be appropriate.

c. Software should be set to protect data to prevent accidental edits.

d. Electronic files should be entered into the database directly using standard file formats and scripts.

8.2.14. Data Security.

a. Typically, the number of people who need to be able to read stored data is greater than the number who are qualified to change the content of a database. Therefore, controlling who has permission to write data to a database is particularly important for data security.

b. Changes to formulas and constants should be documented and carefully applied to only the portion of the data set that it is applicable. Errors in formulas and constants can be difficult to detect and diagnose, so restriction of those permissions is advised.

8.2.15. Data Integrity. Data integrity is the degree to which data are maintained accurately and consistently over time.

a. Responsibility. Responsibility for tasks and products should be assigned to specific people, whether data collection and management is performed manually or electronically. Also, the person who performs a task should be identified. For example, the name of an instrument reader and the name of the person entering or editing the database data should be recorded. This can aid troubleshooting of unusual data.

b. Unusual Data.

(1) Unusual instrument data may be accurate or erroneous. The way people react to unusual data affects data integrity and the responsiveness to a developing problem. Unusual readings should be investigated, explained, and classified as accurate, uncertain, or erroneous.

(2) An unusual reading may indicate a problem with an instrument, data collection system, data collection processes, or may reflect a safety problem. Unusual data can be masked or tagged but should be preserved. Contextual information about the doubtful data should appear in both tabular and graphical formats. For example, data plots should indicate the existence and nature of any masked data, to inform the engineer or geologist analyzing the data.

(3) Data tagged or masked as potentially erroneous should include the name of the engineer, geologist, or technician identifying the potential error. The date and time the potential error was noted should also be recorded within the database.

(4) At times, instrument measurements are accurate but could be misleading unless the conditions affecting the reading are recorded and displayed for the person interpreting the data. Therefore, known causes of erroneous and biased data should be noted in the database, on manual data gathering sheets, and on generated plots.

(5) Instrument maintenance and tests can yield data that can be misinterpreted by someone not aware that an activity was underway at the time of the reading. For example, a falling head test or an instance of inflow of surface runoff into an open standpipe PZ should be recorded.

(6) Although the readings taken during such periods may accurately reflect the water level in the PZ, the readings do not accurately represent the water level in the strata being measured. Rather, the readings reflect recovery from the artificially high levels caused by the falling head test or surface water inflow. These data are valuable and should be kept, but the cause of the unusual reading should be documented.

(7) Erroneous data include readings that are physically impossible or data points that could not be recorded by an instrument. Additional readings should be performed to validate the data when the source of error is unknown. However, most erroneous data should be retained and flagged to facilitate a decision to use the data.

(8) Erroneous data may have merit in identifying problems with equipment, personnel, or other aspects of the monitoring systems. Re-imported data should not be written over incorrect data. Only data that are clearly erroneous should be deleted from the system.

c. Metadata.

(1) Metadata are data about data—providing information on why, where, when, and how data was collected and thereby helping maintain data integrity. Benefits of metadata include confirming data errors and tracking data each time the data are transferred to a new storage medium, file format, or database software.

(2) The metadata for an instrument are the location data, the instrument used to collect the data, the date and time the data was recorded, the engineering units of the measurement, and the name of the person who collected and/or processed the data. The metadata requirements for an instrument reading include the information needed to track responsibility for collection and logging of data.

8.3. Performance Monitoring Procedures.

8.3.1. Data management procedures should be designed to answer questions raised by instrument and observation data and used to assess the performance of the project and support required analyses. An effective database facilitates a variety of monitoring techniques. When developing procedures, consider what data should be collected and the purpose, location, method, and frequency of data collection. Items that should be identified or determined to plan for monitoring include the:

a. Variables controlling instrument behavior.

b. Frequency of change in values of the controlling variables.

c. Frequency of reading providing balance between cost and information.

d. Correct behavior under normal and distressed conditions.

e. Alarm threshold values.

f. Appropriate persons to be quickly notified of an alarm.

g. Previous instrumentation readings to understand thresholds and trends.

8.3.2. Measurements made less frequently than the frequency of change of the input variable can be misleading, although accurate, due to the coincidences of measuring different points in the cycle of the variable. Thus, database products should clearly indicate collection frequencies.

8.3.3. In addition, individuals taking readings should have previous readings available in the field while taking manual readings (including supporting data collected at the same time, such as the river level or reservoir level). That way, additional readings can be immediately taken if unusual readings are obtained to ensure accuracy and limit uncertainty.

8.3.4. The means of notifying appropriate persons of an unusual instrument reading depends on the method of collecting and reporting data. Automated data monitoring systems can notify persons by email, text, or telephone alerts. For manually collected data, readers should be provided with data sheets listing several previous readings and describing how to recognize unexpected and important changes in instrument readings.

8.3.5. Instrument readers should have well documented notification instructions for reporting unusual data quickly and effectively in the event an unusual instrument reading is obtained. Specific actions that need to be taken when unusual readings are encountered should be documented in an emergency action plan.

8.3.6. Once notified of an unusual instrument reading, the engineer or geologist needs to have previous readings available for comparison. Correlation plots and records of baseline instrument response can help engineers, geologists, and field personnel understand the behavior of an instrument and interpret an unusual reading.

8.3.7. A plan to monitor during construction can be quite different from routine monitoring under normal conditions. For example, grouting and the installation of a seepage cutoff wall through an embankment are invasive and may increase the chance that a sudden change occurs in instrument readings. Therefore, the frequency of reading and reviewing of key instruments may need to be increased during construction.

8.4. Data Collection Frequency. Data collection frequency is discussed in subsequent paragraphs.

8.4.1. Effect of Data Collection Frequency on Resolution.

a. A transition from reading an instrument monthly to a higher frequency may reveal that the baseline reactions of the instrument, as understood before the transition, were misleading due to infrequent sampling.

b. An example of how monitoring frequency for an embankment dam can affect analysis is shown in Figures 8.1 through 8.3. Similar correlations can be assumed for levee embankments. The time history plots shown in these three figures are all for the same PZ (C-68R) installed in limestone during a grouting program.

c. Figure 8.1, Figure 8.2, and Figure 8.3 display PZ readings in relation to headwater and tailwater elevations on monthly, daily, and 3-minute display intervals, respectively. Tailwater elevation is plotted at the same scale, but on an offset secondary axis to allow fitting headwater, tailwater, and the instrument data into a compact graph. These three figures display significantly different instrument reactions.

d. The data displayed at a monthly interval does not show clearly that piezometric level is frequently affected by something other than headwater or tailwater elevation and indicates a maximum piezometric level of 617 feet (188.1 m).

e. The data displayed at a daily interval show that, along with a general correlation of piezometric level with high headwater and tailwater levels, frequent spikes in piezometric level occur which must be due to another cause. Moreover, the maximum piezometric level is 624 feet (190.2 m), which is 7 feet (2.1 m) greater than that displayed in the monthly data.

f. The data displayed at a 3-minute interval reveal yet more volatility in instrument reaction than the data displayed at a daily interval. In fact, spikes in piezometric levels caused by grouting operations exceed headwater elevations on multiple occasions.

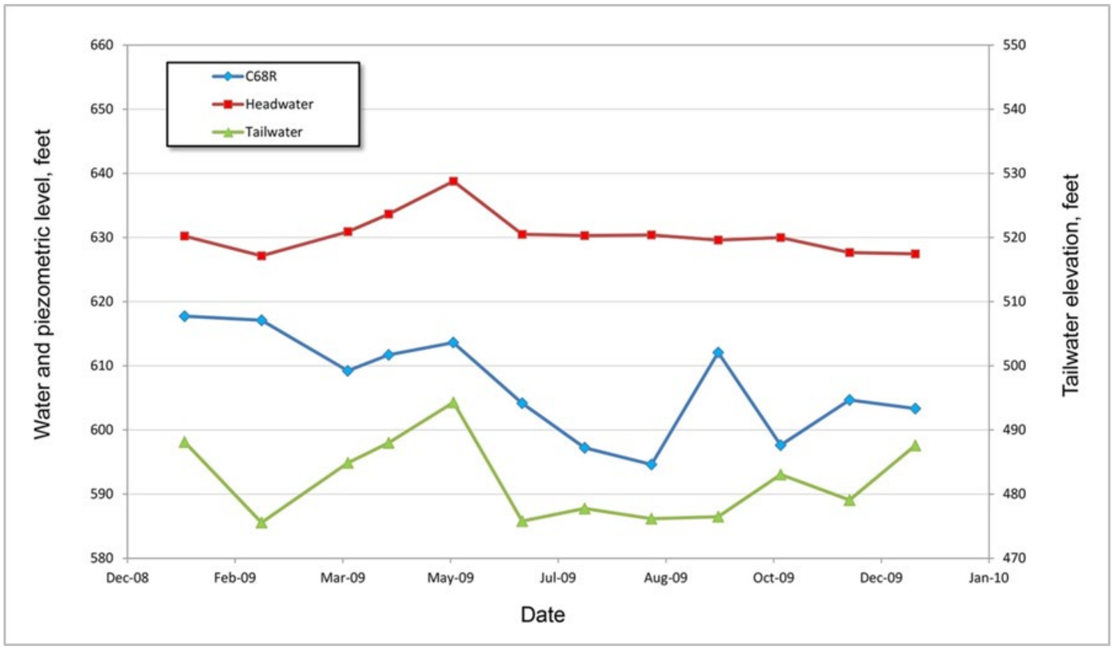


Figure 8.1. Monthly Monitoring Readings for PZ, Headwater, and Tailwater

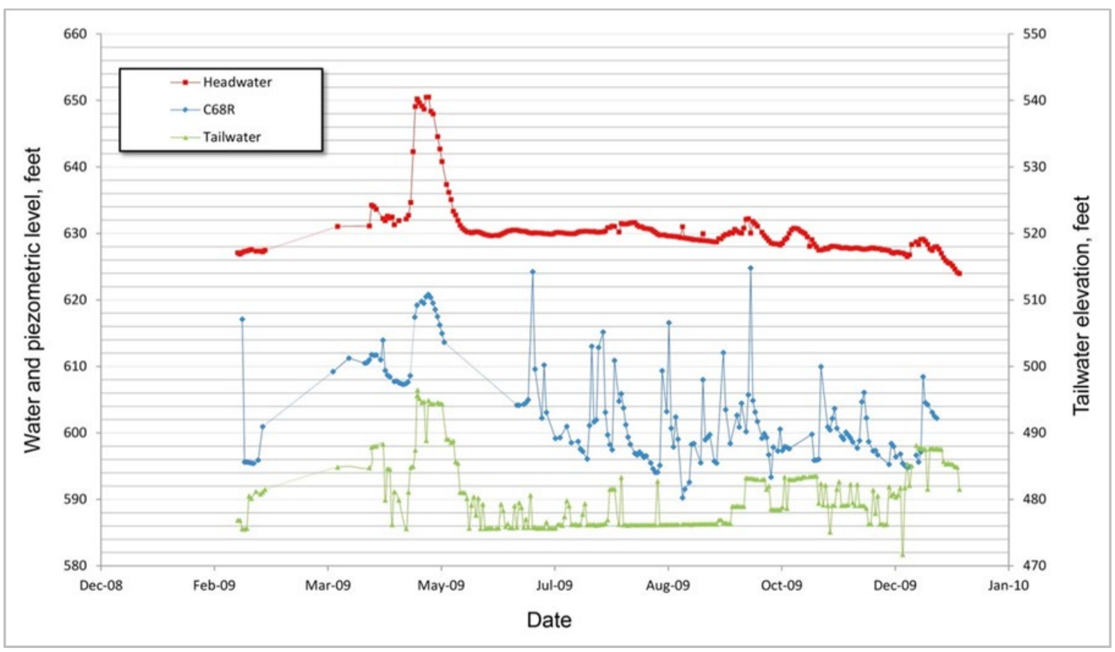


Figure 8.2. Daily Monitoring Readings for PZ, Headwater, and Tailwater

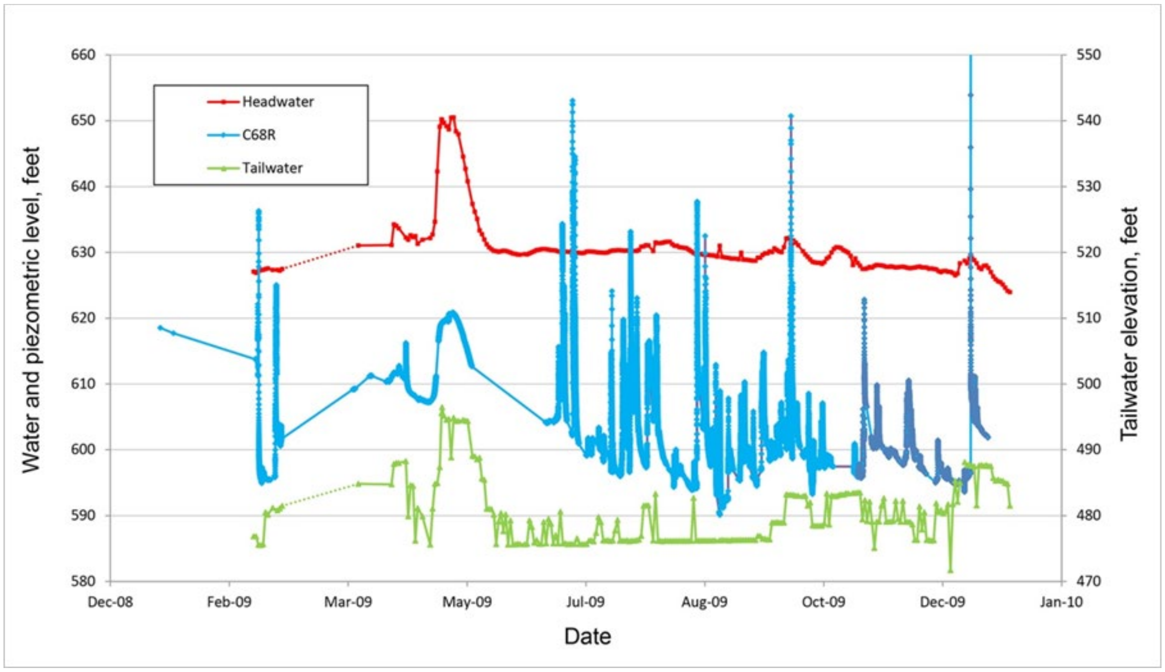


Figure 8.3. Three-Minute Monitoring Readings for PZ, Headwater, and Tailwater

g. Figure 8.4 compares the PZ, headwater, and tailwater readings from Figures 8.1 and 8.2 and illustrates how data resolution may be increased with more frequent readings if an instrument experiences highly variable conditions.

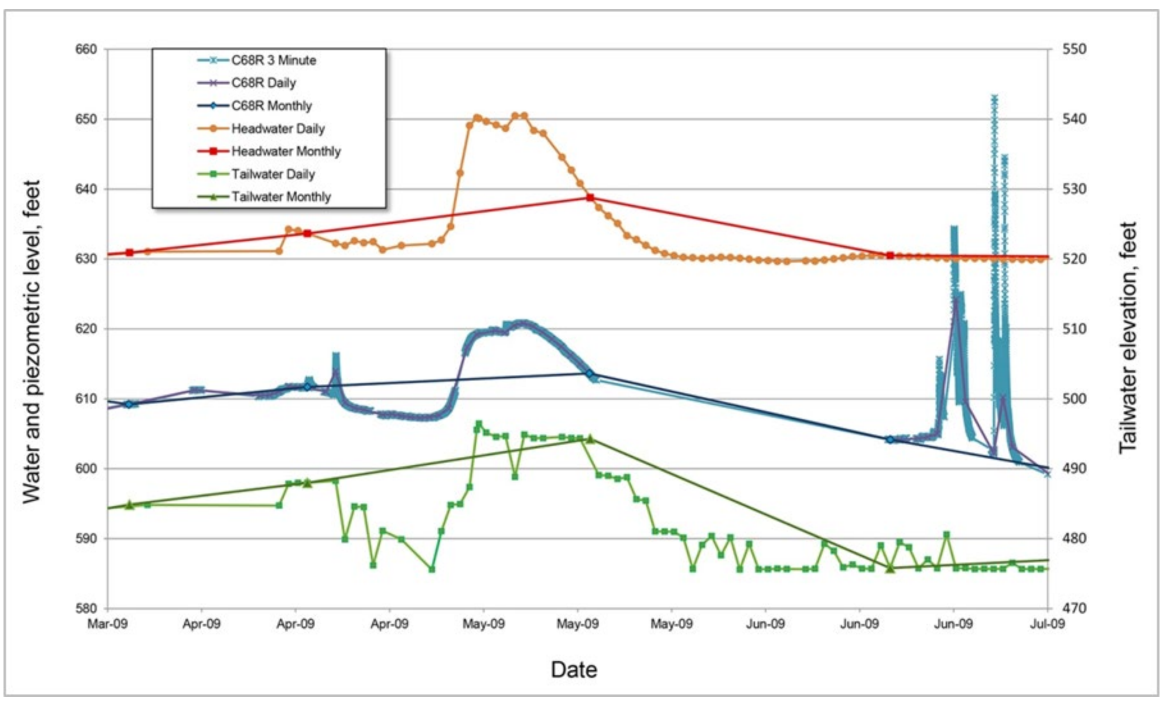


Figure 8.4. Three-Minute, Daily, and Monthly Plots Comparison

h. Infrequent readings can be valuable for ensuring that an instrument is functional, if the risk is low, and for determining long-term trends. A monthly or less frequent reading frequency may also be appropriate for an instrument having a low variability in readings. For example, settlement surveys on an embankment that is not experiencing slope stability problems and has a history of only minor settlement may be performed only as needed to provide up-to-date information for periodic inspections.

i. A monthly reading frequency for the PZ of Figure 8.4 does not adequately monitor the quickly changing reactions caused by pressure grouting in rock and is inappropriate if an understanding of grouting effects on the PZ is needed. A monthly reading frequency is also generally inadequate if statistical analyses are performed on PZ readings, particularly if:

(1) Headwater and tailwater elevations are measured at different schedules than the instrument.

(2) Headwater or tailwater fluctuates significantly during any given day.

j. Monthly plots yield long-term trends, but correlations may be more difficult to discern. For example, tailwater at a dam may change frequently and rapidly due to power generation or spillway operations. The PZ reading should be taken at the same time as tailwater to assess the correlation.

k. Daily readings provide higher data resolution for a rapidly changing instrument. However, a daily interval is typically the highest practical frequency for instruments read manually. Daily readings should typically be performed during a modification project if instruments are read manually, for limited periods of time during high-water events, or for noted unusual instrument readings requiring further investigation.

l. ADASs make possible a much higher instrument reading frequency than is possible from manual reading. Although limitations exist, mostly with respect to telemetry data transmission, reading intervals as short as 15 minutes are often practical. For construction projects expected to produce highly variable instrument readings, such as pressure grouting a karstic foundation, a 15-minute reading interval facilitates good correlation of readings with construction activities.

m. ADAS readings can also be filtered to eliminate duplicate readings or to collect readings at a short time interval but report at a longer time interval. For example, readings may be taken at 3-minute intervals but reported hourly, unless an override is executed due to unusual variability in the readings.

n. PZ readings may change very slowly for a clay embankment. Therefore, monthly readings may adequately reflect variability of such an embankment. However, more frequent readings may still be necessary to adequately monitor for the onset of potential failure modes involving cracking or monitor for construction impacts.

8.4.2. Data Transmission and Storage Limitations. The practical limitations of data transmission rates and data storage should be considered to determine an instrument reading interval. The amount of storage required for instrument data depends on the number of instruments and the frequency of readings.

8.4.3. Recommended Reading Frequency.

a. The reading frequency adopted for any particular instrument or instrument type should be based on the:

(1) Requirements for validating design assumptions.

(2) Support for interim risk reduction measures.

(3) Need to reduce risk by increasing warning time.

(4) Variability expected for that instrument and the variability of the parameters that control the instrument response.

(5) Accuracy of the instrument.

(6) Limitations of personnel reading the instruments.

(7) Data transmission and storage limitations onsite.

(8) Potential failures or distress features the instrument is designed to monitor and how quickly such features may develop.

b. Table 8.1 lists general recommended minimum frequencies for monitoring and for data processing and analysis based on the instrument type, method of data collection, and monitoring condition. Project-specific frequencies for data recording and evaluation should be outlined in a project's emergency action plan. Three monitoring conditions are:

(1) Baseline monitoring is recommended for a project operating under normal conditions with no instrumentation interim risk reduction measures plans (IRRMPs) in effect. Baseline monitoring is not adequate during the occurrence of significant unusual events, expected rapid water-level movements, or while construction or modification projects are underway.

(2) Enhanced monitoring is performed during construction, modification, or when rapid changes in water level may be expected, and is recommended if instrumentation IRRMPs are in effect, if water level is high, and during first loading or rare loading conditions. Enhanced monitoring should also be considered during the design phase of a modification project to better understand embankment conditions and instrument behavior before construction begins.

(3) Emergency monitoring is performed in response to observed, significant distress indicators which indicate developing failure. General recommended frequencies are not listed in Table 8.1 for emergency monitoring, because the appropriate frequency depends on the speed and type of the developing failure and whether the instrument readings would be helpful to responding to the event or investigations following the event.

(4) Frequency of readings during emergency situations should be outlined in a project's emergency action plan.

Table 8.1
General Recommended Frequencies for Monitoring Conditions

Instrument Type	Data Collection Method	Minimum Recommended Frequencies for Monitoring Conditions		
		Monitoring Condition	Reading	Data Processing and Analysis
Crack Pins	Manual	Baseline	Monthly/Quarterly	Quarterly
		Enhanced	Weekly	Weekly
	Automated	Baseline	Weekly	Quarterly
		Enhanced	Weekly/Daily	Weekly
Extensometers	Manual	Baseline	Biannually	Biannual*
		Enhanced	Weekly	Weekly
	Automated	Baseline	Weekly with ADAS	Quarterly
		Enhanced	Weekly	Weekly
Inclinometers	Manual	Baseline	Quarterly/Biannually	Quarterly/Biannually
		Enhanced	Weekly	Weekly
	Automated	Baseline	Weekly with ADAS	Quarterly
		Enhanced	Weekly	Weekly
PZs or Observation Wells	Manual	Baseline	Weekly/Monthly	Monthly
		Enhanced	Daily	Daily
	Automated	Baseline	Hourly with ADAS	Monthly
		Enhanced	Every 15 minutes	Daily
Pressure Relief Wells/Well Points	Manual	Baseline	Weekly/Monthly	Monthly
		Enhanced	Weekly	Weekly
	Automated	Baseline	Daily with ADAS	Monthly
		Enhanced	Daily	Daily
Seepage Measurement Devices	Manual	Baseline	Weekly/Monthly	Monthly
		Enhanced	Weekly	Weekly
	Automated	Baseline	Daily with ADAS	Monthly
		Enhanced	Daily	Daily
Settlement Gauges	Manual	Baseline	Quarterly/Biannually	Weekly/Monthly
		Enhanced	Monthly	Weekly
	Automated	N/A**	N/A	N/A
Surface Monuments, Survey Points	Manual	Baseline	Annually/ >Annually	Annually/ >Annually
		Enhanced	Monthly	Monthly
	Automated	Baseline	Annually/ >Annually	Annually/ >Annually
		Enhanced	Daily	Weekly
Tiltmeters	Manual	Baseline	Quarterly/Biannually	Quarterly/Biannually
		Enhanced	Weekly	Weekly
	Automated	Baseline	Weekly with ADAS	Quarterly
		Enhanced	Daily	Weekly

* Twice a year
** Not applicable

8.4.4. Site-Specific Considerations.

a. Engineering judgment, accounting for the level of risk, is required to determine the appropriate monitoring frequency for an instrument under specific site conditions. Therefore, the recommended minimum monitoring frequencies listed in Table 8.1 should be considered guidelines for dams and levees for a surveillance and monitoring plan to address potential failure modes and risks.

b. Because site-specific monitoring needs vary, monitoring frequencies higher or lower than listed in Table 8.1 may be justifiable. For example, embankments with a low consequence of failure may not need monitoring frequencies as high as those recommended. However, even for an embankment with a low consequence of failure, instrument readings should typically be examined more frequently than the annual gathering of data required to prepare an annual instrumentation report.

8.5. Threshold Values, Alarms, and Alerts.

8.5.1. The development of threshold values for instrument readings should be included in the design phase. Use of threshold values can be a great benefit to the engineer, geologist, or operational personnel for quickly prompting personnel to respond to a developing embankment failure.

8.5.2. Thresholds are also the basis for establishing alarms and alerts to expedite response to dam and levee safety issues. However, automated instrumentation systems should not be tied directly to warning sirens or public alert systems. Alarms and alerts should be verified prior to implementing an emergency action plan.

8.5.3. Criteria.

a. Criteria for establishing instrument threshold values vary by project feature and the type of developing failure the instrument is intended to detect. The criteria that was used to develop the thresholds should be documented to aid future responses to triggered thresholds. Examples of criteria for establishing thresholds are a short-term change in reading, long-term variability, and historical loading. The threshold for an instrument may be triggered if a reading:

(1) Exceeds or falls below a fixed value selected for a variety of reasons, such as a historical maximum or minimum.

(2) Changes more than a fixed amount from one reading to the next or exceeds an allowable rate of change over a longer period of time.

(3) Differs by more than 1 standard deviation from an appropriate statistical index of historical readings.

b. Project-specific thresholds are dynamic in nature and should be re-evaluated periodically as new performance data is documented or when risk assessments are updated.

8.5.4. Multiple Thresholds. Some instruments should have more than one threshold, such that stronger responses follow the crossing of higher thresholds. For example, crossing a lower threshold value based on typical error measurements may trigger verifying the reading, and the crossing of a higher threshold value may trigger halting construction near the instrument.

8.5.5. Recommended Thresholds.

a. The location of an instrument, its surrounding materials, and monitoring frequency should be considered when establishing a threshold value. Changes that may indicate a potential failure should be established before selecting threshold values. Also required are an understanding of:

(1) Historical and seasonal trends,

(2) Tolerable and expected incremental and cumulative changes,

(3) Instrument precision and accuracy, and

(4) Typical instrument and human errors.

b. Reading frequency is critical for the establishment of threshold values. If readings are taken monthly, for example, larger threshold values may be appropriate if they correspond to expected changes in readings. For 15-minute interval readings during construction, it may be advisable to have smaller thresholds for better awareness of rapidly changing conditions. In all cases, decisions on threshold values should be based on sound engineering design and/or analysis.

c. Suggested initial threshold values to trigger a response evaluation are listed in Table 8.2 for 10 types of embankment instruments. These threshold values are based on experience from construction and modification of USACE's inventory of earthen embankment dams and levees.

d. Initial threshold values should be developed with an understanding of subsurface conditions: embankment geometry and materials; abutment contact; foundation materials; and potential failure modes, reservoir loading cycles and riverine flood loading durations are also important to understand when setting reading intervals and threshold values.

e. Following are several hypothetical scenarios to illustrate the development of initial threshold values.

(1) An earthen levee embankment is subjected to long-duration flood loading.

(a) A seepage blanket was constructed on the landside of the embankment to increase seepage path lengths and minimize exit gradients in the pervious sand and gravel foundation materials. Pore pressures equal to or greater than the unit weight of water can cause a quick condition where the effective stress reaches zero.

(b) To address a concern about seepage blow-out at the landside toe of the embankment from high vertical exit gradients, a PZ is installed with the tip and sensing zone contained within the pervious stratum. To provide warning time for effective floodfighting, a threshold value less than the hydraulic pressure necessary for seepage to cause heaving and blowout of the seepage blanket should be established.

(2) An earthen embankment dam is constructed on a foundation susceptible to BEP.

(a) For BEP to occur there must be a source of water, an unprotected or unfiltered exit, erodible material within the flow path, and continuous, stable roof formed allowing a pipe to develop (USBR, 2015).

(b) PZs can be installed to monitor the gradients necessary for BEP. To alert dam safety personnel to the increased risk of BEP, threshold values less than the critical gradient required to initiate BEP should be established.

(3) An I-type floodwall is installed a critical location along the centerline of an earthen levee.

(a) The I-wall is located in a tributary reach which is not gauged and not otherwise monitored. I-walls are particularly susceptible to overturning failure when loaded to the point that lateral hydraulic force creates a wedge-shaped void at the river side face of the wall stem. When water is allowed to into the void, the seepage path is significantly shortened and internal erosion may initiate and ultimately cause an overturning failure of the wall.

(b) A river-side PZ is installed to monitor the hydraulic pressure on the wall. An engineering analysis determined the critical piezometric elevation necessary to cause failure. To provide warning time for evacuation of residents, a threshold value under the critical elevation should be established.

(4) A new earthen dam embankment is constructed between valley slopes of exposed, broken rock.

(a) Survey monuments are installed along the crest to monitor for settlement during and after construction. The designer is concerned that the available clayey sand embankment material may consolidate and undergo differential settlement as a result of irregular contact at the left abutment.

(b) A general rule of thumb allows 3–5% settlement of embankment height during construction and 1–2% post-construction before there is a concern for the potential for internal erosion. Concentrated leak erosion through a defect at the embankment/abutment contact due to differential settlement could progress to eventual breach of the embankment.

(c) To provide ample warning for evacuation downstream of the dam, a threshold value less than 3% of settlement at the crest survey points should be established for monitoring during construction and less than 1% for post-construction monitoring.

f. The reading variations triggering an evaluation are listed for an individual reading and a trend of readings. The risk driving failure modes should be the primary consideration for alerts. Table 8.2 is provided for general guidance, but it is important to know the basis for the threshold to have an appropriate response in the event of exceedance. Utilizing comparisons of data trends with similar types of structures (e.g., embankments that are homogeneous, zoned earthfill, or rockfill) may be helpful in establishing project-specific thresholds.

g. The reference titled, “A Framework for Estimating the Probability of Failure of Embankment Dams by Piping Using Event Tree Methods” by Foster and Fell (1999) is a good resource for data trend for certain types of instruments. For example, it includes construction and post-construction settlement ranges for various types of embankments. Project-specific thresholds and the basis for those values should be outlined in a project’s emergency action plan.

Table 8.2
Example Thresholds to Trigger Response Evaluation

Instrument	Value or Trend	Reading[1]
Crack Pins	Value	+0.1 inch (2.5 mm) between readings
	Trend	3 consecutive readings reaching value
Extensometers	Value	+0.25 inch (6.4 mm) for anchored 1.5 inch (3.8 cm) for magnetic reader
	Trend	3 consecutive readings reaching value
Inclinometers	Value	0.25 inch (6.4 mm) between individual elevation readings
	Trend	3 consecutive readings reaching value 3 consecutive readings with a plot shape indicating shear plane development that may not yet achieve the threshold value
PZs or Observation Wells	Value	2–5 feet (0.61–1.52 m) of change between readings Historically high readings Historically low readings
	Trend	Continued long-term increases or decreases that do not correlate to historic relationship with loading
Pressure Relief Wells/ Well Points	Value	Dependent on design assumptions
	Trend	Continued long-term increases or decreases that do not correlate to historic relationship with loading
Seepage Measurement Devices	Value	More than a 10% change in flow rate for constant loading Historically high readings for the current loading Historically low readings
	Trend	Continued long-term increases or decreases in readings that do not correlate to historic relationship with loading
Settlement Gauges	Value	With no new loading, any reading not indicating decaying rate of settlement
	Trend	3 consecutive readings of a constant rate
Surface Monuments, Survey Points	Value	0.5 inch (1.27 cm) between readings
	Trend	3 consecutive readings reaching value
Tiltmeters	Value	Dependent on design assumptions
	Trend	3 consecutive readings reaching value

[1] Reference **Error! Reference source not found.** corresponding reading intervals

h. For PZs, typical threshold values are rates measured in feet of head and vary based on the medium surrounding the instrument and the monitoring frequency. However, attention should be given to how quickly unusually low or high readings develop. A sudden drop in a PZ level could indicate the enlargement of a seepage path or the opening of a new one.

i. For a downstream PZ in a karst foundation that responds nearly instantaneously to changes in tailwater elevation, a 5-foot (1.52 m) change during tailwater fluctuation of a similar or larger value is not necessarily alarming but a corresponding 5-foot (1.52 m) change in an instrument located within a compacted clay embankment would be.

j. A typical threshold value for a PZ installed in highly permeable material, such as openings in rock or karst, is on the order of 2 feet (0.61 m) of change beyond what the loading change is expected to cause. For PZs in lower hydraulic conductivity material, such as clay, even 1 foot (30.5 cm) of change could constitute a threshold value.

k. In some challenging cases with complex responses to precipitation, the response of a single PZ may need to be evaluated by comparison with the response of a group of PZs. A typical temperature measurement threshold value is a change exceeding what is expected by 2° Fahrenheit (1.1° C).

l. For inclinometers in an embankment, a typical threshold value may be 0.25 inches (6.4 mm) of movement between consecutive readings. However, several consecutive readings each less than the threshold may accumulate to a total movement from a reference position that should constitute crossing the threshold.

m. In cases with a portable probe, the inability to obtain a measurement should be provisionally considered the crossing of a threshold. If an inclinometer probe cannot pass through the inclinometer casing, casing shear may be the cause resulting from embankment or foundation displacement.

n. Extensometers have variable levels of accuracy, depending on the technology used. The accuracy of an extensometer should be considered in determining an appropriate threshold value. Extensometers with transducers connected directly to anchored points have a much higher level of accuracy and precision than those relying on magnetic fields.

o. Settlement monuments on compacted clay embankments typically have small rates of elevation change over long periods of time, typically less than 0.1 inch (2.5 mm) per month for commonly sized compacted embankments more than 2 years old. Consecutive readings that establish a consistent trend of settlement of this magnitude, and especially an acceleration in settlement, should constitute a crossed threshold.

p. Crack pin threshold values should be based on a rate of change and are typically on the order of 0.1 inches (2.5 mm) of enlargement between consecutive readings.

8.5.6. Alarms and Alerts.

a. If an embankment is monitored with an ADAS, alarms can be established for an individual instrument or a group of instruments to notify personnel by various means of the triggering the threshold. Using an automated data transfer system, alarms can be registered on a status screen, as shown in Figure 8.5, or sent to specific persons by email, telephone call, or text message.

b. The status screen shown in Figure 8.5 is organized into zones. The top of the screen is divided into zones of upstream instruments and downstream instruments, and the status of individual instruments is shown. The lower part of the screen is divided into three zones: alarm status, control room horn, and communication system status.

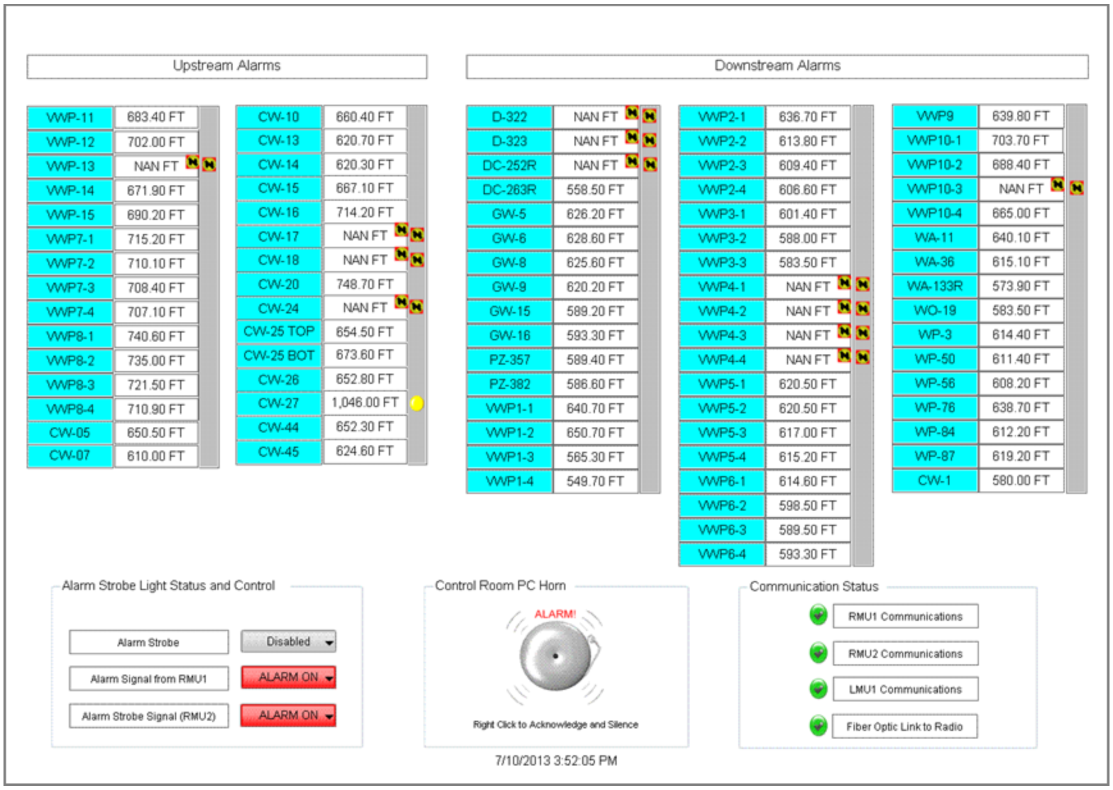


Figure 8.5. ADAS Alert Status Screen

c. Alert levels should be set based on the monitoring condition (baseline, enhanced, or emergency) and the level of risk for each potential failure mode. A project operating under an enhanced monitoring condition can have alert levels established to direct a halt to construction activities if the alert is triggered.

d. An example of an alert level chart based on potential failure modes and risk assessment for a cutoff wall project is shown in Figure 8.6. Levels of risk increase as reactions progress from upstream to downstream and from PZ indications to measured movements to signs of distress visible to the naked eye. Alert levels increase from 1 to 5 as risk increases.

Increasing Risk

Increasing Alert

Alert Level	U/S[1] Piez. Response[2]	D/S[1] Piez. Response[2]	U/S[1] Movement	D/S[1] Movement[3]	Visual Stress Indicator
1	Yes	No	No	No	No
2	Yes/No	Yes	No	No	No
3	Yes/No	Yes/No	Yes	No	No
4	Yes/No	Yes/No	Yes/No	**YES[4]**	No
5	Yes/No	Yes/No	Yes/No	Yes/No	**YES[4]**

1 – In this table, U/S and D/S refer to locations of instruments in relation to the existing barrier wall.
2 – Indicates a piezometer response above established threshold.
3 – Indicates movement beyond established thresholds in inclinometers, extensometers, crack pins, and surface monuments.
4 – Requires automatic work stop when exceedance of threshold occurs.

Figure 8.6. Alert Levels for a Seepage Cutoff Wall Project

8.5.7. Manually Read Instruments. Manually read instruments should have a checklist that includes the threshold values and several previous readings to help the reader notice that a threshold has been crossed. Such supporting information provides context, cues the reader to expected instrument behavior, and in addition helps the reader avoid a data transcription error, reducing the likelihood of a false report of crossing a threshold.

8.6. Data Integrity. Data integrity is discussed in the following sections.

8.6.1. General.

a. Any data management system must be able to apply a calibration and correction factor as needed to each instrument.

b. Calibration factors are applied to provide a comparison of the instrument readings with a standard, known value. Calibration factors are applied at the time of installation (and periodically, as necessary) to account for variations during manufacturing of the instrument. Correction factors account for physical phenomena that change the indicated values (e.g., barometric pressure). Correction factors are typically applied to raw instrument readings to more closely represent actual physical conditions.

c. Data integrity depends on documenting the calibration and correction factors and applying them to the proper instrument. Therefore, all calibration and correction factors should be stored inside the database and in at least one regularly backed up server apart from the database.

d. Care should be taken to ensure calibration and correction factors are applied to the correct instrument. Wiring diagrams and installation photographs can be useful for verifying cable connections to ensure that instruments are identified correct. Manufacturers' user manuals can be a useful tool for troubleshooting suspect data.

e. If an instrument reading is suspect, metadata may be helpful in troubleshooting the problem. The type, position, and calibration of the instrument must be verified along with the reduction equation and any associated constants.

f. Instruments that have developed an intermittent electrical fault or interfering signal from power cables may be problematic. This is a situation where a wiring diagram and metadata could help in identifying the problem. If data problems are subtle, several readings may be obtained before the problem is noticed.

8.6.2. Manual Data Collection.

a. The integrity of manually collected data depends on readers who follow procedures consistently and understand the instrument. A checklist can help a reader perform tasks consistently and react effectively to unexpected instrument behavior. If an anomalous reading is noted, the checklist should prompt the reader to read the instrument.

b. A small diameter standpipe PZ may cause a false reading if the water level indicator reacts to water on the standpipe wall rather than at the actual water level. Knowing what the reading should be should induce the reader to check for a true water surface by lowering the probe.

8.6.3. ADAS.

a. An ADAS makes high frequency readings possible, but its data integrity can be compromised by faulty data cables and loss of electrical power. Instruments are often grouped, with data cables connecting the instruments to a remote monitoring unit which collects and transmits the data. If a data cable is pinched, poorly spliced, placed too close to power cables, or improperly sealed from moisture, erroneous readings can result, such as high or low spikes and trends unrelated to embankment performance.

b. Poor splices and seals in data cables can be particularly difficult to troubleshoot. Loss of electrical power can cause gaps in data that are easily noticed. Therefore, instruments and units that collect or transmit data need battery backup power regardless of the primary means of supplying power to the equipment.

8.7. Construction Data Management.

8.7.1. Much of the data management principles discussed in this chapter can be applied to construction data management systems. Construction can affect instrument readings, and construction management can benefit from instrument readings. Performance monitoring provides staff with data to assess the response of the embankment to construction. Instrument data topics related to construction include:

a. Coordination of instrument and construction data systems,

b. Manual versus automated data management, and

c. Understanding the construction as a whole.

8.7.2. Instrumentation data systems and construction management data systems should be designed so each system can interact with the data of the other, so personnel can benefit from instrument data that may provide information on how the embankment or foundation is reacting to the activities of a construction or modification project.

8.7.3. In principle, correlations between embankment performance and ongoing construction can be accomplished without automation, but in practice instrument data for large construction projects at complex sites are difficult to manage manually, due to the large number of instruments and the high frequency of data collection. The trend is for construction management systems to be increasingly automated.

8.7.4. The personnel who interprets instrument data and relates the data to construction events needs sufficient information to understand the work as a whole. It is necessary to know the full sequence of construction events to judge whether only the most recent work or earlier work has caused an embankment response reflected in instrument data.

8.7.5. The engineer or geologist on an active construction site should have performance monitoring databases that not only include metadata, drawings, subsurface cross-sections and profiles, boring logs, thresholds, but also construction schedules when evaluating embankment performance. It is also crucial to have good communication between all parties to relay changes and concerns.

Chapter 9
Data Processing, Evaluation, and Reporting

9.1. Introduction.

9.1.1. Proper analysis of performance data is crucial to fully understand the behavior of a monitored dam or levee. Analysis cannot only reveal unsafe structural developments but can also indicate the adequacy of the instrumentation system. This chapter presents the fundamentals of processing and evaluating instrument measurements to document assessment of project performance.

9.1.2. Planning all the elements, as discussed in Chapter 4, should be completed before the instruments are installed and monitoring begins. The surveillance and monitoring should take into consideration the parameters associated with the more likely potential failure modes. Analysis and the reporting requirements should match the level of risk of the monitored embankment and the identified potential failure modes.

9.1.3. Chapter 5 describes types of instruments and weaknesses that could lead to erroneous readings. Just as the complexity of the surveillance and monitoring plan should be scaled to the project risk, the data evaluation should reflect that same level of urgency and rigor. Discussion of processing and evaluation of various instrument types is covered in Sections 9.2 to 9.6 and general considerations for data evaluation follow.

9.2. Seepage Flow.

9.2.1. Measurements of water flows that emerge on or downstream of an embankment or floodwall are the most fundamental indicators of performance relative to the retention of water. Seepage flows are measured at seeps, springs, leaks, and drainage outlets. Such discharges may be expected but can compromise safety if not suitably collected, monitored, and safely conveyed from the structure.

9.2.2. All seepage flows, even those that seem too small to measure, should be regularly observed for changes in color/turbidity, quantity, and location over time. If wet areas do not naturally concentrate flow into channels facilitating measurement, then channels should be constructed to facilitate measurement if possible.

9.2.3. Changes in flow trends over time are of primary interest in measuring seepage. Unexpected flow increases should always be considered serious and a justification for increased surveillance. Decreases in flow at a drainage feature could indicate a change in project performance or may be attributed to instrument malfunction that requires maintenance. Variables that may be correlated to seepage flows include:

a. Reservoir surface elevation,

b. River and other discharges,

c. Precipitation,

d. Snowmelt,

e. Water temperature, and

f. Groundwater levels.

9.2.4. Even seepage emerging at a considerable distance from the dam or levee could affect dam or levee safety. Therefore, any seepage that appears to be directly related to the hydraulic load should be monitored irrespective of the distance.

9.2.5. Processing.

a. Conversion formulas are based on the geometry or style of the weir/flume and height of water or time to fill a known quantity container are typically used to calculate flow rate. The conversion factors should be based on as built information and not from design drawings. Using conventional designs and manufactured components promotes accuracy. Any inundation of the measurement device by tailwater or discharge from the device itself should be noted.

b. Velocity meters and timing devices should be calibrated to the range of flows being measured and periodically checked to ensure that calibration is maintained.

9.2.6. Evaluation.

a. Evaluations of measured seepage flows involve not only the magnitude of individual readings but also trends over time. Evaluation of seepage flow should consider information regarding precipitation, pool, tailwater, and observations of sediment transport or dissolution of materials. Evaluation of data trends assess:

(1) Response to loading from pool or tailwater over time,

(2) Whether flow magnitude is within the expected range,

(3) Changes to transmissivity within the foundation or embankment, and

(4) Decreased efficiency of drainage features.

b. The location of identified seepage is important to understand relative to other project features, primarily drainage or cutoff features. Evaluators should ask the following types of questions:

(1) Is seepage surfacing in expected or unexpected locations?

(2) Do the seepage locations indicate a potential failure mode?

(3) Do the seepage locations correlate with other signs of an identified failure mode?

(4) Are all seepage locations mapped for ready location by observers?

(5) Are all measurable seepage locations equipped with proper collection and measuring systems?

c. An evaluation should determine if seepage is continual or transient. The evaluation method depends on the way load is applied to the structure. As a dam experiences first filling or as a levee is wetted after being dry for months or years, time is required to establish an equilibrium seepage response.

d. If a dam embankment or foundation material has very low hydraulic conductivity, the time at which equilibrium is achieved can be years after reservoir filling. In contrast, a steady-state seepage response for a levee may not develop before flood loads cease. For a dam, seepage equilibrium might not be achieved due to fluctuation of the reservoir level. However, saturation of the embankment and foundation typically progresses to near-equilibrium within the bounds of the usual reservoir loading.

e. After the seepage flow record indicates that a steady-state or predictable condition exists, the evaluation should compare measurements against three desirable behaviors:

(1) Sediment load and dissolution remain negligible.

(2) Peak seepage flow and overall volume do not exceed historical levels.

(3) The flow from drainage features does not decrease.

f. The expected seepage quantity should be estimated during design. However, the expected seepage is typically a total flow rather than surfacing or intercepted flows. All observable seepage flows should be monitored throughout the life of the reservoir.

g. Although drainage features are designed for a range of possible flows, estimates of collected flow range across one or two orders of magnitude because of the variability of in situ transmissivity. Thus, the design often over- or under-anticipates actual flow. In addition, geologic features in the foundation may significantly affect the amount and distribution of seepage in unexpected ways.

h. The evaluation should determine if flow remains within the safe capacity of the drainage system during normal and expected peak loadings. Drainage features can be expected to lose effectiveness over time due to corrosion, encrustation, or biological activity. A decrease in the quantity of seepage from drainage features is typically accompanied by increased piezometric level. An evaluation may determine that drain cleaning is necessary to control that pressure.

i. Evaluation of seepage data requires a thorough knowledge of a site geology and how the geology is related to embankment design and construction. Evaluation of a seepage trend is likely to be inaccurate if groundwater, precipitation, and surface runoff are not monitored and considered.

j. Erosion, dissolution, or cracking can produce increased surfacing seepage long after a steady-state seepage condition has developed. Visual observation may note the presence of cracking and internal erosion. The increased effective stress resulting from drawdown may induce settlement that decreases transmissivity and reduces seepage flows.

k. An engineer or geologist performing an evaluation should consider that a change in measured seepage could be produced by flow diversion or rechanneling with no change in total flow. Such flow path relocations can be the result of small-scale surface erosion at a seepage exit point, but may also be due to partial failures of drainage features and animal burrows. However, the possibility should be considered that the redirection results from erosional instability within the embankment or foundation.

l. If relocation of a flow exit is suspected, visual inspection should be performed to find any new exits and to perform measurements that can indicate if total flow has increased. Instruments may need to be read more frequently, and the measurements scrutinized for evidence of a change in seepage.

m. Relatively rapid decreases in seepage flow may indicate that internal erosion or other detrimental deformation has diverted flow and should be immediately investigated. If such a decrease occurs while the loading remains constant or even increases, an evaluation may reveal a possible deterioration of the drainage features.

n. Rapid changes of flow should trigger increased monitoring of all PZs in the vicinity of the flow, and monitoring should continue for some time after the increased flow develops. Any flow increase, or decrease, that is determined to be independent of pool level, groundwater level, and precipitation should be investigated immediately.

o. Even if seepage flow through a dam is primarily a function of reservoir elevation, a change in flow through an abutment or foundation may not closely follow a change in pool level over short periods. Such a lack of close correspondence may be due to layers of high transmissivity, such as gravels, joints, or bedding planes becoming alternately submerged and drained as the pool level fluctuates. Staff not only need to consider geologic features while making an evaluation but to also have complete records of reservoir first filling and high-water loadings.

p. Seepage monitoring during first filling should be more intense because of the potential for rapid emergence and increases in seepage flow. Such increases are anticipated and do not necessarily indicate unsatisfactory performance. However, if increased flow is accompanied by increases in sediment load, a halt to reservoir filling may be required.

q. Significant and/or design loading at some projects may not occur until well into the project lifetime. New pools of record can also generate previously unseen seepage. Engineering judgment is required to evaluate the significance of each new seep. As a dam or levee ages, attention is necessary if:

(1) A new seepage location develops,

(2) A significant unexplained change in flow occurs,

(3) Sediment emerges with flow, or

(4) Modifications are constructed.

r. A typical seepage flow time series, shown in Figure 9.1, is plotted with pool level and daily precipitation. In such plots, presenting other potential causes of seepage, such as groundwater level, can be useful. Plot scales should be selected such that changes 10% or smaller in seepage rates can be recognized.

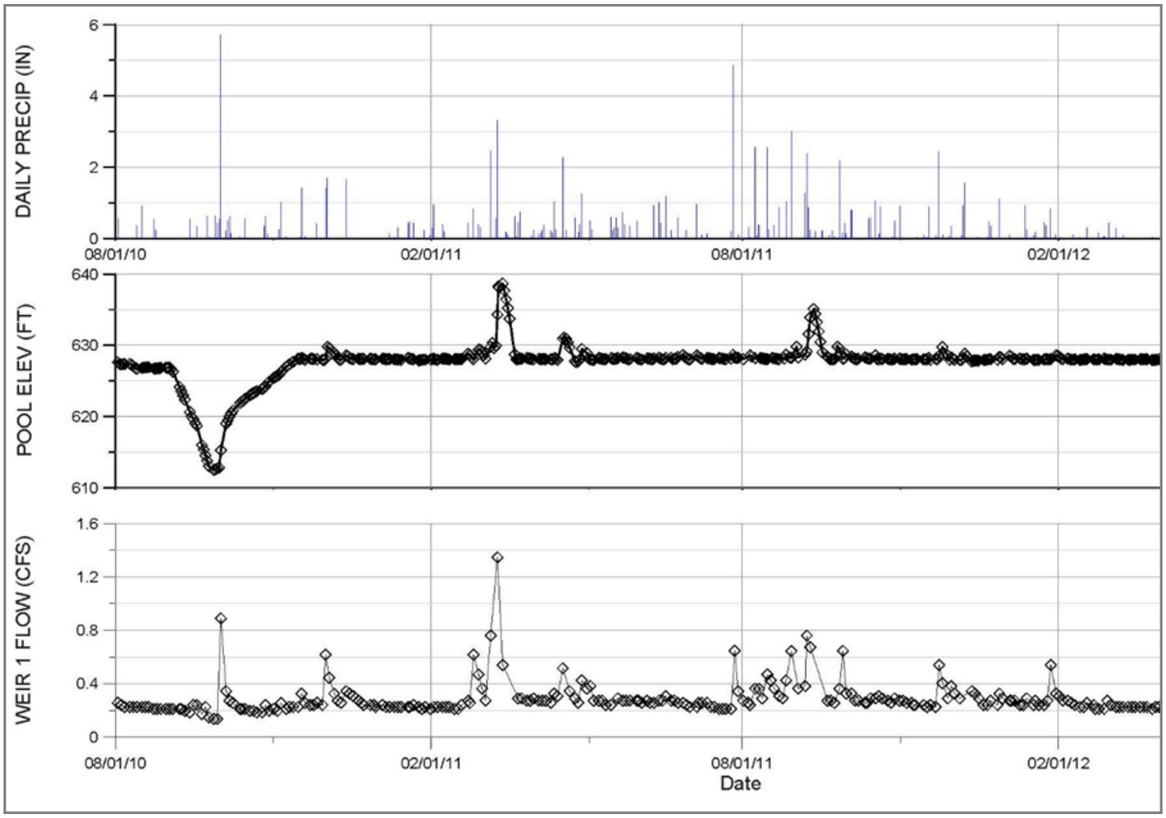


Figure 9.1. Seepage Time Series Plot, Pool Elevation, and Precipitation Depth: 2010–2012

s. Although the area under the seepage flow curve represents the volume of flow, the calculation and direct display of the flow volume rather than rate can be helpful to evaluate trends if large and frequent fluctuations in pool level occur. Comparing the volume of collected seepage flow for a given period to the associated average reservoir level yields a weighted seepage volume useful for comparisons to similar periods of loading.

t. As soil saturates, soil hydraulic conductivity increases as air is forced out of the soil pores. If the change in hydraulic conductivity is significant, an increased drainage potential may result that is typically not detrimental and can be beneficial. For example, under steady loading, an increase in saturation may result in less seepage surfacing, although total seepage is not reduced.

u. Correlation plots comparing pool level versus flow may signal this condition if measured flow decreases while pool level remains steady. These correlation plots of seepage flow versus pool level or river stage can also be useful in identifying elevations in the foundation or embankment that readily convey flow. Figure 9.2 is a correlation plot showing a consistent increase in seepage flow as reservoir pool elevation increases.

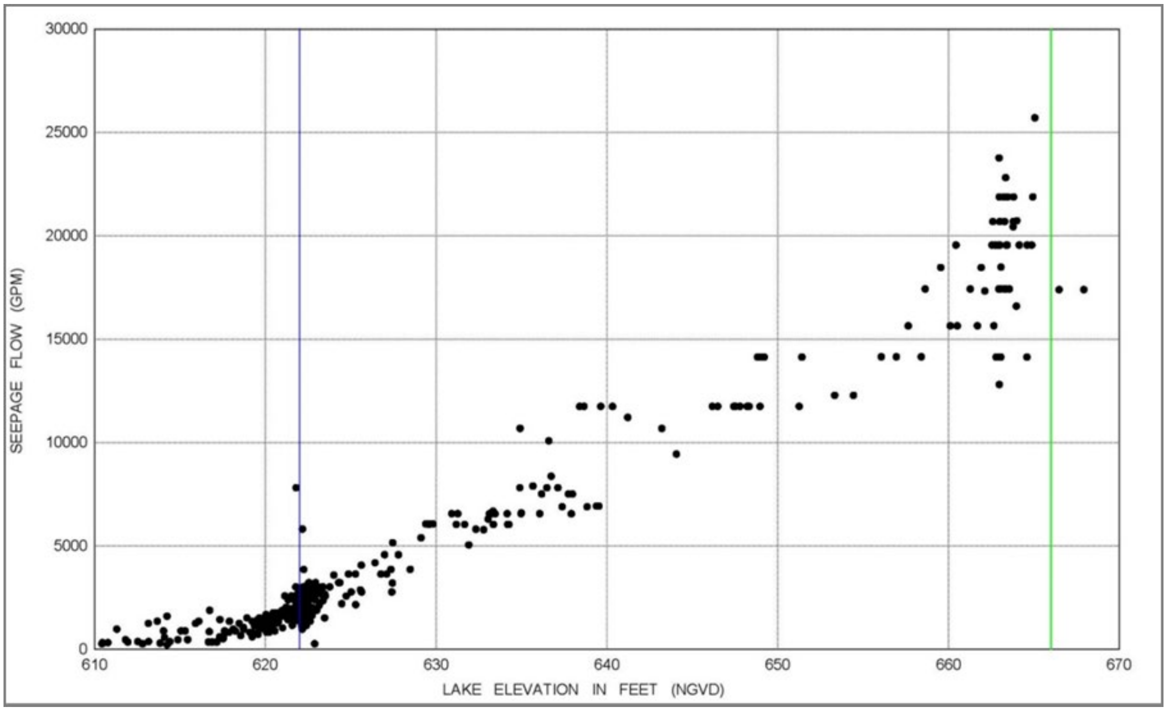


Figure 9.2. Seepage Flow vs. Lake Elevation

v. Correlation plots indicate if seepage flow responds as expected to loading and the degree of correlation. The plots can indicate if more than one loading factor is involved and can show trends if divided into historical periods. Correlation plots can also show hysteresis, or a cyclical lag in response in a measured variable.

w. Comprehensive evaluation of collected seepage flow can be facilitated by evaluating total flow for parts of the embankment, such as left abutment, right abutment, and downstream toe. Plots of the separate findings can be coordinated with respect to elevation, material, geologic structure, or other characteristics.

x. If seepage flow indicates unsatisfactory behavior, assessment of other parameters in addition to seepage is typically necessary to complete the evaluation. Piezometric level may change as flow changes due to a direct connection.

y. Long duration seepage flow may transport solids at an unnoticeable rate, yet the total mass of transported solids may be significant. If significant removal of solids is suspected, the embankment should be routinely inspected for deformation or sink holes.

9.3. Seepage Quality.

9.3.1. Suspended and dissolved solids transportation by significant surfacing seepage flows should be monitored periodically throughout the life of an embankment to:

a. Document the flow of solids,

b. Determine the trend of material transport, and

c. Determine the source of the transported material and/or the source of the seepage.

9.3.2. Even under a steady-state condition, as indicated by flow records and PZ pressures, sediment load and dissolved solid concentration should be monitored. Every seepage flow may transport material as suspended sediment and dissolved solids. As part of an effective monitoring, seepage should be evaluated to determine if the embankment or foundation materials are eroding internally and if the erosion is progressing at a rate that could result in embankment failure.

9.3.3. Additional measurements of suspended or dissolved solid concentrations should be taken if:

a. Significant flow is observed;

b. Erodible foundation materials are not protected by features, such as filters; or

c. Increases in flow indicate that internal erosion is a significant risk.

9.3.4. In addition, water chemistry testing can help determine the source of the seepage by comparing the seepage samples with samples from potential sources, such as groundwater, the reservoir, tailwater, runoff, or adjacent waterways.

9.3.5. Processing.

a. Sediment mass and dissolved salt concentrations are typically determined in a laboratory using standard testing methods. Thus, the interpretation of field measurements is usually not a responsibility of the reviewer of embankment performance. Field techniques used to obtain test samples can, however, significantly affect the evaluation of laboratory results. The engineer or geologist should know where, how, and when a water sample was collected.

b. Sediment deposited in the stilling pool of a weir used to measure seepage flow is typically collected, dried, and weighed. It may be useful to measure the particle size distribution of the collected sediment to help determine the source of the sediment.

c. The laboratory method of determining the weight of the sediment should not concern those interpreting the data. However, the environmental setting of the pool from which the sample was taken is very much an interpretation concern. For example, the pool may collect windblown particles, or observers approaching the pool to collect samples may introduce contamination.

d. The method of sample collection can affect the amount of sediment collected. For example, samples should be collected without disturbing and including accumulated solids that are not intended for collection.

e. The time at which a sample is collected may affect results. For example, surface runoff can introduce additional sediment into a stilling pool. Therefore, if samples are collected during or after a period of surface runoff, the representativeness of the sample should be assessed.

f. In some cases, the approach velocity of the seepage to the collection pool may also be an evaluation factor. For example, periods of high flow are likely to make the pool a less efficient sediment trap or may even flush deposited sediment from the pool. Thus, samples collected during or just after periods of high flow may falsely indicate low concentrations of suspended solids.

g. As a second example, some reservoirs turn over seasonally. If samples are taken during winter or summer stratification, then reservoir pH and temperature may be key to interpreting reservoir chemistry.

h. Both the location and the technique are primary concerns when sampling for dissolved salt content. Samples should be collected as close to the point of emergence and the embankment as possible to reduce exposure to other elements that may contaminate the sample, particularly water-soluble particles. Samples collected during or after a period of surface runoff may require confirmation of representativeness.

i. If water chemistry is used to determine the source of seepage, the physical and chemical characteristics of the path between a potential source and the seepage location should be considered.

j. If the path is short or the travel time is brief, the source and the collected seepage may have nearly identical water chemistries. However, a long seepage path or long travel time and contact with chemically active constituents can significantly alter the seepage water chemistry. Knowing the chemical properties of potential sources and seepage pathways can help determine a seepage source and seepage pathway.

k. In general, monitoring seepage chemistry may be used to evaluate the seepage source if internal erosion is a concern and to determine if dissolution is a potential problem. Interpreting dissolution chemicals can be complex, and if possible, sampling and chemical testing should be limited to periods of lowest and highest driving head.

l. Dissolved solids should be measured as part of a specific geochemical investigation rather than as part of routine monitoring. However, to provide sufficient data to support conclusions, geochemical investigations sometimes must take place over an extended period. Geochemical and geophysical investigations should be carried out by trained specialists. The geotechnical or dam and levee specialist evaluates the significance of geochemical or geophysical findings.

9.3.6. Evaluation.

a. Interpreted data, in the form of sediment load mass and gradations or dissolved solids concentration, helps the evaluator determine the significance of solids transported in seepage. A time series plot of sediment load mass or dissolved solids concentration may be useful, but a plot of mass transported per unit of time is typically more useful.

b. A bar chart like that shown in Figure 9.3 not only illustrates sample weight but also total weight accumulated in a given period. Displaying mass transported per unit time, with time series and correlation plots such as those used for flow quantity help identify trends.

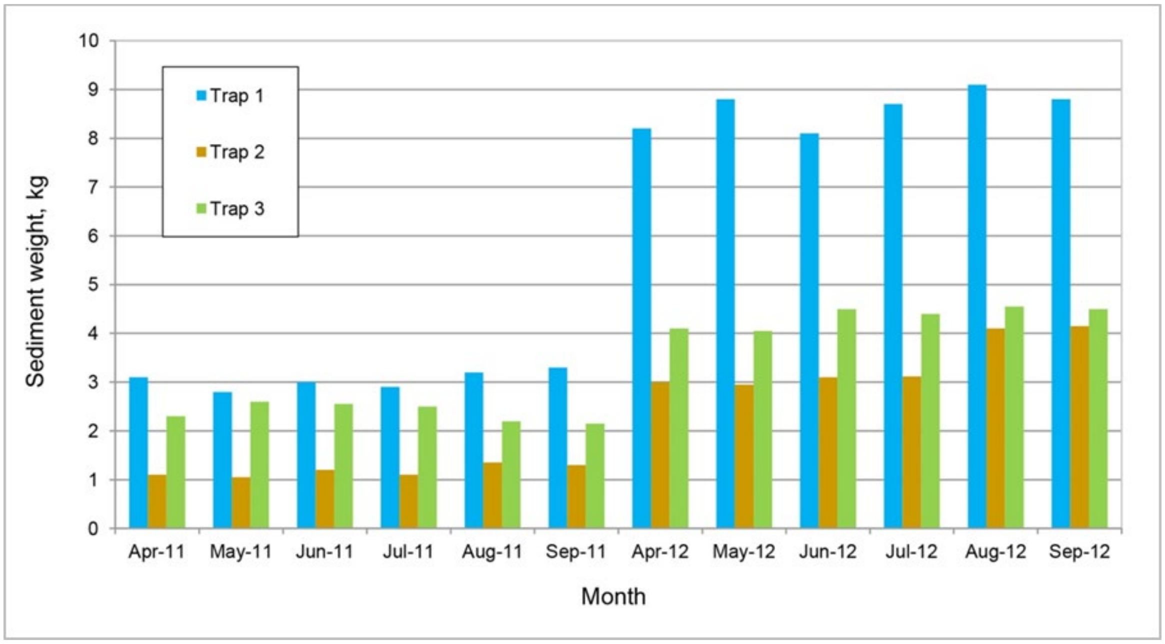


Figure 9.3. Trapped Sediment Weight: 2011–2012

c. If significant transport of suspended sediment is detected, a petrographic analysis is typically required. Evaluation of sediment transport is based upon the quantity of sediment and its origin as revealed by mineralogy. An evaluation should determine if the quantity of transported material is enough to compromise the integrity of the embankment or foundation.

d. Suspended sediment transport is particularly a concern if detected when a hydraulic load is placed upon a newly constructed or modified embankment for the first time. The seepage flow that enters new collection pipes, channels, and drains will likely contain sediments including construction debris. However, good construction control and inspection should limit the quantity of debris. Typically, loadings of a new structure involve holding periods when the load remains constant and the rate of sediment transport decreases quickly.

e. Evaluation involves judgment based on detailed knowledge of the construction history and expected performance of the embankment features. All surfacing flows during initial loading should be observed frequently. Sediment may increase initially as flow increases but should not be expected to continue longer than a day. Increasing or steady sediment transport for more than a few hours requires increased surveillance, and if sustained for more than a day, may require a special investigation.

f. Solids are also transported by seepage dissolution or the dissolving of salts. Although evaluation makes use of the concentration of dissolved solids in a manner similar to evaluating the transport of suspended solids, the evaluation of dissolution data is more problematic than the evaluation of suspended sediment.

g. Not only must the concentration of the seepage water sample be known, but the concentration of dissolved salts at the possible sources must be known as well to estimate the change in concentration associated with dissolution in the embankment or foundation. Even if the chemical nature of the source of seepage water is well understood, chemical changes occurring in water seeping in the embankment or foundation are typically characterized through judgment concerning the nature of the materials through which water flows.

h. Reservoir seepage is typically mixed with groundwater, which naturally is likely to have high concentrations of dissolved salts. Estimates of how much foundation material is dissolving are greatly overestimated if the ratio of seeped water to groundwater is not accurately estimated. In addition, if flow measurement is not carefully monitored, greater flows of lesser concentration may dilute and mask increased material loss. A small volume of dissolved foundation material can be significant if the resulting void is near erodible embankment material.

i. The significance of a high concentration of dissolved salts depends on the groundwater concentration, the reservoir water concentration, the volume of seepage flow, and the relationship of any salt deposits to the embankment. If groundwater concentration is high, seepage samples may appear to have elevated dissolved salt concentration even if no dissolution occurs.

j. A high concentration of salts in reservoir water suggests that seepage will not result in additional dissolution. Alternatively, dissolution is likely if seepage with a high salt content occurs and the reservoir water has a low salt content.

k. If dissolution of limestone is a concern, determining the Langelier Index of each water sample by measuring the in situ water pH, temperature, and alkalinity is recommended. The Langelier Index indicates if water is chemically aggressive toward calcium carbonate, the primary component of limestone, or oversaturated (not aggressive). Reservoir water oversaturated with carbonate is not aggressive toward limestone foundations unless the pH decreases as the water moves along a seepage path.

l. In most cases involving a limestone foundation, very slow dissolution of the limestone rock is less a concern than physical internal erosion of infilling due to an inadequately treated karst formation. Seeps that issue from rock on, beside, or under an abutment could have a very short or a very long seepage path or travel time.

m. The longer the seepage is in contact with the rock, the more likely that water is changed chemically. Some downstream seeps have a comparatively direct connection with the reservoir through karst, but others may be more reflective of groundwater moving through the limestone. Short, direct connections are the most problematic. A high concentration of calcium and carbonate may indicate the dissolution of foundation grout.

n. Evaluation of the concentration of individual salt constituents is typically made by comparing the size and shape of Stiff diagrams. The Stiff diagram is particularly useful for annotating cross-section drawings that show foundation geology. The Stiff diagram shown in Figure 9.4 displays milliequivalents per unit volume and quantifies equivalent weights of the dissolved constituents. The relative amount of equivalent weight of the cations and anions indicates which salt compounds, such as calcium carbonate, dissolved to form the solution.

o. Stiff diagrams indicate if the total equivalent weights per unit of volume for the cations and anions are approximately equal, as should be, providing a check on the laboratory analysis.

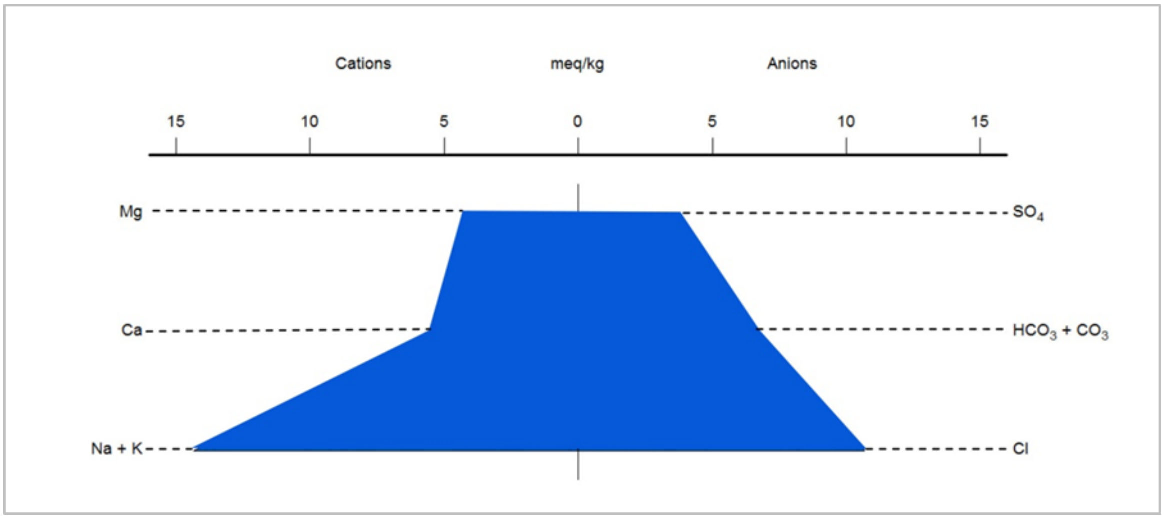


Figure 9.4. Stiff Diagram

p. The radar plot shown in Figure 9.5 and the Piper plot shown in Figure 9.6 displays the six major cations and anions associated with dissolution and allow a comparison of dissolved salt concentrations between samples. The dissolved salt signatures can be useful in delineating the seepage path and source. For example, a set of five sample test results is represented in Figure 9.5 and Figure 9.6.

q. Sample 1 was taken from the collected seepage, Sample 2 from a local groundwater well, Sample 3 from surface runoff, and Samples 4 and 5 from two nearby waterways.

r. As shown in the figures, Sample 3 has a much different composition than the other samples and is probably not the source of the seepage. Samples 4 and 5 have similar compositions but have greater concentrations of Cl and lesser concentrations of HCO_3 and SO_4 than the seepage sample. Sample 2 has a chemical composition very similar to Sample 1 and was determined to be from the primary source of the seepage.

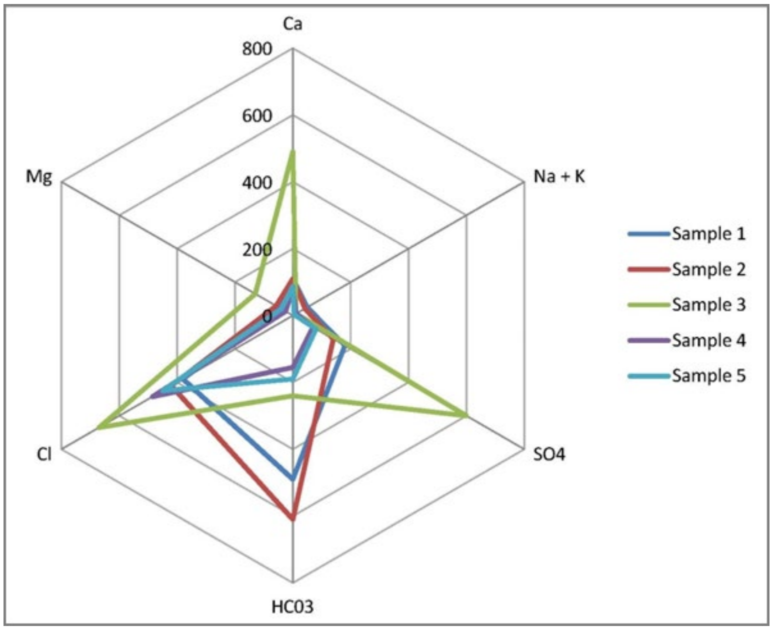


Figure 9.5. Radar Plot of Five Water Samples

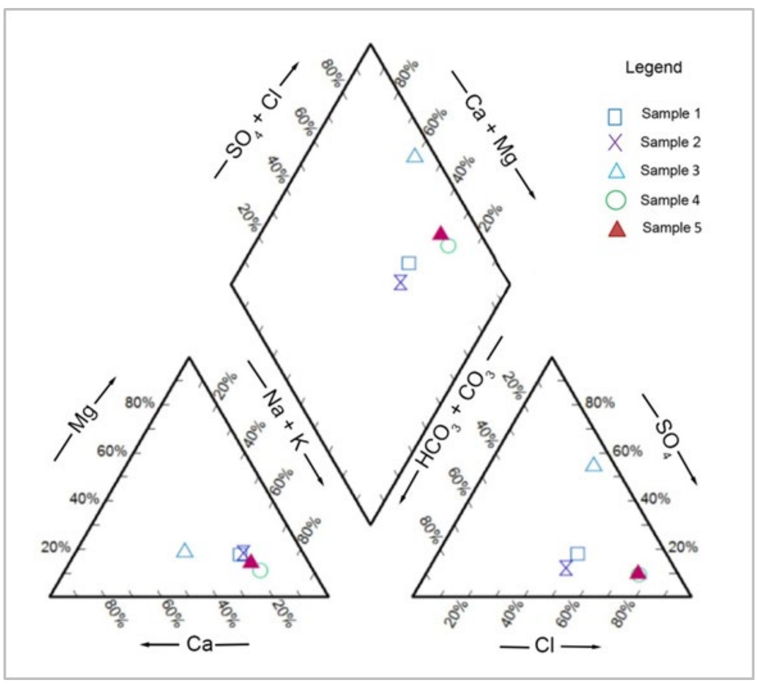


Figure 9.6. Piper Plot of Five Water Samples

9.4. Deformation Monitoring.

9.4.1. Vertical and horizontal displacements in an embankment, floodwall, or foundation can lead to cracking, settlement, heave, or slumping on the surface and can also displace parts of rigid structures, affecting joints and causing tilt. Evaluation of subsurface or surface deformation should determine if design function or shear strength is diminished or if deep-seated erosion has occurred.

9.4.2. Most deformation occurs during or soon after construction. Even without a water load, significant deformation can be expected to continue for decades after construction has been completed, especially for clay embankments. Normal deformation is slow, within the limits predicted by analysis of primary and secondary material consolidation, and, as long as there are no significant differential settlements related to abrupt slope or material changes, should not be a dam/levee safety concern.

9.4.3. Consolidation can sometimes be accelerated by installing a temporary surcharge load to preload underconsolidated fill. Preloading (or precompression) may be used to minimize consolidation settlement for new construction.

9.4.4. Monitoring of the settlement using extensometers and dissipation of excess pore water pressure using PZs provides information useful to assess design assumption. Additionally, any subsurface or surface deformation related to a significant potential failure mode requires monitoring. Such deformation may be related to slope stability, settlement resulting in loss of freeboard, cracks in the embankment or floodwall, development of a potential seepage path, and damaged appurtenant features.

9.4.5. Displacement should be reviewed and analyzed to determine if:

a. Cracking can occur, compromising a seepage barrier, drainage feature, or resulting in internal erosion.

b. Accelerating displacement indicates developing structural failure.

9.4.6. An evaluation should account for changing load over time, correlated deformation, the date when monitoring of a feature or location began, and the date when any surface deformation was visually detected.

9.4.7. The magnitude, rate of change, and direction of displacement can be determined using subsurface and surface instruments. Subsurface instruments typically used to measure vertical deformations include extensometers, settlement gauges, base plates, and subsurface monitoring points. Horizontal subsurface deflections are typically monitored with inclinometers, TDR cables, shear detectors, or extensometers.

9.4.8. Surface monitoring of a large area is typically performed with surveys based on conventional geodetic methods (triangulation, trilateration, and level surveys), GPS, or a combination of both to determine absolute displacement. The method of survey, desired accuracy and confidence limits, and associated monumentation should be coordinated between the surveyor and the engineers evaluating the data. Large-scale structural deformation surveying is discussed in the USACE publication EM 1110-2-1009: Structural Deformation Surveying.

9.4.9. Surface monitoring of smaller areas is typically performed with direct mechanical measurements or sensors measuring relative displacement, such as LVDTs, crackmeters, strain gauges, tape extensometers, tiltmeters, and alignment targets. Relative displacement data should be incorporated into the system of tracking positions and deformation across the entire embankment.

9.4.10. Processing.

a. Deformation data cannot be interpreted without detailed knowledge of the measurement device installation, operation, and the method of data collection and processing. Relative displacements due to deformation may be measured directly or, typically, calculated from angular displacement, vibration frequency, force, voltage, electrical resistance, and wavelength change.

b. To determine the amount and significance of deformation, raw data from each type of sensor or transducer must be reduced to magnitudes and directions and be referenced to elevation, embankment axis, or a stable structural element such as concrete founded on competent rock for subsequent performance evaluation.

c. Measurement of displacement can be relative or absolute. With the exception of tiltmeters, which reference the vertical datum, most instruments measure relative displacement from one part of an instrument to another part of the instrument. For concrete structures, simply determining if a crack or joint is stable or widening may be sufficient. However, the determination of absolute displacement at different points on the project can yield much more diagnostic information than an isolated relative measurement.

d. Absolute displacements are obtained by having an element of the monitoring instrument fixed at a stable reference point. The chosen reference point should be stable throughout the life of the instrument. Such anchor points may be the bottom of boreholes, where the instrument heads are set in concrete, or benchmarks where the distance from the instrument is periodically measured.

e. An instrument casing is not perfectly vertical and straight at the time when installation is completed. If an instrument such as an inclinometer is intended to provide information about movement from the date of installation, the casing deflection magnitude and inclination direction should be recorded at the time of completion. For example, casings typically tilt downslope toward the valley center over time as the embankment and foundation settle.

f. Recording the direction of inclination can serve as a check on the orientation (or polarity) of readings. The general tilt direction should not vary between reading profiles. Rock-bound inclinometer casings are likely to have a small arbitrary initial tilt acquired during installation, and the tilt should not be expected to change beyond the expected precision.

g. A typical method to verify reading quality is by comparing two sets of readings obtained in the same session but at 180° from each other. Except for orientation, the two readings obtained at a given depth should agree within manufacturer guidelines and reading pairs that fall outside of suggested limits should be measured again. Typically, the cause of a disagreement is due to taking the pair of readings at different depths or not allowing the probe to reach temperature equilibrium for one or both of the readings.

h. An evaluation can be enhanced by interpreting vertical deformation components in terms of changing elevation and horizontal components in terms of changing offsets from the structural axis. Initial readings should be obtained at the time the instrument installation is completed, and the significance of changes in subsequent readings is determined during evaluation.

i. The efficacy of interpreted data for subsequent evaluation depends on the attention devoted to sensor calibration, installation orientation, and the polarity used in data processing. The polarity associated with mechanical measurements may be a function of the positioning of the observer. Therefore, standard positioning procedures should be followed for each reading.

j. When reviewing data from survey crews, the reading polarity should be checked for conformance with the polarity used with non-geodetic measurements and the embankment in general. Portable measuring devices are subject to loss of accuracy due to handling and transport. Engineers and geologists interpreting data should determine if the data has been collected with compromised equipment.

k. Secondary loading factors, such as temperature or hydraulic loading, may influence data interpretation and should be considered. For example, hydraulic loads such as pool level in the lock chamber, may affect the tilt of a monolith wall. Proper processing should indicate when and how any secondary factors influence data.

9.4.11. Evaluation.

a. Evaluation of embankment deformation is concerned with:

(1) Locating the source of deformation potentially compromising the embankment,

(2) Recognizing differential displacements large enough to cause cracks that may lead to internal erosion, and

(3) Identifying trends indicating developing instability.

b. Evaluation should consider if unexpected results are indications of problems, shortcomings in the surveillance and monitoring, and/or the result of a lack of understanding of design, construction, or materials.

c. The process of identifying deformation problems is relatively routine following effective data processing and display. Both the magnitude and rate of deformation are of concern and should be evaluated to determine if either exceeds design expectations. Deformation rate can be more significant than magnitude because a rate that is not decreasing may be an early indicator of developing instability, possibly requiring an immediate adjustment of embankment operation. In addition, the overall pattern of deformation should be interpolated between measured areas to estimate deformation of unmeasured areas.

d. Unusual embankment deformations are likely to be an aspect of differential settlement or deflection and should be evaluated. These differentials are functions of foundation shape, material compressibility, material shear strength, and internal erosion.

e. Deformation Rate.

(1) All earthen embankments deform to some extent. Embankment deformation, including surface deformation, settlement, compression, and lateral spreading can be due solely to consolidation, and nearly all expected deformations of this type exhibit a logarithmic decay rate in the early years after construction. Normal deformation plots as a straight line against a logarithmic time scale, indicating a steady reduction in the deformation rate.

(2) An increase in deformation rate is easily discerned with semi-logarithmic graphs as shown in Figure 9.7. Figure 9.7 shows the settlement of two settlement plates over a 30-year period, one installed in the embankment fill (shown in blue) and one installed in the foundation (shown in red). Periods of greater settlement rate are highlighted in yellow shading.

(3) Settlement was minor in the embankment and foundation during the first 60 days of monitoring. However, on approximately Day 150, fill was placed, raising the embankment height and increasing the rate of settlement. Settlement exhibited a decreasing rate of creep for the next 2 years.

(4) Between Day 900 and 1,300, a second placement of fill occurred, again resulting in an increased rate of settlement. Thereafter, the embankment exhibited a continuous trend of primary settlement, until nearly Day 10,000, when secondary consolidation apparently began to occur.

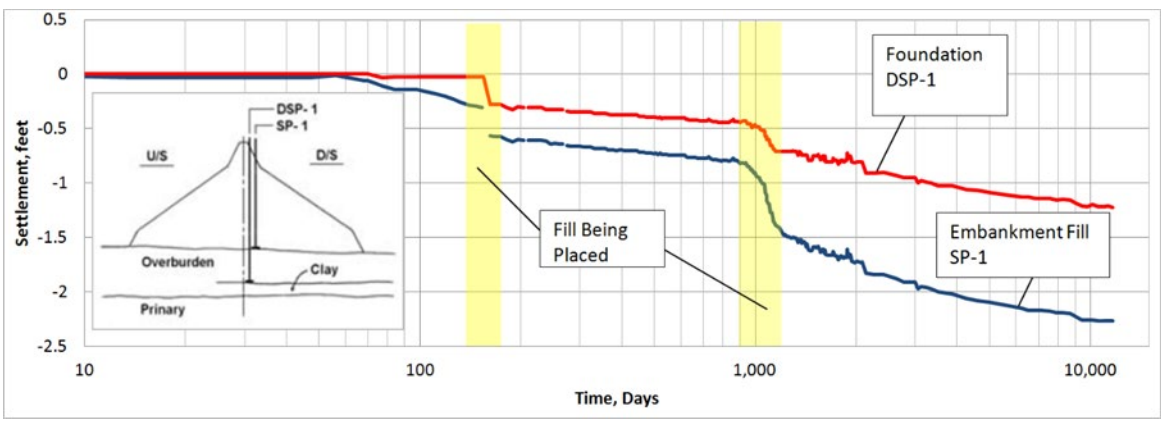


Figure 9.7. Settlement vs. Time

(5) For older embankments, small amounts of deformation may continue for years due to secondary consolidation. If an embankment does not exhibit a gradually decreasing settlement rate or a small constant settlement rate, monitoring and investigation using inspection and instrument measurements should be performed to determine the cause. For an increasing deformation rate, consideration should be given to the possibility that internal erosion or strength failure is occurring.

(6) If the rate of settlement, especially at the crest but also on the slope of an embankment, begins to accelerate, the cause may be the development of a shear failure surface in the embankment. If so, rapid displacement may result from continued loss of shear strength along the failure surface.

(7) As a shear surface progressively fails and a mass of soil in the embankment moves significantly, features may form such as cracks, head scarps, side scarps, and heave at the base of the slope.

(8) Embedded devices such as inclinometer casings and shear indicators are effective for locating the depth to the shear surface and can be installed after surface deformation is detected. Inclinometer data can be used to determine the rate of shearing by comparing multiple internal horizontal displacement profiles over time.

(9) Inclinometer readings can also be used to determine a general direction of movement. An inclinometer helps the engineer or geologist monitor a slowly developing shear surface before surface deformation is apparent. If an inclinometer reading indicates the formation of a shear surface, further investigations should be conducted immediately.

(10) Rockfill embankments featuring an internal impervious barrier can exhibit deformation of the rockfill on both the downstream and upstream sides of the barrier. For example, rockfill downstream of the barrier may become more compact under the influence of the load transmitted through the barrier. Rockfill on the reservoir side of a barrier may deform in stages due to cycles of loading and unloading as the reservoir level rises and falls.

(11) At a given reservoir level, the weight of submerged rockfill is partly supported by the buoyant force of the water in the interstices of the rocks. However, the weight of free-draining rockfill above the level of the reservoir is supported entirely by the solid stone, and the stress is greatest at the contact points of rocks.

(12) The strength of the contact points can be reduced by cycles of wetting and drying accompanying changes in reservoir level. Therefore, the increase in stress on the contact points as the reservoir level falls can cause crushing of the contact points. The gross effect of the crushing is the deformation of the rockfill mass. Typically, each cycle of buoyant weight change of equal magnitude should produce less deformation than the cycle before.

f. Vertical Deformation.

(1) Settlement of embankments results from consolidation of the fill and the foundation. If seismic activity is not a factor, the magnitude of settlement is directly related to the weight of overlying fill and the compressibility of underlying materials.

(2) Typically, settlement is greatest where fill height is greatest. Heave is only to be expected at the downstream toe of the embankment. Heave is typically due to normal slope slumping but may be due to uplift caused by excess piezometric levels or instability due to low embankment or foundation shear strength. Heave associated with excess piezometric level or instability may occur at a constant or increasing rate.

(3) Normalizing settlement in terms of percentage of fill height is an effective means of identifying fill or foundation zones of low density. A greater than expected magnitude of settlement is not detrimental if the settlement does not compromise design function, such as lowering the crest elevation below a minimally acceptable elevation, cause cracking, or progress at an increasing rate, indicating impending structural failure. Evaluation of settlement addresses three questions:

(a) Is the location critical?

(b) Is deformation expected to continue?

(c) Is the deformation rate increasing?

(4) A decreasing rate of deformation may indicate increasing stability. If the settlement is differential, a partially developed internal seepage feature may become more fully developed during a future significant hydraulic loading, threatening the stability of the embankment.

(5) Figure 9.8 shows the relationship between fill height changes and settlement response. The upper graph shows the changes in fill height at a specific location in an embankment over the course of approximately 3 years. The fill height was increased from 5 to 44 feet (1.52 to 13.4 m) during the first year of monitoring. Near the beginning of the second year (Day 740), 25.5 feet (7.8 m) of fill was removed to achieve a height of 18.5 feet (5.6 m) for construction purposes.

(6) The fill was replaced in the middle of the third year of monitoring (near Day 1,300). The lower graph shows trends of settlement rates corresponding with times of fill height increases and rebound trends during times of fill height reductions. When the fill height was reduced by 18.5 feet (5.6 m), a corresponding rebound of 0.14 feet (4.3 cm) occurred until the fill was replaced, after which the settlement trend continued before holding generally steady.

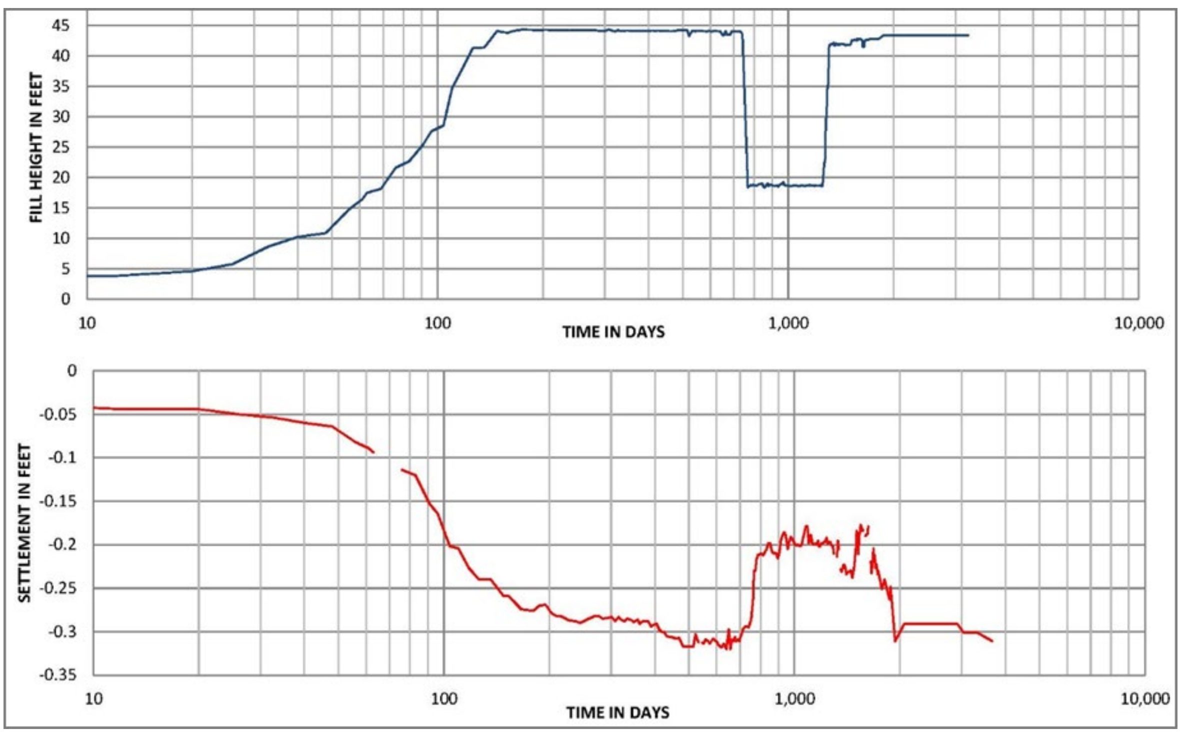


Figure 9.8. Settlement and Fill Height Change vs. Time

(7) Displaying a percentage settlement change from the original reading provides a percentage compression, for which many published discussions of embankment performance exist. For example, a maximum compression for a 10-foot (3.05 m) interval at the lowest level of a modern compacted earthen embankment could be expected to be approximately 3%.

(8) A compression much less than 3% for the lowest interval indicates an unusually dense and possibly brittle fill. Alternatively, a compression much greater than 3% may indicate insufficient compaction.

(9) While evaluating internal settlement in terms of percentage compression, data evaluators should be aware of the pattern of individual settlements. For example, Figure 9.9 is a plot of percentage compression versus elevation for an embankment 30 years old and 122 feet (37.2 m) high.

(10) The graph indicates that the lower third of the embankment structure has less than 3% compression. Unlike neighboring layers, the zone near elevation 928 feet (282.9 m) has a compression of 0.5%, indicating a much denser layer of material. The middle third of the embankment shows much variability between zones, with the interval near elevation 953 feet (290.5 m) having greater than 3% compression although the interval near elevation 963 feet (293.5 m) has a compression of only 1%.

(11) The compression of the interval near elevation 953 feet (290.5 m) may be a result of poor compaction efforts. The upper third of the embankment shows a more consistent compression of approximately 0.5%. The overall trend in compression in the embankment is for percentage compression to decrease as elevation increases.

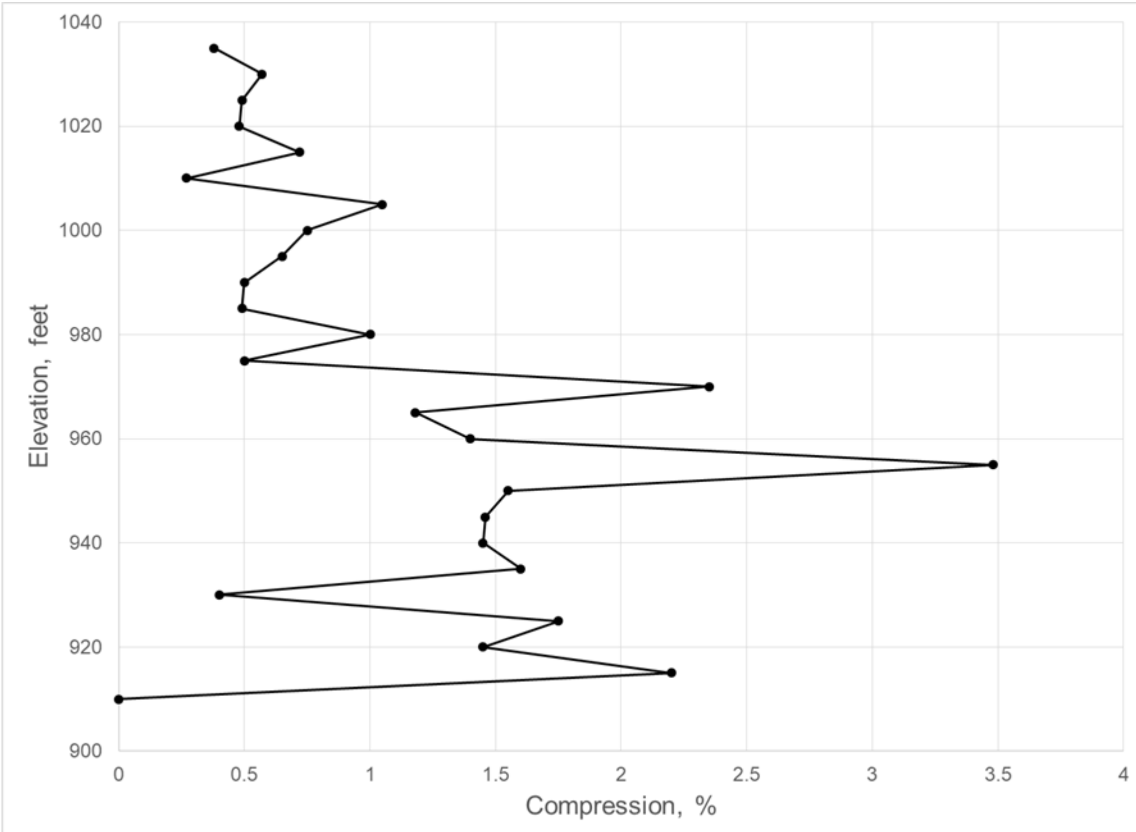


Figure 9.9. Compression vs. Elevation

(12) Maximum settlement does not typically occur at the same elevation as maximum compression but rather occurs near and often below mid-height where the combined effects of overburden and underlying compression are maximized, as illustrated in Figure 9.10. Therefore, patterns of settlement that do not increase uniformly from crest and foundation to mid-height may indicate problems with construction control regarding placement and compaction of fill or be associated with historical river sediment deposits.

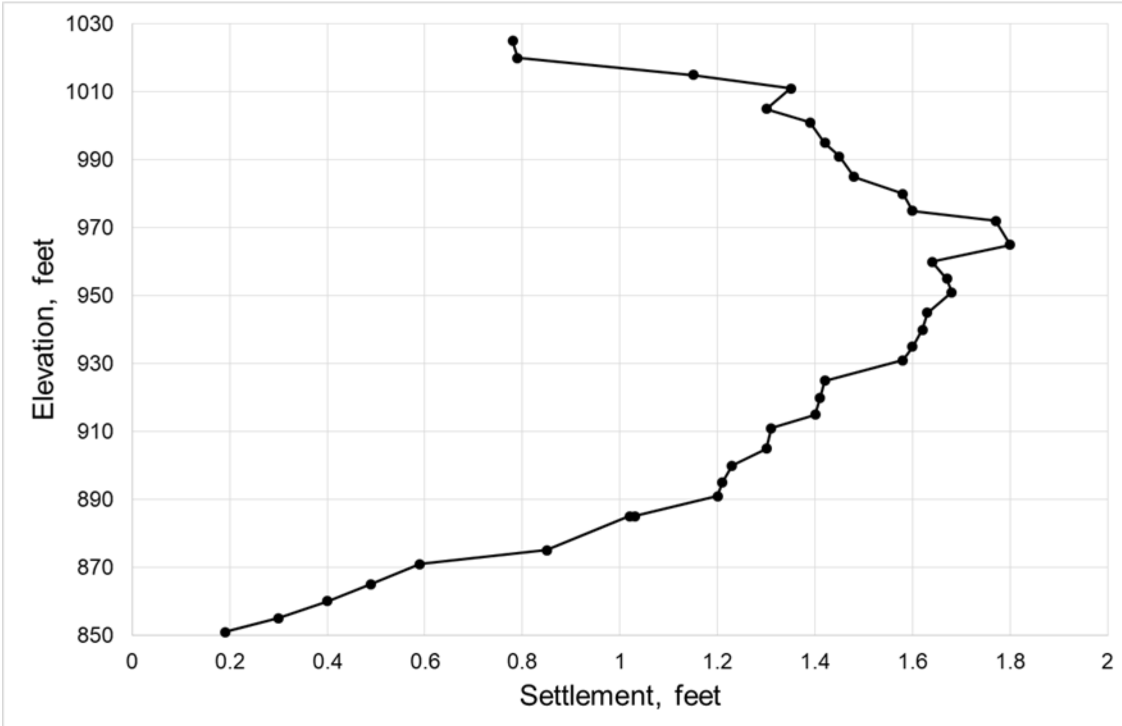


Figure 9.10. Settlement vs. Elevation

g. Horizontal Deformation.

(1) For the three types of instruments discussed below, basic evaluation compares the most recent instrument profile measurements to previous and baseline profiles. Cumulative and incremental displacement plots are used to assess the depth and direction of movement and time displacement plots provide a measure of the rate of deformation.

(2) Steady deformation is of less concern than accelerating deformation, no matter how slight the acceleration may appear. Detailed discussion about horizontal deformations related to slope instability can be found in EM 1110-2-1902.

(3) Inclinometer.

(a) To determine the magnitude and location of movement, inclinometer data are plotted in profile such that the length of the inclinometer is along the vertical axis and the cumulative or incremental displacement is along the horizontal axis. The vertical depth scale, horizontal displacement scale, and number of profiles shown on the graph should be arranged such that displacement magnitudes and locations can be easily comprehended.

(b) If biaxial instruments are used, both axial directions should be evaluated to determine the direction of movement. Cumulative horizontal displacement plots are best used to identify zones of shear, and general patterns of relaxation or tilt. However, because each interval includes the cumulative displacement of the interval beneath it, localized errors can accumulate over the depth of the inclinometer and affect the overall results.

(c) Incremental plots depict the movement at each sensor or measurement location and can be used to check for any incremental displacement errors affecting the cumulative displacement results.

(d) Another method of limiting error is to plot the checksums from the measurement data sets. The checksums compare two readings of opposite polarity taken at the time of measurement. The difference between the checksums should be within the manufacturer's guidelines for a valid measurement. Good practice is to review checksums to evaluate the quality of the data before analysis.

(e) Performing spiral surveys and applying correction factors for casing deviations helps reduce error in the displacement results although a spiral correction factor is normally not necessary for casing lengths less than 100 feet (30.5 m). Spiral corrections, usually obtained once prior to collecting the initial deflection data, should be applied with caution because the spiral survey equipment is generally more delicate than the ordinary inclinometer probe and can be misoperated.

(f) For example, if a spiral survey shows a twist that is physically implausible, such as more than 5 to 10 degrees of twist per 100 feet (30.5 m) of casing, the spiral survey should not be considered reliable. Therefore, spiral survey data should be checked for excessive twist before it is used to determine a correction factor for a particular inclinometer instrument installation.

(g) Two types of typical inclinometer displacement profiles are shown in Figure 9.11. A bulge in a plot may be an indication of both vertical and horizontal displacements. Supplementary instrumentation data such as surveys should be evaluated to help understand the nature of the deformation.

(h) Embankment inclinometer casings typically tilt along the long axis of the embankment toward the deeper fill near the valley center where more consolidation occurs. Casings also typically tilt away from the crest toward the nearest slope toes. Verification of the overall tilt eliminates the possibility that the probe orientation used to collect readings was incorrectly identified.

(i) Installations in rock typically exhibit an initial tilt due to the circumstances of installation and grouting but should not exhibit progressive tilt afterward. Tilt may indicate normal long-term consolidation. However, if tilt becomes pronounced, it may reflect other influences. Those influences can be due to:

- A nearby excavation or fill placement,
- Surface displacement of the top of the inclinometer casing caused by very shallow surface sliding, or
- Other construction-related disturbance.

(j) A shallow surface slide and construction traffic disturbance would only affect the inclinometer casing at a shallow depth. However, excavation and fill placement have the potential to affect the casing at greater depths. If the degree of tilt is continuous from the top to the bottom of the casing, a bias shift error in the data may be the cause.

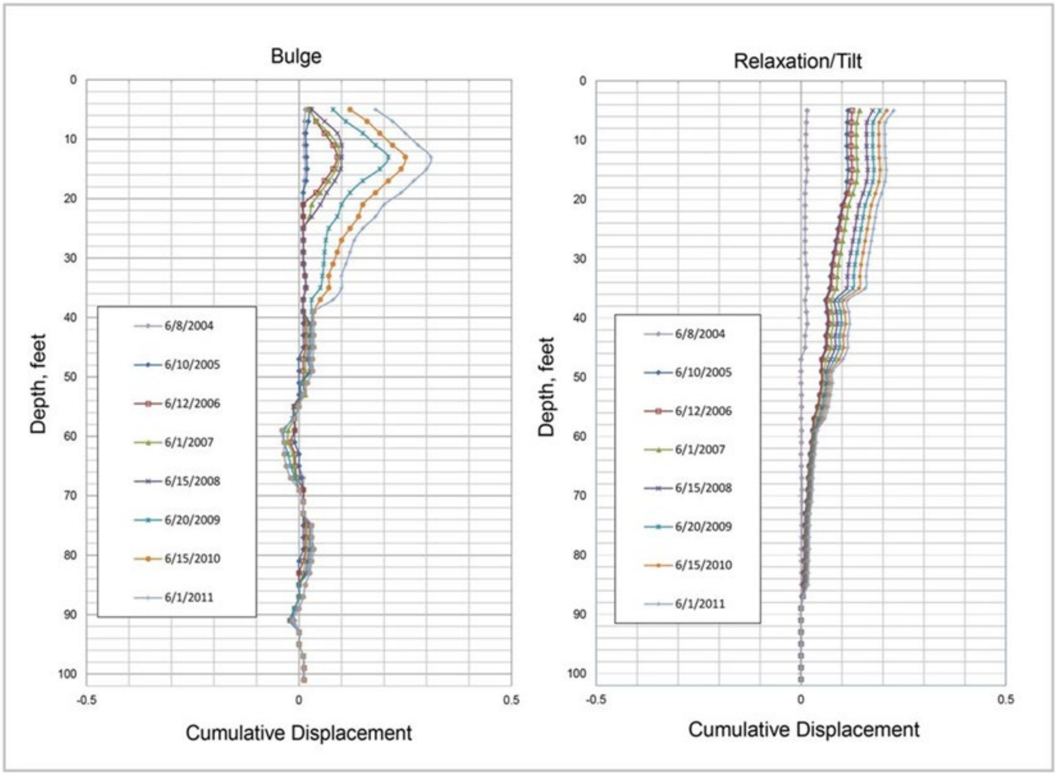


Figure 9.11. Inclinometer Plots Indicating Bulging and Creep

(k) Shear appears in profile as an obvious or pronounced offset that occurs in a distinct zone of displacement. Figure 9.12 shows typical inclinometer cumulative displacement plots of the two axes being monitored.

(l) Axis A shows the cumulative displacement within the expected plane of primary movement and Axis B shows the cumulative displacement perpendicular to the primary movement. This plot indicates a shear zone near a depth of 230 feet (70.1 m) below ground surface and increased tilt near the ground surface that is likely due to traffic, most notably in the Axis-A plane.

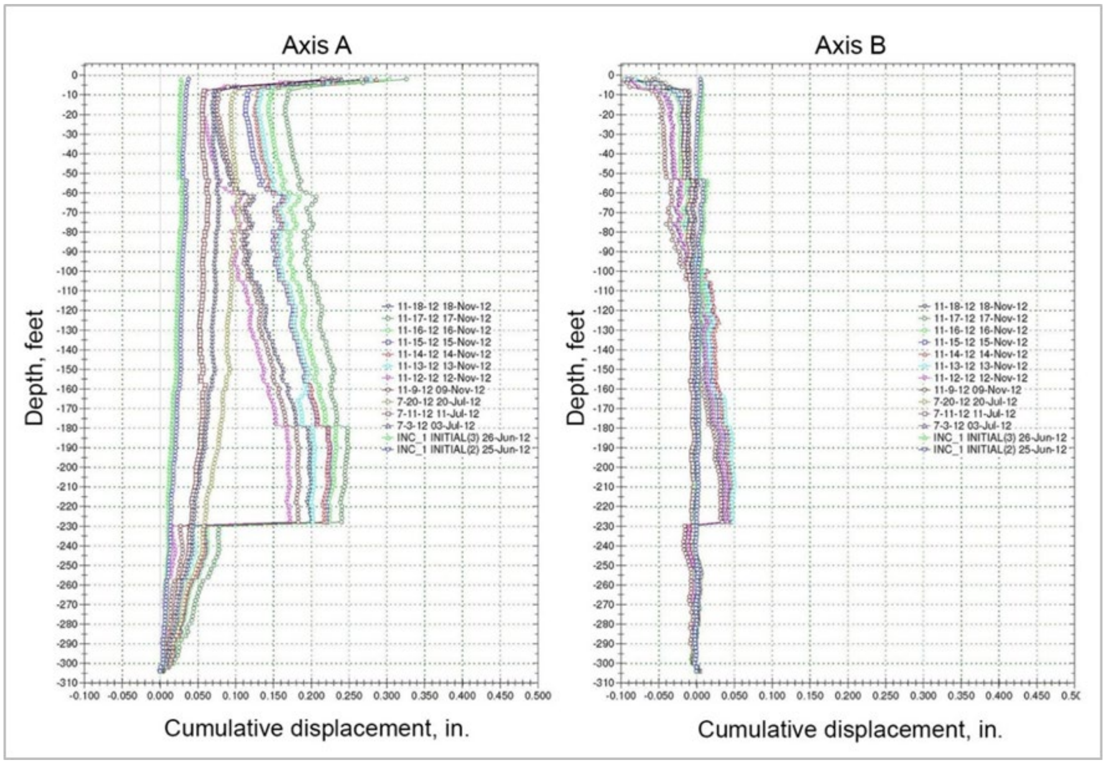


Figure 9.12. Inclinometer Plots Indicating Shear

(m) An inclinometer plot should be considered in the geological, material, and construction contexts. Displacement plots can be included in a drawing of the embankment cross-section to conveniently display displacements or potential failure surfaces in relation to embankment or floodwall structural features and materials as shown in Figure 9.13.

(n) Figure 9.15 shows embankment details, geology, and construction features needed to understand inclinometer movement—note the work platform and adjacent sheet pile wall on the upper upstream slope. Slope inclinometers in the plot are annotated as “SI”. The plot illustrates three types of displacement:

- SI-52E, the farthest upstream, illustrates the characteristic bulge associated with new fill consolidating and deforming down slope and near surface tilt toward traffic.

- SI-04 illustrates creep of material causing tilt in the casing—in this case, due to displacements of a sheet pile wall.

- SI-18 illustrates tilt developing in an inclinometer casing due to normal spreading of the downstream slope toward the downstream embankment toe.

(o) Although not shown in Figure 9.13 due to limited space, all inclinometer cumulative displacement plots should have the date of the reading clearly marked, as illustrated in Figure 9.12.

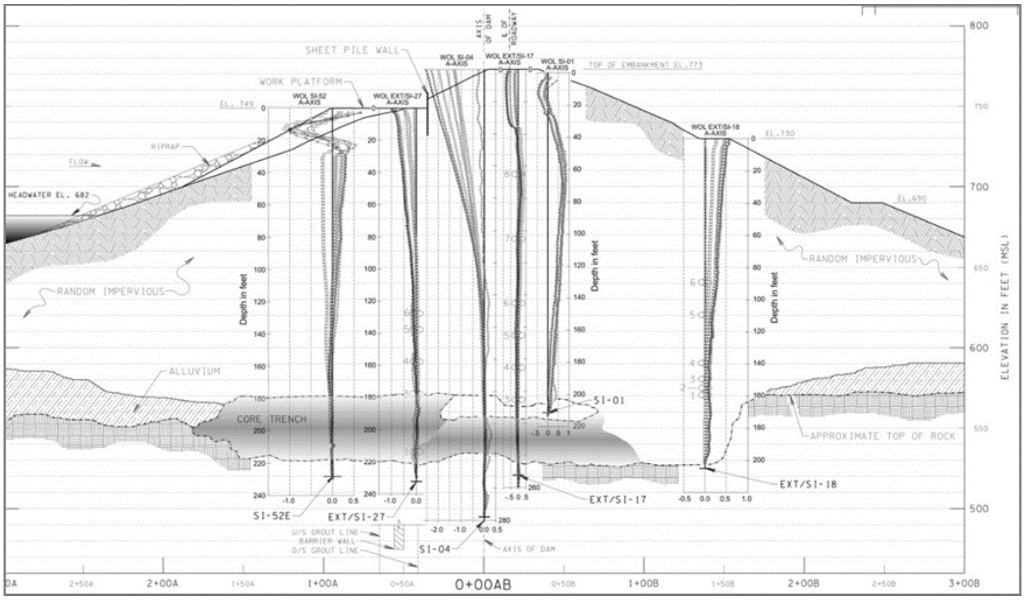


Figure 9.13. Section View of Embankment and Inclinometer Displacements

(p) For inclinometers embedded in an embankment during embankment construction, the casing length progressively increases until construction and installation are complete. As the embankment is raised and inclinometer profiles are measured, different sections of casing serve as a temporary top of the inclinometer. In such a case, good practice is to evaluate the deflection of each individual profile from the vertical rather than the change from the initial profile.

(q) Although incremental and cumulative displacement plots can show the displacement profiles of multiple records over the monitoring period, the rate of displacement is best visualized with the use of time history plots of individual intervals. For example, Figure 9.14 is a time series plot of in-place inclinometer displacement as measured by six sensors. Each sensor monitors only the displacement of a particular interval along the casing length.

(r) The plot also includes precipitation data over the same period plotted on the right-hand y-axis. The graph indicates greater displacement at the three intervals between 10 and 21 feet (3.05 to 6.4 m) below ground surface (bgs), indicating a shear zone in that interval, and within the interval of 15 to 18 feet (4.6 to 5.5 m) bgs.

(s) Precipitation events are correlated with displacement increases at the sensors, although the amount of displacement varies with depth. The two shallow sensors, monitoring the intervals of 0 to 5 feet (0 to 1.52 m) bgs and 5 to 10 feet (1.52 to 3.05 m) bgs, show an increase in displacement during heavy precipitation that rebounds during periods of little to no precipitation.

(t) The three sensors monitoring the intervals between 10 to 21 feet (3.05 to 6.4 m) bgs show displacements that do not rebound, indicating plastic deformation and possibly indicating a zone at which precipitation accumulates, increasing the water content of the soil and decreasing the soil shear strength and possibly causing the shear displacement.

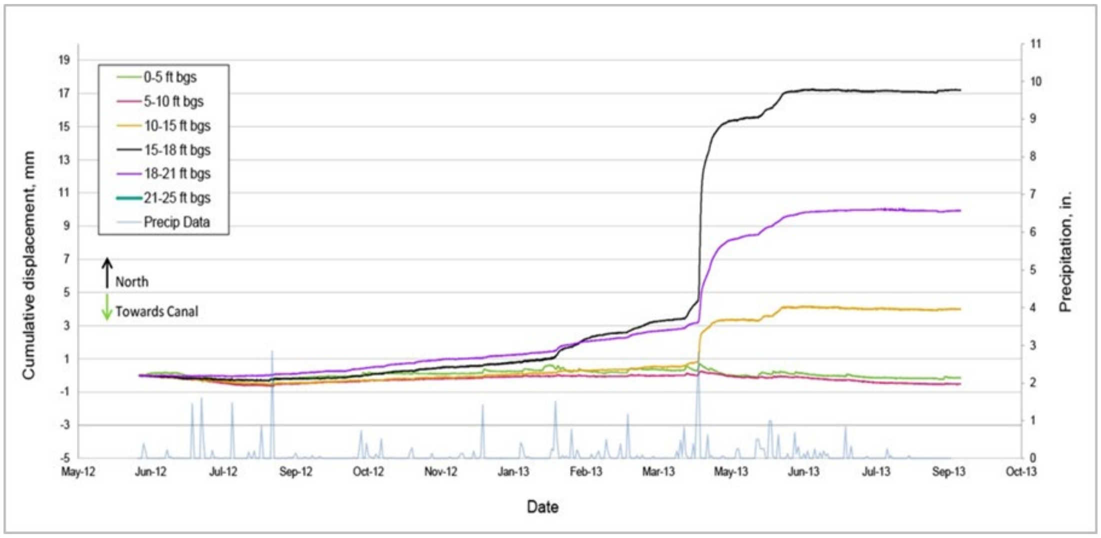


Figure 9.14. Displacement and Precipitation: 2012–2013

(4) Extensometer.

(a) Evaluation of extensometer data is concerned with extension or compression between anchor points. Within drill holes, extension occurs if a shear between anchor points becomes active or compressive stress along the axis of the drill hole is relieved. Typically, the change in length of extensometers in rock and fill differs in magnitude.

(b) Both rock, concrete, and earthfill are weak in tension. Extension displacements between anchor points in those material may be significant, depending on the relation between location, material properties, and the potential failure modes of concern.

(c) Borehole extensometers installed in a structure indicate axial extension and compression parallel to the axis of the drill hole. One end of an extensometer is anchored and considered a fixed reference point. Other anchored points are considered moveable, and distance to the fixed point for each is measured. A change in the measured distance is a measure of displacement along the extensometer axis.

(d) Accumulated extensometer displacement should be displayed as a function of depth and time. Evaluation should identify a correlation between any displacement change and occurrences including hydraulic or other load changes, construction activities, and seismic events.

(e) Figure 9.15 is a plot of an MPBX showing the displacement trends of three anchors over time. This MPBX was monitored during excavation and periodic blasting on Bench IV. All three anchors show displacement increases that correlate with blasts. The instrument head was used as the reference point in this case although, depending on the application, the stable reference point may be the deepest anchor. The small fluctuations in readings are due to diurnal temperature fluctuations. Displacement between blast events is due to creep.

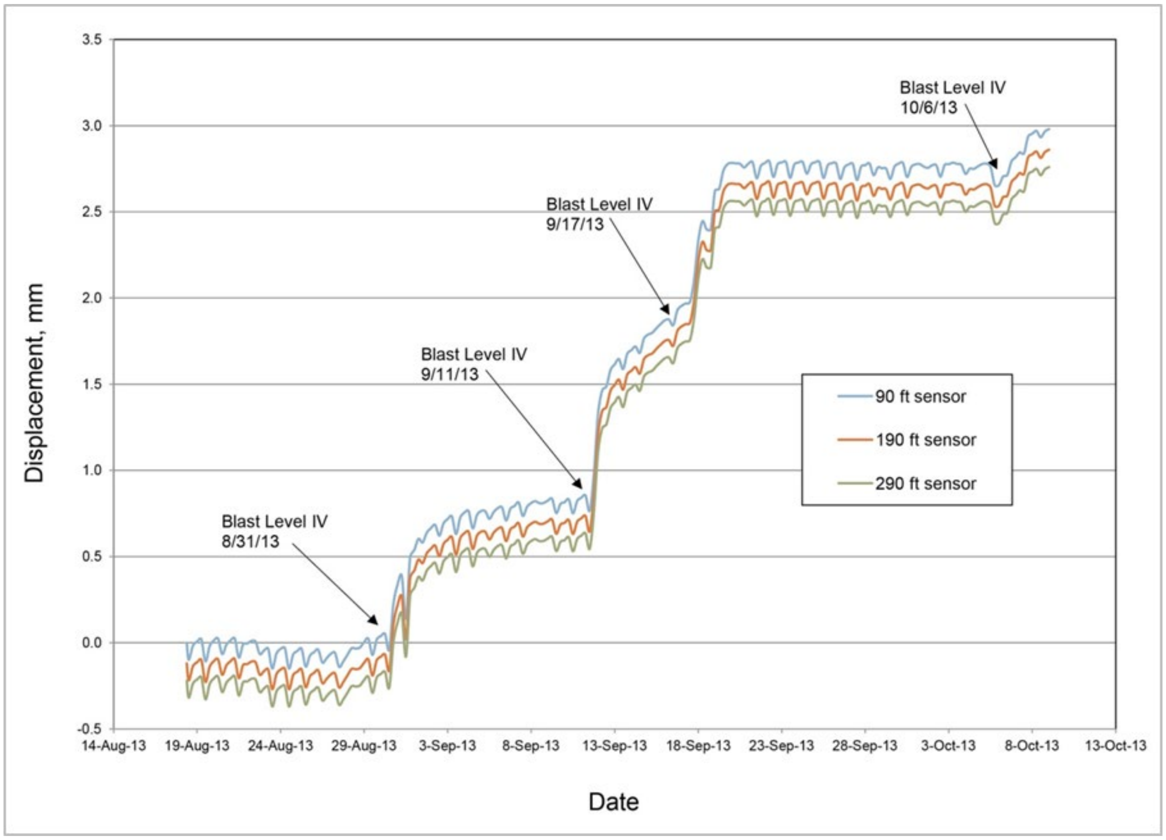


Figure 9.15. Extensometer Displacement: 2013

(f) The instrument head should be checked regularly to determine if it has been disturbed. For borehole extensometers that use wire between anchors, the possible elongation of the wire, unrelated to true displacement, should be considered. Extensometers using rods that are slender relative to the distance between anchors may buckle.

(g) Minor buckling may affect reading accuracy and disguise the earliest indications of displacement. Such instrument behavior may result in small, abrupt indications of displacement change. Some extensometers avoid these complications by directly measuring the distance between anchors with downhole sensors. Complications associated with rod corrosion should be considered for the drill hole air-water interface.

(5) TDR Cable.

(a) TDR cable measurements are obtained from a read-out measuring relative reflectance, and the results of a measurement can be plotted. The length of the cable is defined on the y-axis in terms of depth or elevation, and the relative reflectance is plotted on the x-axis.

(b) The vertical and horizontal scales should be adjusted to minimize noise signals and emphasize true reflective differences. A spike in the relative reflectance corresponds to a point of altered cable tension, indicating displacement.

(c) The direction and magnitude of displacement cannot be measured from the TDR signature. However, relative magnitudes and rates can be estimated, and the location of the zone of displacement can be accurately determined. An example read-out indicating a shear location at a depth of 134 feet (40.8 m) is shown in Figure 9.16. The depth is equal to the elevation minus 105 feet (32.0 m).

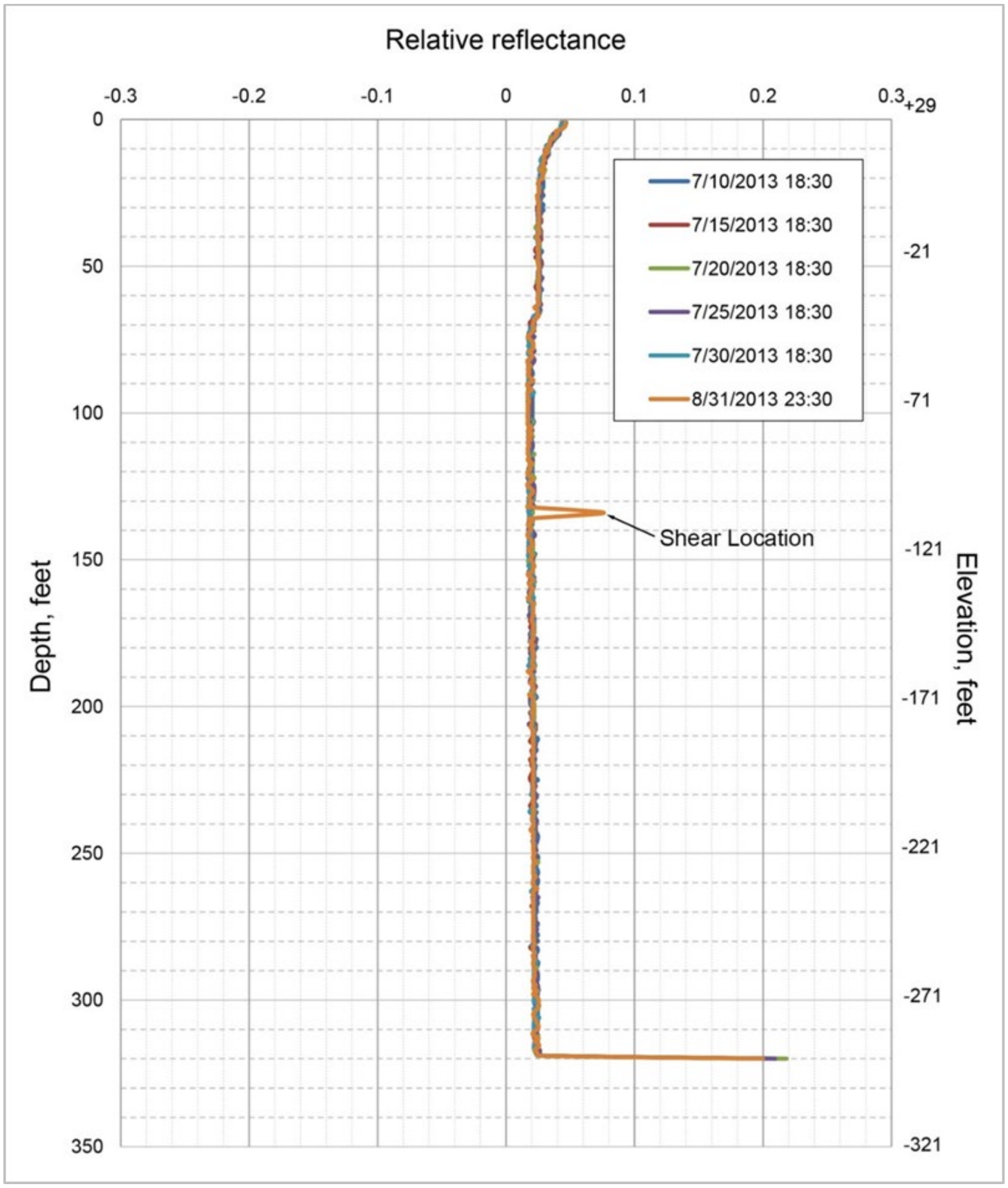


Figure 9.16. TDR Relative Reflectance vs. Depth

h. Surface Deformation on Appurtenant Structures and Floodwalls.

(1) Joint opening, offset, and crack expansion on concrete structures, such as parapet walls, spillway chutes, and stilling basin walls, could be associated with underlying weak or

eroding zones. Data evaluators should know if such displacements are increasing and if the increase is irregular, steady, or accelerating. Appropriate evaluation determines if periods of increased displacement correlate to hydraulic loading, precipitation, seepage increases, groundwater level fluctuations, or activities such as construction.

(2) Given any associated potential failure modes and anticipated directions of displacement, the evaluation should also ascertain if the instrument orientation was correct. In addition to knowing the behavior and propagation of individual cracks, the distribution of cracks should be mapped. An evaluation should direct attention to changes in crack distribution and propagation.

(3) Crack displacement may be plotted with correlated precipitation and construction events as shown in Figure 9.17, which indicates an overall trend of crack aperture growth through the entire monitoring period. However, correlated events suggest influences on crack expansion and contraction. In Figure 9.18, heavy precipitation events (highlighted in green) and cool months (highlighted in orange) correlate with an increase in crack aperture and rate of displacement.

(4) Construction activities also appear to have influenced an increase in crack growth rate. However, during warm months (highlighted in yellow) aperture size tends to remain constant or decrease slightly. The average rate of decrease in aperture size in warm months is less than the average rate of increase in cool months.

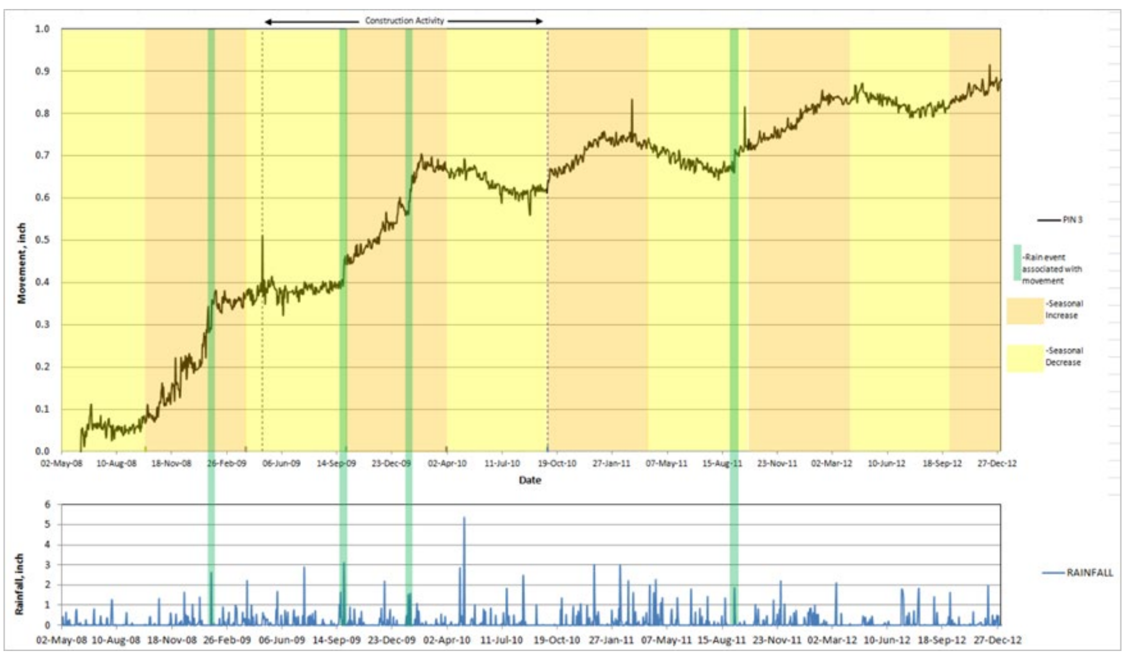


Figure 9.17. Crack Cumulative Displacement: 2008–2012

(5) In the evaluation of joint displacement, the general magnitude of measurements is typically clear. However, determining which side of a joint exhibits the greater absolute movement might be less evident. If one side cannot be reasonably assumed fixed, then geodetic surveys may be required.

9.5. Piezometric Level.

9.5.1. Piezometric level is typically measured to determine the pattern of dissipation of seepage induced pressure. Some structures and foundations feature low hydraulic conductivity cores, cutoff walls, cutoff trenches, grout curtains, and other barriers designed to significantly reduce downstream piezometric level and potentially associated seepage.

9.5.2. PZ use includes confirming design and construction effects on seepage flow direction, pressure gradients, relative transmissivity between zones of materials, and the adequacy of drainage. PZs can also facilitate the observational method of design and construction, allowing more economical construction rates and slope placement.

9.5.3. Drainage features such as chimney, blanket, and toe drains do not reduce seepage but do collect and discharge seepage in a safe, controlled manner, lowering piezometric level in specific areas. PZs in the foundation and embankment can be located to measure the effectiveness of a seepage barrier or drain. A PZ in the downstream slope of an embankment can also indicate the effect seepage or porewater pressure has on the stability of the downstream slope or abutment. An array of PZs in an embankment and foundation can indicate the seepage gradient across material interfaces, to help assess seepage related potential failure modes.

9.5.4. The dissipation of piezometric level may be broadly and evenly distributed across an embankment and foundation or may be highly concentrated in one or more comparatively small areas. The piezometric gradient may be steep or mild. A structure expected to be subjected to a high piezometric gradient should be designed to withstand the forces exerted. Little to no excess pressure should exist downstream of the toe of an embankment, which could lift the ground surface or initiate piping.

9.5.5. Pressure dissipation within the fill and foundation weakens the material subjected to the hydraulic gradient. In the plan view, PZs only provide pressure information at points. However, if embankment design and foundation geology are well understood, interpolation between individual PZs can provide contours of the pressure distribution across part of the embankment site.

9.5.6. Processing.

a. Piezometric data are most often displayed on a time series plot for comparison of the response to loading between different periods. In addition, a correlation plot displaying piezometric level against a possible driving variable, such as pool elevation, can be used to check if the response to loading is changing. Contour maps of the pressure distribution are used to identify seepage paths, sources, and areas of transition.

b. In PZ standpipes or tubes open to atmospheric pressure, direct measurements of water elevations or water depth relative to a reference point are made. The more common measurement is the depth to the water surface below the top of casing.

c. The vertical distance between the reference point from which a measurement is made and the lowest point (tip) elevation at which water can enter the PZ should be known. The standpipe top of riser and tip elevations should be periodically verified. Typically, for open standpipes, an accuracy of 0.1 feet (3.05 cm) is adequate.

d. The processing and evaluating of readings for a closed PZ bears some relation to that of an open PZ. For example, the need to know the location of tip, seals, and influence zone is much the same as for an open PZ. However, the closed PZ also requires correct labeling of tubing and cabling used to identify the PZ.

e. A precise barometric correction applied to data from an unvented, closed PZ can help the engineer identify the very beginning of a trend. Without a barometric correction, an initial apparent change should exceed approximately 0.5 psi (3.4 kPa) before being considered a true response to a change in loading.

f. Such early detection is typically desirable during first filling of a reservoir or at critical locations with very high risk of failure. Otherwise, the normal, frequent fluctuations of atmospheric pressure tend to reveal the nature of the change. If timely barometric correction beyond that available from local meteorological records is desired, a site barometer read by the same system used to read the PZs may be effective.

9.5.7. Evaluation.

a. Embankment PZs are typically used to answer at least one of three questions:

(1) How well are seepage barriers performing? (These barriers include impervious cores, cutoff trenches, grout curtains, and various types of embedded walls.)

(2) What are the pressure gradients at exit locations such as the downstream toe, across the foundation contact, and at potential locations of internal erosion?

(3) How are porewater pressures distributed within specific embankment sections or at interfaces such as the abutments? (Ascertaining the spatial and temporal pressure distributions supports a determination of long-term resistance to internal erosion and shear strength failure in the embankment and foundation.)

b. Ideally, an evaluation of piezometric level measurement should determine if conditions are satisfactory and stable, but some embankments are not in equilibrium with respect to developing seepage flow. For example, the internal pressurization of an embankment and foundation may continually increase for years after the initial external hydraulic loading is fully developed.

c. Even dams that have been loaded by full reservoirs for decades may not have developed steady-state seepage because the embankment materials have a very low hydraulic conductivity. Likewise, sudden drops in pool elevation, as when pool restrictions are implemented, might not be reflected in pressure measurements because of insufficient time to reach a new steady state.

d. Some embankments are frequently dewatered for extended periods and have never attained a steady-state internal pressure under a full hydraulic load. In addition, the susceptibility of the PZ to precipitation or surface runoff should be known.

e. Open PZs typically feature comparatively long vertical intervals, or influence zones, from which groundwater may enter the casing. To accurately interpret PZ readings, one should know the type and location of impervious seals installed to define the PZ influence zone. The seals should be able to seal tightly against the in situ materials for the intended life of the instrument.

f. The length of the influence zone should be determined as the resultant piezometric reading is determined by the various localized pressures exerted upon the cylindrical periphery of the influence zone. Piezometric level is typically assigned to the midpoint elevation of the influence zone. However, the influence zone may intercept a crack, joint, or bedding plane that can significantly influence the PZ reading, and it may be appropriate to associate the piezometric level with that interception location.

g. Time Series Plots.

(1) A time series plot of piezometric level or elevation, shown in Figure 9.18, is the most fundamentally informative display of pressure data. A time plot shows pressure fluctuations and trends and response rate to a change in loading, such as reservoir level. Loadings that affect piezometric values are typically shown on a time plot of piezometric data. Response at a PZ is usually a function of the difference between the elevations of headwater and tailwater. A load and response of comparable magnitude should be plotted on the same scale.

(2) Whatever scale is chosen, piezometric variations as small as 0.5 foot (15.2 cm) should be apparent, allowing barometric change to be evident and the response rate to be estimated by eye. Data obtained from multiple PZs can be plotted on a single plot if the individual plots can be distinguished. However, PZs plotted on one plot should have something in common such as PZ tip elevation and location and/or monitoring purpose such as drainage system or seepage barrier performance.

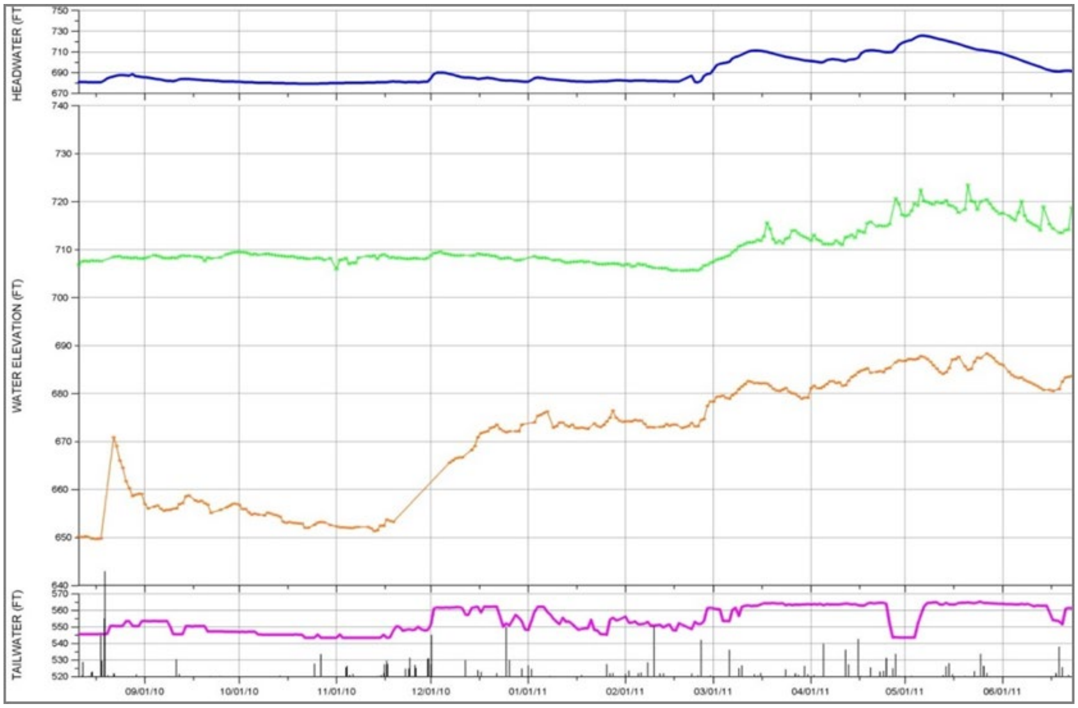


Figure 9.18. Time Series PZ Levels and Headwater and Tailwater Elevations

h. Correlation Plots.

(1) A correlation plot of response versus loading can be helpful in evaluating a PZ. If an instrument reading has been highly correlated with a variable, such as headwater, and that correlation changes drastically with time, in situ conditions or the instrument itself may have changed. Correlation changes over the range of the response may point to elevations at which the embankment or foundation transmissibility are greater.

(2) To be truly useful, correlation plots should clearly indicate extent of time and separate plots for particular periods. A single plot identifying different periods with different line types or colors may be necessary. Annotation should be included indicating the data progression over time.

(3) Properly devised correlation plots may show that other factors besides the presumed loading influence the PZ. Therefore, correlation with other possible loadings such as tailwater, groundwater, and precipitation should be considered in evaluating the instrument. Increases and decreases in the response rates can be shown clearly and quantified using the correlation plot.

(4) Correlation plots can also pinpoint how changes in response rates may be associated with particular hydraulic loading elevations. For example, when a rising pool surface elevation reaches the elevation of a foundation gravel layer, downstream PZs may exhibit an increased rate of response. Linear regression may predict the response of the instrument beyond its maximum historical loading.

i. Seepage Barriers.

(1) Pervious embankments, foundations, and abutments may include seepage cutoffs to reduce seepage flow and pressure. A seepage barrier may be a wall, grout filled fissure, or a backfilled trench and is intended to decrease the hydraulic gradient by forcing most seepage to follow a longer path.

(2) If the seepage barrier is efficient, the piezometric level on the upstream/waterside face of the seepage barrier is almost as great as the source pressure, and the piezometric level on the downstream face or landside slope may be much lower than the source pressure. This results, however, in an increased gradient across any flaws that may exist within the seepage barrier.

(3) Figure 9.19 is a time series plot for an example dam. Figure 9.20 includes two correlation plots of the PZ responses downstream of a seepage cutoff wall. The PZ, located downstream of a newly installed cutoff wall, correlated well with headwater prior to cutoff construction.

(4) Immediately following construction, the head loss across the wall increased to 30 to 57 feet (9.1 to 17.4 m) from the 20 to 26 feet (6.1 to 7.9 m) of loss that was typical prior to construction. The PZ shows a significant decrease in the response to headwater post-construction.

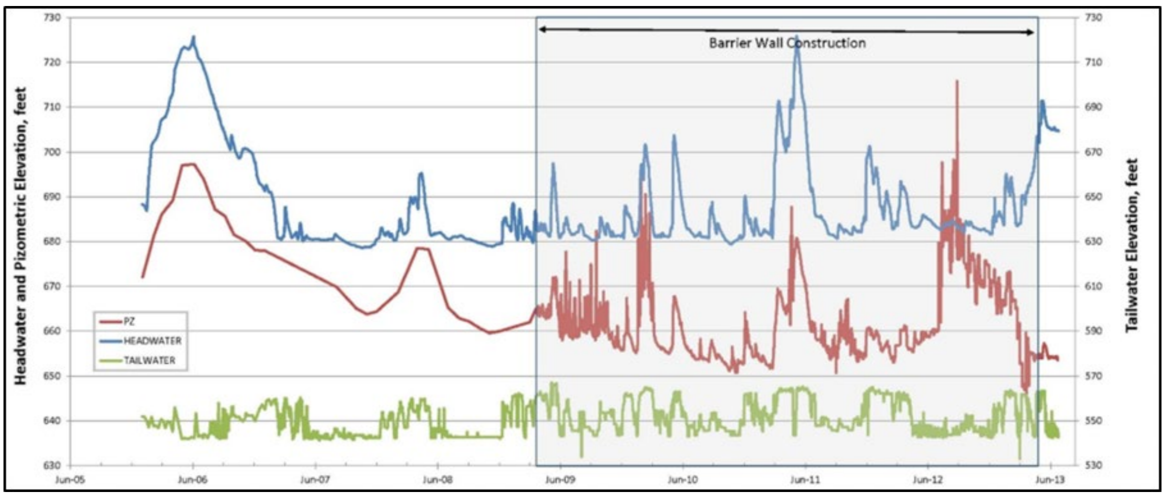


Figure 9.19. Piezometric Drop After Cutoff Wall Installation

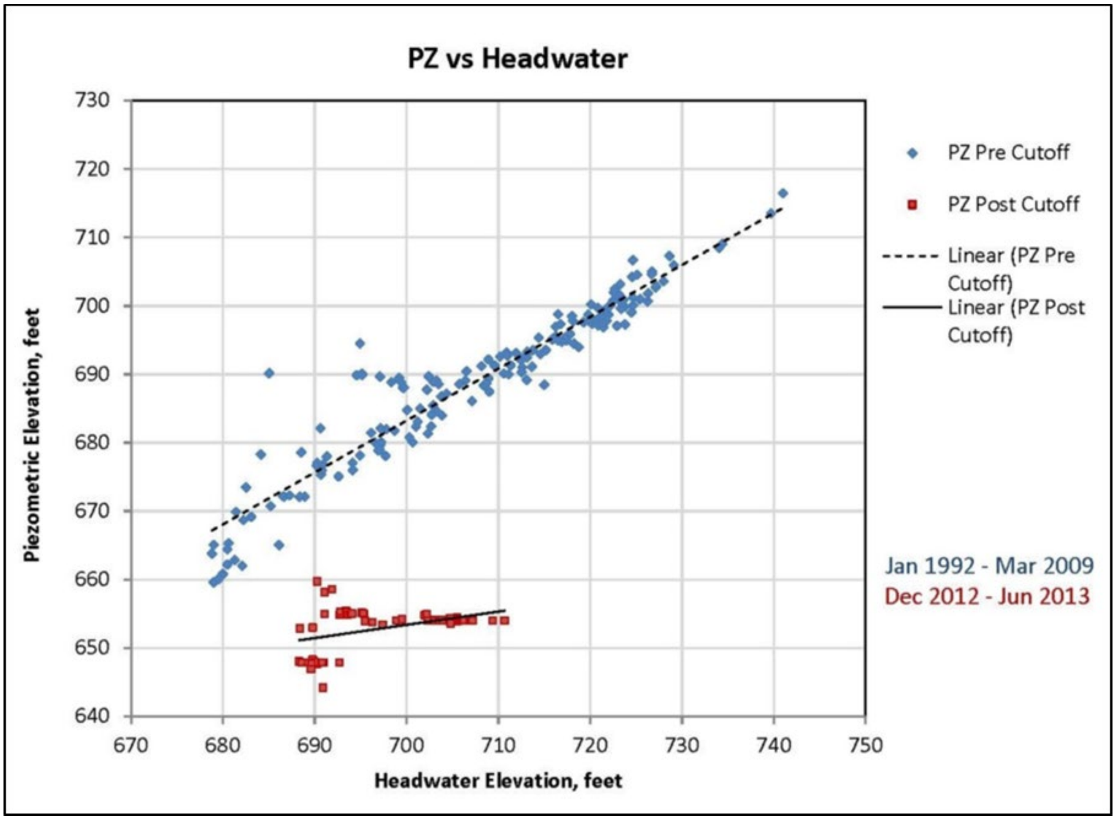


Figure 9.20. Correlation Plots Pre- and Post-Cutoff Wall Installation

(5) An evaluation of a newly installed seepage barrier should determine the amount of pressure reduction achieved. Seepage barrier efficiency is expressed as a percentage reduction of the head in excess of the base level, which is usually the tailwater/landside toe elevation. A significant reduction in seepage flow requires a seepage barrier efficiency greater than 50% (Cedergren, 1967).

(6) Grout curtain cutoff efficiency may be difficult to evaluate because a PZ may not be clearly upstream/waterside or downstream/landside of the grouted zone. For example, a PZ located within a wide grouted zone may indicate that pressure has not been significantly altered by grouting. However, the seepage quantity may have been greatly reduced.

(7) Figure 9.21 shows how grout distribution may not affect foundation piezometric level even though the cutoff is effective in reducing seepage flow. For the narrow curtain in Figure 9.22 (b) a large piezometric level drop indicating an effective cutoff is shown. The wide curtain shown in (c) may also be an effective cutoff, but it has no significant effect on foundation pressure distribution.

(8) Evaluation of the curtain efficiency should consider the possibility of pressure measurements being made inside rather than outside the grouted zone. A PZ intended to monitor grout curtain efficiency should be installed after the barrier is completed because the results are more conclusive. For example, the PZ drill log should note where grout material is encountered, providing information needed to judge the location of the PZ with respect to the grout curtain.

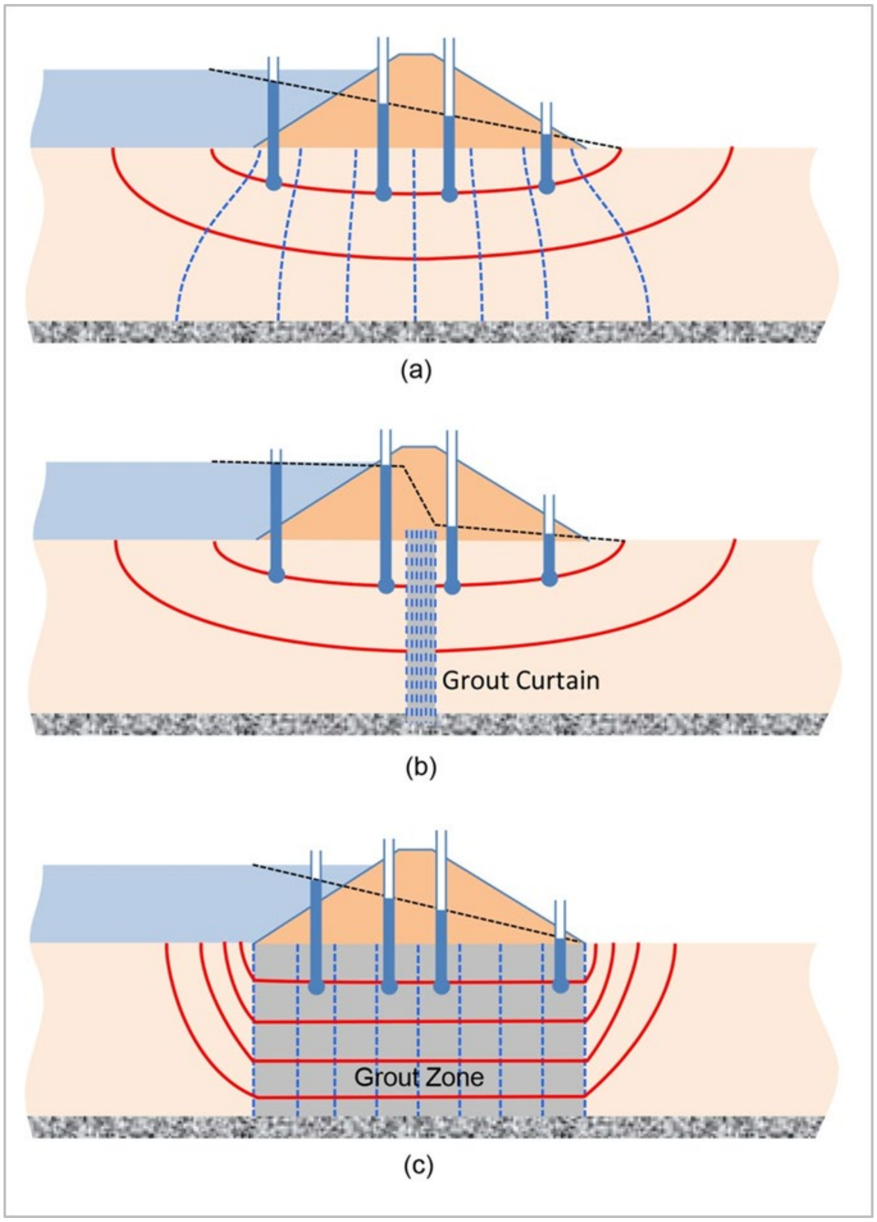


Figure 9.21. Section View of Piezometric Response to Grouting

j. Seepage Collection.

(1) PZs located near or in seepage collection drains can be used to judge drain adequacy. An evaluation should consider whether the monitoring PZ is inside or outside of the drain. If a PZ is installed in a drain consisting of an open pipe or pervious fill, the piezometric level should not significantly exceed elevation of the drain and the drain should not be full of water unless the drain is pressurized by design.

(2) For a PZ installed outside a drain, the pressure near the drain should only slightly

exceed pressure in the drain. However, if the drain capacity is exceeded, pressure will increase in the drain and surrounding material. The internal flow from a clogged drain may be less than normal while an increased pressure exists near the drain. Therefore, effective monitoring of seepage collection drains is based on both pressure and flow data.

(3) Relief wells are typically vertical drains placed to relieve uplift pressure at the downstream toe of an embankment. PZs are typically located equidistant between two wells. Under maximum hydraulic load, the piezometric level at the downstream toe of an embankment should be lower than the ground surface.

(4) EM 1110-2-1914: Design, Construction, and Maintenance of Relief Wells specifies the minimum permissible vertical interval between the ground surface and the piezometric level. The permissible interval varies with the type of foundation material. An increase in the pressure response to a given load over time indicates an eventual need for well maintenance. The approximate date at which maintenance should be performed can potentially be forecasted by extrapolation.

k. Cracking and Internal Erosion.

(1) PZs may be located in areas of an embankment or a foundation where uncontrolled drainage associated with cracking or internal erosion is a concern. Whether or not pressure existed previously, changes occurring independently of the loading may indicate internal erosion or cracking. Pressure distributions should be evaluated for a significant change from the normal pattern. The ratio of the transmissibility of the areas immediately downstream and upstream of the PZ determines the piezometric level indicated.

(2) If a crack or void develops and cannot drain freely, it may be subjected to increased pressure. An evaluation should determine if unexpected changes in PZ readings are due to an increase in transmissivity of material downstream or upstream of the PZ. Either case should be evaluated.

(3) Too often, staff are overly concerned with monitoring for pressure increases. In fact, depending upon the location, the formation of a crack or void may cause an unexpected lowering of pressure. More embankments have failed due to internal erosion than to slope failure.

(4) Piping, as shown in Figure 9.22, is a consequence of uncontrolled drainage. As piping progresses, drainage continues. Therefore, unexpected pressure decreases can be a sign of piping. A crack or erosion pipe that may or may not resist erosion can be expected to lower pressure upon connection with the downstream end of a seepage path.

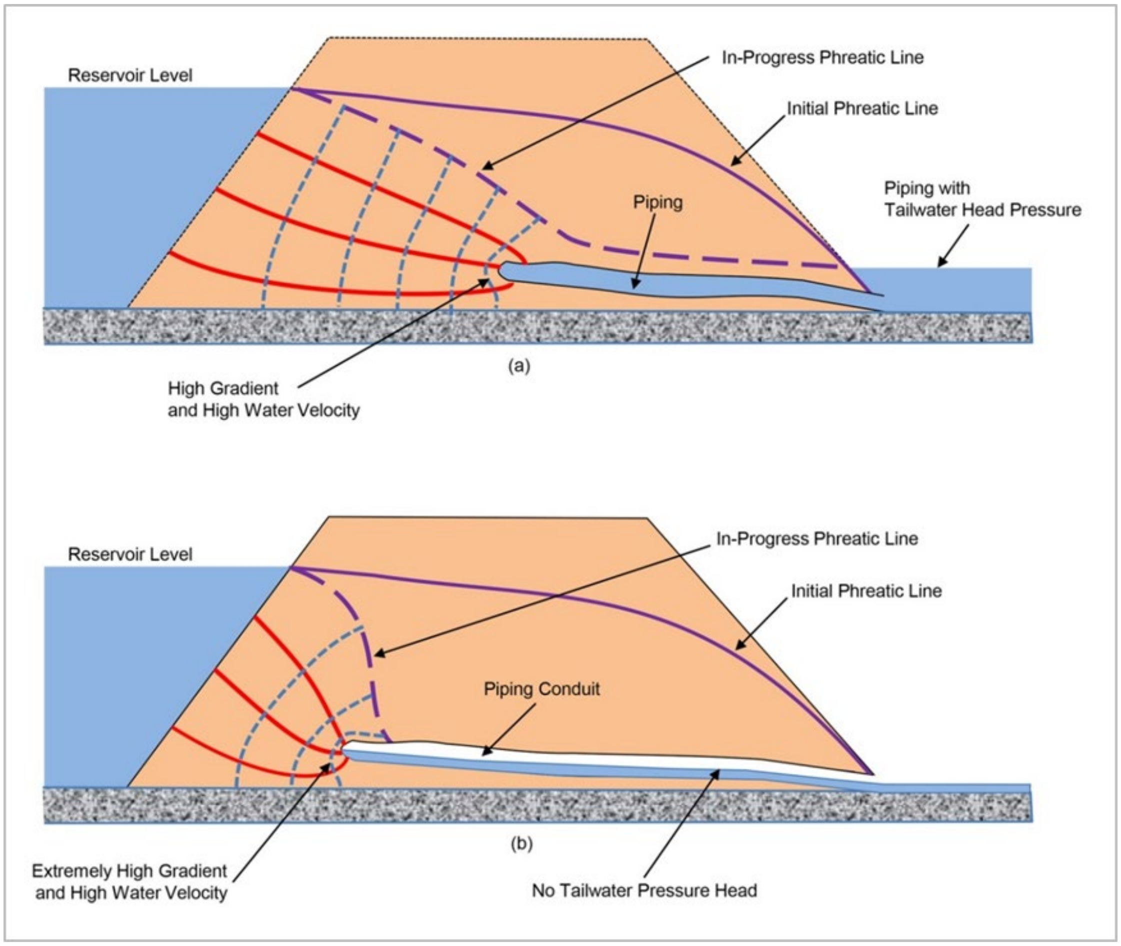


Figure 9.22. Section View of Piezometric Level Decreases Indicative of Piping

l. General Pressure Distributions.

(1) Placing PZs in an array facilitates mapping a general seepage flow pattern. An array of PZs can monitor the efficiency of cutoffs and drains and the relative transmissivity of adjacent embankment zones and foundation layers over a large area. Data collected from a PZ array can:

(a) Verify design assumptions regarding material properties and feature designs.

(b) Aid evaluation of abnormal data.

(c) Allow continuation of monitoring even if some instruments become inoperable.

(d) Evaluate water tightness.

(e) Estimate the vertical to horizontal hydraulic conductivity ratio.

(f) Evaluate the degree to which one zone or feature drains another feature.

(2) Pressure Distribution in Section.

(a) With adequate coverage, the data from a PZ array can be used to generate diagrams of pressure distribution and flow nets for a section of embankment. Figure 9.23 shows an example of piezometric data used to generate or calibrate equipotential lines in the development of a flow net for seepage through a central core.

(b) Connecting points on the figure that have equal sums of pressure and elevation generates the equipotential lines as illustrated at two locations in the figure. Even without transforming vertical and horizontal sectional scale to reflect the ratio of vertical to horizontal hydraulic conductivity, information to determine whether flow is directed into the foundation or from the foundation into the embankment is readily available.

(c) If needed, using a properly transformed section to generate a flow net with flow lines perpendicular to the equipotential lines yields a better estimate of flow direction and the pressure gradient across material boundaries. If known, gradients across foundation and embankment material boundaries should be reported to facilitate estimates of the potential for internal erosion.

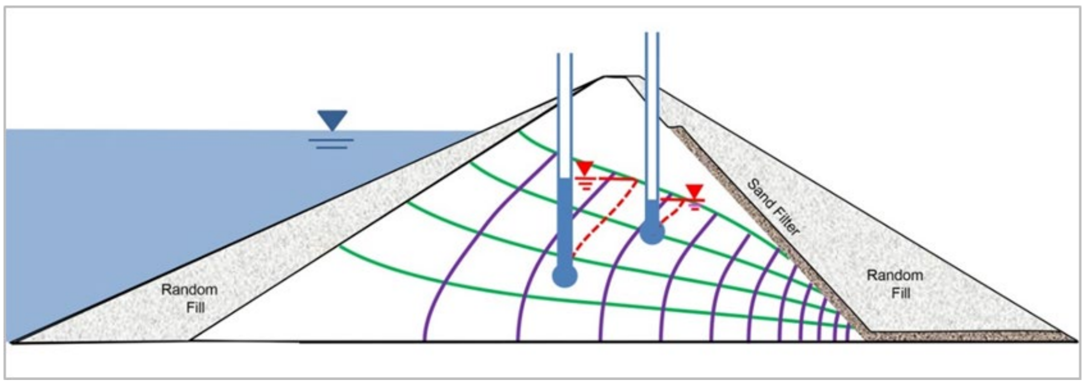


Figure 9.23. Section View of Piezometric Data and Flow Net

(3) Pressure Distribution in Plan.

(a) Figure 9.24 is a plan view of an embankment and shows lines connecting points of equal pressure. In this example, all PZs are tipped at similar elevations within the foundation material. The PZ contours are shown for a pool elevation of 1,400 feet (426.7 m). Such pressure contours are best used to develop a sense of where seepage drains to and also how shear strength may be affected along a particular elevation.

(b) In Figure 9.25 it appears that seepage flows toward the maximum section and downstream within the maximum section. Where embankment stability is a concern, cross-section and profile views of the pressure contours are needed.

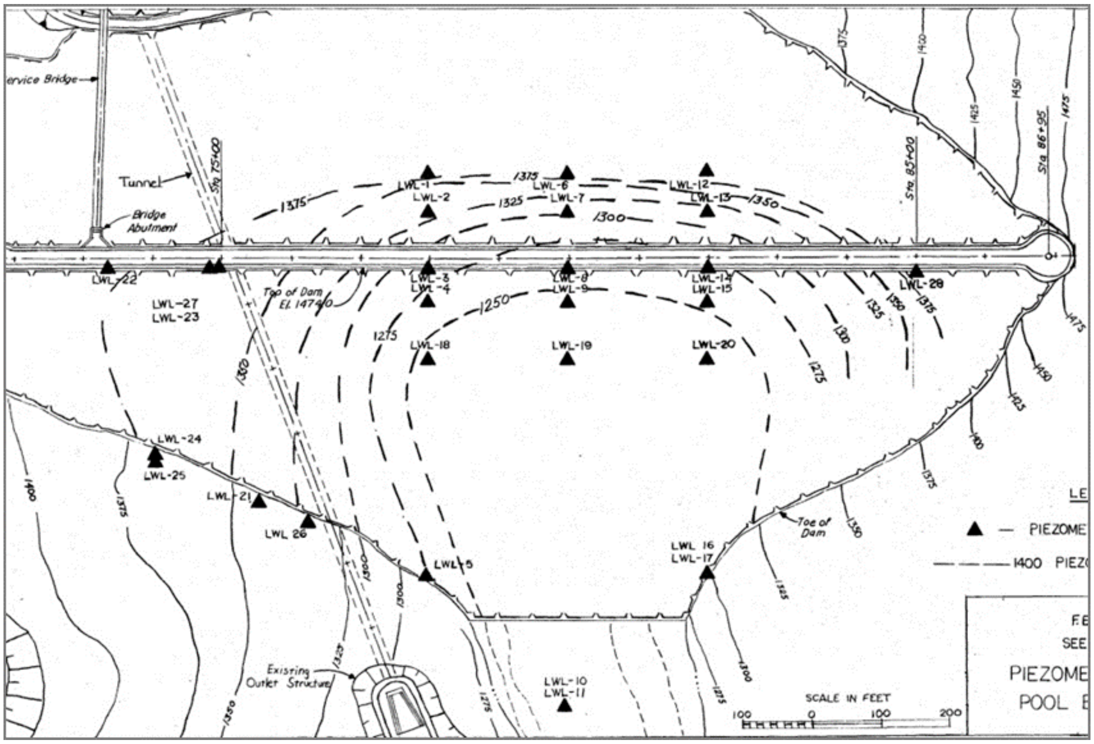


Figure 9.24. Plan View of Embankment and Pressure Diagram

(4) Construction.

(a) Earthfill placed with a high-water content may saturate during the first phases of consolidation that occur as the embankment is raised or shortly after completion of construction. Saturation during consolidation could lead to excessively high porewater pressure and slope failure. Therefore, monitoring porewater pressure may be necessary to ensure that the rate of fill placement is not excessive.

(b) Proper evaluation requires knowledge of the construction stage stability analyses, which estimate the maximum acceptable porewater pressures and determine if pressures will exceed limits for a specified construction sequence. If pressure measurements indicate a danger of exceeding the porewater pressure limits, slowing or temporarily halting fill placement may be required. After the porewater pressure has stabilized or dissipated, the fill placement may resume with continued pressure monitoring.

(5) Slope Stability.

(a) Concern about pressure increases are historically related to slope stability. Typically, the critical section is the section of maximum height but could be a section of lesser height having questionable foundation strength. The monitored pressures indicate the porewater or neutral stress portion of total stress.

(b) Typically, a PZ array is installed primarily in the downstream shell but occasionally extends to or includes the upstream shell if a rapid decrease in the external hydraulic loading is a concern. Once instruments indicate steady-state seepage and the embankment has endured maximum loading or unloading, the value of continued monitoring should be evaluated.

(c) If the results of a stability analysis are available, the piezometric levels used in the calculations should be compared to pressure readings. The comparison and evaluation should state whether the values used in the calculations are exceeded and, if so, the magnitude of exceedance. Updated analysis may be needed if calculated pressures are significantly exceeded.

(d) Seepage and stability analysis could be used to inform selection of instrument thresholds. If seepage has not reached a steady state, pressure trends (of at least the higher-pressure PZs) should be extrapolated to estimate the potential of exceeding the values used in the stability calculations.

(e) A rapid lowering of the hydraulic load on an embankment can be critical if the upstream slope material or foundation does not drain freely. Pressure in the upstream slope may destabilize the embankment as the reservoir pool is lowered. If the potential for failure exists, monitoring internal pressures in the upstream slope may be required during drawdown of the pool. An evaluation should take place during a drawdown by comparing the measured porewater pressures to the porewater pressures used in a stability analysis.

(f) Many embankments feature a downstream random or miscellaneous fill zone to increase slope stability. Either the material is free draining or drains are installed to prevent the material from becoming saturated. An evaluation of the degree of drainage for various loadings can be made with PZs located in the lower portions of the zone to indicate the presence and depth of water.

(g) Evaluations of piezometric level often involve extrapolating the existing response to design loading. Typically, linear regression of pressure versus pool level is satisfactory. If time lag is significant due to low hydraulic conductivity, a series of flow nets can be a helpful evaluation tool. The flow nets assume a steady state for each loading level of interest.

(h) In addition to magnitudes, historical readings and trends, evaluations of individual PZs should establish how porewater pressures relate to the elevation of natural and structural features. Such features may include ground level, structure base, the embankment/foundation contact, zone interfaces, drain margins, weak or pervious layers, and normal piezometric levels for maximum loading.

(i) If PZs provide data on both sides of an earthen boundary, the pressure gradient across the boundary should be estimated. Internal seepage pressure gradients should be estimated for materials thought to be susceptible to erosion or supporting piping paths.

m. Temperature.

(1) Subsurface temperature data can aid the evaluation of piezometric level data. Vibrating-wire PZs include a temperature measuring element. PZs deeper than 60 feet (18.3 m) can generally be expected to experience constant temperature. Therefore, for embankments that hold water, fluctuations in temperature at those depths are likely to be related to changes in seepage flow.

(2) Depending upon the site latitude, PZs embedded between 10 to 60 feet (3.05 to 18.3 m) below the ground surface can be expected to experience consistent annual temperature cycles, although seepage from impounded water can diminish or expand those cycles. Instruments installed within 10 feet (3.05 m) of the ground surface are likely to experience daily temperature variations difficult to evaluate.

(3) Although the relationship between temperature and depth is a function of rock type and ground surface conditions, predictions are usually determined with temperature sensors installed below the depth of diurnal variation and at points where seepage does not influence temperature. Confirmed variation from a predicted temperature pattern is likely caused by seepage flow.

(4) In higher latitudes including most of the continental United States, seepage can either deliver heat to or remove heat from in situ materials, depending on the season. Drilling or grouting operations in rock may temporarily change the temperature of the rock if surface water is used in the work and the temperature of the surface water is different from the temperature of the rock. A temperature variation that is contrary to that expected for the season or coincides with intrusive construction activities in the foundation may need to be evaluated.

(5) Seepage flow need not be in contact with a PZ to affect recorded temperature. An evaluation should consider if PZ temperature appears to vary between an in situ temperature unaffected by seepage and the temperature of a possible seepage source such as the reservoir.

n. Tides.

(1) At and near the coast, oceanic tides can significantly affect dams and levees seepage flow and pressure. Tides can periodically increase internal piezometric level, cause oscillations of the seepage pressure gradient, and even reverse the direction of foundation seepage. Therefore, understanding tidal patterns and associated effects can be key to the safe operation of embankments near sea level on the coast. For example, awareness of tides can prevent misguided interpretation of instrument readings.

(2) Tides may be diurnal (one cycle per day) or semidiurnal (two cycles per day). In North America, the semidiurnal pattern is prevalent on the Atlantic and Pacific Coasts. However, the Gulf Coast has essentially a diurnal cycle. Tide schedules are available from the National Oceanic and Atmospheric Administration (NOAA). NOAA provides accurate predictions for particular stations. Due to topography, the tide magnitude and timing between the project and nearest NOAA station can be expected to vary somewhat; however, the tidal patterns will be similar.

9.6. Total Stress. Total stress is typically measured to determine the stress on a potential shear plane within the fill or between the fill and an embedded structure. Effective stress can be determined by subtracting piezometric level from total stress.

9.6.1. Processing. A total stress cell, either structural or embedded, measures pressure within a fluid filled disk. Some of the concerns regarding the processing of piezometric measurements apply also to total stress cells.

a. Stress History.

(1) Stress history plots show changes in stress over time. Identifying pre- and post-construction periods on a stress history plot is useful. Post-construction changes in stress are likely to be smaller than changes during construction. Therefore, the vertical scale of stress history plots for the construction and post-construction phases may need to be different.

(2) Figure 9.25 illustrates the history of soil stress as a dam is constructed and the reservoir is filled. The horizontal axis is time from the year 1983 through 1988. The left vertical axis is the elevation in meters of the embankment fill and the reservoir pool. The right vertical axis is soil pressure in kilopascals. The soil pressure increases as the embankment is raised, but the pressure does not change after placement ceases and continues unchanged as the reservoir level rises.

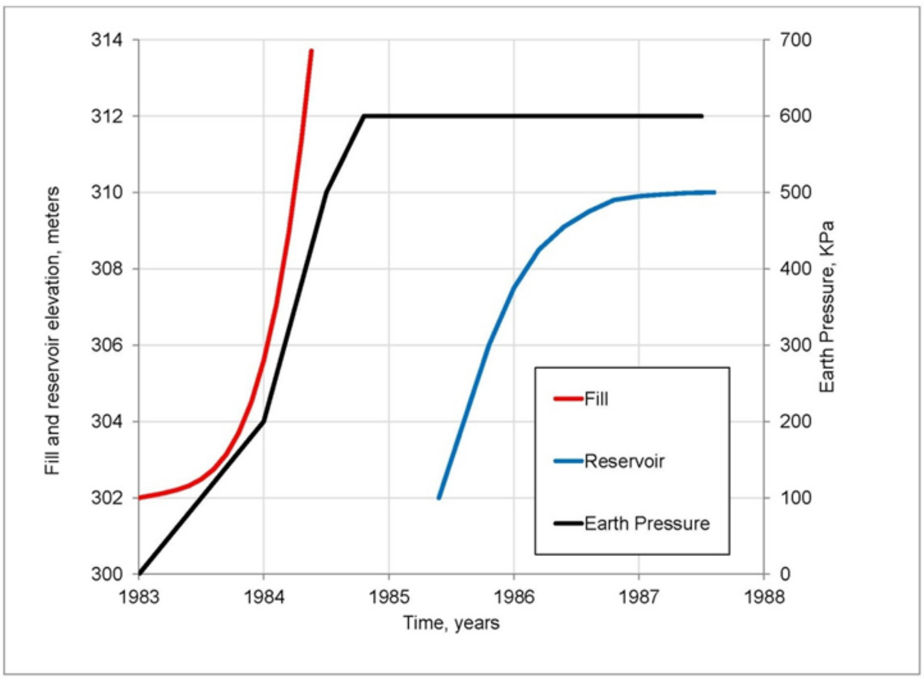


Figure 9.25. Pressure vs. Time (Adapted from ASCE Guidelines, 2000)

b. Correlation. If stress variations occur, correlating associated loads such as pool surface elevation or height of fill during construction may help interpret the indicated change. For example, a steadily dropping cell pressure may indicate a leaking cell disk. If the changing readings are not correlated to operational loads and no other indications of changing stress or deformation exist, those readings may not be reflective of project performance.

c. Installation Records. Installation records should be reviewed for structural contact pressure cells. Readings from cells that are not flush with a structural surface may be misleading. Cells set within the structure surface are likely to under-register contact stress. Alternatively, cells that protrude from the structural surface may over-register stress. For both types of cells, if the density of the encompassing specially compacted fill does not equal the general fill density, then under- or over-registering of stress occurs.

d. Verification. Because a field measurement of stress can be inaccurate, assessments of the inaccuracy should be based on control cells placed at locations where stress can be reliably estimated by theory. For example, a cell that solely measures overburden weight at a point of geostatic stress provides a check on the reliability of stress readings for other orientations. For embedded and structural contact stress measurements, several cells should be located where the readings are expected to be the same.

9.6.2. Evaluation.

a. Contact stress measurements should be compared to the stresses expected. Evaluation of vertical total stress is a simple comparison of the cell readings to the calculated weight of overburden. The weight of overburden cannot be accurately estimated unless the unit weight and dimensions of the overburden are accurately known. If the fill density surrounding the stress cell conforms well to the general fill density, the calculated and measured pressure should agree within approximately 10%.

b. Measures obtained from multiple total stress cells in a location help quantify random errors although systematic errors, such as the stress field disturbance due to installation of the instrument, may still exist. The variance between the readings of multiple cells in locations of equal stress helps quantify measurement accuracy.

c. An estimate of principal stress is useful for an evaluation of stability. Principal stress can be calculated from a rosette of three or more embedded pressure cells by constructing a stress ellipse, which yields both a principal stress magnitude and a direction. Plotting pool surface elevation against effective stress indicates how shear strength varies with reservoir operation. Judgment is required to determine if the calculation of principal stress is appropriate based on the degree to which total stress readings conform to expected values.

d. Lower than expected effective stress may be related to seepage and a potential for internal erosion. Low contact stress between an embankment and a rigid structure may be a serious problem if seepage occurs. Low effective stress may be reduced to zero by piezometric level related to seepage. If effective stress becomes zero, a contact between an embankment and a structure may open, increasing seepage flow along the structure.

e. The total stress measured by a pressure cell is typically constant once fill placement is complete if live loading is insignificant. If the effective stress component is reduced to zero, an increasing piezometric level registers on the total stress cell. At that point, the cell begins to behave as a PZ.

f. If structural displacement is monitored, earth pressure cell data may be correlated to displacement to assess structural response to the applied stress.

9.7. Data Evaluation General Considerations.

9.7.1. A thorough evaluation includes both an evaluation of the project performance and monitored potential failure modes as well as the effectiveness of instrumentation and monitoring program. The surveillance and monitoring plan should be checked and updated as needed during the annual instrumentation review, periodic inspections, high-water performance report, and any risk assessment.

9.7.2. Any data evaluation should include consideration whether studies, repairs, rehabilitation, or replacement are recommended for the structure or instrumentation and monitoring program. A significant change in instrument response or signs of distress may also require an evaluation of the monitoring system as a whole.

9.7.3. Although data evaluations for individual instruments are likely to provide examples of the strengths and shortcomings of the general system, a summary of the individual assessments and an evaluation of the system is needed. System evaluations assess if data and visual observations accurately indicate the performance of the project and if the information is collected, recorded, and reviewed appropriately.

9.7.4. Each instrument location and purpose should be discussed thoroughly. The evaluation should state if the documentation of the system purpose, construction, installation, and rehabilitation of devices, as-built conditions, and data acquisition are appropriate.

9.7.5. An effective evaluation addresses data quality, the complete history of each instrument, the significance of periods of poor or missing readings, and other anomalies. The system evaluation should address visual observations, data collection, reviews, and ongoing data evaluation.

9.7.6. Data Statistics.

a. Descriptive statistics help the engineer or geologist grasp the range, central tendency, and variability of the data and can be quickly and accurately determined with computer software. At times, an experienced member of the project staff can immediately detect significant or faulty data by glancing at a summary table of descriptive statistics such as the maximum, minimum, mean, and standard deviation.

b. The development of inferential statistics typically requires the involvement of skilled staff. An extrapolated trend line of the values of a variable is an example of a simple interpretive statistical product easily calculated by a computer, but the skilled professional is needed to judge the appropriateness of the type of curve selected for the trend line, the determination of the physical limits associated with the curve, and the range of values the curve represents accurately.

c. More sophisticated types of interpretive statistics include multivariable linear regression. Consideration should be given for tailwater or landside water surface backing up on the toe of the embankment which may change the correlation trend during higher loadings.

9.7.7. Unusual Readings.

a. Unusual readings should be verified by ensuring that reading procedures were properly followed and that the instrument is functioning properly. In instances of unexpected and obviously impossible readings, data collection and processing should be checked and a visual inspection performed in the vicinity of the instrument. Faulty instruments should be identified and repaired, replaced, or abandoned as appropriate. The instrumentation data should be evaluated to determine if:

(1) Additional data are collected to verify the correctness of anomalous data, and the information is immediately reported through established channels for further investigation.

(2) Data or observations appearing to indicate unusual or unacceptable dam/levee performance are thoroughly investigated and documented by specialists in collaboration with personnel from the office that submitted the data.

(3) Systemic or random error of the instruments can be attributed to the unusual readings.

b. If monitoring thresholds are exceeded or other unusual readings, the DSO/LSO and Operations staff must be notified and additional investigations should be communicated and documented. Refer to USACE Engineer Circular 1110-2-6074 for additional incident reporting requirements.

9.7.8. Recordkeeping.

a. Records needed for evaluation include:

(1) Historical performance monitoring data;

(2) Calibration and maintenance activities;

(3) Description of modifications and repairs;

(4) Other existing performance monitoring review reports; and

(5) Environmental factors that affect embankment and instrument behavior, such as precipitation, temperatures, and upstream/waterside flood elevation.

b. All instrumentation drawings showing locations and subsurface conditions pertinent to future monitoring should be reviewed and a determination made whether new drawings are needed, and, if so, the existing drawings can be revised. Annual instrumentation reviews should reassess if additional drawings, cross-sections, or analyses are needed.

c. The surveillance and monitoring should be evaluated to determine if:

(1) All performance data, visual observations, and supporting information are recorded and maintained as official records.

(2) Procedures are established such that instruments and systems are periodically inspected, maintained, and calibrated and that the performance of those activities is documented as official records.

(3) All records of essential details concerning repairs, modifications, and the installation of new instruments are maintained.

(4) Effective ongoing comparisons are made between collected data and visual observations and relevant previous data and observations.

(5) All measurements and observations obtained by personnel conducting performance monitoring activities at the site are compared with previous data and reviewed for unexpected changes or anomalies.

(6) Comparisons and reviews are made as data are collected at the site or as soon as practicable.

(7) Performance monitoring records are reviewed by a qualified and experienced embankment and instrumentation specialist.

9.7.9. Conclusions. Conclusions should state the adequacy of identification and installation details such as surveyed locations and as-built drawings for all instrumentation. Conclusions should state whether:

a. The project is performing as per design or if there are any indications of concern.

b. There is any indication of potentially concerning data related to monitored potential failure modes.

c. Significant potential failure modes are adequately monitored or include a recommendation for additional instrumentation or adjustment to monitoring frequencies.

d. Data are collected from properly functioning and calibrated instrumentation systems conforming to accepted standards and practices and at the prescribed frequencies.

e. Procedures are established ensuring monitoring devices and systems are periodically inspected, maintained, and calibrated.

f. Execution of procedures is documented and reported.

g. Repairs, rehabilitations, and replacements are timely.

h. All essential and significant details concerning repairs or modifications to existing devices or the installation of new devices are recorded and maintained as official records.

i. There are any recommended actions to be performed.

9.8. Reporting Instrumentation Results.

9.8.1. Effective communication of data evaluation is essential for assessing and managing risks associated with the project.

9.8.2. For newly completed or a major modification projects, a report documenting the performance of the structure during and after construction. The report must comply with ER 1110-1-1901: Project Geotechnical and Concrete Materials Completion Report for Major USACE Projects.

9.8.3. In addition to the detailed data evaluation reports included within Periodic Inspections and risk assessments, and annual reporting required by ER 1110-2-1156: Safety of Dam: Policy and Procedures, more frequent routine ongoing evaluations should be made and recorded in a timely manner.

9.8.4. The frequency of evaluations and level of detail should be commensurate with the level of risk associated with the project. Routine evaluations primarily address whether performance parameters are within expected limits and note trends. An effective instrumentation report alerts operators of impending problems and states whether readings, observations, and evaluations have been performed at the expected frequency.

9.8.5. The report typically includes graphs comparing the most recent readings to those of the past 5–10 years and states if the response to driving variables is consistent. Driving variables, such as reservoir level, precipitation, and temperature should also be shown, along with the response data on time series plots.

9.8.6. If significant performance deficiencies are not apparent, a narrative associated with the graphs is typically one or two pages in length depending on the size of the project and number of instruments. Ongoing evaluation is most effective if the reader can reference more detailed documentation that documents instrument types, use, location, and expected parameter limits. See Appendix D for guidance on developing an instrumentation report.

Chapter 10
Instrument Maintenance

10.1. Introduction.

10.1.1. Instruments require maintenance to be accurate, reliable, and have the longest possible life. Maintenance is required even though an instrument has been thoroughly checked before, during, and shortly after installation. Careful attention to factory calibration, pre-installation acceptance tests, installation, and post-installation acceptance tests should ensure that a recently installed instrument is operating well. However, even the best instruments may go out of calibration or deteriorate.

10.1.2. Instrument maintenance is typically the responsibility of the project owner or as specified in the project partnership agreement. Maintenance requirements and procedures should be planned during the instrumentation system design and outlined in a project O&M manual. An O&M manual should include or refer to the instrument manufacturer's instruction manuals. Maintenance procedures should be based on the manufacturer's instruction manual and on specific project site conditions.

10.2. Maintenance Schedule. A schedule should be established for the regular budgeting of instrument maintenance. Maintaining instruments on schedule increases project operational reliability. An appropriate maintenance frequency depends on the type of instrument, the application of the instrument, and the operating environment. The specific benefits of regular instrument maintenance may include:

10.2.1. Detecting or preventing the collection of faulty data.

10.2.2. Identifying the need to repair an instrument before a critical operational condition occurs.

10.2.3. Avoiding the loss of an instrument.

10.3. Maintenance Contract.

10.3.1. Instrument maintenance may be accomplished by the personnel of the agency owning the project or by contract. Some instruments are readily serviced by project personnel, but other instruments require specialized skills and equipment that a manufacturer or contractor may be better suited to provide.

10.3.2. Purchase contracts for instruments and components may include a limited warranty period. Instrument maintenance contracts with the manufacturer may be a wise investment if the complexity of the maintenance justifies long-term support by specialists. Contracts for regular maintenance and recalibration of instruments should define:

a. Maintenance schedules.

b. Scope of work.

c. Procedures.

d. Contractor qualifications and experience.

e. Technician qualifications and experience.

f. The requirement of an O&M manual, including troubleshooting guidance, as a deliverable for a new instrument.

10.4. Service Recordkeeping.

10.4.1. Keeping a service record for an instrument supports tracking the performance of maintenance, planning periodic maintenance, and determining the needed inventory of spare parts. A service record for an instrument or component should be provided to persons responsible for the technical management and operation of the project. The types of servicing noted in the record should include maintenance, recalibration, repair, modification, and replacement.

10.4.2. The types of information entered in a service record for a given instrument should include the date of service, observations, serial numbers, calibration factor, problems encountered, corrective procedure employed, and persons involved.

10.5. Spare Parts. Safe and efficient operation of a project requires that monitoring not be unduly interrupted by the failure of an instrument. Because instruments do fail from time to time, a supply of spare parts and units is necessary to resume monitoring as quickly as possible. Economically maintaining a stock of spare parts requires methodical inventorying, budgeting, and purchasing by:

10.5.1. Including spare parts and instrument units in an inventory list.

10.5.2. Keeping the inventory list at the project site and at a more central location for instrumentation and monitoring management, and including the lists in documents such as an O&M manual or a troubleshooting guide.

10.5.3. Keeping spare read-out units.

10.5.4. Stocking at the time of a new instrument installation those spare parts that are essential or are likely to be needed.

10.6. Recalibration.

10.6.1. Instruments in service may require recalibration. Test equipment used to recalibrate components of an instrumentation system should also be maintained. Standard methods used to recalibrate instruments are set forth by the National Institute of Standards and Technology (NIST). The NIST makes special use of the term “traceable.” Recalibration and testing equipment used on a USACE project should be traceable to the NIST.

10.6.2. Routine recalibration is best performed by the persons who collect data. Recalibration should be coordinated with the persons who analyze the data. Manufacturer recommendations should be referenced to develop a recalibration schedule, but site conditions, extreme operating environments, and other factors should be considered as well.

10.6.3. Intricate components, such as inclinometer probes, data loggers, or strong-motion accelerographs, should only be recalibrated by the manufacturer. Procedures, warranty information, and manufacturer technical support contacts needed for recalibration should be included in an O&M manual.

10.6.4. Recalibration Indications.

a. Changes in instrument readings may indicate the need for an unscheduled recalibration. Loss of calibration may be indicated by:

(1) An abrupt change in readings unrelated to project operation.

(2) Progressive deviation in readings over time unrelated to field conditions.

b. Some instruments, such as an embedded transducer, or direct-burial devices sealed or grouted in place, cannot be recalibrated. The data evaluation procedures discussed in Chapter 9 may be used to detect poor calibration in inaccessible instruments.

10.6.5. Portable Read-Out Units.

a. Portable read-out units may require both scheduled and unscheduled calibration. For example, rough handling may put a portable read-out unit out of calibration. Read-out units may be calibrated by project personnel, the manufacturer, or contractors as appropriate.

b. Frequent field calibration checks may be accomplished on some portable instruments. By taking readings at a controlled location of known performance. For example, inclinometer probes may be checked by examining repeatability at the bottom immovable segment of an installed inclinometer casing.

10.6.6. Retrievable Components. Retrievable components are removed for recalibration or replacement. Retrievable components should be installed with all fittings, such as shutoff valves or disconnect plugs, positioned to facilitate removal and replacement.

10.7. Twin-Tube Hydraulic PZ Maintenance.

10.7.1. Compare current PZ readings with previous readings to determine a trend. The two gauges connected to a single tip may be compared to determine if the pressures agree within established limits. If the pressures are not similar, a need for maintenance by means of flushing, rehabilitation, or replacement is indicated.

10.7.2. Periodic maintenance includes flushing, testing pressure gauges, and ensuring freeze protection and terminal drainage:

a. Flush with de-aired or distilled water to clean the lines and manifold system of any mineral deposits and to remove accumulated gas bubbles.

b. Check water discharged during flushing for discoloration, gas, sediment, or mineral deposits.

c. Check individual pressure gauges against a single larger and more accurate master gauge.

d. Maintain and recalibrate master gauges and individual instrument gauges, replacing inaccurate or damaged gauges.

e. Protect a PZ located in a terminal structure from freezing by heating the structure and maintaining the heating, ventilation, drainage, and moisture control equipment in the structure.

10.7.3. Additional guidance is included in the Bureau of Reclamation 2005 publication Operation and Maintenance Guidelines for Hydraulic PZ Installations at Dams.

10.8. Open Standpipe PZ Maintenance.

10.8.1. Operational precautions include avoiding introducing contaminants into the PZ with the read-out probe while taking a reading.

10.8.2. Maintenance indications are based on the response time, or hydrostatic time lag, of an open standpipe PZ. Therefore, a PZ requires periodic testing of response time, as discussed in Chapter 5. The response time of a PZ should be determined at the time of installation by performing a rising head (bailing) test, or a falling head test if the hydraulic conductivity of the embankment zone is greater than 10^{-5} cm/sec.

10.8.3. Test data may be used to calculate a hydrostatic time lag for the PZ by the method described in Appendix C. If clogging is suspected, the hydrostatic time lag for the PZs should be determined periodically and compared to the original time lag.

10.8.4. An automated system may be able to obtain rising and falling head test information remotely from a water level indicator located within a standpipe. An automated system can also obtain diagnostic information, such as changes in signal-to-noise ratio, peak signal amplitude, and signal decay ratio, which may indicate a failing instrument or wiring. Automated equipment may be programmed to notify operators of a developing failure.

10.8.5. Periodic maintenance may include flushing, bacterial treatment, and ensuring freeze protection by:

a. Sounding periodically to determine if the bottom contains sediment.

b. Flushing sediment from the bottom of the PZ if sediment is found.

c. Treating with disinfectant if necessary to suppress bacterial growth.

d. Maintaining any equipment required to prevent freezing—if the standpipe diameter permits, consider placing a seal with an underlying transducer below the frost depth to protect the PZ from freezing.

10.9. Observation Well Maintenance.

10.9.1. The maintenance of an observation well is generally the same as that for an open standpipe PZ. However, larger standpipe diameters may allow the use lifting or flushing to remove accumulations from intake openings. Well rejuvenation using lifting or flushing should be used cautiously and comply with Section C.2 of Appendix C.

10.9.2. Jetting is sometimes used to maintained observation wells in rock. However, jetting is not always appropriate, and the method should be properly evaluated to protect against damage to the instrument and the surrounding materials.

10.10. Extensometer Maintenance.

10.10.1. Maintenance of probe extensometers includes:

a. Keeping probe extensometer read-out devices clean and free of grit.

b. Keeping measuring tapes and cables dry and free of kinks.

c. Checking reading cables for any damage at the connectors that may affect depth readings.

d. Checking probe cables annually for stretching, using an invar measuring tape to ensure the probes measure elevation accurately.

e. Flushing casing as needed to remove silt, corrosion, or plugging material.

10.10.2. Maintenance of fixed borehole extensometers includes:

a. Keeping extensometer reference head free of dust, grit, and moisture.

b. Removing transducers within the extensometer head as necessary for calibration and replacement.

c. Resetting the reference head according to the manufacturer's instructions if head displacement has occurred.

d. Protecting the instrument head from direct sunlight, to avoid thermal distortion.

e. Protecting the instrument head from physical displacement or damage due to falling objects or traffic.

10.11. Inclinometer Maintenance.

10.11.1. Ongoing inspection and maintenance of the probe, cable, and casing is necessary for inclinometer installations. Periodic recalibration of the inclinometer probe should be performed as recommended by the manufacturer. Usually, the probe, cable, and read-out unit are returned to the manufacturer for factory calibration or repair. Recalibration of the inclinometer probe is typically needed if the bias (zero offset) error is not within manufacturer's specifications. Consult the operator's manual for the allowable deviation.

10.11.2. The probe should be cleaned, dried, and the wheels lubricated after each use. During that process, the wheel yoke should be inspected to determine that the yoke is laterally stable and free to return to the fully extended position. If the wheel yoke does not function satisfactorily, it may be necessary to replace the pivot pin, spring, or entire wheel assembly. O-rings and electrical connectors should be inspected for wear before each use.

10.11.3. The cable should be inspected for breaks, twists, gouges, and worn markings. The connector and its O-rings should be inspected for wear and corrosion. Faulty connectors and O-rings should be replaced before use.

10.11.4. Inclinometer casings may require cleaning, including:

a. Flushing to remove debris or sediment deposits from the bottom of the casing.

b. Mechanical brushing to remove biological or chemical residue, or incrustation from the casing grooves.

10.11.5. A deteriorated casing may be able to support continued inclinometer measurements by being relined. Where that is not feasible, replacing the sensor with a cable or strip running the length of the casing can allow for the implementation of TDR technology or shear strip technology to at least determine the location of shear. These methods of continuing to use a deteriorated inclinometer casing are normally much less expensive than drilling a new hole and installing a new inclinometer casing.

10.11.6. Inclinometer casings may corrode or otherwise deteriorate to the extent that an inclinometer reading cannot be obtained. In some situations, the casing may be relined with a smaller diameter inclinometer casing, allowing continued use of a probe.

10.12. Seepage Instruments Maintenance. Maintenance of instruments measuring seepage flow, such as weirs, flumes, or seepage pipes, typically includes cleaning, servicing mechanical and electrical components, and maintaining the elevation and flow section of hydraulic components.

10.12.1. Channel and Structure.

a. An open channel may be required to convey flow to or from a structure in which flow is measured. Such a channel may become clogged with sediment, debris, or vegetation and affect flow measurement. The flow measurement structure may also become fouled. Therefore, the channel and the structure should be maintained by:

(1) Keeping the channel bed and sideslopes free of sediment, debris, and excessive vegetation and keeping records of the nature and quantity of sediments potentially resulting from internal erosion.

(2) Keeping the surfaces of the structure, and any components such as scales or staff gauges, free of dirt, mineral deposits, biological deposits, and vegetation growth.

b. Cleaning a channel or a structure may briefly affect flow rate through the instrument. Therefore, after cleaning, flow should be allowed to return to a steady state before obtaining a flow measurement.

10.12.2. Flow and Velocity Meters. Flow meters require mechanical or electrical maintenance and require periodic calibration and adjustment by:

a. Keeping all moving parts of meters clean, lubricated, and free of corrosion and grit.

b. Cleaning film buildup from the electrodes of electromagnetic instruments.

c. Checking calibration of meters and adjusting as needed.

10.12.3. Weirs and Flumes. Weirs and flumes depend on accurate elevation control and exact section geometry to measure flow accurately. Therefore, maintenance of weirs and flumes includes:

a. Checking that the structure is level and that the crest is at the same elevation as the zero reading on the staff gauge.

b. Checking section dimensions.

c. Checking weir notch or crest for nicks or dents that may affect accuracy.

d. Dressing or repairing nicks or dents if the shape of the section can be maintained.

e. Replacing a structure or structural components that cannot be repaired.

LEFT BLANK INTENTIONALLY

Appendix A
References

A.1 Required Publications (Referenced).

a. USACE. Levee Owner’s Manual for Non-Federal Flood Control Works. The Rehabilitation and Inspection Program, Public Law 84-99, pp.16–17, March. https://www.nae.usace.army.mil/Portals/74/docs/Emergency%20Operations/USACE_NonFed%20Levee%20Owner's%20Manual_Mar06.pdf.

b. U.S. Army Corps of Engineers (USACE). ER 1110-1-1807: Drilling in Earth Embankment Dams and Levees. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-1-1807.pdf.

c. USACE. ER 1110-2-103: Strong Motion Instruments for Recording Earthquake Motions in Dams. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-103.pdf?ver=2013-09-08-233412-823.

d. USACE. ER 1110-2-1156: Engineering and Design: Safety of Dams—Policy and Procedures. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-1156.pdf.

e. USACE. EM 1110-1-1009: Structural Deformation Surveying. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1009.pdf.

f. USACE. EM 1110-1-1804: Geotechnical Investigations. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-1-1804.pdf.

g. USACE. EM 1110-2-1913: Design and Construction of Levees. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1913.pdf.

h. USACE. EM 1110-2-1914: Design, Construction, and Maintenance of Relief Wells. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1914.pdf.

i. USACE. EM 1110-2-2300: General Design and Construction Considerations for Earth and Rock-Fill Dams. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2300.pdf.

j. USACE. EM 1110-2-3506: Grouting Technology. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-3506.pdf.

k. USACE. EM 1110-2-4300: Instrumentation for Concrete Structures. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-4300.pdf.

l. U.S. Department of Homeland Security, Federal Emergency Management Agency. Training Aids for Dam Safety (TADS). https://www.fema.gov/media-library/assets/documents/13602.

m. U.S. Department of Interior, Bureau of Reclamation. Embankment Dam Instrumentation Manual. https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/EDamInst.pdf.

n. U.S. Department of Interior, Bureau of Reclamation. Water Measurement Manual (revised reprint of the 1997 version). https://www.usbr.gov/tsc/techreferences/mands/wmm/WMM_3rd_2001.pdf.

o. U.S. Department of Interior, Bureau of Reclamation, and USACE. Dam and Levee Safety Risk Analysis Best Practices Training Manual. https://www.usbr.gov/ssle/damsafety/risk/methodology.html

p. U.S. Environmental Protection Agency. Handbook of Suggested Practices for the Design and Installation of Groundwater Monitoring Wells. https://www.epa.gov/sites/production/files/2015-06/documents/fieldsamp-wellshandbook.pdf

q. American Society of Civil Engineers. (2018). Monitoring Dam Performance: Instrumentation and Measurement. https://ascelibrary.org/doi/book/10.1061/9780784414828.

r. Bassett, R. (2012). A Guide to Field Instrumentation in Geotechnics—Principles, Installation, and Reading, CRC Press, Boca Raton, Florida.

s. Casagrande, A. and Fadum, R.E. (1940). "Notes on Soil Testing for Engineering Purposes," Harvard University Graduate School of Engineering Publication 268, 74 pp.

t. Cedergren, H.R. (1989). Seepage Drainage and Flow Nets, 3rd ed., John Wiley and Sons, New York.

u. Das, B.M. (2007). Principles of Geotechnical Engineering, International ed. of 6th. revised ed., Nelson Engineering. http://dl.icdst.org/pdfs/files/f26ec24b602af7971800c8c327a3b3bd.pdf.

v. Dave, T.N. and Dasaka, S.M. (2011). "A Review on Pressure Measurement Using Earth Pressure Cell." International Journal of Earth Sciences and Engineering, International Standard Serial Number (ISSN) 0974-5904, Volume 4, No. 06 SPL, October, pp. 1031–1034. https://www.civil.iitb.ac.in/~dasaka/papers/trudeep_dasaka_IJESE_2012.pdf.

w. Driscoll, F.G. (1986). Groundwater and Wells, 2nd. ed., Johnson Division, SES, Inc., St. Paul, Minnesota.

x. Dunnicliff, J. (1990). "Twenty-Five Steps to Successful Performance Monitoring of Dams." Hydro-Review.

y. Dunnicliff, J. (1993). Geotechnical Instrumentation for Monitoring Field Performance, Wiley-Interscience, New York.

z. Foster, M., and Fell, R. (1999). “A Framework for Estimating the Probability of Failure of Embankment Dams by Piping Using Event Tree Methods.” UNICIV Report (Draft), April. https://vm.civeng.unsw.edu.au/uniciv/R-377.pdf.

aa. Geokon. (2019). Instruction Manual, Model 6300, Vibrating Wire In-Place Inclinometer. https://www.geokon.com/content/manuals/6300_In-Place_Inclinometer.pdf.

bb. Holtz, R.D., Kovacs, W.D., and Sheahan, T.C. (2010). An Introduction to Geotechnical Engineering. Prentice-Hall, Englewood Cliffs, New Jersey.

cc. Hvorslev, M.J. (1951). Time-Lag and Soil Permeability in Groundwater Observations. Bulletin No. 36, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi. https://www.csus.edu/indiv/h/hornert/geol%20500%20spring%202014/Week_3_slug_tests/Hvorslev%201951.pdf.

dd. Johansson, S. and Sjödahl, P. (2004). “Downstream Seepage Detection Using Temperature Measurements and Visual Inspection—Monitoring Experiences from Røsvatn Field Test Dam and Large Embankment Dams in Sweden.” Procs. Stability and Breaching of Embankment Dams, EBL, Oslo, p. 20. https://www.sensornet.co.uk/wp-content/uploads/2016/05/Oslo_2004-_Downstream_Seepage_Detection_using_Temperature_Me.pdf.

ee. Kinemetrics. (2013). EpiSensor ES-T, Force Balance Accelerometers. http://www.kinemetrics.com/uploads/PDFs/EPISENSOREST.pdf. Accessed 26 June.

ff. Lazebnik, G.E. and Tsinker, G.P. (1998). “Errors in Measuring Soil Contact Pressures by Means of Nonstiff Pressure Cells.” Monitoring of Soil-Structure Interaction, pp. 79–111.

gg. McKenna, G.T. (1995). “Grouted-in Installation of Piezometers in Boreholes.” Canadian Geotechnical Journal, 32: pp. 355363.

hh. Mikkelsen, P.E. and Green, G.E. (2003). “Piezometers in Fully Grouted Boreholes.” Symposium on Field Measurements in Geomechanics, FMGM 2003. Oslo, Norway, September. https://www.zeminas.com.tr/docs/ZET_2.pdf.

ii. Penman, A.D.M. (1960). “A Study of the Response Time of Various Types of Piezometers.” Proceedings, Conference on Pore Pressures and Suction in Soils. Butterworth, London.

jj. Solinst. (2013). Dataloggers and Level Measurement Devices, http://www.solinst.com/Prod/Prod.html, accessed 23 October.

kk. Teledyne Isco. (2006). Open Channel Flow Measurement Manual, 6th ed.

ll. Terzaghi, K., Peck, R., and Mesri, G. (1996). Soil Mechanics in Engineering Practice. 3rd ed., Wiley-Interscience, New York.

Appendix B
Case Studies

B.1. This appendix presents three case studies of decades-old USACE embankment dams that required remedial construction to reduce seepage and increase safety. The case studies focus on the period of construction and stress the importance of:

B.1.1. Project history.

B.1.2. Having the proper types and numbers of instruments in the proper locations.

B.1.3. Using instruments to determine the need for and the success of remedial construction.

B.1.4. Adjusting instrument reading frequency to the circumstances.

B.1.5. Evaluation of data, including the use of linear regression.

B.1.6. The establishment of operational thresholds, alerts, and alarms.

B.1.7. Communication and coordination among USACE and contractor personnel.

B.2. The three case studies are:

B.2.1. Case Study 1: Francis E. Walter Dam, Pennsylvania.

B.2.2. Case Study 2: Wolf Creek Dam, Kentucky.

B.2.3. Case Study 3: Wood River Flood Protection Project, Upper Wood River Levee System, Illinois.

B.3. Case Study 1: Francis E. Walter Dam, Pennsylvania.

B.3.1. Francis E. Walter Dam (Walter Dam) is located in eastern Pennsylvania near White Haven on the Lehigh River in the upper part of the river watershed. The dam crosses a reach of the river forming the boundary between Carbon and Luzerne Counties. The project was constructed in the late 1950s, and the project authorized purposes are flood control and recreation.

B.3.2. The project consists of an earthen embankment, a gated outlet works, an ungated spillway with an ogee-shaped concrete weir, and a saddle dike. The crest of the 234-foot (71.3 m) high embankment is at elevation 1374 feet National Geodetic Vertical Datum (NGVD) 29 (418.8 m). The embankment is 2,200 feet (670.6 m) long and has a crown width of 30 feet (9.1 m).

B.3.3. In section, the embankment has a semi-pervious upstream zone, a central impervious zone, a pervious inclined blanket drain, and a random fill downstream zone. Both the upstream and downstream slopes are protected with riprap or rockfill. The crest of the spillway ogee weir is at 1,350 feet elevation (411.5 m).

B.3.4. A low conservation pool at 1,300 feet elevation (396.2 m) is the normal reservoir surface elevation, although the pool elevation has been held at higher elevations several times for approved, temporary purposes such as drought storage. Due to the location of the dam high in the Lehigh River watershed, the pool elevation commonly rises as much as 70 feet (21.3 m) as the result of a single storm. The pool can also be safely evacuated in a relatively short period. The record high pool elevation was 1441.73 feet NGVD 29 (439.44 m) on 29 June 2006.

B.3.5. The case study focuses on an assessment of piezometric data and is described under the headings of:

a. Instrumentation.

b. Right abutment grouting program.

c. Instrument assessment.

d. Conclusion.

B.3.6. Instrumentation.

a. The locations of the geotechnical instruments at Walter Dam used to help monitor potential failure modes are shown in the plan view in Figure B.1. The instrumentation consists of:

(1) 36 open-tube PZs tipped in different zones of the embankment, as well as in the foundation and in the downstream area.

(2) 2 inclinometers.

(3) 6 crest settlement pipes.

(4) 18 settlement pipes on the downstream slope.

(5) 47 surface motion points on the upstream slope.

(6) 9 surface motion points on the downstream slope.

b. Monitoring of the PZs and weirs supports the assessment of seepage-based potential failure modes such as internal erosion. Monitoring of the inclinometers, settlement pipes, and surface motion points supports assessment of movement-based potential failure modes such as instability.

B.3.7. Right Abutment Grouting Program.

a. The grouting program has a history of investigation beginning in the early 1980s. In a 1983 seepage study report, seepage was reported along the right abutment-embankment contact groin and near the downstream toe.

b. The report indicated that sustained seepage was present whenever the pool was held above 1,370 feet elevation (417.6 m). The seepage was described as clear, indicating no significant progression of internal erosion. The seepage was monitored by measurement and observation of flow through the weirs and by monitoring the PZs.

c. In 2000, a geotechnical investigation of the right abutment was conducted, consisting of five borings, one of which was angled. Concerns with the condition of the original grout curtain were identified, and the borings were grouted upon completion.

d. Funding for improving the grout curtain was allocated as part of the American Recovery and Reinvestment Act of 2009. The grout curtain along the right abutment was not only improved throughout the extent of the original grout curtain but deepened. Figure B.2 depicts the work performed, which was completed in 2010.

B.3.8. Instrumentation Assessment.

a. PZs provide an indication of groundwater pressure. The locations of the PZs and a table of general information about the PZs, including threshold values, are shown in Figure B.1. Walter Dam is equipped with an Automated Geotechnical Data Acquisition System (AGDAS). The original system was installed in 1995–1996.

b. Numerous problems were encountered with the original system, including damage due to several lightning strikes. The system was upgraded in 2001. Nevertheless, the system still suffered damage due to electrical surges. The system was upgraded again in 2010 and has performed satisfactorily since.

c. Piezometric data and pool elevation are collected daily, and the daily readings are automatically uploaded to an instrumentation database. The data collected at daily intervals provides the true instrument response and is far superior to data collected at monthly intervals.

d. Piezometric response as represented on time-series plots is dependent upon the location of the PZ tip, and changes in piezometric elevation are compared to changes in pool elevation and amount of precipitation. Time-series plots for individual PZs and for groups of PZs aligned in an embankment cross-section were plotted and assessed.

e. Correlation plots of piezometric elevation versus pool elevation were also plotted and assessed. These correlation plots provided an indication of how well a PZ reacted to changes in pool elevation, to provide insight on the effectiveness of the grout curtain.

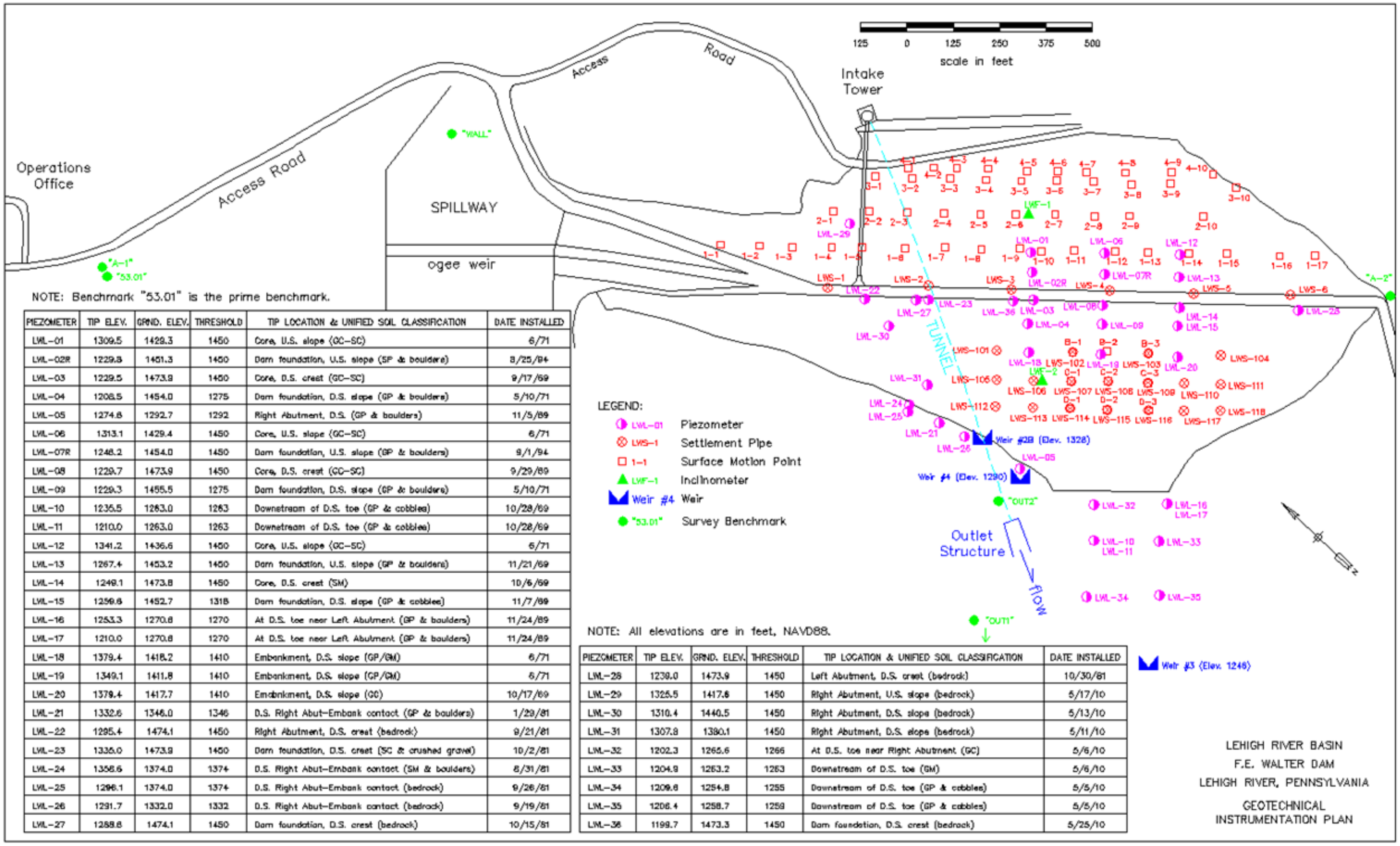
f. Because only PZs LWL-22, LWL-23, and LWL-27 were located in the critical area both before and after the grouting program, assessment of the correlation plots of the three PZs could indicate the effectiveness of the grouting program. Figure B.3 is a simplified version of Figure B.2, showing the tip locations of PZs LWL-22, LWL-23, and LWL-27. Figures B.4 through B.6 are annotated correlation plots for the three PZs.

g. Figure B.4 is the correlation plot for PZ LWL-22 and shows no change due to the grouting program. However, as shown in Figure B.3, the tip for PZ LWL-22 is well below both the original and improved grout curtains.

h. PZ LWL-23 is tipped near the top of the grout curtain. Figure B.5 is the correlation plot for PZ LWL-23 and indicates that LWL-23 reacted to changes in pool elevation before the grout curtain improvement, but did not react to such changes after grout curtain improvement. The ability of LWL-23 to provide the post-construction data was due to not grouting the PZ.

i. PZ LWL-27 is tipped near the bottom of the grout curtain. Figure B.6 is the correlation plot for PZ LWL-27 and indicates that before the grout curtain improvement, LWL-27 reacted to changes in pool elevation. However, after the grout curtain improvement, LWL-27 was less correlated to changes in pool elevation.

B.3.9. Conclusion. The instrumentation system was successful in providing adequate data to evaluate the effectiveness of the grouting program. The assessment of the correlation plots indicates that the grouting program has reduced seepage through the right abutment.



PIEZOMETER	TIP ELEV.	GRND. ELEV.	THRESHOLD	TIP LOCATION & UNIFIED SOIL CLASSIFICATION	DATE INSTALLED
LWL-01	1309.5	1429.3	1450	Core, U.S. slope (GC-SC)	6/71
LWL-02R	1229.8	1451.3	1450	Dam foundation, U.S. slope (SP & boulders)	8/25/94
LWL-03	1229.5	1473.9	1450	Core, D.S. crest (GC-SC)	9/17/69
LWL-04	1208.5	1454.0	1275	Dam foundation, D.S. slope (GP & boulders)	5/10/71
LWL-05	1274.8	1292.7	1292	Right Abutment, D.S. (GP & boulders)	11/5/89
LWL-06	1313.1	1429.4	1450	Core, U.S. slope (GC-SC)	6/71
LWL-07R	1248.2	1454.0	1450	Dam foundation, U.S. slope (GP & boulders)	9/1/94
LWL-08	1229.7	1473.9	1450	Core, D.S. crest (GC-SC)	9/29/69
LWL-09	1229.3	1455.5	1275	Dam foundation, D.S. slope (GP & boulders)	5/10/71
LWL-10	1235.5	1263.0	1263	Downstream of D.S. toe (GP & cobbles)	10/28/69
LWL-11	1210.0	1263.0	1263	Downstream of D.S. toe (GP & cobbles)	10/28/69
LWL-12	1341.2	1436.6	1450	Core, U.S. slope (GC-SC)	6/71
LWL-13	1267.4	1453.2	1450	Dam foundation, U.S. slope (GP & boulders)	11/21/69
LWL-14	1249.1	1473.8	1450	Core, D.S. crest (SM)	10/6/69
LWL-15	1259.6	1452.7	1318	Dam foundation, D.S. slope (GP & cobbles)	11/7/89
LWL-16	1253.3	1270.8	1270	At D.S. toe near Left Abutment (GP & boulders)	11/24/89
LWL-17	1210.0	1270.8	1270	At D.S. toe near Left Abutment (GP & boulders)	11/24/89
LWL-18	1379.4	1418.2	1410	Embankment, D.S. slope (GP/GM)	6/71
LWL-19	1349.1	1411.8	1410	Embankment, D.S. slope (GP/GM)	6/71
LWL-20	1379.4	1417.7	1410	Emabnkment, D.S. slope (GC)	10/17/69
LWL-21	1332.6	1346.0	1346	D.S. Right Abut-Embank contact (GP & boulders)	1/29/81
LWL-22	1295.4	1474.1	1450	Right Abutment, D.S. crest (bedrock)	9/21/81
LWL-23	1336.0	1473.9	1450	Dam foundation, D.S. crest (SC & crushed gravel)	10/2/81
LWL-24	1358.6	1374.0	1374	D.S. Right Abut-Embank contact (SM & boulders)	8/31/81
LWL-25	1298.1	1374.0	1374	D.S. Right Abut-Embank contact (bedrock)	9/26/81
LWL-26	1291.7	1332.0	1332	D.S. Right Abut-Embank contact (bedrock)	9/19/81
LWL-27	1288.8	1474.1	1450	Dam foundation, D.S. crest (bedrock)	10/15/81

PIEZOMETER	TIP ELEV.	GRND. ELEV.	THRESHOLD	TIP LOCATION & UNIFIED SOIL CLASSIFICATION	DATE INSTALLED
LWL-28	1239.0	1473.9	1450	Left Abutment, D.S. crest (bedrock)	10/30/81
LWL-29	1325.5	1417.6	1450	Right Abutment, U.S. slope (bedrock)	5/17/10
LWL-30	1310.4	1440.5	1450	Right Abutment, D.S. slope (bedrock)	5/13/10
LWL-31	1307.8	1380.1	1450	Right Abutment, D.S. slope (bedrock)	5/11/10
LWL-32	1202.3	1265.6	1266	At D.S. toe near Right Abutment (GC)	5/6/10
LWL-33	1204.9	1263.2	1263	Downstream of D.S. toe (GM)	5/6/10
LWL-34	1209.6	1254.8	1255	Downstream of D.S. toe (GP & cobbles)	5/5/10
LWL-35	1206.4	1258.7	1259	Downstream of D.S. toe (GP & cobbles)	5/5/10
LWL-36	1199.7	1473.3	1450	Dam foundation, D.S. crest (bedrock)	5/25/10

Figure B.1. Plan View of Walter Dam and Instrument Locations

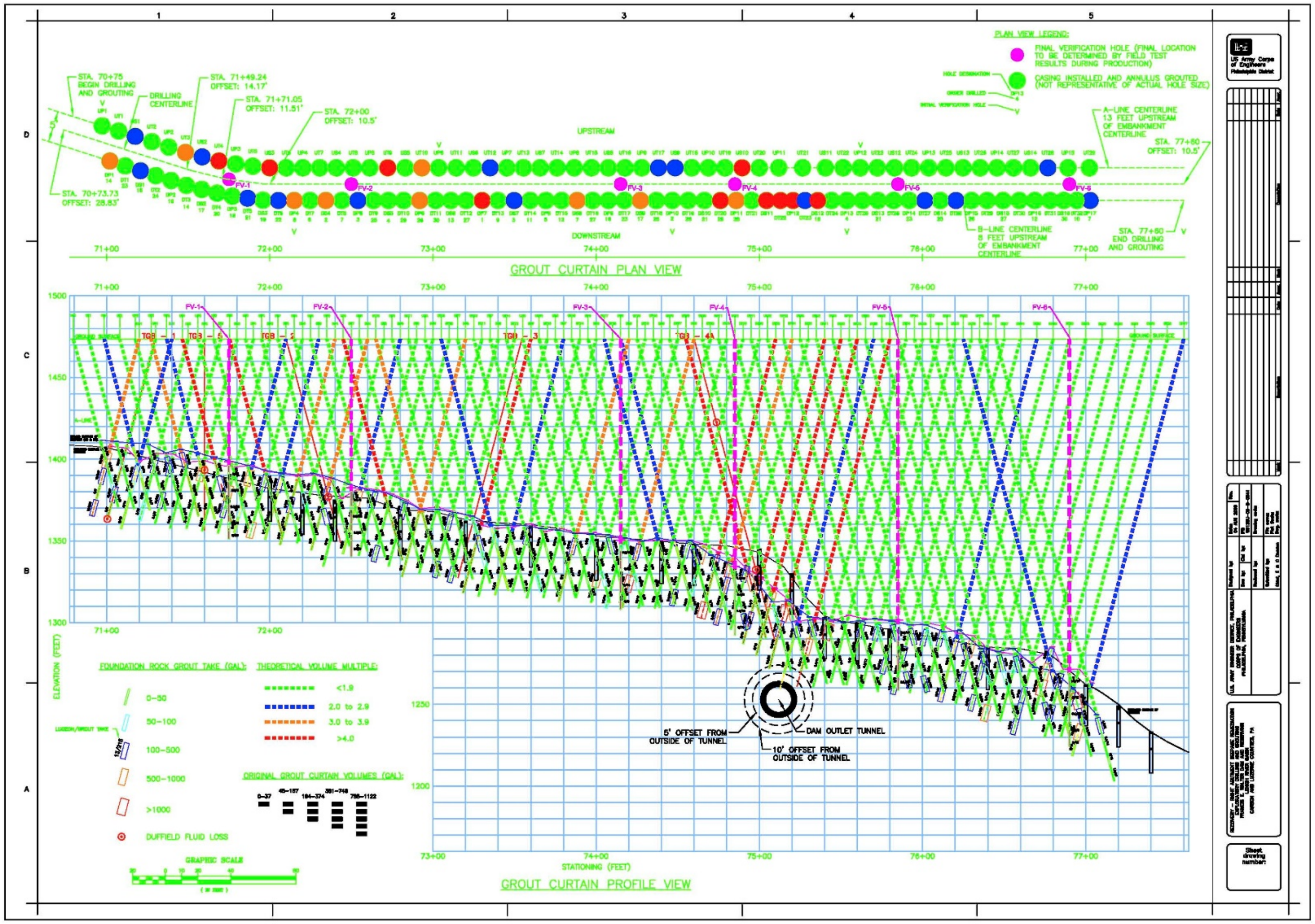


Figure B.2. Plan and Profile View of Grouting Performed

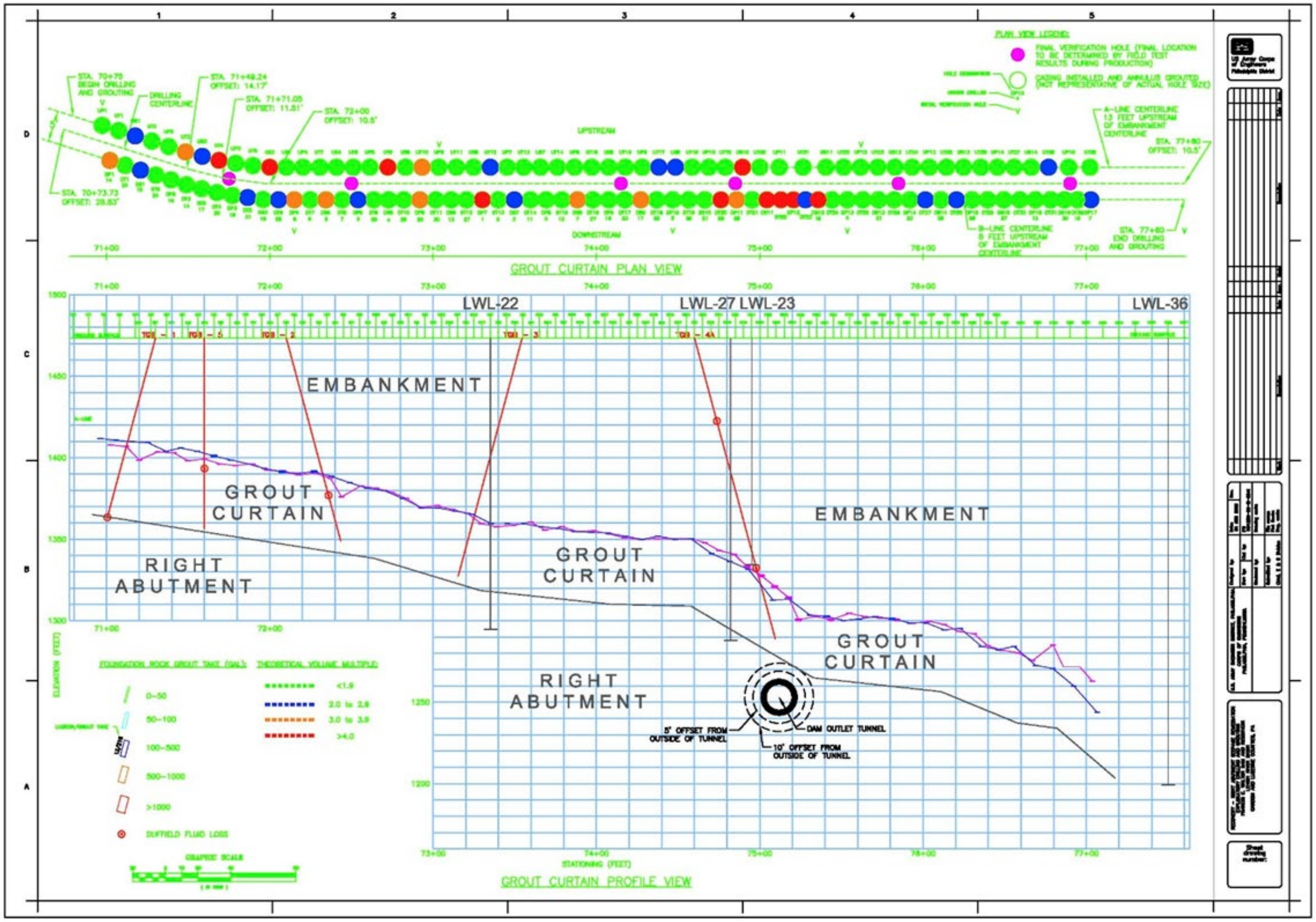


Figure B.3. Plan and Profile View of PZ Tip Locations

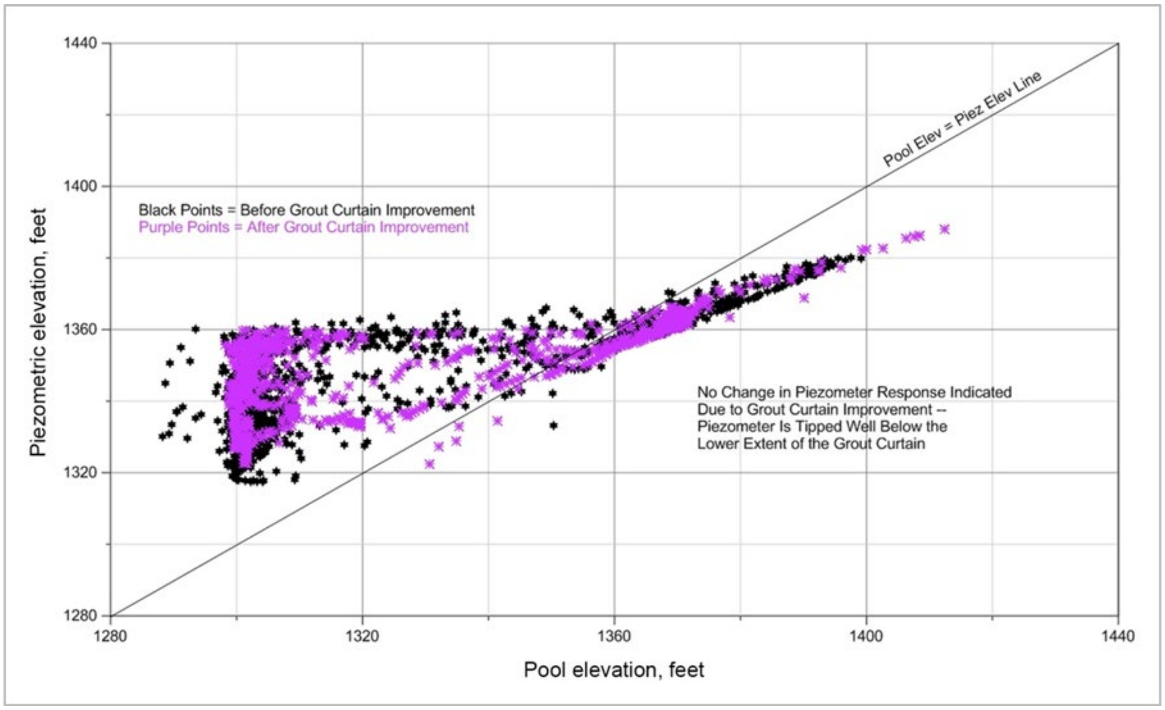


Figure B.4. PZ LWL-22 Level vs. Pool Elevation

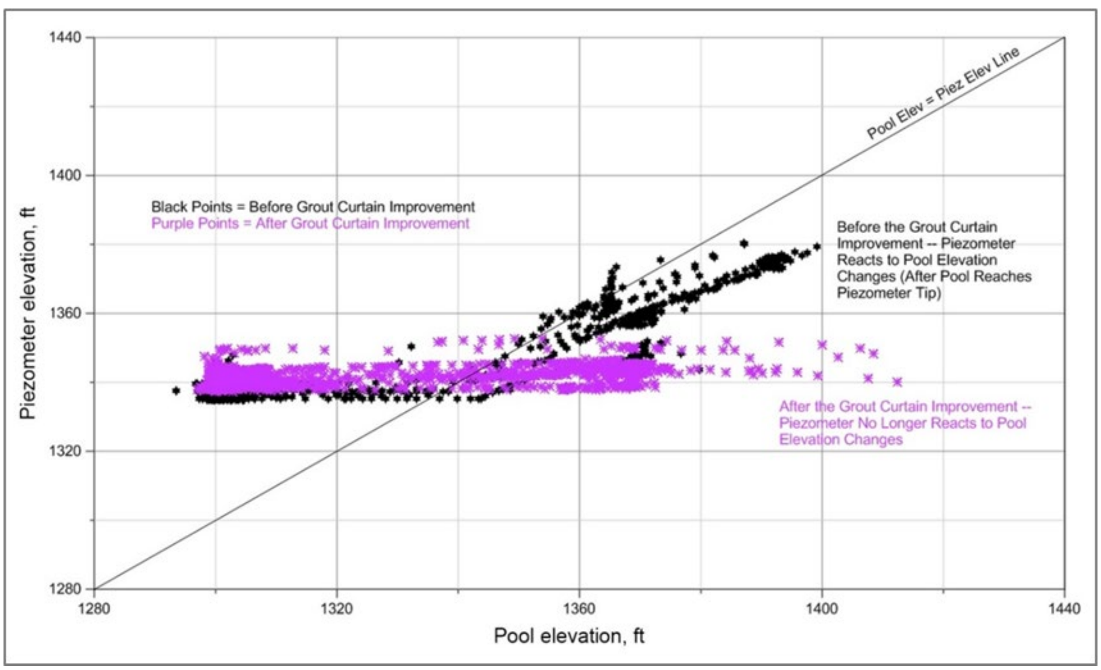


Figure B.5. PZ LWL-23 Level vs. Pool Elevation

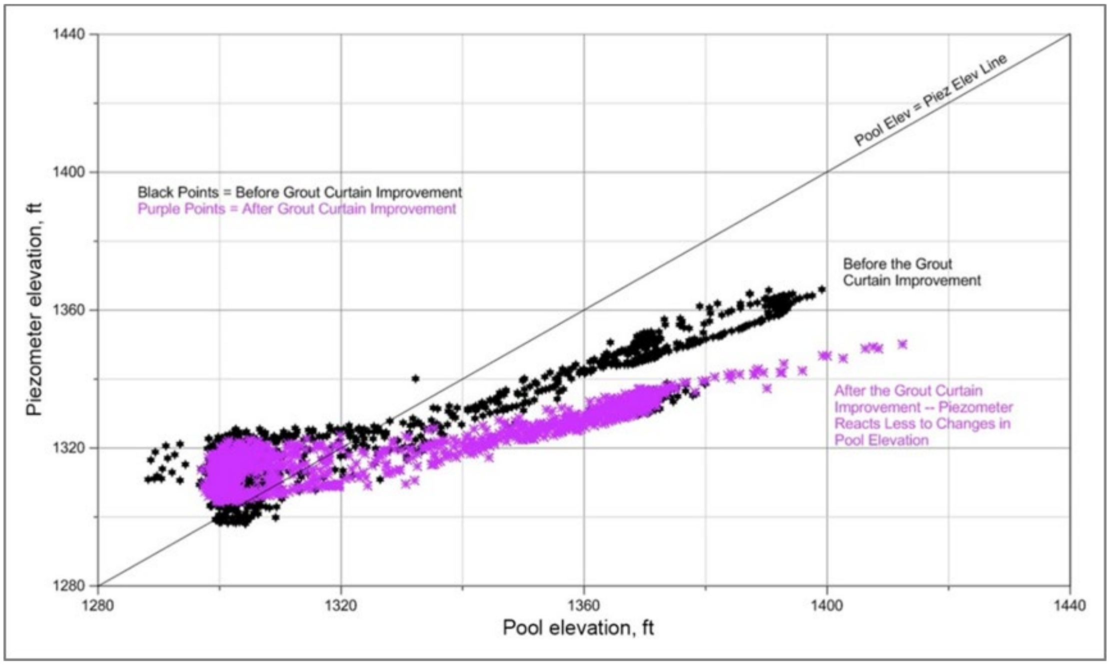


Figure B.6. PZ LWL-27 Level vs. Pool Elevation

B.4. Case Study 2: Wolf Creek Dam, Kentucky.

B.4.1. Wolf Creek Dam is an earthen embankment placed on an untreated karst foundation, has hundreds of installed instruments and has a history of multiple mitigation efforts, including grouting and seepage cutoff wall installation. Therefore, a complete assessment of the instrumentation reactions to the most recent barrier wall installation is beyond the scope of a case study. However, important lessons learned on this project may bear on other instrumentation dam safety and construction monitoring. The case study stresses four topics:

a. The use of a PZ to verify the need for a seepage cutoff wall.

b. The use of a PZ to verify the success of an installed cutoff wall.

c. The value of appropriate monitoring frequency.

d. The use of thresholds to assess instrument reactions.

B.4.2. The case study is presented under the headings of:

a. Project description.

b. Instrument reaction.

c. Thresholds and reading frequency.

d. Conclusions.

e. References.

B.4.3. Project Description.

a. Wolf Creek Dam impounds Lake Cumberland in southeastern Kentucky along the Cumberland River. Shown in Figure B.7, the combination earth and concrete gravity dam has a maximum height of 258 feet (78.6 m), is 5,736 feet (1748.3 m) long, and has a total storage of more than 6 million acre-feet (7.4 billion cubic meters) of water.

b. Wolf Creek Dam is the largest dam east of the Mississippi River providing flood control, hydropower, recreation, water quality, and water supply benefits. Catastrophic failure of the project would result in deaths, widespread flooding, and economic damage to downstream communities, including Nashville, Tennessee, possibly exceeding $3 billion (USACE, 2013).

Figure B.7. Aerial Photograph of Wolf Creek Dam

c. Built in the 1940s, the earthen embankment was placed directly on alluvial soils overlying a limestone foundation with extensive karst development. No foundation treatment was performed outside of the upstream core trench or the original grouting zone (Zoccola, Haskins, and Jackson, 2009). Figure B.8 shows the typical design cross section of the dam, with a narrow cutoff trench constructed on the upstream side and an alluvium layer left in place beneath the embankment.

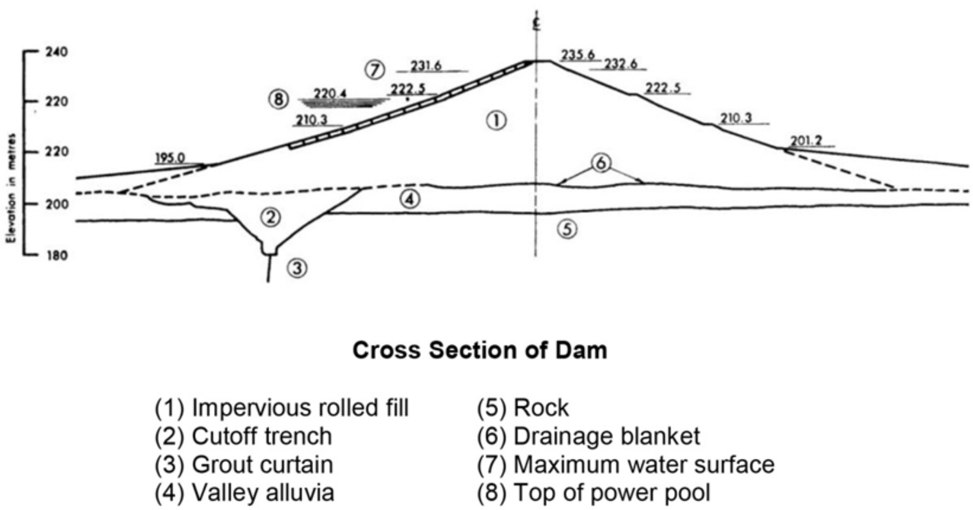


Figure B.8. Design Section of Embankment (Fetzer 1979)

d. The alluvial material constituted a serious flaw in the foundation, resulting in susceptibility to seepage, piping, and sinkhole formation. Numerous distress indicators—such as muddy flow in the tailrace (1967), sinkhole formations (1967 and 1968), persistent wet areas, and unsatisfactory PZ readings—led USACE to perform a large emergency remedial grouting program in the 1960s.

e. The first seepage cutoff walls were also installed—commonly called the ICOS or diaphragm wall (the ICOS company performed the construction and was later acquired by TREVI to form TREVIICOS in 1997)—through the main dam at the crest and in the switchyard area in the 1970s (Simmons, 1982).

f. Additional distress indicators appeared in the decades after the ICOS wall construction (USACE, 2005; Zoccola, Haskins, and Jackson, 2009) resulting in the design and construction of a more comprehensive seepage cutoff wall and supporting grouting program on the upstream side of the dam, completed in March 2013.

B.4.4. Identification of Critical Instruments.

a. The risk driving failure modes were used to identify critical instruments to be monitored more frequently during construction. Instruments were classified as primary, secondary, or tertiary. Figure B.9 shows PZ priorities classified as primary (red), secondary (blue), or tertiary (yellow).

b. The PZs were installed to monitor structurally significant features or locations in the foundation, embankment, and abutments where indications of a developing problem could be detected. The primary PZs were installed along identified potential seepage pathways in the foundation and at the foundation and embankment interface, and were automated, recording levels hourly.

c. To facilitate data interpolation between primary PZs, the secondary PZs were installed near the potential seepage pathways and were monitored manually, once daily. Tertiary PZs were installed in the embankment and were monitored manually, as needed, to indicate the bounds of the primary and secondary PZ responses.

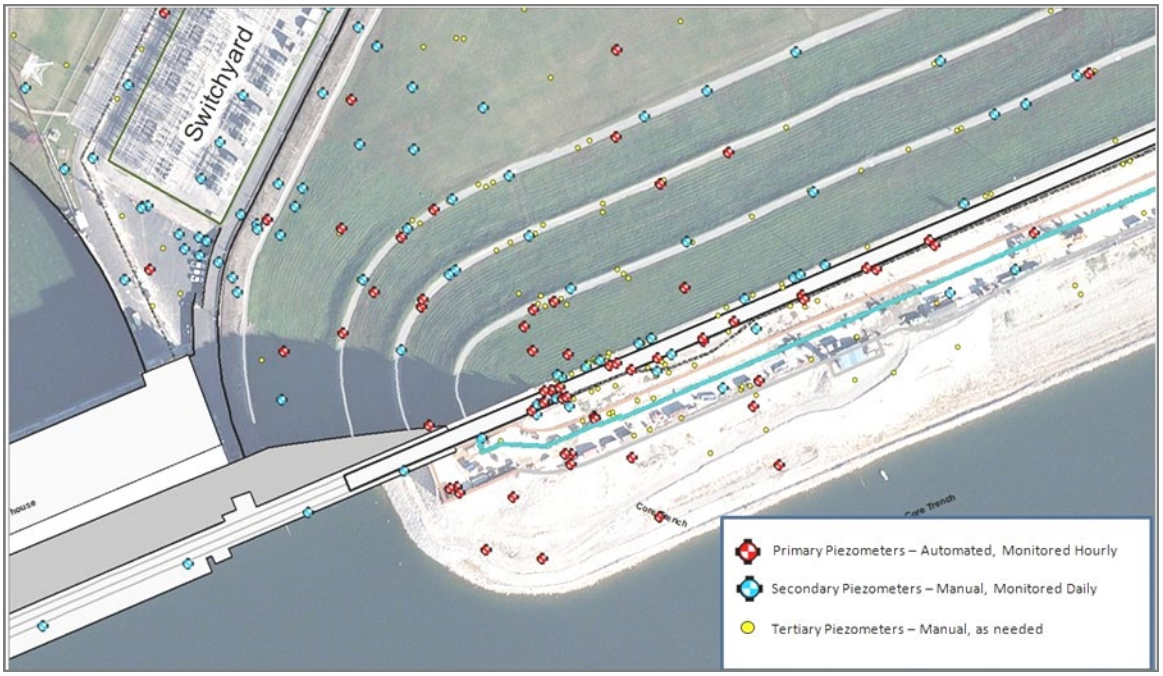


Figure B.9. Color-Coded PZ Priority

d. Thresholds and Reading Frequencies.

(1) No discussion of instrumentation can be considered complete without a discussion of monitoring frequencies and thresholds. The accuracy of the data and the reliability of an instrument can be reduced if monitoring occurs at the wrong frequency.

(2) An overwhelmingly large amount of data can mask a problem as easily as a small data set or infrequently collected data. Therefore, threshold values are necessary for an extensively instrumented embankment to ensure that readings indicating a safety problem or needed to guide construction are scrutinized immediately. Thresholds and reading frequencies are discussed under the headings of:

(a) Monitoring frequencies.

(b) Alert levels, reading frequencies, and thresholds.

(c) Visual distress indicators.

(d) PZs.

(e) Inclinometers.

(f) Extensometers.

(g) Crack pins.

(h) Settlement monuments and survey observation points.

(3) Monitoring Frequencies.

(a) Statistical and visual correlations related to time series plots are difficult to evaluate at low monitoring frequencies. One of the lessons learned from Wolf Creek Dam is that monitoring frequencies can affect the engineer's or geologist's understanding of an instrument. Monthly readings may not accurately reflect the reactivity of a PZ in certain cases, and the behavior of PZ WA-59R is further examined as an example.

(b) Figure B.10 shows the location of the instruments to be discussed, and Table B.1 lists information about WA-59R and other PZs.

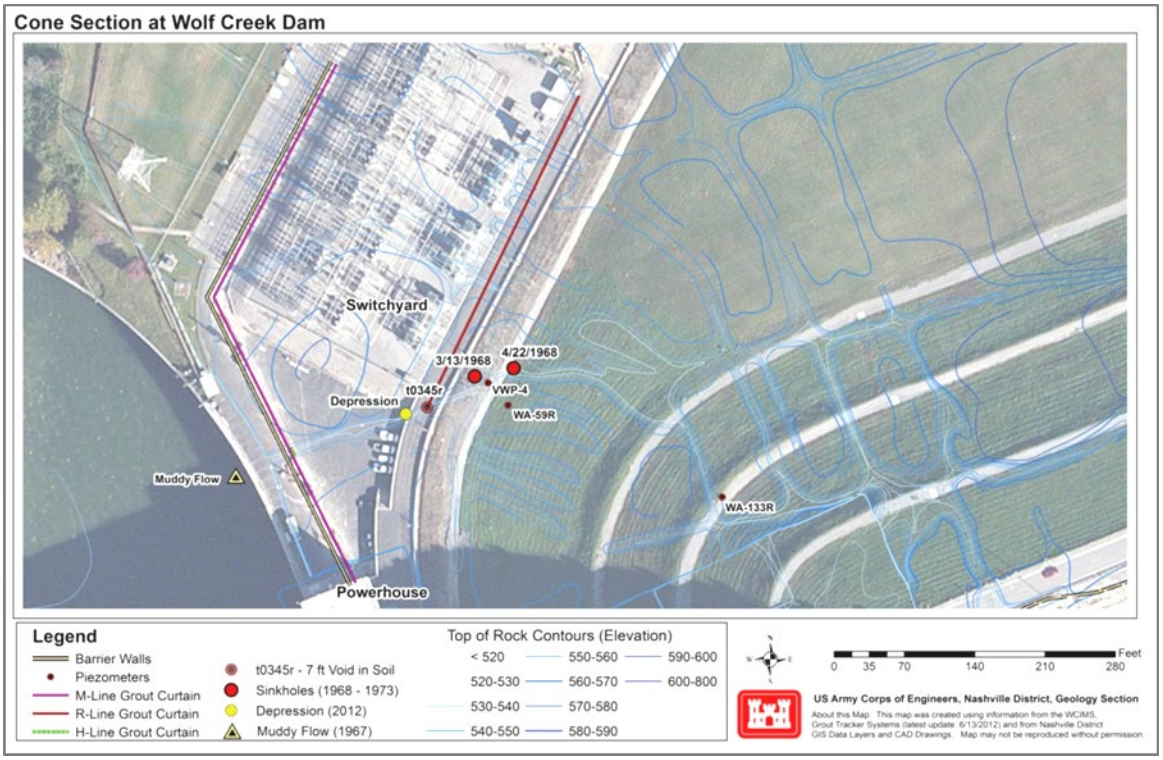


Figure B.10. Plan Location of PZs Used to Illustrate Monitoring Frequency Effects

Table B.1
PZ Information

PZ	Station & Offset	Tip Elev.	Medium	Automated	Type
WA-59R	32+40L 3+81B	502.8	Rock	N	OS
WA-133R	34+03L 2+16B	512.4	Rock	Y	OS w/ T
VWP4-1	32+30L 4+09B	560.0	Top of Rock	Y	T
VWP4-2	32+30L 4+09B	550.0	Rock	Y	T
VWP4-3	32+30L 4+09B	542.0	Rock	Y	T
VWP4-4	32+30L 4+09B	532.0	Rock	Y	T

OS: open standpipe, OS w/ T: open standpipe retrofit with a transducer, T: transducer.
All automated instruments were monitored hourly.
Non-automated instruments were monitored daily.

(c) PZ WA-59R shows the strongest correlation to changes in tailwater, as shown in Figure B.11. The strong correlation is not due to sampling effects, but accurately reflects the strong physical connection that the instrument has to tailwater through karst features. If WA-59R had been monitored more frequently, the correlation with tailwater elevation might have been even stronger.

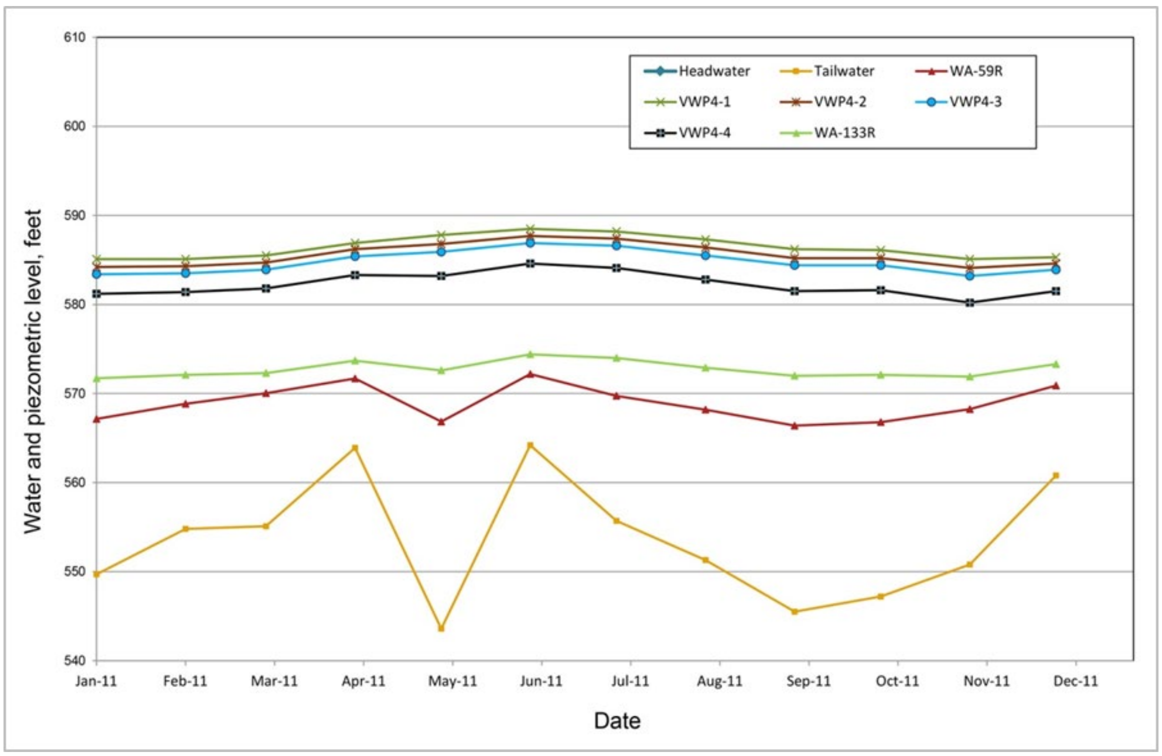


Figure B.11. Monthly Levels for Six PZs: January 2011–December 2011

(d) Figure B.12 is a plot of the full data set, with PZ WA-59R monitored daily and all other instruments monitored hourly. In this data format, PZs WA-59R, WA-133R, and VWP4-4 exhibit a strong and immediate reaction to tailwater elevation.

(e) The strong and nearly instantaneous reaction of both VWP4-4 and WA-133R with WA-59R and tailwater elevation indicated a seepage path that could endanger the embankment if not blocked. This was the very failure mode identified by the 1960s grouting program and the reason that an ICOS wall was built in the 1970s in the switchyard (Simmons, 1982). Moreover, the evidence of a seepage path supported the decision to construct a supplementary new barrier wall in this location.

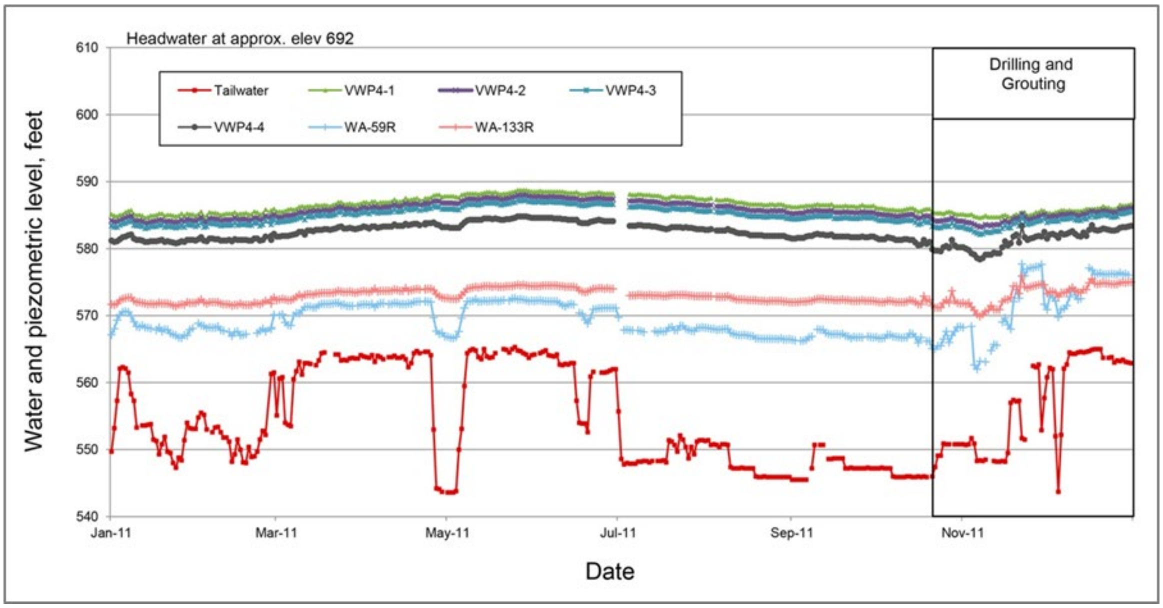


Figure B.12. Hourly Levels for Six PZs: January 2011–December 2011

(f) The example provided in this section indicates that the engineer or geologist should determine the data resolution needed to perform an analysis and should be aware of the reactivity of an instrument to variables influencing readings. If a driving variable changes quickly and an instrument reacting to the variable is read infrequently, data resolution and understanding can be unnecessarily limited.

(4) Alert Levels, Reading Frequencies, and Thresholds.

(a) The threshold value of an instrument should be based on:

- Instrument type.
- Instrument accuracy.
- Rate of change of an independent variable correlated with instrument readings.
- Potential failure mode or distress indicator monitored.

(b) The disturbance of constructing the seepage cutoff wall at Wolf Creek could conceivably have temporarily increased seepage along an existing pathway or could have caused other sudden changes in the foundation. Therefore, readings were obtained at a much higher frequency than would be typical for a similar embankment not undergoing construction.

(c) Given the consequences of a catastrophic failure of Wolf Creek Dam and the many instruments monitored during construction, a surveillance and monitoring plan was implemented with the contractor. The Joint Instrumentation Monitoring Plan (JIMP) clearly defined responsibilities for reading instruments, sharing data, submitting reports, and setting threshold values for the instrumentation.

(d) These threshold values were established by a joint team of Nashville District engineers and geologists, contractor representatives, and the advisory panel experts of both the district and the contractor. The success of this process led the Nashville District to implement the same process for the seepage cutoff wall construction project at Center Hill Dam on the Caney Fork River in Tennessee.

(e) JIMP alert levels were established for various instruments (as shown in Figure 8.6). Alert levels were established to ensure that anomalous readings were examined without delay to avoid a dam safety risk. The large number of instruments at the site and the need to make quick decisions regarding construction activities made established alert levels invaluable.

(f) Five alert levels were set. The lowest alert level (Level 1) only involved evaluating reactions of PZs upstream of the existing barrier wall. In contrast, Levels 4 and 5 required an automatic stoppage of work.

(g) Automated alerts were set for all automated PZs, notifying project staff members by email and text message if any threshold values were exceeded. Exceedance of an instrument threshold triggered an evaluation by both the project team and the contractor team to determine the cause of the exceedance and to assess the level of risk posed to dam safety.

(h) Regarding manually read instruments and visual observations of distress, instrument readers were trained to recognize the exceedance of a threshold level and to communicate an exceedance or observation of distress promptly. Oral notification was required in all cases to verify that the proper person had been informed and was taking action.

(i) A list of contacts was provided, and all were available on-call 24 hours. In the event of a threshold exceedance or the observation of potential distress, the observer was required to start at the top of the telephone contact list and continue downward until live contact was made. The JIMP also contained the emergency action plan contact list and defined the notification responsibilities within the Dam Safety Chain.

(j) For alert Levels 1–3, USACE and Contractor Data Managers will immediately inform each other of any data indicating a threshold exceedance and will proceed as if this measurement represents accurate data until checked, as outlined below:

- Instrument integrity should be verified by:

- Re-reading the offending instrument.
- Verifying that the instrument is reporting correct data; for automated PZs this may involve looking at the temperature measurements. If the temperature measurement was dramatically different, this can indicate an electrical problem.
- Verify that the instrument was not damaged.
- Check for appropriate device calibration.
- Verify if a new instrument reader took the measurement, and if necessary, re-take the measurement with a more experienced reader to verify reading.

- Perform a quality-control check on the data reduction calculations.
- Confirm the time and location of specific construction activities; this is necessary to evaluate what might be causing the instrument reactions.
- Read all instruments surrounding the offending instruments deemed necessary for corroboration of the threshold exceedance, determine the extent of the response, and assess the threat level to dam safety.
- Perform visual inspections of the embankment, work platform, and the lake.
- Evaluate any climatological factors that could be affecting the instrument. One of the most common reasons for threshold exceedance in an open standpipe PZ is an instrument that takes on surface water.
- Assess the problem and attribute a suspected cause.
- Share results immediately with the project team and the contractor.

(k) The JIMP specifically stated that USACE could unilaterally change or stop any construction activities that were suspected of threatening the integrity of the dam. Likewise, the contractor could unilaterally suspend construction activities suspected of threatening the dam.

(5) Visual Distress Indicators.

(a) Any observation indicating a failure in-progress would automatically trigger the highest level of alert. These events included, but were not limited to:

- A whirlpool in the lake.
- A sinkhole upstream or downstream.
- A depression in the upstream or downstream embankment slope.
- Settlement or cracking in the embankment crest.
- The sudden appearance of muddy water in the tailrace.
- The sudden appearance of grout downstream.
- Increased seepage or boils transporting solids.

- Slope instability.

(b) Observations of existing wet areas were regularly made, and inspectors were trained to immediately report any changes or the existence of new wet areas. A small depression was noted in the corner of the switchyard during construction, and a 7-foot (2.1 m) void in the soil was also noted near PZ WA-59R along the switchyard exploratory grouting.

(c) Cracking in the crest on the immediate upstream side of the ICOS wall was also observed during the construction, and crack monitors were added to quantify movement. The dam was inspected at least once during the day shift and once during the night shift by USACE personnel. A checklist was used to verify that the inspectors were checking the same areas and achieving comprehensive coverage.

(6) PZs.

(a) All automated PZs were monitored every 15 minutes, but due to data transmission limitations when the system was established in 2009, only the hourly readings were retained. Alerts were set using the 15-minute readings, which could be retained with manual intervention if needed. Later improvements to data transmission equipment made the higher frequency reading interval possible before completion of the barrier wall in 2013, but the hourly frequency was maintained to avoid costly upgrades near the end of the project.

(b) All manually read PZs were monitored daily, which was judged to be the practical limit due to the labor required. Later instrumentation analysis showed that this was sufficient when combined with the automated readings from other instruments, but the daily readings did not show the extent of all PZ reactions.

(c) The threshold limits were set based on an assessment of instrument behavior and location:

- A 5-foot (1.52 m) change in consecutive readings for a PZ on the upstream side of the existing ICOS barrier wall.

- A 2-foot (0.61 m) change in consecutive readings for a PZ on the downstream side of the existing ICOS barrier wall.

- Any temperature change exceeding ± 2.5° C for consecutive readings measured by a vibrating-wire PZ.

(7) Inclinometers.

(a) Inclinometers were read weekly and all inclinometers at the site were manual. More frequent readings were impractical due to the large number of instruments and the accuracy of manual readings in comparison to the small magnitude of the expected and measured movements—more frequent readings would have been more reflective of noise than actual movements. Any high PZ reading was to trigger additional readings of inclinometers surrounding the PZ. Inclinometer threshold limits were exceeded if:

- 0.5 inches (1.27 cm) of horizontal deformation occurred between consecutive weekly readings.

- Three consecutive readings established a consistent trend of movement, and the total movement over the three readings equaled or exceeded 0.5 inches (1.27 cm).

- A probe was unable to travel freely or to advance beyond a certain point, requiring an immediate report.

(b) An inclinometer probe used to read instruments onsite was also checked during the same week by taking a reading in the inclinometer calibration hole.

(8) Extensometers. The extensometers at the site were also read weekly, but did not yield very accurate data. The threshold limits were exceeded if:

(a) Vertical movement between consecutive weekly readings exceeded 1.2 inches (0.1 feet or 3.05 cm).

(b) Three consecutive readings established a consistent trend of movement, and the total movement over the three readings equaled or exceeded 1.2 inches (0.1 feet or 3.05 cm).

(9) Crack Pins. Crack pins were installed to monitor cracking in the crest immediately upstream of the old ICOS wall. The inclinometers in the ICOS wall showed little movement, but noticeable displacement occurred in the upper portion of the embankment at the crest. Although not deep, the movement was a safety concern. Therefore, the crack pins were monitored daily. The threshold limit was exceeded if:

(a) Horizontal movement between consecutive daily readings exceeded 0.1 inch (2.5 mm).

(b) Vertical movement between consecutive daily readings exceeded 0.25 inches (6.4 mm).

(10) Settlement Monuments and Survey Observation Points. A sheetpile wall was installed to retain the embankment adjacent to the work platform. Settlement monuments and survey observation points were established to monitor the embankment and the sheetpile wall. Thresholds were exceeded if:

(a) The change between consecutive low-level surveys performed 6 months apart exceeded 0.6 inches (1.52 cm) vertically or horizontally.

(b) Three consecutive surveys established a consistent trend of movement, and the total movement over the three readings equaled or exceeding 0.6 inches (1.52 cm) vertically or horizontally.

e. Conclusions.

(1) Long-term surveillance and monitoring will continue after construction is completed at Wolf Creek Dam to determine the long-term effectiveness of the seepage cutoff wall. The complicated karst foundation makes this evaluation complex. However, as shown in the examples, initial instrument data indicates success.

(2) The robustness of the construction monitoring justifies confidence that the wall conforms to exact specifications and unprecedented tolerances. The surveillance and monitoring plan implemented during construction contributed to the success. Quick comparison of instrument reactions during construction kept project engineers and geologists informed, ensuring that the wall was constructed safely.

f. References.

(1) Fetzer, C. (1979). "Wolf Creek Dam: Remedial Work Engineering Concepts, Actions, and Results." Commision International Des Grandes Barrages, pp. 57–82.

(2) Mackey, S. and Haskins, T. (2012). "The Initial Grouting in the Embankment Foundation at Wolf Creek Dam Near Jamestown, KY." Grouting and Deep Mixing , pp. 1324–1334.

(3) Simmons, M. (1982). "Remedial Treatment Exploration, Wolf Creek Dam, KY." *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, pp. 966–981.

(4) U.S. Army Corps of Engineers (USACE). (2005). Seepage Control Major Rehabilitation Evaluation Final Report. USACE Nashville District, Nashville, Tennessee.

(5) USACE. (2013). Wolf Creek Dam Safety Major Rehabilitation, KY: Fact Sheet. Retrieved on July 8, 2013, from USACE Nashville District, http://www.lrn.usace.army.mil/Media/FactSheets/FactSheetArticleView/tabid/6992/Article/6242/wolf-creek-dam-safety-major-rehabilitation-ky.aspx

(6) Zoccola, M., Haskins, T., and Jackson, D. (2009). "Seepage, Piping, and Remediation in a Karst Foundation at Wolf Creek Dam." 29th Annual United States Society of Dams Conference Proceedings, pp. 1467–1474.

B.5. <u>Case Study 3, Wood River Flood Protection Project, Upper Wood River Levee System, Illinois</u>.

a. The Wood River Flood Risk Reduction project was originally authorized by the 1938 Flood Control Act. Between 1949 and 1960, the St. Louis District issued numerous construction contracts to build/improve levees in the Wood River Drainage and Levee District. As originally authorized, the project includes approximately 21 miles of mainline levee, 170 relief wells, 26 closure structures, 41 gravity drains, and 7 pump stations.

b. Figure B.13 presents the official project map. The levee is authorized to provide flood protection from a stage height equal to or greater than 54 feet on the St. Louis Mississippi River gauge, which corresponds to a maximum design flood elevation of 443.5.

c. The entire system exists in Madison County, Illinois, situated on the left descending bank of the Mississippi River in three separate levee districts.

d. The Wood River Drainage & Levee District (D&LD) Upper Levee System (National Levee Database System ID 5605470002) is approximately 5.2 miles long. The project originates near the intersection of Langdon and Front Streets (U.S. Highway 67) in Alton, Illinois, at Mississippi River Mile 203. From there, it extends downstream past the Melvin Price Locks and Dam (approximate river mile 200.8) to the mouth of the Wood River at Mile 199.4. There it proceeds along the right descending bank of the Wood River for 1.6 miles to the project terminus.

e. The levee system originally included nine closure structures, of which seven serviced railroad tracks and two serviced roadways. Most of the closure structures have been recently rehabilitated with gate repairs or replacement and other mechanical and structural repairs. Railroad Closure CS-9 at Station 271+50 was abandoned and removed in 2011.

f. The levee system includes four gravity drains, all of which have been rehabilitated with grouted high-density polyethylene (HDPE) liners and mechanical and structural repairs. The current East Alton pumping station is located at Project Station 133+70 and represents a penetration through the levee.

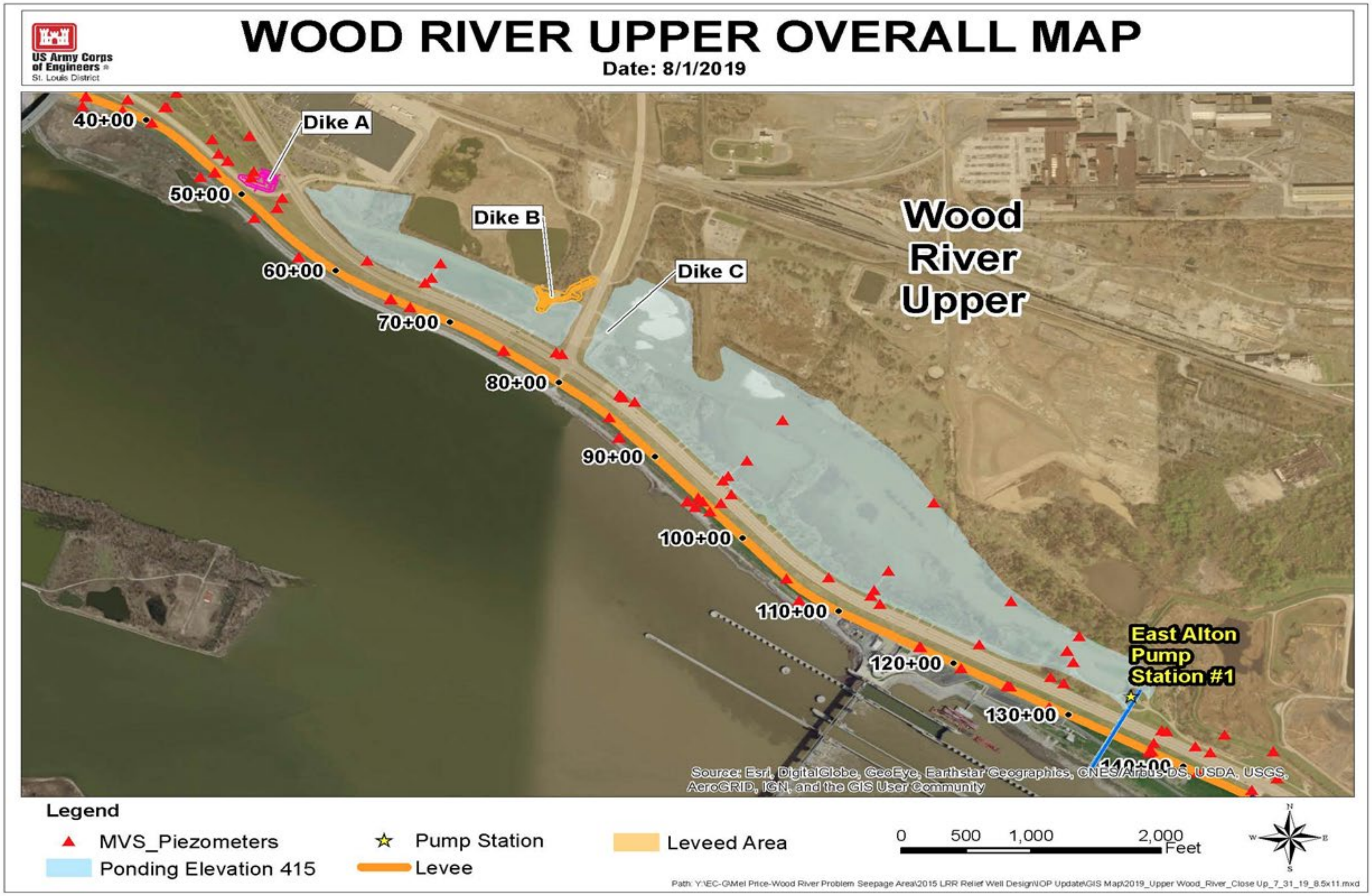


Figure B.13. Upper Wood River Project

g. The following case study and discussion outlines the use of instruments and instrument data in a risk-informed context for a levee system within the USACE portfolio. The focus is on assessment of piezometric data in order to optimize the operational actions of the interim risk reduction measures (primarily relief well pumping and landside ponding) and is included in the following paragraphs.

B.5.1. Surveillance and Monitoring Plan.

a. The surveillance and monitoring plan for the Upper Wood River Levee System was developed in coordination with the Risk Cadre that was assigned to perform the risk assessment for the system. This cadre consisted of Risk Management Center, Mississippi Valley Division, and South Pacific Division staff form various disciplines related to geotechnical engineering and geologic engineering.

b. The instrument monitoring supports assessment of seepage-related potential failure modes, particularly backwards erosion and piping. The main purpose of the surveillance and monitoring plan was to better monitor the groundwater regime and pressures associated with the normal river levels and provide information for any operational changes of the IRRMs at elevated river levels.

c. Gathering and evaluating this data could then allow for an enhanced understanding of the probable failure modes associated with internal erosion and BEP that were identified during the PFMA session.

d. Based on an assessment of the most pertinent probable failure modes, PZs were selected to better monitor groundwater pressures and evaluate gradients. The monitoring of the PZs would allow for a better understanding of the risk as well as adjustments of any interim risk reduction measures while a more permanent remediation was being developed.

e. The Risk Cadre along with MVS staff identified three critical locations utilizing 2D seepage modeling, past performance indicators (e.g., sand boils) and other geomorphologic factors to select these locations for the instruments. The team designed three lines of PZs with four landside PZs extending away from the levee toe a distance of up to 1,000 ft.

f. A greater density of the PZs were selected to be installed in and around the levee toe. Generally, the intent was to be able to identify the piezometric grade line in the immediate landside of the levee. A few of the lines included PZs near the levee centerline and even on the riverside in order to better identify the seepage entrance.

g. MVS contracted for the installation of 10 new PZs in the seepage area exhibiting the most critical conditions. Eight of these were arranged in 2 ranges of 4 PZs located downstream of Cpl. Belchik Road as follows:

(1) First PZ at the landside levee toe along the existing line of relief wells.

(2) Second PZ located approximately 70 feet landside of the well line.

(3) Third PZ located approximately 170 feet landside of the well line.

(4) Fourth PZ located approximately 700 feet landside of the well line.

h. Sensors were added to these PZs to automate the collection of the piezometric data. The sensor are the LevelLogger-Gold as manufactured by Solinst. The cabling from each sensor (on the levee side of the drainage ditch) was run back toward the levee toe and terminated in weather-proof housings. This enables the technician to poll multiple sensors from one location. Due to their relatively remote locations, PZs PZ-07, PZ-08, and PZ-Ponding must be individually visited.

i. The instruments were programmed to obtain and store a reading every 4 hours. The data is collected on a weekly basis and can be collected more frequently as necessary. Initial operational difficulties (icing and flooding) were overcome, and staff has been collecting the piezometric data on a regular basis.

j. Figures B.15 through B.17 present plots of measured piezometric elevations, Melvin Price pool and tailwater elevations, and landside ponding with respect to time for piezometric lines 1, 2, and 3. Study of the data shows that the groundwater regime landside of the levee responds to both changes in the Melvin Price pool elevation and changes in the landside ponding elevation. It is difficult to separate the exact contribution of both of these phenomenons.

B.5.2. Instrumentation Assessment & IRRMs.

a. PZs provide an indication of groundwater pressure. In Upper Wood River, the PZs indicate an artesian condition at the landside roughly at a river level with an AEP of 99%. In other words, during “normal” river conditions (i.e., non-flood stages) the levee foundations are hydraulically loaded by the Mississippi River.

b. During elevated river levels, above roughly elevation 419 NAVD88, the PZs responds fairly readily with the river and the rate of groundwater pressure increase is about equal to the rate of river rise. This can be seen in Figures B.15 through B.17.

c. Information about the geology in the area confirms that the highly pervious foundation aquifer beneath the levee can carry large amounts of flow beneath the levee foundation. Additionally, information about the near surface geology confirms that a very thin impervious layer of silts and clays confines this highly pervious aquifer.

d. With the increase in river levels and subsequent increase in groundwater pressure, the likelihood of an internal erosion failure mode (particularly BEP) is increased. This has been confirmed by the instruments, but also by visual observation of sand boil and seepage activity at the landside levee toe. Deterministic calculations of exit gradients also confirm low Factors of Safety (< 1.0) when projecting piezometric readings along the entire height of the levee.

e. One of the main functions of the instrumentation at the Upper Wood River Levee is to measure effectiveness of IRRMs. As seen in the figures below, both the landside ponding and the air lifting of relief wells influence the groundwater pressure and thus reduce the likelihood for initiation of BEP. In particular the effectiveness of the airlifting can be observed by the nearly sudden drop of the piezometric level once the air lifting compressors are turned on, as shown in Figure B.15 and Figure B.17.

B.5.3. Conclusions.

a. The results of the instrumentation assessment has indicated in certain instances that the different IRRMs can be modified or delayed depending on the projections and forecasts of the different flood levels. This has allowed for a more effective use of limited funds for the operation of the IRRMs. In addition, the instrumentation data was helpful in informing quantitative risk assessments utilized to inform future and further risk management measures.

b. Together with risk assessment tools and methodologies, the instrumentation at the Upper Wood River levee has not only served to better inform our understanding of the levee performance and levee risk, but it has allowed for a more effective management of the IRRMs being implemented at the levee.

Figure B.14. Instrumentation Plan View

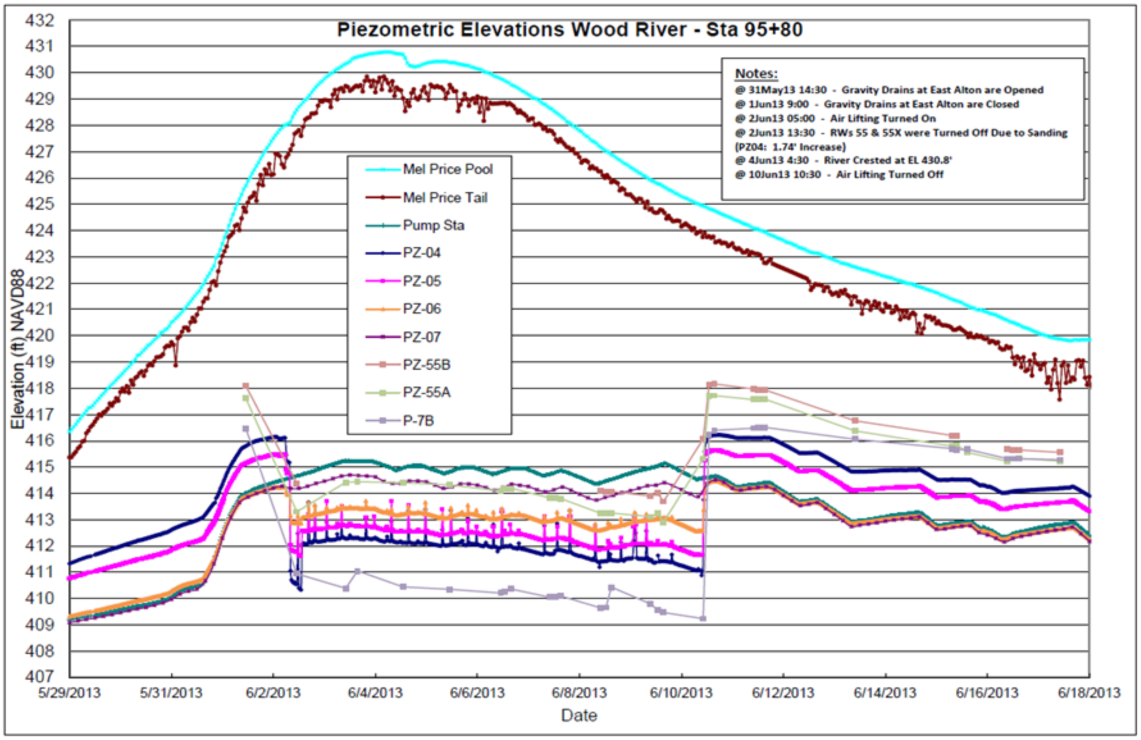


Figure B.15. Impact of IOP on Seepage Pressures: Station 95+80

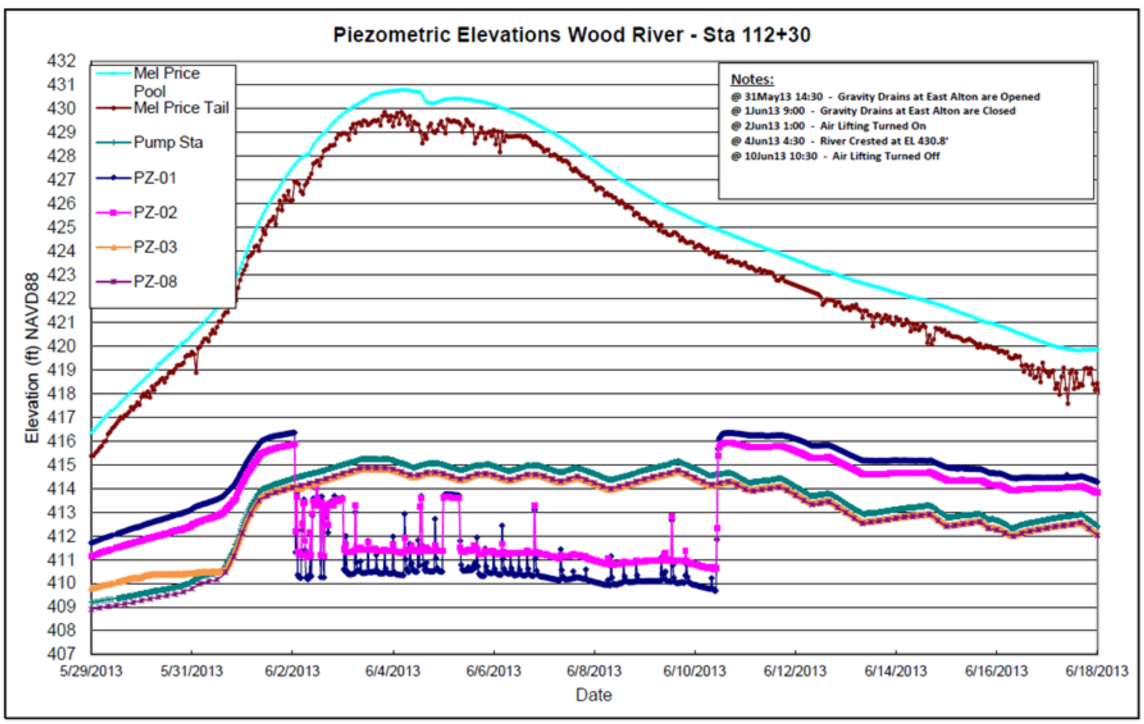


Figure B.16. Impact of IOP on Seepage Pressures: Station 112+30

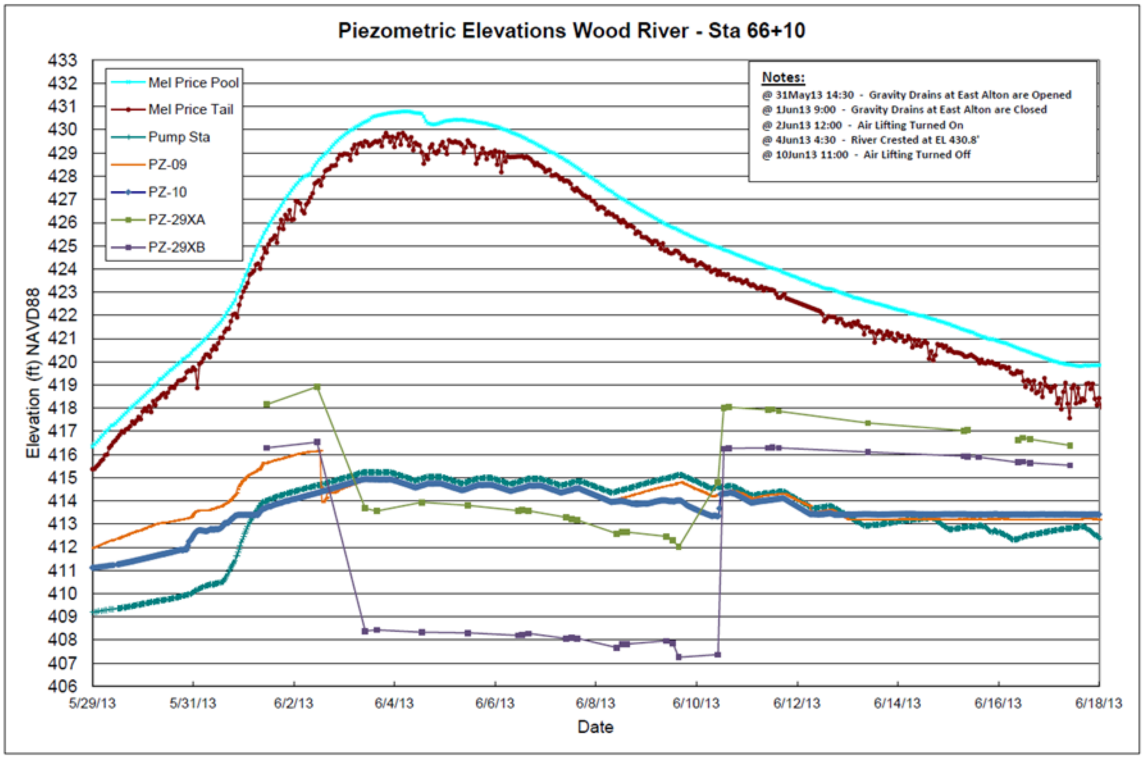


Figure B.17. Impact of IOP on Seepage Pressures: Station 66+10

Appendix C
Open Standpipe PZ Response Tests and Rejuvenation Procedures

C.1. This appendix describes procedures for testing and rejuvenating open standpipe PZs. The procedures are general and may be modified or adapted to suit site-specific conditions. Response tests determine the performance of a PZ. PZs with unsatisfactory performance are rejuvenated.

C.2. A PZ response test determines the time required for the water column in a PZ to equilibrate. The greater the delay, or response time, the less accurately the PZ level keeps pace with changing porewater pressure in hydraulically confined zones of the embankment or foundation.

C.3. Resistance to flow in the filter and screen contributes to the delay. Therefore, rejuvenation consists of reducing hydraulic resistance in the filter and screen. Open standpipe rejuvenation methods are similar to water well development and rejuvenation methods, and may include bailing, air lifting, surging, bleach or acid treatment, detergents, or brushing.

C.4. Criteria in ER 1110-2-1807: Drilling in Earth Embankment Dam and Levees Rejuvenation should be reviewed prior to rehabilitation or rejuvenation to determine if a drilling plan is required. Response testing and rejuvenation efforts can damage the embankment, foundation, and/or PZ.

C.5. Increased water, grout, or air pressure should be applied only with full awareness of the consequences, particularly for PZ tips in the impervious core or in the foundation beneath the core. Water used for flushing or response testing PZs should be potable to avoid bio-fouling.

C.6. Open Standpipe PZ Response Test Procedures.

a. Open standpipe PZ response tests are performed by quickly raising or lowering the water column in the PZ and determining the time required for the water column to achieve equilibrium with the porewater pressure. If water is added to the PZ, the water column falls to achieve equilibrium (a falling head test). Therefore, a falling head test, or slug test, is performed by quickly adding a measured quantity of water and measuring the depth of the falling water surface over time.

b. In contrast, if water is removed from the PZ, the water column rises to achieve equilibrium (a rising head test). Therefore, a rising head test is performed by quickly removing or bailing water from the standpipe and measuring the depth of the rising water surface. The movement of the water column continues at a decreasing rate as the test proceeds.

c. Falling and rising tests provide the same information. However, bailing water is preferred to adding water because bailing provides a water sample and draws water into the standpipe, possibly flushing debris out of the screen. The water sample can be examined for solids. Unlike bailing, adding water could force solids accumulated in the PZ into the screen, plugging the openings. Bailing does require special equipment, and a small-diameter standpipe or obstructions in the standpipe can make bailing impossible.

d. Open standpipe response test procedures are discussed under the headings of:

(1) Preparation.

(2) Sounding.

(3) Falling or Rising Head Test: Procedure 1.

(4) Falling Head Test: Procedure 2.

C.6.1. Preparation.

a. Preparation for PZ response tests includes assembling records, determining a typical response time, and checking filter criteria.

b. Good records include:

(1) Instrument identification number.

(2) A well or borehole log including device (standpipe, casing), backfill column (diameter, seals), and stratigraphy.

(3) Standpipe diameter.

(4) Elevation of the top of standpipe, ground surface, and screen tip.

(5) Construction details (e.g., dates drilled or extended during construction).

(6) Soil or rock influence zone.

(7) Screen tip details (e.g., slot or pore size, length, diameter, and filter materials).

c. A typical response time should be determined for comparison with the test results. The typical response time is based on experience of the response times of other instruments of the same type installed in the same type of earth or rock. Table 5.1 lists estimates of PZ response times.

d. The planning of PZ rejuvenation should include a check of filter criteria for preventing the migration of soil particles. Filter criteria are checked by determining screen slot or pore size, filter material gradation, and soil gradation surrounding the filter and performing filter calculations.

e. If the filter parameters are not known or the filter calculations indicate migration may occur, then the decision must be made whether to proceed with the rejuvenation. If the decision is to proceed, in some cases, the work can be made safer by limiting the head change that the filter is subjected to.

f. A second restriction may be to limit the concentration of soil fines in the water. For example, if a filter is functioning properly, the water should not become cloudier during rejuvenation. Therefore, if the water does become more and more cloudy, soil migration is indicated. Particularly if the filter parameters are not known, detecting increasing cloudiness in the water can justify a halt to the work.

g. The restrictions of limiting changes in head and the concentration of fines are examples that may prove effective in some cases. However, the engineers and geologists planning the rejuvenation of a PZ should determine what restrictions permit the safe rejuvenation of a PZ with a filter of doubtful adequacy.

C.6.2. Sounding.

a. Persons testing and maintaining a PZ need to know the depth of the water column in the PZ because accumulated solids in the bottom of a PZ can affect test results. Due to an ongoing accumulation of solids, the actual depth at the time of testing may not be the same as the as-built depth or the depth at the time of a previous test or maintenance.

b. Sounding is the measurement of the depth of a water column by lowering a weight tied to the end of a line or tape and can determine the depth to the top of a layer of accumulated solids.

c. The depth to the tip of the PZ should be sounded and recorded. The top of a layer of accumulated solids may not be firm. Therefore, sounding with a comparatively blunt weight affixed to the end of a measuring tape may afford a better feel of the bottom softness than an instrument such as a water-level indicator.

d. If the sounded depth is less than the as-built or previously sounded depth, then material may have accumulated in the PZ tip. PZ logs should be maintained to document measured depth and any removal of solids.

e. Solids may accumulate due to:

(1) A broken riser pipe.

(2) A displaced seal in the riser.

(3) Filtration through the screen.

(4) Vandalism via an unlocked cap.

(5) Inundation at the surface.

(6) Biological debris.

f. Solids accumulation due to a broken pipe is difficult to remedy. If the accumulation of solids is continual and the amount of accumulation indicates a broken pipe, the PZ may need to be abandoned. In some cases, a camera inspection can be performed to verify that a pipe is broken.

C.6.3. Falling or Rising Head Test: Procedure 1. As a general guideline, a PZ responds accurately with minimal time lag if the responsiveness is greater than 80% in 5 minutes, although the guideline may be adjusted for specific sites. The Falling or Rising Head Test: Procedure 1 is a quick test of PZ lag and is described in eight steps.

a. Measure initial depth to water in the PZ riser tube.

b. Determine if a space exists between the PZ riser tube and an outer casing. If a space exists, use a water-level indicator to measure the water level in the annulus. The falling or rising head test should not have an impact on the water level in the annulus. If the annulus water levels change as a result of the test, a leak in the PZ system is indicated.

c. Start the timer and immediately add water to (or bail water from) the PZ riser pipe.

d. Measure and record depths to water in both the standpipe and annulus at 1-, 2-, 5-, and 10-minute intervals. However, a measurement that indicates the initial water depth has been restored is final.

e. Determine the drawdowns of the depth measurements as the difference between the initial, static water level and the measured water level at the times of measurement.

f. Plot the drawdown data and project the curve, if necessary, to estimate the 90% response time.

g. Determine the responsiveness of the PZ using Equation C.1:

$$R = 100\ (D_5/D_0) \qquad \text{(Equation C.1)}$$

where:

R = responsiveness, percent
D_0 = Initial depth of water surface, ft
D_5 = Depth of water surface at 5 minutes, ft

h. If the responsiveness is less than 80%, calculate the response time using Equation C.2. If the field check for responsiveness is vastly different from the t_{90}, then use Procedure 2.

$$t_{90}=3.3\text{x}10^{-6}\,\frac{d^2 \ln\left([L/D]+\sqrt{1+\left(\frac{L}{D}\right)^2}\right)}{kL} \qquad \text{(Equation C.2)}$$

where

t_{90} = time at 90% response, days
d = inside diameter of standpipe, cm
L = length of screen and sand pack, cm
D = diameter of PZ tip (screen) or sand pack as appropriate, cm
k = permeability of soil, cm/sec

C.6.4. Falling Head Test: Procedure 2.

a. Falling Head Test: Procedure 2 is a more advanced method of determining PZ lag time than Procedure 1 and should be applied if the results of Procedure 1 indicate the need. The theoretical response is based on the assumption of a PZ installed in homogenous, isotropic soil. Therefore, the response curves may be unrealistic if applied to a PZ installed in jointed rock. The response curves may also provide information about the PZ and the associated representation of the groundwater.

b. The information required to apply Procedure 2 includes:

(1) Riser diameter.

(2) Effective well radius (diameter of sand pack or screen).

(3) Top of riser elevation.

(4) Length of screen and sand pack.

c. The water level must be above the screen and sand pack if the Procedure 2 equations are to correctly estimate the initial rise or fall due to adding or bailing water.

d. Procedure 2 is based on an equation for PZ response obtained by equating a Darcy equation for water draw from a well to the water volume required to fill the standpipe. The equation may be derived for radial or spherical flow patterns. The form of Equation C.3 describes a radial flow pattern to or from the PZ screen. In contrast, a variant of Equation C.3 describing a spherical flow pattern simply substitutes 4π for π in Equation C.3.

$$\Phi_i=\Phi_\infty+(\Phi_o-\Phi_\infty)e^{-\left(\pi r_w k/A_p\right)(t_i-t_o)} \qquad \text{(Equation C.3)}$$

where:

Φ_i = water level in PZ at time, t_i
Φ_∞ = water level in PZ after infinite recovery time (or the initial water level before adding or bailing water)
Φ_o = water level in PZ at initial reading time (immediately after adding or bailing water)
r_w = effective radius of the PZ
k = permeability
A_p = cross-sectional area of the PZ riser pipe
t_i = time at which PZ is read
t_o = initial reading time (immediately after adding or bailing water)

e. Procedure 2 is described in nine steps.

(1) Measure the initial depth to water to the water surface in the PZ riser tube.

(2) Determine if a space exists between the PZ riser tube and an outer casing. If a space exists, use a water level indicator to measure the water level in the annulus. The falling head test should not have an impact on the water level in the annulus. If the annulus water levels change as a result of the test, a leak in the PZ system is indicated.

(3) Start the timer and immediately add water to (or bail water from) the PZ riser pipe.

(4) Measure and record the water volume added or bailed.

(5) Measure and record the depth to the water surface at intervals that plot well on a logarithmic time scale. Typically, doubling each successive cumulative time works well (e.g., 1-, 2-, 4-, 8-, 15-, 30-, 60-minute intervals).

(6) If necessary, obtain additional readings on subsequent days until 80% of the initial change in head is recovered.

(7) Plot the field data water levels on the Y-axis and time on a logarithmically scaled X-axis, as shown in Figure C.1.

(8) Use Equation C.3 to calculate the theoretical water levels in the PZ at the times the depth to the water surface was measured.

(9) Plot the theoretical PZ water levels and the field data as shown in Figure C.1. Iteratively adjust the value of the hydraulic conductivity in Equation C.3 until the theoretical response curves bound the field data curve of PZ response. A computer spreadsheet can facilitate the iteration.

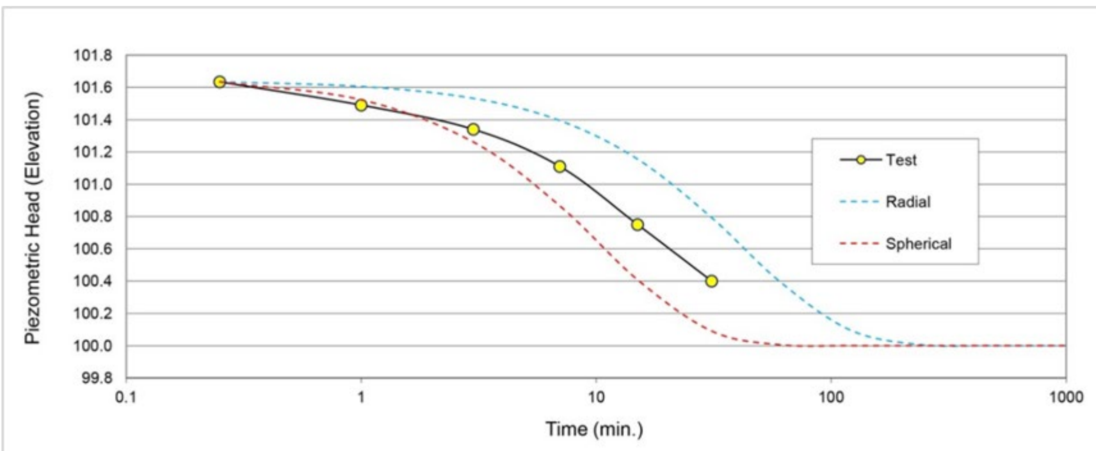


Figure C.1. Procedure 2 Response, PZ Head vs. Time

C.7. <u>Open Standpipe PZ Rejuvenation Procedures</u>. Open standpipe response test procedures are discussed under the headings of:

a. Suction lifting.

b. Flushing.

c. Air lifting.

d. Water lifting.

e. Documentation.

C.7.1. Suction Lifting.

a. Suction lifting depends on atmospheric pressure to force water and solids up the PZ and is not effective for water depths approaching or exceeding 33 feet (10.1 m), which is equivalent to one atmosphere of pressure. Suction lifting for a shallow PZ can be accomplished by:

(1) Air suction hose.

(2) Centrifugal pump.

(3) Peristaltic pump.

(4) Pitcher pump.

b. An air suction hose inserted in the PZ can convey water and solids. Although centrifugal pumps can be used for suction lifting, peristaltic pumps are recommended because the flow produced by a peristaltic pump is generally less than that of a centrifugal pump and more controllable. The pitcher pump is a common hand-operated piston pump for shallow water wells. Pitcher pumps can be used to purge shallow water-bearing PZs with a suitable pipe and screen size. The casing must be air-tight and able to support the pump.

C.7.2. Flushing.

a. Flushing is the use of a surface pump and water supply to inject water into the PZ until the circulation discharges from the top of the riser pipe at the surface.

b. PZs installed in the embankment core or in the foundation beneath the core should not be flushed due to the possibility of causing hydrofracturing as the water column in the PZ rises to the surface of the embankment. However, PZs near the embankment toe can be flushed with potable water to lift sediment out of the riser.

c. Flushing should be performed using flexible tubing with a tip that does not damage the PZ. Low pressure pumps, such as manual diaphragm pumps or peristaltic pumps, are preferred to prevent damage to the PZ. The pump capacity should be matched to the standpipe diameter and water losses into the aquifer.

d. The pump hose should be inserted to the tip of the PZ, the pump manufacturer's instructions for priming the pump should be followed, and water should be pumped into the PZ standpipe, increasing the flow rate until water discharges at the surface and sediment is lifted out of the PZ.

e. Flushing should be followed by a test of responsiveness. However, some time should elapse before the test begins to allow the water level to recover and suspended sediment to settle in the PZ. Bailing can shorten the recovery time. After sounding, the falling or rising head test is repeated to determine the PZ responsiveness.

C.7.3. Air Lifting.

a. Air lifting is the use of an air compressor or blower at the surface to inject air through a tube into the PZ tip so the bubbles lift the column of water and sediment. Air lifting is only feasible for standpipes of approximately 0.75 inches (1.91 cm) in diameter or less.

b. High pressure air lifting should not be applied to PZs in the embankment core or in the foundation beneath the core. Likewise, quickly injecting air at the base of a water column in the PZ could cause hydrofracturing.

c. For risers 0.75 inches (1.91 cm) in diameter or less, flexible tubing with a tip that does not damage the PZ should be used.

d. Air pressure should be adjusted to lift the column of water at a moderate rate, allowing capillary action between the air bubbles and water to lift the water to the surface. The unit weight of water is approximately 62.4 pcf, which translates to 0.43 psi per foot of head (9.7 kPa/m). Based on increasing the air pressure slightly above the static water pressure to initiate movement, an air pressure of 0.50 psi per foot of head (11.3 kPa/m) can be adopted, leading to the rule of thumb for air lifting pressure expressed in Equation C.4:

$$P = H/2 \qquad \text{(Equation C.4)}$$

where:

P = air pressure, psi
H = head, feet

e. The air hose should be inserted to the tip of the PZ and air injected until water emerges from the standpipe.

f. If the riser pipe diameter permits, an eductor nozzle should be used to avoid blowing high-pressure air through the screen. The eductor nozzle should be configured to move material from the bottom of the pipe and lift water.

g. Time should be allowed for the water level to recover and suspended sediment to settle in the PZ. Potable water may be added, if necessary, to shorten the recovery time. After sounding the PZ, the falling head test is repeated to check the PZ responsiveness.

C.7.4. Water Lifting.

a. Water lifting uses a small diameter down-hole pump to remove water from the PZ until the circulation discharges from a tube extending from the top of the riser. Water lifting is primarily useful for pervious soils that recharge the PZ quickly.

b. For low hydraulic conductivity soils, water is evacuated until the PZ is dry. However, for deep PZs, emptying the PZ could result in such a greatly unbalanced pressure that the PZ standpipe collapses. Deep PZs can be manually replenished with potable water to avoid excessive drawdown and standpipe collapse. The maximum allowable water drawdown depth for casing stability or screen filtration should be determined, and the pump intake should be set at or above the drawdown depth limit.

c. For inertial pumps, the tube depth should be adjusted so the limit of the down stroke is maintained safely above the PZ screen to avoid damage. Marking the tubing depth increments with tape is good practice. Inertial pumps are most commonly available for small diameter PZs.

For larger diameter PZs, other types of applicable pumps include submersible, helical rotor, bladder, piston, and gear pumps.

d. The pumped water should be collected in a bucket. The water should be checked for solids and odor. Flushing should continue until the water is clear.

e. After water lifting is complete, time should be allowed for the water level to recover and suspended sediment to settle in the PZ. Potable water may be added, if necessary, to shorten the recovery time. After sounding, the falling head test is repeated to determine the PZ responsiveness.

C.7.5. Documentation. PZ tests and maintenance should be documented. Figure C.2 is a blank sample form suitable for documenting the flushing of a PZ. Entries include measurements of depth and elapsed time, qualitative measures of sediment in the return flow, and remarks. The form is divided in five parts:

a. Heading.

b. Before flushing.

c. Flushing.

d. After flushing.

e. Remarks.

PIEZOMETER FLUSHING REPORT

Project:	Date:
Piezometer No.:	Flushed By:

BEFORE FLUSHING

Depth to Water: Bottom of Hole:

FALLING HEAD TEST

Time	Elapsed Time (minutes)	Depth to Water (feet)	Comments
	0		
	1		
	2		
	5		

FLUSHING

Start Time: Stop Time: Elapsed Time:

PRESENCE IN RETURN FLOW:

	A lot	Some	Little	None
Silt	_____	_____	_____	_____
Sand	_____	_____	_____	_____
Particulate Matter	_____	_____	_____	_____

AFTER FLUSHING

Bottom of Hole:

FALLING HEAD TEST

Time	Elapsed Time (minutes)	Depth to Water (feet)	Comments
	0		

Remarks:

General Comments: (needs painted, concrete collar chipped, etc . . .)

Figure C.2. Sample PZ Flushing Form

Appendix D
Instrumentation Evaluation Reporting

D.1. Purpose/Objective.

D.1.1. This appendix provides guidance and procedures for documenting, in a report, the result of instrumentation evaluation.

D.1.2. The effective communication of the information contained in the instrumentation data is essential for evaluating the performance of a dam or levee, and its foundation, and for estimating risk associated with the presence of the structure.

D.1.3. The objective of this appendix is to provide guidance and outline the tasks for evaluating, interpreting, portraying and reporting the information contained in this data. Proper communications of these data enables high quality evaluation of dam/levee and foundation performance and reduces uncertainty in the risk estimates.

D.1.4. For a new structure or when there is a major modification of a dam/levee, a report documenting performance during and post construction will be done in support of the Project Geotechnical and Concrete Completion Report and the follow-on Periodic Inspections.

D.1.5. If there are existing documents that address information required by the guidance in this appendix, then first review and evaluate those documents. Summarize the findings of those reviews and any pertinent information in this document and then refer to the appropriate locations in those documents for any detailed information that may be needed by a risk cadre or reviewer.

D.2. Background.

D.2.1. The goal of the instrumentation data documentation is to provide the information necessary for a thorough evaluation of instrumentation data and observed performance for the full monitored history of the structure.

D.2.2. The process of reviewing, compiling for presentation, interpreting, and evaluating instrumentation data and then assimilating it into a useful and concise format is extremely important for understanding the performance of a dam/levee and its foundation.

D.2.3. Observed performance will be compared with established performance thresholds based on the design assumptions and criteria and with related potential failure modes to ensure the dam/levee is performing as intended. Reviewing and summarizing this information should confirm that the data is being collected and managed in a proper manner that ensures a high level of data quality is achieved year after year.

D.2.4. The compiling of and clear presentation of this data can also provide critical information for decisions when unusual conditions occur over the life of the structure.

D.3. Instrumentation Data Management.

D.3.1. Present the district level and project level surveillance and monitoring plan and programmatic documentation that governs the data management and the data quality management procedures that are in place. Document the chain of responsibility for and the administration of the project instrumentation and monitoring for the district and for the specific dam or levee.

D.3.2. This documentation referred to below is typically some form of a generic ISO type documentation of the policies, processes, and procedures related to these activities. These documents might be district level or project specific, but should be fairly static once they are generated. Reference to these documents is adequate; do not repeat them in this report. Provide a summary of project specific documentation if it exists. If this district-level and project-specific documentation does not exist, then it must be developed.

D.3.3. Evaluation of any project level automated data collection and handling procedures.

D.3.4. Typical questions to be addressed in this part of the dam/levee performance documentation report are listed below.

a. What are the policies and procedures in place, and do they ensure the appropriate level and type of data to properly assess the performance the dam/levee?

b. What are and do the policies and procedures in place ensure the data is properly collected and in such a manner as to ensure the proper level of data quality?

c. What are and do the policies and procedures in place ensure the collected data is managed properly to prevent loss of the data, to ensure the data quality, and to provide the appropriate level of access to those that collect, evaluate, and use the data.

d. Is there a project-specific surveillance and monitoring plan in place? Present a summary of the requirements and refer to the surveillance and monitoring plan for details.

e. Does the project-specific surveillance and monitoring plan reflect increased surveillance needs according to pool/waterside level and seismic events?

D.4. Summary of Site-Specific Surveillance and Monitoring Plan.

D.4.1. Present, in summary form, descriptions of the type and condition of all performance monitoring instrumentation including survey monuments and the supporting documentation.

D.4.2. For each instrument document its measurement history: the date installed, repairs, changes, system upgrades, and any other significant event impacting the specific instrument. Show what and how parameters are monitored, the frequency of monitoring and review along with all significant, favorable and unfavorable, aspects of the measurement and observation record. Indicate significant events such as initial fillings, historical low- and high-water events, modification to the dam/levee, changes in operations, etc.

D.4.3. Inventory of Instrumentation.

a. What instruments are at the dam/levee? List all instruments at the site to include the model and serial number. This is best in a tabular format.

b. Where are the instruments? Show on the plan, map, and cross-sections. On the map or plan, show the dam/levee and all appurtenant features and structures. Show on the cross-sections the details of the dam/levee and the foundation geology: soils and rock.

c. What is the purpose of each instrument? State what the instrument is monitoring, what parameters are being measured, and how the instrument relates to a potential failure mode, or a particular aspect of performance.

d. Show the pertinent installation details for each instrument. Provide the coordinates and elevation of the instrument and for any monitoring points or sensing zones. State the degree of precision of the survey data and the datum and coordinate system used.

D.4.4. Data Acquisition Schedules.

a. What is the reading frequency? Does the surveillance and monitoring plan call for appropriate changes in frequency of data collection and visual observations in relation to pool/waterside level, seismic events, or specific performance of a given feature?

b. Provide the required and actual frequency of observation and reading for each monitored parameter.

c. Are other observable or measurable parameters monitored that trigger changes in the surveillance and monitoring plan?

D.4.5. Visual Observations.

a. Describe the location of visual observations, the parameter being monitored, the relationship that the parameter and location of observation have to potential failure modes, associated instruments, and the performance of the dam/levee.

b. Provide the required and actual frequency of observation and reading for each monitored parameter.

D.4.6. Automated Systems.

a. Present a summary of the automated system to include the component description including make and model, telemetry data transfer mechanisms and standards, application software and how it is tailored to the site, history of system upgrades, listings of equations and constants used for data reduction, status if instrument is active or inactive, and the reason for inactive status.

b. If an existing standalone document exists for the automated system with this information, then refer to that document and present a summary of the information in the instrumentation report.

D.4.7. Inventory of Available Data. Describe the quantity, period, and continuity of recordings and complete period of use for each measurement/observation.

D.4.8. Use of Plans and Sections.

a. As-Built Drawings. Include all original construction drawings related to instrumentation. Usually these include plans or schedules showing location, and typical construction details. Plan, profiles, and sections are to illustrate not only instrumentation location, but also features in common such as the instrumentation type, monitoring purpose, dam behavior parameters, criticality, and installation period.

b. Stratigraphy Details. Include plans, profiles, and sections of the instrumented features. PZs and other instruments should include sections showing the ground surface, stratigraphy, and measurement devices. Identify construction materials and when needed, cross-reference drill hole and instrument ID numbers. They are to reference the supporting borehole logs and as-built drawings as appropriate. Datum used for construction and data processing must be listed.

c. Post-Construction Mapping.

(1) When available and applicable, there may be a wide range of products available for specific projects, often dependent on previous studies or investigations. Some projects may include GIS products, such as instrumentation arrays overlaid on aerial photography.

(2) Previous investigations may have included cross-sections displaying stratigraphy, instrument measurement points, and measured water levels. Related instrumentation plan, profile, and section drawings should be cross-referenced. If tables summarizing location information are not included on plan, section or profile drawings then a separate cross-referenced table drawing must be created.

d. Boring and Well Logs.

(1) When available and applicable, include logs from drilling and installation for PZs, inclinometers, or other instruments installed in a bore hole. The logs should establish where the instrument is set in relation to geologic stratigraphy and constructed features. Drafted logs are preferable, but field logs will suffice. Any drawings should clearly document installation and construction details, when available.

(2) For unusual or unique instrumentation, provide details and descriptive information that is not readily available elsewhere.

D.5. Presentation of Data and Evaluation of the Data Quality.

D.5.1. Type of Data. Data to be presented typically will include visual observations, surfacing seepage/leakage and drainage, structural and foundation activities, piezometric and groundwater levels, and other measurements or behaviors such as reservoir operation, precipitation, seismic events, and landslide deformations.

D.5.2. Presentation Format.

a. Present the data in the appropriate graphical format: the time/history; deformation over time; deformation in relation to a point of reference; readings versus the influencing variables, response to pool, and air temperature; and measured parameter in relationship to performance thresholds.

b. For visual observations in general, report overall results of all visual observations using maps and available photographs.

c. For critical elements, scale drawings (typically with a 1:1 aspect ratio) depicting the dam's geometry, materials, and foundation geology that also show selected results from measured performance.

d. Provide miscellaneous figure details for unusual, unique or uncommon instrumentation for which ready access to descriptive information is not available elsewhere.

D.5.3. Data Quality Evaluation.

a. Present an evaluation of the quality of the data for each instrument and its value in relation to the intended purpose for the instrument. Add notations to data plots to include notes on the quality of instrumentation data, history of associated maintenance, calibration date and method, and the skill level and experience of the person making the instrument reading or observation to aid evaluation of performance. Note unusual aspects of the data record, such as periods of questionable or missing readings.

b. Discuss installation details and activities that may have affected measurements associated with a significant anomaly. Report whether or not reference points (e.g., survey control, inclinometer base, extensometer/standpipe head) are stable.

c. Report the accuracy standards (order and/or confidence limits) used for geodetic-based surveys.

D.5.4. Cause and Effect.

a. Present an evaluation of the data for each instrument in relation to "cause and effect" and changes in the dam/levee or foundation over time. Consider initial saturation phase during first-filling of the reservoir or the first levee high water event, changes in reservoir or watershed operation, major modifications to the dam/levee or its foundation, and changes in observed performance after high-pool/high-water events or drought.

b. For example, correlations based on the entire period of record for a PZ/weir are not appropriate for forecasting future performance if a seepage berm (i.e., a major modification) was installed 10 years ago that altered the performance.

D.6. Data Interpretation and Performance Evaluation Narratives.

D.6.1. General.

a. Bring the various types of data together and present the interpretation and evaluation of this synthesis of data as it relates to potential failure modes and general performance.

b. Data interpretation and evaluation includes a narrative assessment that places all data in a meaningful structural and geologic context that addresses both the behavior of the dam/levee and the instrumentation system. Use drawings, tables, and graphical displays that illustrate the observations and findings should be included. Graphical displays should be used for nearly all data and should be self-explanatory. Unsatisfactory performance, especially behavior related to potential failure modes, must be highlighted.

c. Describe and interpret the leakage, deformation, and piezometric pressure behavior of the dam that is indicated by measurements and observations.

d. Evaluate the effectiveness and performance of individual instruments and the instrument arrays. Highlight elements associated with those measurements that can provide an advanced warning of failure mode initiation.

e. If a potential failure modes analysis has been completed, make conclusions concerning the future usefulness of all monitoring elements based upon instrument and dam performance in view of the potential failure modes.

D.6.2. Visual Observations. For the visual observations report, in general, overall results of all visual observations to include items such as adequacy of coverage, and intermittent phenomenon such as wet spots. Highlight unexpected observations and include available photographs. Report how unexpected observations have been addressed.

D.6.3. Seepage, Leakage, and Drainage.

a. Report all flow quantities that are, or potentially are, a function of pool/waterside surface elevation. Discuss how other influencing factors, such as tailwater/landside water elevation, precipitation, snow melt, and groundwater affect measured monitored flows. Describe whether or not the relationship between influencing factors and the resulting flow quantity, quality, or location is consistent over time, and highlight increasing flow responses.

b. Report whether or not drainage features appear to lose capacity or convey less expected flow over time.

c. Report any measured or evidence of material transport by seepage flow. Where erosion of water-soluble salts may be a concern, report whether or not flow water quality differs from reservoir/waterside water quality and whether or not reservoir/waterside water quality is aggressive toward any foundation materials.

D.6.4. Structural and Foundation Deformation and Displacement. Highlight any indications of steady, accelerating, or otherwise unstable displacement rates. Report whether or not cyclic deformations remain predictable as functions of season or reservoir operation. Report any instances of differential deformations that could lead to cracking. For individual project features, report whether or not the maximum and minimum deformations or displacements occur at the expected locations. Highlight deformations that exceed expected magnitudes or the expected rate of progression.

D.6.5. Uplift, Piezometric Pressure, and Groundwater Level.

a. Highlight pressures or water levels that are greater than expected. Report whether or not the distribution of pressure conforms to the expected pattern. Report trends of increasing or decreasing pressure responses to reservoir/waterside surface level. Indicate the effectiveness (and any change in effectiveness that has occurred over time) of seepage cutoff features that affect the measurements.

b. Indicate the effectiveness (and any change in effectiveness that has occurred over time) of drainage features that affect the measurements. Estimate seepage exit gradients in erodible materials where pressures are measured near seepage exits. Estimate uplift on planes of suspected weakness such as lift lines, joints, shale seams, and foundation contact.

D.6.6. Seismic Monitoring. Present the evaluation of the performance of the dam/levee for any seismic loadings the dam has experienced. Present any measured ground motion records and their evaluation. Present and discuss any observed physical displacements or cracking due to seismic loadings.

D.7. Conclusions.

D.7.1. State any conclusions derived based on the evaluation of the district-level programmatic policies and procedures and the project-specific surveillance and monitoring plan pertaining to the amount and type of data and data quality to allow adequate evaluation of performance.

D.7.2. State any conclusions on the quality of the data actually collected. Based on the data interpretation and performance evaluation conducted, state any conclusions arrived at concerning the performance as related to any specific potential failure mode. Include conclusions derived from the interpretation and evaluation of any unexpected performance.

D.8. Recommendations. Present recommendations for additional instruments, removal of instruments from the regular reading schedule, increased or decreased monitoring, and different or revised graphics based on the evaluations and conclusions presented. Address such items as needed maps/drawings such that the locations of monitoring elements are fully documented. Highlight any weaknesses and make recommendations for improvements to monitoring, documentation, and the plans for responding to unexpected performance.

D.9. Instrumentation Report Format.

D.9.1. Summary.

a. A succinct summary must be written to describe the most important findings, implication, and conclusions pertaining to risk estimates. This summary should provide reviewers and decision makers fast access to the most significant information and interpretations.

b. This is not a lengthy discussion of the instrumentation data interpretation and evaluation, but a focused summary to pull only the most significant information together to quickly describe the performance of the dam/levee for evaluating risks associated with the credible and significant potential failure modes.

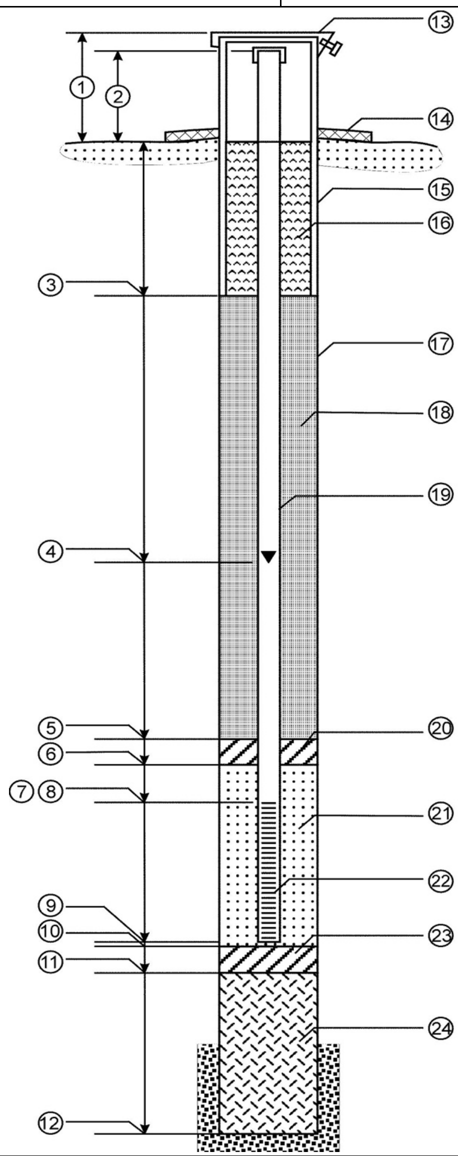
D.9.2. Report Format. There is no set format for this report, but it must contain the following: a title page, table of contents, appropriate district quality control and agency technical review documentation, the sections presented above in this appendix, a list of references cited, and any required figures, tables, charts, plots, and appendixes to support the instrumentation report.

LEFT BLANK INTENTIONALLY

Appendix E
PZ Installation Log

(next page)

Piezometer/Hole Number:		**Piezometer Installation Log**		LOCATION: Northing: Easting:
Drilling Contractor:		Project Name:		Date/Time Started:
Geologist:		Project Number:		Date/Time Completed:
Driller:	Drilling Method:	Ground Surface Elev. **ft.msl**		Top of Piezometer Casing Elev. **ft.msl**



1. Stick-Up of Protective Casing (If Any) Above Ground: ____________ft.
2. Stick-Up Piezometer Casing Above Ground: ____________ft.
3. Depth to Base of Surface Seal/Protective Casing: ____________ft.
4. Depth to Static Water Level: ____________ft.
5. Depth to Top of Screen Interval Seal: ____________ft.
6. Depth to Bottom Screen Interval Seal/Top of Sand Pack: ____________ft.
7. Depth to Top of Piezometer Screen: ____________ft.
8. Total Length of Blank Piezometer Casing: ____________ft.
9. Depth to Bottom of Piezometer Screen/Cap: ____________ft.
10. Depth to Base of Sand Pack/Top of Bottom Hole Seal: ____________ft.
11. Depth to Base of Bottom Hole Seal: ____________ft.
12. Depth to Base of Hole: ____________ft.
13. Protective Casing: ☐**Yes** ☐**No** **Locking Cap:** ☐**Yes** ☐**No**
14. Concrete Pad: ☐**Yes** ☐**No**
15. Type of Protective Casing: ____________Diameter: ______in.
16. Type of Surface Seal: ____________
17. Borehole Diameter: ____________in.
18. Type of Annular Seal: ____________
19. Type of Piezometer Casing: ____________Diameter: ______in.
20. Type of Screen Interval Seal: ____________
21. Type/Size of Sand Filter Pack: ____________Diameter: ______in.
22. Type of Screen Material: ____________Slot Size: ______ in
23. Type of Bottom Hole Seal: ____________
24. Borehole Backfill Material and Thickness ____________

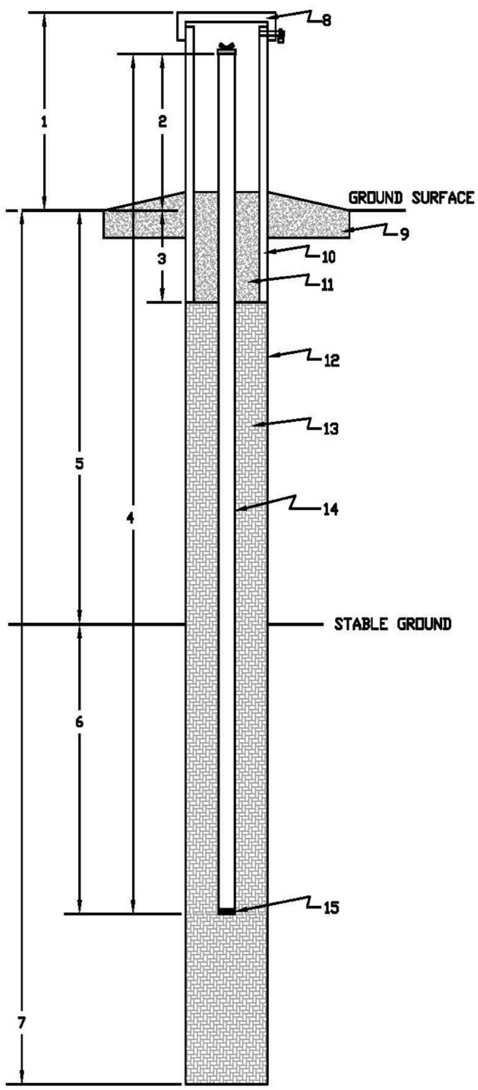
Recorded By	Date	Checked By	Date

Appendix F
Inclinometer Installation Log

See next page.

Inclinometer/Hole Number:	**Inclinometer Installation Log**	LOCATION: Northing: Easting:
Drilling Contractor:	Project Name:	Date/Time Started:
Geologist:	Project Number:	Date/Time Completed:

Driller:	Drilling Method:	Ground Surface Elev. **ft.msl**	Top of Inclinometer Casing Elev. **ft.msl**



1. Stick-Up of Protective Casing (If Any) Above Ground: ________________ft.
2. Stick-Up Inclinometer Casing (If Any) Above Ground: ________________ft.
3. Depth to Base of Surface Seal/Protective Casing: ________________ft.
4. Total Length of Inclinometer Casing: ________________ft.
5. Depth to Stable Ground Level: ________________ft.
6. Embedment Depth into Stable Ground: ________________ft.
7. Depth to Base of Drill Hole: ________________ft.
8. Protective Casing: ☐ **Yes** ☐ **No** **Locking Cap:** ☐ **Yes** ☐ **No**
9. Concrete Pad: ☐ **Yes** ☐ **No**
10. Type of Protective Casing:____________________Diameter:________in.
11. Type of Surface Seal:____
12. Borehole Diameter:____in.
13. Type and Depth Interval of Annular Backfill:__________________________

13a. Grout Mix (if used):____

13b. Grout Stage 1 Depth Interval (if used):__________________________

13c. Grout Stage 2 Depth Interval (if used):__________________________

13d. Grout Stage 3 Depth Interval (if used):__________________________

13e. Grout Stage 4 Depth Interval (if used):__________________________

14. Type of Inclinometer Casing:____________________Diameter:________in.
15. Bottom Cap:______________________________Diameter:________in.
16. Calculated Volume of Hole to Grout:______________________________
17. Actual Volume of Grout Used:______________________________
18. Orientation of Inclinometer Casing Grooves: ______________________

Recorded By	Date	Checked By	Date