of the potential for loss of fines and for poor conditions for compaction. Scarifying, discing, and compacting the subgrade are common foundation-preparation practices.

Provisions for seepage control in the foundation vary. Some embankment dams have no cutoff. Others have a simple cutoff trench filled with compacted impervious fill. High-embankment dams customarily have a grout curtain or a cutoff wall backfilled with concrete or impervious soil material. How well the cutoff is constructed will affect its performance and the potential for undesirable seepage. The decisions made and actions taken during foundation preparation and cutoff construction affect long-term performance.

Good-quality embankment construction requires careful control of material properties, moisture, and compaction. Weather is an important factor. The right materials that meet the designer's specifications must be placed in the right zones. Moisture control is important, both at the borrow area and on the fill to achieve the design density. Fill placed too dry may settle excessively upon wetting. Fill placed too wet may crack or may cause excess construction pore pressures. Frozen subgrade or fill is not allowed. Sufficient effort must be applied to compact soil to the specified density. Clean separation without contamination between zones is important to the proper function of the zones. It is important that filters and drains not become contaminated with excessive fines. Where coarse soils are used as fill, segregation of coarse particles must be avoided in areas where rock pockets may form and provide a path for undesirable concentrated seepage.

SOLUTIONS 4.0

Other construction practices may affect performance:

- Overworking of fill is avoided by carefully establishing construction traffic lanes.
- Contamination between zones is avoided by carefully establishing protected crossing areas.
- Compaction around penetrations and against structures is performed with care.
- Completed elements of the embankment are protected against the degrading effects of weather.

In summary, construction practices strongly affect long-term embankment performance.

Opportunities to monitor performance differ between prospective and existing embankment dams. The differences are illustrated in the figures in this chapter; the major difference is that some instruments such as liquid-level settlement devices, fiber optic cables for distributed-temperature sensing, total pressure cells, soil extensometers, and many instruments in the foundation cannot be installed in an existing embankment but are routinely installed during construction of a new embankment.

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10.3 INSTRUMENTED MONITORING

After the embankment is constructed, its performance indicators from settlement (change of shape), water pressures (reservoir and adjacent groundwater), and seepage (amount, clarity, and distribution) are measured and compared to expected performance. If responses equal or exceed expected responses, a dam is judged to meet performance criteria because it meets the required margin of safety. If a measured performance indicator suggests that a failure mode may be developing, then evaluation is required.

The level of effort involved to evaluate depends upon the significance of the difference between observed and predicted behavior. Several performance indicators of behavior may require measurement:

- Movement to assess change of shape,
- Water pressure,
- Stress, and
- Seepage and leakage to assess changes in flow quantity and quality.

Chapters 5 and 6 provided a palette of instrument and survey means and methods suitable for measuring performance indicators. Table 10-1 summarizes the possibilities.

Not all the properties listed in the table are, or necessarily should be, monitored for all embankment dams. For a concrete-face rockfill dam, the horizontal and vertical movements of the face, the settlement of the crest, and the quantity of seepage through the face may be more important than other performance indicators.

Detailed internal distribution of piezometric pressures may be very important in a homogenous embankment because the entire cross section may develop pore pressures that could cause slope instability.

Some of the performance characteristics that are considered important for the most common types of embankment dams are shown in Table 10-1. As discussed previously, the monitoring program for a particular embankment dam requires careful design to address the specific performance indicators of that dam.

Existing and prospective dams and typical locations of instruments to monitor their performance are shown in Figs. 10-4 to 10-9, with the key to the symbols representing the various instruments shown in Fig. 10-3.

Care must be exercised when making penetrations for installing instruments in an existing dam. Proper filter criteria and compaction around an instrument are needed to avoid opening a seepage path. Borings to install instrumentation, especially in a core or at the toe, can create a shortcut through filters that may fracture the core. Installing instruments in a core

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Table 10-1. Instrument and Survey Methods.

Performance indicator	Measurement location	Instruments - methods
External movement	Crest or other surface location of interest	Visual observations, geodetic survey
Internal movement	At points of interest within embankment	Inclinometer, settlement gauge, extensometer
	At points of interest within foundation	Inclinometer, extensometer, settlement gauge
Water pressure	Within embankment	Piezometer, observation well
	Within foundation	Piezometer, observation well
Stress	Within embankment	Total pressure cell
Seepage quantity	Where measurable	Calibrated container, weir, flume, flow meter
Seepage quality	Where measurable	Turbidity meter, water chemistry
Flow	At toe	Weir, flume, fiber optic cable for distributed temperature measurement
Earthquake response	Crest and adjacent bedrock	Seismograph

requires great care and is generally avoided unless there is a question that cannot be answered otherwise.

In new dams, current practice is to avoid installing instruments that extend upward through subsequent embankment lifts. Where extensions such as a pipe cannot be avoided, special care is required to fully compact the soil around the extension to reduce the potential of creating a seepage path. An extension at a lift surface is an obstruction for construction traffic. Marking is required to avoid damage.

The scope and procedures of installing an instrument penetrating an embankment requires special care. Design and peer review by experienced geotechnical engineers is required. Installation requires qualified personnel experienced with installation of similar instruments in embankments.

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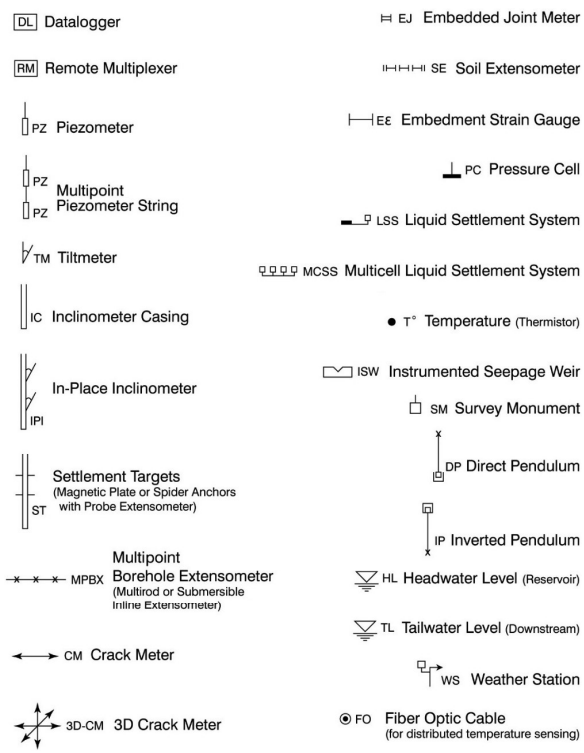

SOLUTIONS 4.0

Fig. 10-3. Key to symbols

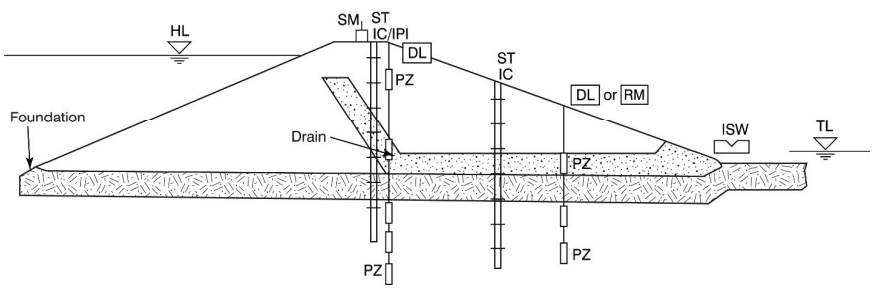

Fig. 10-4. Existing homogeneous embankment dam

10.3.1 Settlement

During the design of embankment dams, the engineer typically estimates the magnitude of foundation and embankment compression owing to the weight of the embankment. Compression of the embankment materials owing to self-weight consolidation is generally limited to about 1% of the

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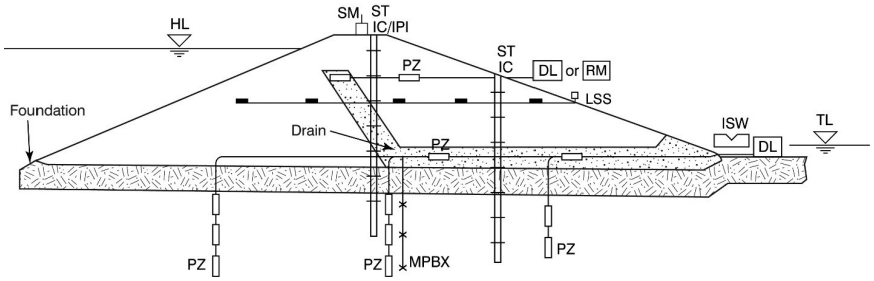

Fig. 10-5. Prospective homogeneous embankment dam

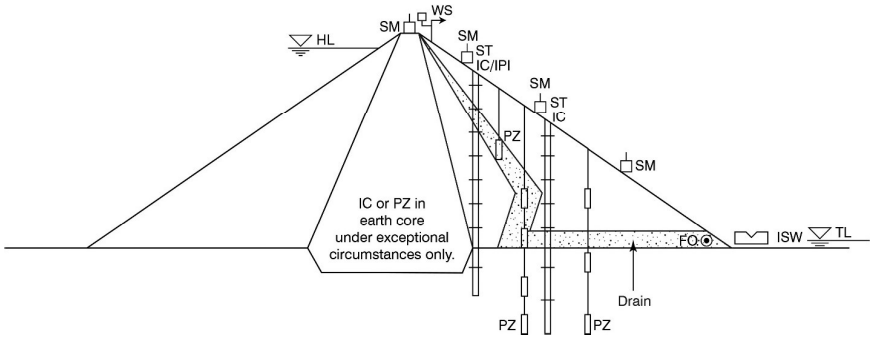

Fig. 10-6. Existing zoned embankment dam

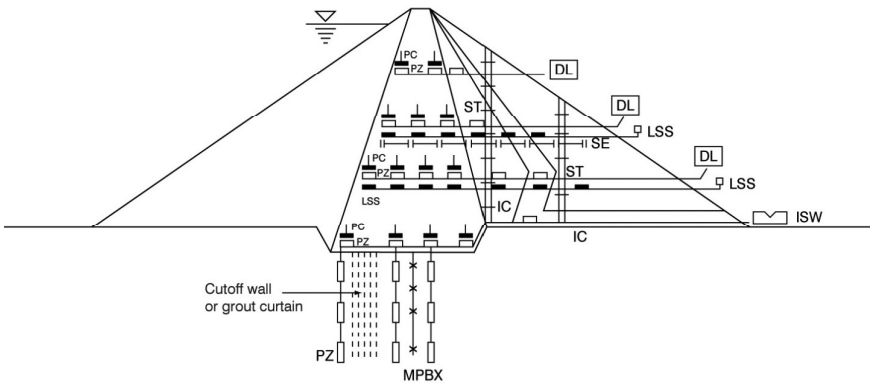

Fig. 10-7. Prospective zoned embankment dam

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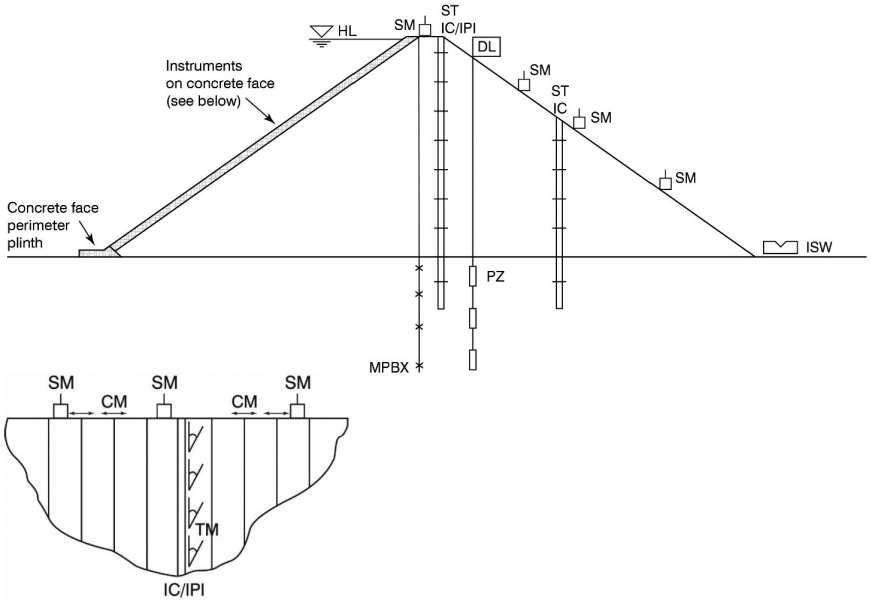

Fig. 10-8. Existing membrane (CFRD) embankment dam (section and plan of face)

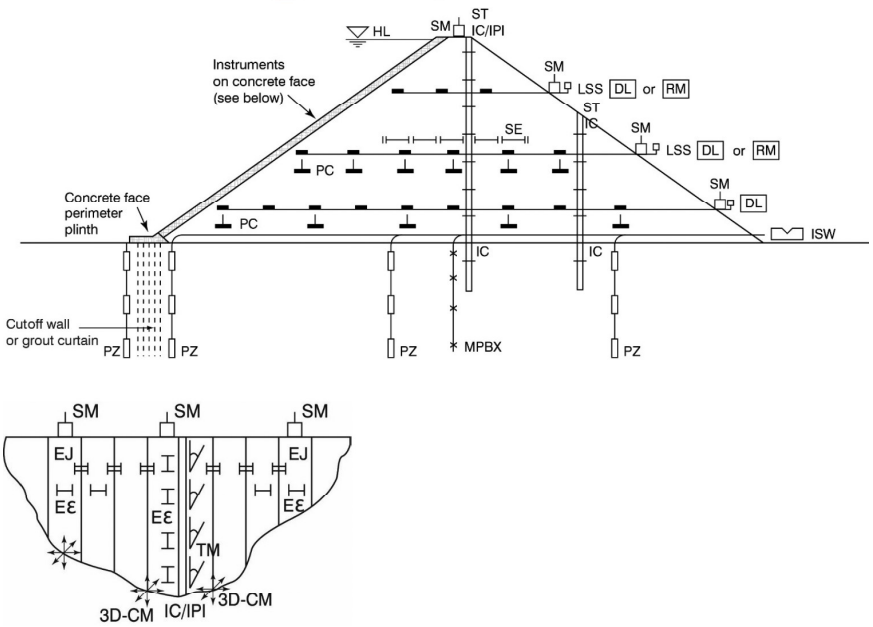

Fig. 10-9. Prospective membrane (CFRD) embankment dam (section and plan of face)

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embankment height in modern compacted embankment dams. Compression of relatively softer foundations can result in increased amounts of settlement. During an earthquake, shaking may cause embankment settlement or liquefaction of loose sands in the foundation. When establishing a design crest elevation for the dam, the estimated crest settlement resulting from the compression must be taken into account and the crest appropriately "cambered." This involves building the crest to an elevation higher in the middle tapering to the abutments to accommodate anticipated settlement. The desired result is that the settled crest elevation does not dip below the design elevation. This is extremely important with respect to the ability of the dam to maintain freeboard and withstand a flood without overtopping.

During construction, settlement platforms can be used to monitor settlement as embankment fill is placed. Settlement platforms are generally located at the maximum embankment section, the location of the maximum thickness of compressible foundations, and at other locations of interest to the designer where settlement magnitude has been estimated. Settlement platforms are particularly useful because the vertical riser pipe can be extended upward as fill is placed, allowing measurements during and after construction. Settlement is indicated by changes in the surveyed elevation of the top of the riser pipe with time.

Borros points and borehole extensometers are also used during construction. During construction, they can be used to isolate the settlement that has occurred in individual compressible foundation layers (below the construction subgrade). They are installed in cased holes made by rotary drilling methods.

After construction, settlement of an earthfill embankment dam is typically monitored along the crest and/or on benches of the downstream slope utilizing monuments that are periodically surveyed for changes in elevation. Monuments are typically concrete piers, usually flush with the ground surface, with a punch mark or "X" on a brass cap (as shown in Fig. 10-10) to indicate the exact survey point. These monuments may follow one of the designs presented in Chapter 6. Spacing of the settlement platforms and survey monuments depends on the size and length of the dam, as well as on the composition of the embankment materials, foundation and abutment conditions, and estimated magnitude of settlement.

If the foundation of the dam rises steeply at a stiff abutment contact (such as a bedrock abutment) and the main foundation consists of relatively compressible soil, then differential settlement caused by this abrupt change in shape and stiffness can cause transverse cracking perpendicular to the dam axis. This type of cracking can cause seepage problems internally within the dam and possibly damage filters that guard against internal erosion. Therefore, when there are steep abutment contacts, it is important to have a continuous settlement profile across the zone of abrupt shape change. Such a profile can be monitored using a differential settlement gauge (liquid-level

Fig. 10-10. Embankment crest monument
Source: Courtesy of DG-Slope Indicator, reproduced with permission.

SOLUTIONS 4.0

or overflow settlement cell), horizontal inclinometer, or hydraulic settlement profiler, which is best installed during construction of a dam.

Horizontal inclinometers can in some cases be installed after construction; however, this involves drilling a horizontal hole that requires more specialized drilling equipment. Access to achieve the proper orientation may be difficult. When installing during construction, care must be taken in the compaction of the lifts directly above the instrument so as not to damage it due to overstressing. Also, strain on the instruments, for example shear or tension due to stretching, which can occur owing to differential settlement, must be considered to avoid broken tubing, casing, cabling, or conduit. Differential settlement gauges can also be installed during construction and require the same care in installation and protection.

Installation of cabling and tubing in embankment dams, shown in Figs. 10-11A and 10-11B, presents vulnerabilities that must be considered in design and construction.

Cabling or tubing is installed in trenches to provide sufficient burial to avoid damage due to subsequent lift placement. Cable trenches passing entirely through the core of an earth or rockfill dam can lead to cracking and internal erosion and therefore, must be avoided. It is also usually desirable to minimize the cable runs and daylight the cable at the ground surface

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Fig. 10-11A. Horizontal cabling and overflow settlement cell installation in an embankment dam
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Fig. 10-11B. Vertical cabling, magnetic settlement target, and inclinometer casing installation in a CFRD dam
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in as short a distance as practicable. Horizontal routing of the cabling should avoid areas where differential settlement could induce damaging tension or shear. Vertical routing also has vulnerabilities in areas of significant settlement; the vertical tubing runs of instruments installed during construction on the Pyramid Dam in Southern California failed due to compression of a shear zone that consolidated excessively. Adequate compaction around vertically routed tubing, conduit, cables, or casing is an important consideration. Selection of flexible and ductile materials and installation procedures can have a large impact on the resistance of the instrument installation to overstressing due to compression, settlement, or compaction. The design life of the tubing is also very important. Tubing that becomes brittle can become a conduit of unintended water flow and pore pressure that can threaten core and filter zone material. Installation and protection of cabling/tubing during construction are discussed in Chapter 5. They are also discussed by Dunnicliff (1988), the U.S. Army Corps of Engineers (1995), and the U.S. Bureau of Reclamation (1987).

Permanent settlement of the embankment dam surface can also occur owing to deformation resulting from seismic response to an earthquake. Survey monuments on the downstream slope and crest of the embankment dam are typically used to monitor permanent settlement at the surface of the dam from seismic activity. In seismically active areas, it may also be advisable to place internal vertical movement devices, such as Borros point anchors or borehole extensometers, at strategic locations within the embankment to monitor continuity of the filters. These locations include the interface of the embankment fill with the foundation or at chimney or blanket drain interfaces. At these locations, permanent earthquake-induced settlement could damage drains and filters, possibly causing internal erosion or pore pressure build-up and associated slope instability owing to strength loss. Borros point anchors and borehole extensometers are discussed in Chapter 5.

ICE SOLUTIONS 40 – 51

Settlement observed or measured along abutment or structural contacts or above pipes and conduits passing through the embankment dam can be a sign of internal erosion of fines from the dam. Such observations require closer monitoring and appropriate investigation. Also, small, localized sinkholes on the crest or the upstream or downstream slope of an embankment dam can be a sign of internal erosion. It usually is not practical to specifically place survey monuments or internal instruments at all potential internal erosion locations. However, the design of the settlement monitoring system needs to consider the location of conduits.

Zoned earthfill dams are similar to homogeneous earthfill embankment dams except that embankment materials are placed in zones to meet stability and seepage control requirements. Zoned dams generally have a fine- (or finer)-grained core section flanked by more granular soil materials

upstream and downstream of the core. Appropriately graded filters designed to prevent potential erosion of core fines and possible development of internal erosion separate these major zones. Of particular concern for zoned earthfill dams is the difference in the compressibility of the different zones. This can result in a heightened potential for differential settlement across filter or drainage zones and the potential for longitudinal cracking along the embankment. To assess this potential for longitudinal cracking, it may be desirable to supplement traditional surface survey monuments, which monitor only surface settlement, with internal vertical movement devices such as Borros anchors, soil extensometers, or borehole extensometers. Such devices can be used to monitor settlement of deeper layers. Full-profile settlement gauges such as horizontal inclinometers, full-profile hydraulic settlement gauges, settlement plates (Fig. 10-12), or differential settlement gauges may also be useful. The use of these more sophisticated instruments is governed by the size and hazard potential of the dam and the degree to which differential settlement is perceived to be a potential problem.

Hydraulic fill dams are constructed by sluicing embankment materials into place with water. Although not generally used anymore for water-retaining structures, hydraulic fills are occasionally constructed to retain mine tailings. Hydraulic fill dams are more susceptible to liquefaction and/or slope movements during earthquakes. Regular settlement measurements and extra measurements after earthquakes are important.

In a zoned embankment, core materials are generally fine-grained soils, and the upstream and downstream shells consist of various gradations of rockfill. The core and the outer shells are typically separated by graded filters to allow development of a phreatic surface without internal erosion of core fines into the coarser downstream shell materials. A chimney and blanket drain downstream of the core draws down the phreatic surface and

Fig. 10-12. Settlement plate
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

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keeps the downstream shell materials relatively dry. Upstream filters guard against slope failure during drawdown.

Zoned rockfill dams are similar to zoned earthfill dams except that the difference in grain-size distribution between the zones of materials is much greater. These dams are generally constructed in mountainous regions where overburden soils are at a premium and the majority of available dam construction material is excavated and crushed bedrock. Differential settlement increases the potential for longitudinal cracking parallel to the crest. Settlement may result from relatively softer core material "hanging up" on the shell as it settles, which can result in a reduction of vertical stress in the lower portions of the core material, thus increasing the potential for hydraulic fracture. An abrupt change in abutment slope is a worthy candidate for measuring differential settlement because the effective stress in the embankment changes quickly over a short distance, leading to the potential for core cracking (Fig. 10-13). This mechanism was postulated as a contributing factor in the failure of Teton Dam in 1976.

Monitoring of zoned rockfill settlement can be performed in a similar fashion to earthfill embankment dams and zoned earthfill dams.

Embankments with upstream membranes are different because the water-retention capabilities are provided by a low-permeability barrier on the upstream slope. The seal may be constructed of concrete, asphalt, steel, or a geomembrane that is connected to the foundation and abutments at the heel with a concrete plinth and a grout curtain into bedrock. Modern designs commonly utilize a concrete upstream face and are referred to as concrete-faced rockfill dams (CFRD). The composition of dams with upstream membranes may include two transition zones between the upstream membrane and the general rockfill.

CFRD dams are not immune to developing problems. They are vulnerable to failure by internal erosion at the foundation level if they do not have adequately filtered internal drainage layers. Leakage can also occur owing to deterioration or cracking of the face caused by aging or differential settlement of the structure. Settlement and downstream movement of the embankment in the maximum section area, especially from any rock particle breakdown under load, bends the membrane, creating tension near the abutments and causing leaks. A concrete upstream face is sensitive to panel damage above the reservoir level during an earthquake that may crack concrete or tear waterstops, requiring repair before the reservoir can be filled.

At membrane dams, it may be important to supplement periodic surveying of surface monuments with appropriate measurements to verify satisfactory performance of the membrane itself (Teixeira da Cruz et al. 2010). Instruments may consist of internal movement devices installed in the membrane, such as embedment strain gauges and joint meters and at the perimeter of the membrane, with submersible 3D crack meters to

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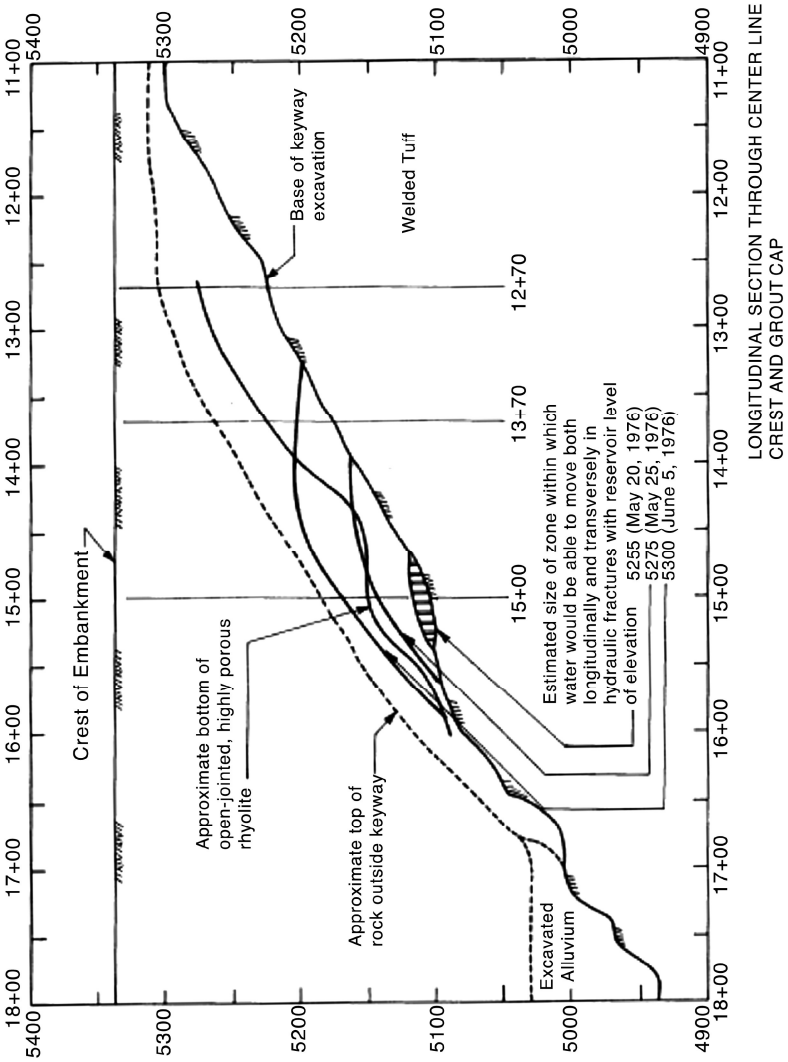

Fig. 10-13. Example of core cracking
Source: U.S. Bureau of Reclamation.

measure the relative movement between the membrane and the perimeter plinth, and on the membrane, such as inclinometers aligned to the upstream slope angle or a series of single-point submersible tiltmeters.

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10.3.2 Slope Movement

Measuring movement of embankment dams can be performed at the surface and in the subsurface. Instruments to measure slope movement include survey monuments on the crest or benches, manual or in-place inclinometers, and tiltmeters.

Well-designed and -constructed embankment dams are expected to settle. However, slope movement may not be the result of ordinary settlement. If slope movement measurements exceed expected movement, then a slope stability failure may be developing and require investigation of the cause.

10.3.2.1 Surface Measurements. Surface monitoring of slope movement is most often done by surveying the same monuments used to monitor settlement. These monuments are embedded in the embankment surface or mounted on structural components such as a parapet wall or concrete facing, in the case of a CFRD dam. Survey monuments can also be placed along the benches of an embankment dam or at other strategic locations where movement could be critical to the overall performance of the dam.

Successive measurements taken over time and plotted against reservoir elevation will show movement trends, if any. Data can be geo-referenced. Where possible, a survey traverse is made to distribute closure errors.

ICE SOLUTIONS 42.30

A temporary, quick, rough form of surface monitoring is by use of slope stakes. These are stakes for temporary use that can be put in a line across the slope both on and outside of a known area of movement. Continued movement can be assessed by sighting (or running a string line) across the stakes and noting any relative movement from previous observations. In addition, existing guard rails, fences, and utility poles along the crest of the dam make useful "qualitative" lateral movement monitors. Note that as with all instruments, bumps by construction or other traffic can break the continuity of the monitoring data. As such, instruments must be protected from disturbance.

10.3.2.2 Subsurface Measurements. Surface monitoring measures slope movement only at the surface of a slope. In contrast, subsurface monitoring instruments are designed to measure movement at one or more depths within a dam. Inclinometers and tiltmeters are described in Chapter 5. Inclinometers are located to measure movement at any depth of interest. If movement in the foundation is expected, then measuring at the toe of the slope will be the best location. Where the embankment slope is expected to fully contain a failure surface, measuring at the crest and at

depth intervals down the slope is best to intercept any potential slip surface.

Typical subsurface measurement locations for inclinometer casings and borehole tiltmeters are shown in Fig. 10-14.

Inclinometers are difficult or impossible to install in or through rockfill. Instead, visual observations detect slope changes but are not able to judge subsurface movement. Judgment is required to decide if a failure mode is developing.

Parapet walls and CFRD faces may crack in response to movement in underlying fill. Cracks can be measured. The distance between simple marks or rebar set on either side of an identified crack can be measured. Gridded plastic devices that fasten across a crack and electronic crack meters are also available.

10.3.3 Seepage and Leakage

Seepage flows through, around, and under all embankment dams. Acceptable seepage is benign seepage, being adequately controlled by the dam's drainage system. Observed seepage can be measured by several devices. Seepage that emerges at the toe may be acceptable if it runs clear without evidence of material transport or erosion of the embankment slope and does not increase (independent of reservoir level) with time. Seepage emerging on a downstream slope must be interpreted carefully because an elevated phreatic surface may be the prelude to slope instability. Seepage emerging along embedded conduits or concrete walls often is worrisome because internal erosion is more likely to occur along these interfaces than at other

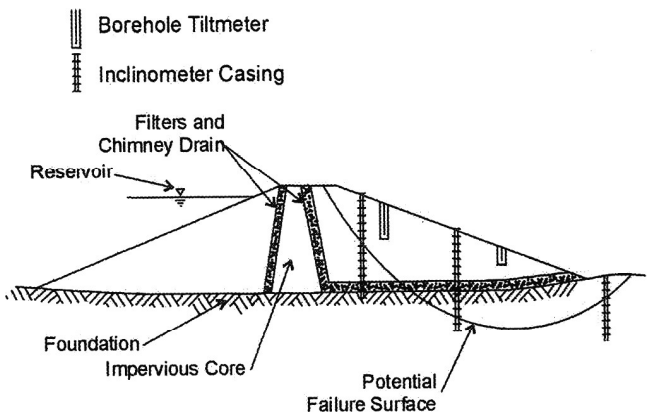

Fig. 10-14. Potential inclinometer and tiltmeter locations
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

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V-Notch Weir Parshall Flume

Fig. 10-15. Weir and flume

locations of a dam. Decreasing flow may indicate declining effectiveness of a chimney, blanket, or toe drain. Rainfall and reservoir level affect seepage. Plotting flow, rainfall, and net head versus time aids in effective interpretation. Seepage measurements can be compared with design assumptions and with prior measurements to decide if the measured seepage is evidence of a developing failure mode. Flow rate and pressure can be measured.

Many dams have a drain system that discharges at the toe. The toe drain system is designed to collect seepage flowing through both the embankment and foundation. Seepage is measured at any location of interest where seepage can be channeled. Increasing flow, particularly flow carrying sediment, may be a sign of internal erosion. Any measurement suggesting that internal erosion is occurring is worrisome and requires evaluation by a geotechnical engineer experienced in internal erosion. Typical locations to measure flow are at the toe and along groins.

Common types of seepage flow measurement instruments are weirs, flumes, and calibrated containers. A pair of weirs is shown in Fig. 10-15. Downstream topography and expected flow range are important considerations when selecting the seepage flow instruments. Weirs are appropriate for modest flow where a minimum of 0.2 ft (0.061 m) of head upstream of the weir is likely. In flatter areas where there is less head loss or at locations where higher flow exceeds weir capacity, flumes are used. Typical arrangements of weirs and flumes are described in Chapter 5. Care is required to isolate a weir from rainfall and surface runoff. Low flows can also be measured by the time it takes for flow from a discharge point, such as from an outlet pipe, to fill a container of a known volume.

10.3.4 Pressure

Hydraulic pressure often is measured in an embankment and its foundation by piezometers. Their performance indicators are pressure and water

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surface elevation, and they serve to detect pore pressures, which may indicate a developing slope failure. They may also be used to determine flow conditions through the embankment. Pore pressures in excess of those anticipated may reduce slope stability to the point where a slope begins to move.

Piezometers are also used to measure induced pore pressures during construction, particularly in low-permeability soil. After construction, piezometers measure pore pressures for comparison with design. Rising pore pressures and reduced flow may indicate changes in internal drainage efficiencies, possibly blockage of drains. This unfortunate situation may be the consequence of improperly graded filters and drains—a situation with no simple remedy.

Piezometers are also used to monitor the performance of a cutoff, such as a slurry wall or grout curtain. In this case, piezometers are placed upstream and downstream of the cutoff as an indication of the hydraulic gradient across the cutoff. In an embankment, pressure may be measured at any point of interest. For cases where the foundation is of concern (for example, foundations on pervious sand and gravel or confined artesian zones), measurements are made in isolated foundation zones. Both embankment and foundation piezometers can be located to measure hydrostatic pressure upstream and downstream of chimney, blanket, and/or toe drains to assess effectiveness.

Measurement data travels from a piezometer via flexible cables, tubes, or conduits installed during embankment construction or after construction in drilled boreholes. A sand filter is often backfilled around a piezometer's transducer, porous filter, or slotted screen to provide the hydraulic connection to pore water and provide a filter to protect the piezometer from plugging. An unfortunate problem with poorly designed standpipes occurs where the specified slot size is too wide, allowing infiltration of finer soil, eventually plugging the tip and requiring replacement.

In borehole installation, a layer of swelling bentonite pellets or chips is placed above the collection zone to isolate the instrument within the zone of interest. Bentonite pellets are generally preferred over bentonite chips or powder that are more likely to "bridge" in the borehole during placement. When bentonite pellets are used, they are added a few at a time to avoid bridging.

The response time of a piezometer is an important selection consideration. Standpipes or piezometers with sand filters are not desirable in lower-permeability soils because they are slow to respond to changes in pore pressure. If an accurate, responsive reading of changing pore pressures is desired, particularly in less permeable soils, a small displacement transducer is used. Then, wide changes in pressure require very little volume change in the transducer to accurately measure pressure. If a transducer with a larger displacement or a standpipe (which has significant storage in

the riser pipe) is used, it will take more time for sufficient water to pass through the filter zone to accurately reflect the pore pressure at the point of interest. Standpipes and large displacement transducers are only used in pervious stratum or in situations with slow changes in reservoir elevation.

When piezometers are installed during construction, care must be given to protect the transducer and cabling and tubing from damage during subsequent lift placement. Also, because cabling and tubing usually involve long horizontal runs to extend to the readout station, the effects of differential settlement must be considered (Fig. 4-1 in Chapter 4). Selection of flexible or ductile materials and how the instruments are installed can have a large impact on the resistance of the instrument installation to settlement-induced overstressing.

10.3.5 Stress

There are occasions when stress monitoring is desirable. Stress monitoring can be done using total pressure cells typically installed during construction or, alternatively, using a self-boring pressure meter after construction. Total pressure cells can be installed to verify overburden and lateral stresses. Total pressure cells cannot be installed after construction without disturbing the stress field of interest. Although somewhat exotic, the self-boring pressure meter provides an alternative that can be used postconstruction. Total pressure cells are located at any point of interest where the designer is concerned with soil pressures against a particular structure or conduit. Because the cells measure total pressure, they can be used in combination with a piezometer to interpret effective stress. Total pressure cells require care during installation to provide accurate measurements. Installation of a total pressure cell is depicted in Fig. 10-16.

Fig. 10-16. Total pressure cell
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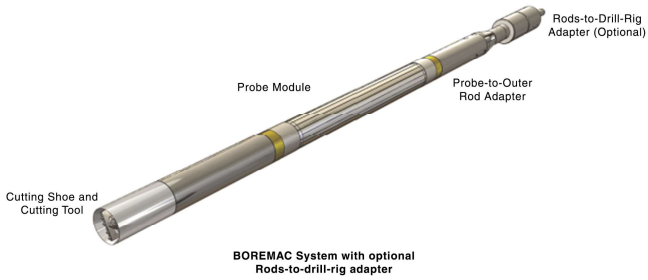

Fig. 10-17. Self-boring pressure meter
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A self-boring pressure meter (SBP) requires special equipment and expertise for installation. Equipment and trained operators are available at a few universities with advanced geotechnical research programs or through a few highly specialized practitioners. Although these devices have been used primarily for university-level research, their ability to accurately measure in situ stress is proven. The instrument essentially tunnels itself into the soil without significant disturbance or stress relief, allowing an accurate stress measurement. The device measures total stress; however, effective stress estimates can be made if testing includes accurate determination of in situ pore pressure. The device is ideally suited for use in sand and clay without stone content, and as such it is limited to stress measurement in finer filters or core material. Fig. 10-17 is a photograph of an SBP meter without its protective shield.

In situations where the stress imposed by the embankment overburden on buried conduits and structures or retaining structures is of concern, a total pressure cell can be recessed into the face of the structure to measure the pressure. The core block in the Oroville Dam, a 770 ft (235 m) high embankment in Northern California, was an example of where stress cells were used to monitor the stress imposed on an internal structure owing to embankment placement.

An example where stress measurement might prove to be a performance indicator would be an embankment with a steep-sided soft core and a relatively stiffer shell. With differential settlement between the core and shell materials, it is possible that the core could "hang up" on the shell. If this occurs, it could tend to reduce the effective stress in the lower portions of the core. If the pore pressures are sufficiently high, hydraulic fracture of the core could occur, opening the potential for internal erosion of the core. This would be a special case where lateral stress in the lower portions of the core could be measured using an SBP meter drilled down from the crest. An SBP meter can also be used to measure lateral stress beside conduits and retaining walls.

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10.3.6 Earthquake Response

Only large embankments in areas of moderate-to-high seismic activity are monitored for earthquake response. Agencies such as the USGS and the California Division of Mines and Geology contract for installation and maintenance of strong-motion instruments at dams. The advantages of this arrangement to dam owners are avoiding highly specialized installation, operation, and maintenance and processing of the acceleration records obtained. Similar offerings from government agencies are also available in a number of countries with moderate or high seismic activity, although some large dam owners choose to develop their own expertise. Measuring earthquake response serves two purposes: (1) design verification of expected versus actual response during a seismic event and (2) adding to the state of knowledge about how dams perform in earthquakes. For example, did the dam deform as expected, or did it deform excessively?

Ground motions are measured by accelerographs (strong-motion machines). Accelerographs are often located on bedrock at the downstream toe or abutment and on the crest of the embankment. Earthquake energy is carried away from the source of the earthquake through bedrock, transferring the energy to the dam foundation. By monitoring the bedrock in the dam foundation, the energy being transferred to the dam can be measured. Monitoring the dynamic response of the dam is typically done by means of one or more accelerographs at strategic points on the embankment. During an earthquake, the oscillatory motion of the earthquake causes a sympathetic oscillatory response in the embankment. The magnitude of the response is dependent on the resonant frequency of the embankment dam and the frequency spectrum of the earthquake.

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Unless there is significant strength loss (liquefaction), embankment dams tend to amplify the peak acceleration of an earthquake. This is exemplified by the fact that peak acceleration on the crest tends to be higher than the peak acceleration in the foundation bedrock. This magnification effect is typically greater for earthquakes with smaller peak accelerations, and the amplification fortunately tends to be less for larger earthquakes.

Permanent earthquake deformation falls into two general categories—liquefaction and deformation. If an earthquake causes pore pressure to rise in the foundation or embankment to the level of the intergranular stress, the affected zone of soil will lose most of its strength. Poorly graded granular soils of low relative density in the embankment or foundation may liquefy. Embankment dams of modern compacted lift construction are resistant to liquefaction; however, if left untreated, loose foundation soils may be vulnerable even with modern construction methods. Measuring earthquake deformation is performed by geodetic survey using the same techniques that are used for measuring settlement.

10.4 PERFORMANCE INDICATORS

Measurements and visual observations of performance indicators are tailored to answer the question of whether or not an embankment dam is performing as expected.

Acceptable measurements of performance indicators depend on the size of an embankment, the properties of its materials, the care exercised during construction, age, and the dam's hazard classification. For example, one foot of settlement at the center of the crest of a 15 ft (4.75 m) high embankment during its first year of operation would be cause for concern, whereas the same amount of settlement of a 450 ft (137 m) high earth and rockfill dam may have been predicted.

Settlement of the crest is expected immediately after first filling. The amount of settlement will reflect the height of the dam, the materials used, and the care taken during construction. Typical designs provide camber to accommodate settlement at the highest section of an embankment. An acceptable value of settlement upon first filling is estimated during design. Measuring settlement provides a basis for comparing actual performance versus estimated design performance. Vertical settlement decreases over time for consistent loads. Measured settlement exceeding 1% of embankment height requires explanation, the character of the embankment materials notwithstanding.

Geodetic surveys or inclinometers may measure movement of an embankment or its foundation under various loading conditions at a variety of locations. The actual magnitudes of movement will vary with the size of the dam and the loading conditions in the context of the dam design and construction. There is no generally accepted amount of allowable movement for an embankment. Again, it is a case-specific measurement. Accelerated rates of movement at zones of interest on and within the embankment may be indicative of a developing failure surface.

Pore pressure within the core of a zoned embankment dam may be higher immediately after construction because of compaction and consolidation from the weight of the overlying embankment. However, excess pressures should dissipate over time and should shift to a steady-state condition. The phreatic level estimated to maintain stability can be compared with the design level to decide if the performance indicator points to stability or instability.

The designer of an embankment dam estimates the magnitude of seepage expected from the dam and designs a drainage system to collect and discharge seepage safely from the dam to minimize the potential for erosion or saturation of the downstream slope. The performance indicator to measure seepage requires monitoring weirs or flumes to measure the quantity of the seepage and to examine the clarity of the discharge. Measuring seepage quantity over time for each weir enables a review of any trends

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that might indicate a developing failure mode. There is no standard value or range of values of seepage quantity that can be considered as acceptable. Seepage and leakage are evaluated on a case-by-case basis with respect to reservoir level and precipitation. If the dam designer predicts a seepage quantity from the dam under full reservoir head and that quantity is exceeded prior to the reservoir reaching that elevation, or if seepage becomes cloudy, immediate attention and careful evaluation are required.

In the design of a monitoring program, design criteria are reviewed to set threshold limits for measured performance indicators. Threshold limits are based on engineering analyses and predicted magnitudes of behavior made during the design of the dam or upon long histories of performance of existing dams.

Threshold limits are established to focus attention on measurements outside the limits of the predictions and readings that require evaluation to explain the behavior. Both overall magnitude and rate of change thresholds may be developed to raise awareness of when evaluation is required.

Evaluation could lead to one or more of the following actions:

- Alerting the responsible engineer;
- Confirming the reading (measurement) by checking it again and, if possible, confirming the proper operation and calibration of the monitoring instrument;
- Inspecting the dam for the cause of the measurement;
- Increasing the frequency of measurements to provide data for further evaluation;
- Revising the threshold limit;
- Installing additional instruments to verify a measurement that exceeds a threshold;
- Implementing remedial measures such as cleaning foundation drains, repairing damage, or modifying the dam in some fashion; and
- Determining whether implementing emergency measures is required.

The performance indicators and threshold limits for embankment behavior are established by careful review of the design, construction, and historic performance records. Experienced geotechnical engineering is required because one approach does not fit all embankments. Engineering judgment is required to properly interpret the results.

The next chapter is a review of performance indicators for concrete dams.

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CHAPTER 11
CONCRETE DAMS

> There exists everywhere a medium in things, determined by equilibrium.
>
> Dmitri Mendeleev

Chapter 3 introduces monitoring considerations for concrete dams from the perspective of their potential failure modes. This chapter discusses concrete dam vulnerabilities and behavior, and it provides guidance about means and methods to measure the performance indicators that answer the question, "Is the dam behaving as expected?"

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11.1 DESIGN

Concrete dams exhibit a wide variety of sizes and shapes, all of which are variations on three basic themes—gravity, arch, and buttress. A gravity dam transfers its reservoir load on the upstream face directly to its foundation, relying on its weight for stability. An arch dam transfers its reservoir load relying on arch action to transfer load into its abutments. An arch dam requires less mass for stability than a gravity dam of equivalent height.

A buttress dam transfers the reservoir load on its inclined upstream face through piers and into the foundation. Buttress dams have fallen from favor, and there are far fewer of them than gravity and arch dams. This chapter discusses instrumented monitoring of gravity and arch dams. Buttress dams are discussed in Chapter 12.

For many years, all concrete dams were constructed of conventional concrete. The advent of roller compacted concrete (RCC) introduced a new

technique for constructing gravity and arch dams. It was developed to promote rapid construction and reduce cost, and as a result it has won global acceptance and is widely used.

Common factors affecting the performance of gravity, arch, and buttress dams are the structural properties of the concrete and the foundation stability. Important concrete properties are

- Loading conditions (normal, unusual, extreme),
- Concrete components (aggregates, cement, admixtures, water) and their proportions (mix design),
- Strength (compressive, tensile, and shear),
- Elastic properties (modulus and Poisson's ratio),
- Unit weight, and
- Seasonal temperatures.

For all concrete dams, abutment geometry and foundation character influence abutment shaping, requiring an understanding of

- Loading conditions (normal, unusual, extreme),
- Proper angle of incidence with the abutment,
- Rock properties such as permeability, weak zones, and presence of discontinuities with the potential to affect stability,
- Rock strength (compressive, tensile, and shear),
- Rock elastic properties (deformation modulus and Poisson's ratio),
- Stress history, and
- Groundwater or artesian conditions.

11.1.2 Gravity Dams

The design of gravity dams must consider the following:

- Shape must be adequate to transfer load to maintain moment and sliding equilibrium while maintaining concrete stresses within acceptable limits.
- The dam must be proportioned to meet acceptance criteria requiring margins of safety for normal, flood, and earthquake loads.
- The dam must have stable foundations and abutments. Particular care must be taken to ensure that blocks of rock when loaded by the mass of the dam and reservoir pressure are not displaced. Loss of foundation support is a mode known to cause gravity dam failure by sliding.
- Gravity dams are suitable for wide gently sloping valleys, as well as for steep valleys. Steep valleys may promote three-dimensional load sharing, a positive factor for stability.

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11.1.3 Arch Dams

Arch dams are shaped to satisfy one basic requirement while maintaining concrete stresses within acceptable limits: safely transfer load into the abutments and foundation.

Implied in this requirement are foundation and abutment stability, with particular care to ensure that blocks of rock under load cannot move, robbing the arch of the ability to successfully redistribute stress. There have been few arch dam failures, and the only mode known to have caused failure was loss of foundation support.

Arch dams are best suited to valleys with width-to-dam height ratios of less than 6:1 to keep concrete proportions as low as possible yet achieve efficient load transfer to the abutments.

Chapter 3 describes the vulnerabilities of concrete dams and introduced sliding and overturning or combinations of both motions as potential failure modes.

11.2 PERFORMANCE

Concrete is the medium common to gravity and arch dams. How those dams perform depends upon the care taken at every step of the design and construction processes. Recognizing a dam's vulnerabilities and associated performance indicators related to failure modes provide the basis for monitoring performance.

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11.2.1 Concrete

A blend of cement, water, fine and coarse aggregate, and admixtures, concrete derives its strength from the qualities of its various constituents. Concrete technology is quite advanced, and how the constituents are proportioned to produce strong concrete for a wide range of conditions is well understood, either for conventionally placed concrete or RCC. When suitably proportioned, batched, placed, consolidated, and cured, concrete performance seldom requires measurement as an indicator of failure mode in modern concrete dams except in cases where concrete is composed of aggregates reactive with the alkalis in cements.

Older (pre-1930s) dams may exhibit symptoms of freeze–thaw damage, a consequence of water freezing and thawing in the open concrete pores. In the 1930s, admixtures were developed that employed a liquid similar to detergent to create tiny bubbles in the concrete mix. The bubbles provided the concrete with the ability to absorb freezing strain without cracking or spalling. For the most part, freeze–thaw damage is cosmetic (Fig. 11-1), requiring occasional maintenance to restore lost concrete. Other than visual

Fig. 11-1. Freeze–thaw damage

inspection to determine depth of damage, measurements are not likely to be required.

Quartz is a common rock-forming mineral. Some species of quartz found in igneous and metamorphic rocks may react adversely with the alkalis in cement, forming a gel that swells and cracks the aggregate. Ultimately, it may crack the concrete, as shown in Fig. 3-17 in Chapter 3.

This process is variously referred to as an alkali–silica silicate reaction (ASSR), alkali–silica reaction (ASR), alkali [Mg]–carbonate reaction (ACR), or alkali–aggregate reaction (AAR). Many existing concrete dams suffer from these reactions, some with consequences that demand monitoring to indicate whether action is required to reduce the effects that cracking has on structural capacity. Appropriate measurements for "healthy" and ASR-affected dams may include

- Movement (x, y, z),
- Total stress,
- Crack aperture and depth of penetration,
- Temperature, and
- Water pressure within the dam body.

Fig. 11-2 shows plots of changes in crest elevation with time at Kariba Dam caused by swelling from ASR.

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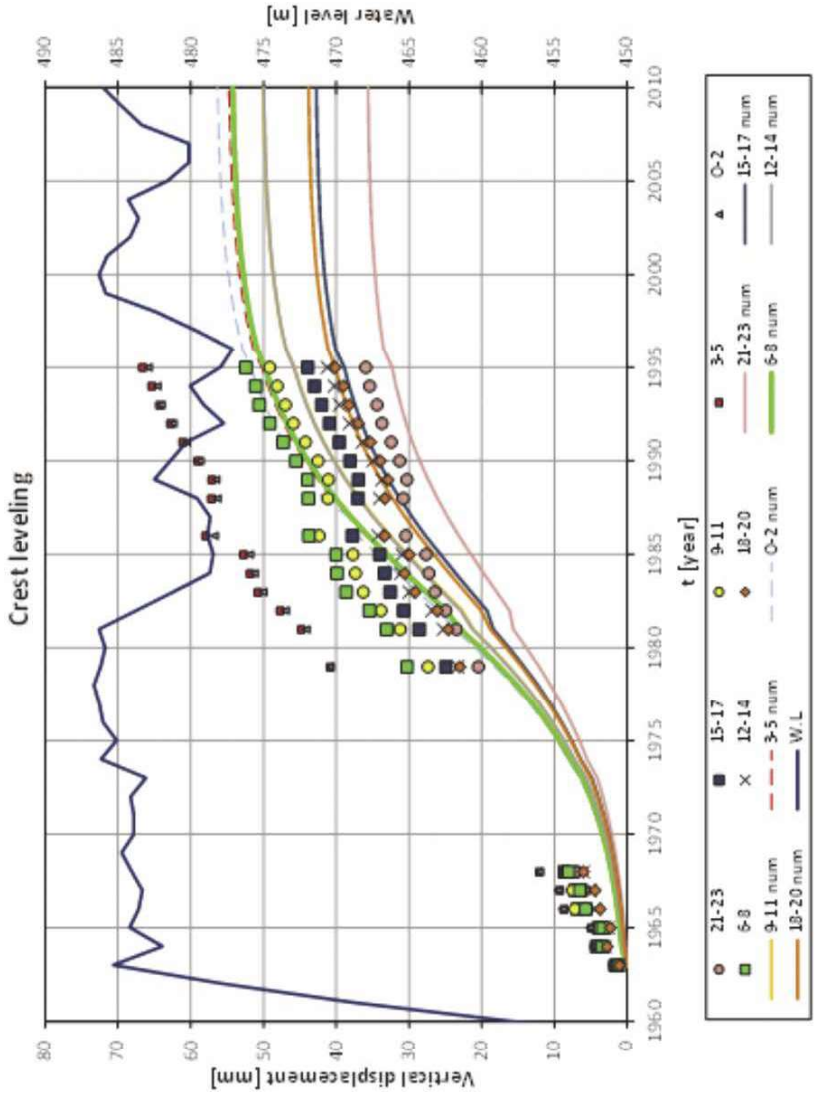

Fig. 11-2. Kariba Dam crest movement caused by ASR

11.3 INSTRUMENTED MONITORING

Chapters 5 and 6 describe instruments and techniques to measure these performance indicators. Geodetic surveys measure deflections along the crest.

Figs. 11-3 and 11-4 illustrate existing and prospective concrete gravity dams, respectively, and typical locations of instruments to monitor its performance. The figures show broad arrays of instruments that may be installed in concrete gravity dams. However, the actual type and number of instruments to be installed depends on the specifics of the project, including its design, size, and hazard potential.

Where the instruments are placed and what measurements are made varies somewhat among the three basic types of concrete dams, but the basic instrument types are the same for all three.

The key to the symbols representing the various instruments in the cross sections is the same as that in Chapter 10 (Fig. 10-3).

11.3.1 Temperature

Concrete temperature is a concern not only during construction but also over the life of the dam. During construction, temperature changes and shrinkage in the concrete introduce secondary stresses. Mass concrete mixes are carefully designed. Fly ash (pozzolan) is employed to reduce the heat of hydration. Heat may be controlled by cooling the aggregate and adding ice to the mixing water. Embedded cooling coils are used to control the

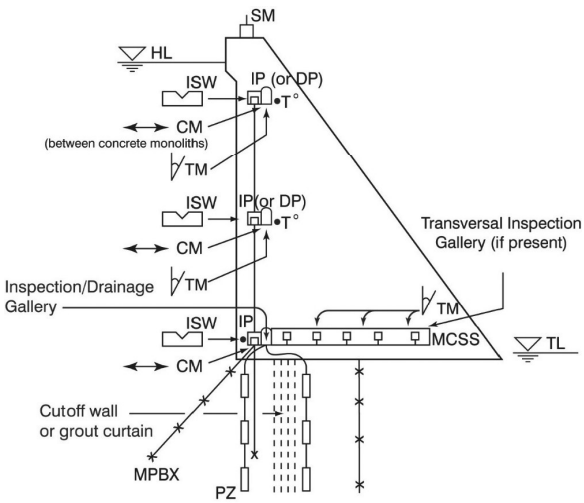

Fig. 11-3. Instrument locations for an existing concrete gravity dam

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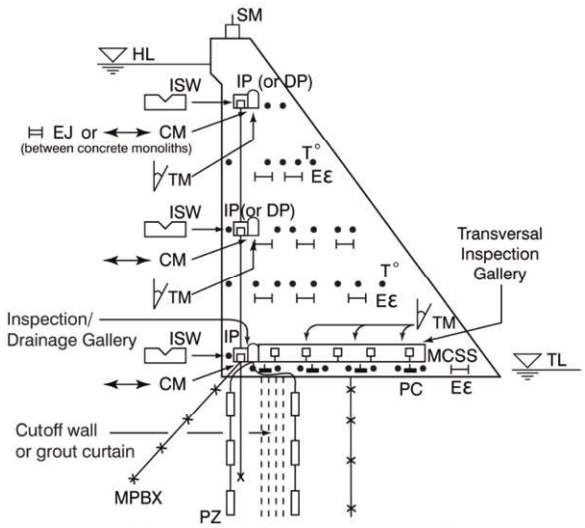

Fig. 11-4. Instrument locations for a prospective concrete gravity dam

secondary stresses by limiting the heat rise, thereby avoiding undesirable thermal cracking. Field adjustments to mixes and construction methods are made based on concrete temperature measurements.

Over the life of a dam, particularly an arch dam, temperature measurements may be important for a proper understanding of both dam behavior and the behavior of instruments. Arch dams and buttress dams are more responsive than gravity dams to movements caused by temperature changes. Temperature changes cause concrete volume changes that apply horizontal forces, resulting in load transfer across contraction joints and into the abutments. The abutments may thus experience increased load. Seasonal temperature changes cause a concrete dam to deflect downstream in winter and upstream in summer.

When it is desirable to measure temperature for a dam under construction, temperature-measuring devices can be placed strategically in fresh concrete. Thermometers and thermistors such as those shown in Chapter 5 are the most commonly used temperature-measuring devices. For existing dams, temperature-measuring devices can be surface mounted or installed in drilled holes.

Knowing the ambient air and reservoir water temperature is important to sorting out the dam's response to temperature changes, and both can be measured with thermometers or thermistors. Heating of concrete from solar radiation is usually considered qualitatively.

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11.3.2 Movement

Movement refers to either translation or rotation within a frame of reference. For example, movement occurs at the crest of a concrete dam as a result of changes in the reservoir load and the temperature of the dam's concrete, air, and reservoir water. Movement is measured relative to a fixed point on a rigid body.

Deformation is movement that changes the shape of a solid body relative to its unloaded shape or its shape at a particular load or stress. Reservoir filling at a concrete gravity dam causes its axis to deform as a curve. Expansion and contraction due to concrete temperature changes also cause deformation. Measurements are related to a line or a plane.

The fundamental design requirement for gravity dams is to resist reservoir load by self-weight. Measurements are made to understand dam movement relative to the movements estimated in design. Periodic surveys of crest monuments provide the appropriate measurements. Because a gravity dam is designed to maintain both moment and sliding equilibrium, all three movement components (x, y, z) are surveyed relative to benchmarks (nonmoving monuments), as described in Chapter 6. The positions of the crest monuments require accuracy to at least 0.01 ft (2.5 mm). This accuracy is easily obtained. Movements for an existing dam are often measured monthly or quarterly. Problem dams may require more frequent measurement. Survey frequency during first filling is important and may be scheduled on the basis of reservoir filling increments rather than by regular time increments.

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The fundamental design requirement for arch dams is efficient load transfer from the arch into the foundation and abutments while maintaining acceptable stresses in the concrete. Surveys of crest monuments using abutment benchmarks or primary survey control networks give a quantitative measure of movement that may be compared to expected values from design or from performance of similar dams.

Measurements of three-dimensional (x, y, z) movement within the dam body are less common than two-dimensional (x, y) movement and are tailored to the specific dam and foundation conditions. Two-dimensional (x, y) measurements may be made with a plumbline, inclinometer, or a series of tiltmeters installed in inspection galleries at different elevations. Horizontal movement within the foundation relative to the dam may be measured with an inverted pendulum installed into the foundation, anchored at depth in the foundation, with movement measured within the dam.

Concrete dams are constructed as discrete blocks separated by contraction joints. Upon completion of construction, the contraction joints are grouted to form a continuous structure. To verify the dam is functioning monolithically, the relative movement, if any, across the joints can be mea-

sured. Block movements are measured with points between contraction joints located in galleries or across contraction joints at the surface. Embedded joint meters, crack meters, and extensometers are designed to measure movement across joints.

The dominant gravity dam load causing block movements is reservoir stage. As the reservoir water surface changes, the relative movement between adjacent blocks is expected to be very small, less than a few hundredths of an inch.

For an arch dam, the normal operating loads are both reservoir stage and concrete temperature. For variations in either load, the relative movement between blocks is likely to be very small, with joints between adjacent blocks closing as the dam goes into compression and opening as the dam extends.

Swelling caused by AAR is measured with surface or embedded strain gauges, crack meters, extensometers, or geodetic techniques presented in Chapters 5 and 6.

With the advent of electronic distance measuring equipment and LiDAR, deformation can also be measured using surface monuments on the downstream face, total stations, and least-squares adjustment. Synthetic aperture radar allows targetless measurements of movement along the entire downstream dam face. Fiber optic arrays can detect and measure very small movements of exposed concrete surfaces.

Most mass concrete structures crack. The cracks are often cosmetic (nonstructural). Cracks are usually observed on the crest, the downstream face, and in the drainage gallery. When cracks may be related to structural integrity, movement across the crack is measured. These movements can be made at the surface using pins or crack meters or at depth using joint meters. Usually the crack width or relative slip is measured, depending on the location and orientation of the crack. Another method to monitor a crack is to mark and date both ends of the crack to track changes in its length. Generally, cracks form during and immediately after construction, during first filling, and as a result of strong ground shaking. Cracks in most concrete dams move only a few hundredths or thousandths of an inch with no net long-term movement. During design, cracking may be anticipated at certain locations owing to unique features of a particular dam and its site. If needed, instruments such as multipoint extensometers or joint meters are appropriate to measure crack aperture and extension.

11.3.3 Uplift

Uplift exerted by the reservoir opposes self-weight and reduces the effective normal force acting on the base or along any horizontal plane within the dam body. The reduction of effective normal force reduces both frictional and moment resistance. The simplified diagrams (Fig. 11-5) illustrate the

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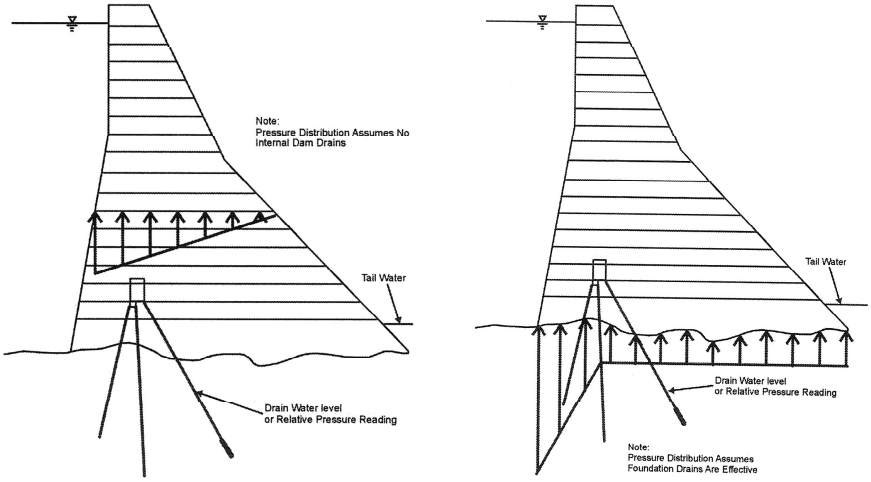

Fig. 11-5. Uplift within the dam body and along the base
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

typical pressure diagram associated with uplift on horizontal planes within the dam body and on the contact between the dam and foundation.

If a dam has no internal or foundation drains, the uplift pressure through a horizontal section of a dam varies linearly from reservoir pressure at the heel to tailwater pressure at the toe.

Uplift is managed with seepage reduction measures and drainage. Most, if not all, concrete dams have grout curtains designed to reduce seepage by lengthening the seepage path across the dam base. Downstream of the grout curtain, seepage at the foundation is drained by drilled or formed drains to reduce uplift. Within the dam body, seepage is intercepted in drilled or formed drains. Galleries above foundation level provide an ideal location to drill down for foundation drains and up for dam body drains.

The simplest flow-measuring device employs a calibrated container and a stopwatch (Fig. 11-6). The container is placed at the discharge of a drain. The operator then measures the amount of time it takes to fill the container. Generally, the longer it takes to fill the container, the more accurate the reading; selection of a container of sufficient volume to result in a minimum filling time of at least 30 s is recommended. The volume of the container divided by the time it takes to fill the container is a measure of flow rate. In areas where there is not a defined discharge pipe, a vertical drop can be made by installing a fabricated plate with a small discharge pipe through the plate to create the vertical drop necessary to fill the container. Small weirs such as those shown in Fig. 11-7 also may be useful in measuring flow.

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Fig. 11-6. Calibrated container and stopwatch

Fig. 11-7. Small measuring weir

11.4 PERFORMANCE INDICATORS

Uplift correlates directly to the margin of safety and, consequently, to the potential for failure by sliding or overturning moment. Key performance indicators are measurements of drain pressure and volume of drain flow. Unexpected increases in drain pressure or flow may indicate a deterioration

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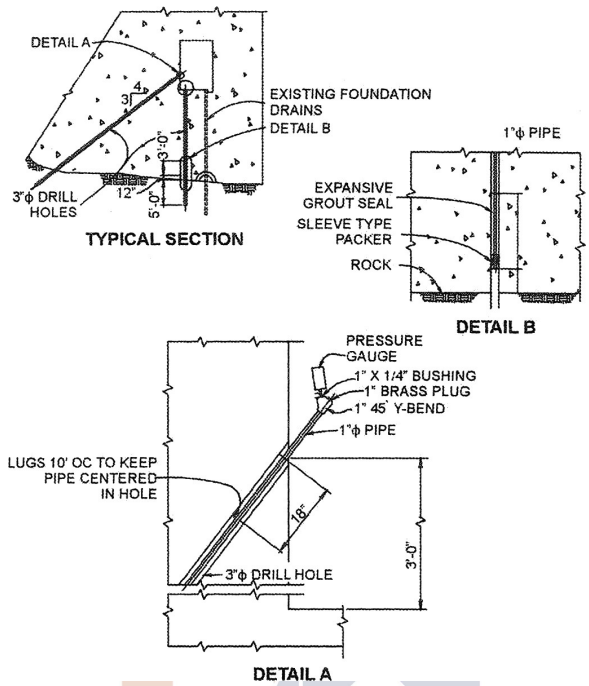

Fig. 11-8. Pressure check
Source: Courtesy of Alabama Power Company, reproduced with permission.

of the grout curtain. Instruments similar to those employed for an embankment are suitable—piezometers for pressure and weirs for flow. Fig. 11-8 illustrates a method to measure hydrostatic pressure mostly for upstream but also downstream of the existing grout and drainage curtains.

Where instruments are placed and measurements are made varies somewhat among the three basic types of concrete dams, but the basic instrument types are the same for all three. A typical arch dam monitoring arrangement is shown in Fig. 11-9, and typical measurements and instruments are shown in Table 11-1.

Decreasing drain flow may signify plugging. Foundation drains are susceptible to both chemical deposition and bacterial growth. Dissolved solids such as calcium carbonate may precipitate to coat the walls of drains in rock, retarding flow and requiring periodic cleaning. Similarly, bacterial growth caused by organisms that thrive on iron or sulfur content in water in an oxygenated environment may retard flow. This phenomenon is characterized by distinctive orange or brown slime at drain discharge. Careful measurements will disclose loss of drainage efficiency. Drains that exhibit plugging are candidates for cleaning, reaming, or redrilling in an alternate location.

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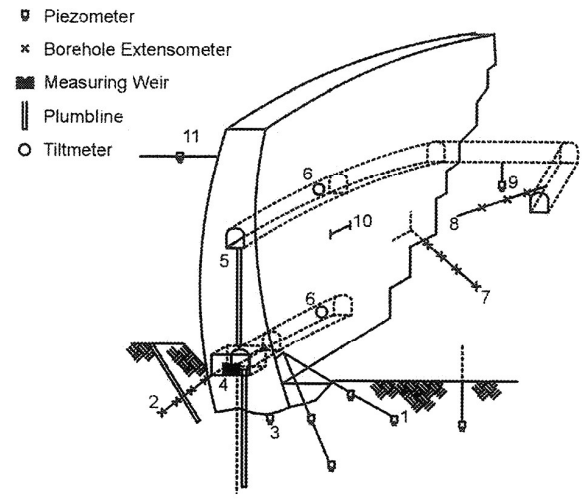

1 - Monitoring effectiveness of grout and drainage curtains by pore pressure measurement

2 - Monitoring movement of fissure openings, using borehole extensometers

3 - Pore pressure measurement at concrete-rock interface

4 - Seepage flow measurement

5 - Plumbline to measure movement

6 - Fixed tiltmeter

7 - Monitoring of foundation compressibility, using borehole extensometers

8 - Monitoring of specific geological features, using borehole extensometers

9 - Pore pressure measurement for monitoring effectiveness of grout curtain and abutment stability

10 - Measurement of strain to determine stress

11 - Measurement of reservoir level

Fig. 11-9. Typical arch dam monitoring arrangement
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

Uplift directly correlates to the margin of safety and, consequently, to the potential for failure by sliding or overturning moment. Key performance indicators are measurements of drain pressure and volume of drain flow. Unexpected increases in drain pressure or flow may indicate deterioration of the grout curtain. Instruments similar to those employed for an embankment are suitable—piezometers for pressure and weirs for flow.

Uplift is an important consideration in the design of gravity dams. Uplift is often neglected in arch dam design because base widths are small in comparison to dam height and the foundation and abutments are capable of resisting the imposed loads; however, uplift may adversely affect foundation stability. Notwithstanding the minimal effect that uplift may have on arch dam behavior, many arch dams have drains and periodic measurements are made.

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Table 11-1. Monitoring Locations and Methods.

Performance indicator	Measurement location	Typical instruments or methods
Movement (upstream-downstream)	Crest or other surface location of interest	Geodetic survey
Movement (rotation)	Within concrete	Inclinometer, pendulum, tiltmeter
Differential movement	Across joints or cracks	Strain gauge, extensometer, joint meter
Differential movement	Within foundation	Extensometer
Water pressure	Within dam body	Piezometer
Water pressure	At foundation contact	Piezometer, pressure gauge
Water pressure	Within foundation	Piezometer
Stress and strain	Within foundation	Total pressure cell, load cell, strain meter, flat jack
Stress and strain	Within concrete	Total pressure cell, strain meter
Internal temperature	Within concrete	Thermocouple, RTD, thermistor
Seepage quantity	Where measurable	Calibrated container, weir, flume, flow meter
Seepage quality	Where measurable	Turbidity meter
Anchor load retention	Anchor head	Load cell, jack and pressure gauge
Earthquake response	Crest, free field, or other surface location of interest	Strong-motion accelerometer

11.4.1 Stress

Design criteria for both gravity and arch dams require the dam shape to be adequate to transfer load to maintain stability while maintaining concrete stresses within acceptable limits for all anticipated loading conditions.

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The state of stress and resultant strains can be important in understanding the behavior of concrete dams and their foundations. Stress may be measured directly; however, strain measurements are more common because the instruments are easier and less expensive to install. Converting strain measurements to estimates of stress requires knowledge of the mechanical properties of concrete (modulus of elasticity and Poisson's ratio), together with the concrete's creep, shrinkage, and response to temperature changes. Occasionally, direct stress-measuring instruments are used to verify the results of strain-measuring instruments. Loads can be estimated from measured or inferred stresses and those loads compared with design estimates.

Where to measure stress or strain depends on the loads to be estimated. For any concrete dam, locations where tensile stresses may develop are candidates for measurement. Another reasonable location is the maximum section to attempt to confirm the maximum load. Arch dam instrumentation often includes strain meters or stress meters oriented to measure stress in the direction of arch thrust.

Foundation stresses and strains may be measured similarly to concrete stresses and strains; however, the instruments are not embedded in fresh concrete but rather, they are installed in drill holes or sawed slots.

Strains also may be measured precisely with geodetic techniques by measuring between fixed points on a structure.

Measuring stress or strain in an existing dam can be accomplished by drilling a hole to the point of interest and securing an appropriate instrument in position. Stress also may be measured by inserting a pressure cell or flat jack into a sawed slot.

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There are many instruments and techniques for measuring stress and strain. Flat jacks, Carlson concrete stress meters, WES pressure gauges, Gloetzel stress cells, and vibrating-wire stress meters comprise part of the array of stress-measuring instruments. Electrical strain gauges, Carlson strain meters, mechanical strain-measuring devices, linear variable differential transformers, and vibrating-wire strain gauges complement the field of strain-measuring instruments. These instruments are described in Chapter 5.

Strain gauges are usually of the vibrating-wire type. The length of their measuring base must be longer than three to four times the coarse aggregate size. For this reason, gauge lengths of 25 to 30 cm (250 to 300 mm) are often specified. The use of a dummy gauge as described in Chapter 5 is also recommended.

Upon installation, the strain gauges are strongly tied to rods or brackets that will keep them in place during concrete placement. The rods or brackets are small enough to not interfere with the stress field surrounding the strain gauges. Concrete is not placed directly on the gauges but rather beside them and then vibrated. If necessary, the coarse aggregates can be removed from the concrete to comply with the requirement of gauge length equal to three or four times the aggregate size.

Signal cabling must possess adequate quality and strength for direct embedment in concrete, as it is not recommended to use embedded cable protective ducts that could later become a preferential path for water circulation. It is also important to consider the location where the electrical cables come out of the concrete. This is the location where electrical cables often get severed because they are exposed. Some strain relief is needed at this location, for instance, by the use of a short length of plastic duct that will protrude from the concrete or alternatively by a recess in the concrete where the cable can be placed.

11.4.2 Alkali–Silica Reaction (ASR)

Stress measurement is often used in concrete affected by ASR to assess the extent of the effects, monitor long-term effects, and verify the effectiveness of the remedial measures such as stress-relief dam sawing. Stress measurement has an important role in monitoring of dams experiencing ASR.

In the case where existing strain gauges are already embedded in the dam and are located at suitable positions with respect to the areas affected by ASR, they are obviously the best instruments to monitor the evolution of deterioration. Postconstruction, embedding strain- or stress-monitoring instruments in the concrete becomes necessary.

For this purpose, two approaches are generally preferred. The first is to use vibrating-wire stress inclusions, as described in Chapter 5. These inclusions are installed in small-diameter, usually 32 mm (1.5 in.), boreholes with a special installation tool that wedges the instrument in the hole at the proper depth, orienting it so that the direction of the change of stress measurement will be known. Up to three instruments can be installed in the same borehole with measurement directions rotated 60 degrees from each other. This is done to obtain the true state of stress in the plane perpendicular to the borehole or, more precisely, the change of stress, because the initial stress in the concrete is usually not known when the instrument is installed. Instruments have been installed successfully up to 20 m (65.6 ft) or more into a borehole. When the direction of stress to be measured is known, only one instrument will provide sufficient information.

A second type of inclusion that has been used to monitor changes of stress related to ASR is an instrumented cylinder as shown in Fig. 11-10. The cylinder consists of three or six vibrating-wire strain gauges oriented respectively in a two- or three-dimensional rosette configuration so as to obtain a plane state of strain or the complete three-dimensional state of strain. Short strain gauges approximately 100 mm (3.93 in.) length are generally used so that the rosette-configured assembly of gauges can be precast in a concrete cylinder 140 mm (5.51 in.) in diameter. The modulus of the concrete can be adjusted by use of suitable aggregates to approximate that of the AAR-

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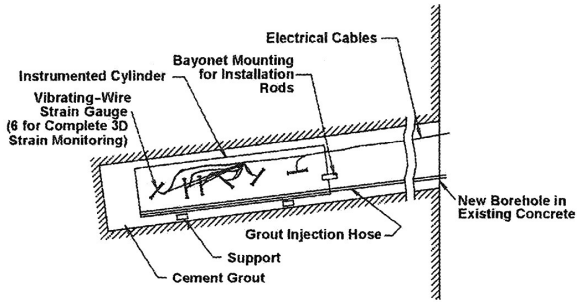

Fig. 11-10. Vibrating-wire strain gauge-instrumented cylinder

affected concrete in which it will be installed. The instrumented cylinder is an inclusion with modulus adjusted to be in the same range as the enclosing material. This procedure maintains the state of stress around the borehole in which it will be installed and grouted.

To install the cylinder, a 152 mm (6 in.) borehole is drilled in the concrete and the instrumented cylinder is pushed in the borehole by means of installation rods that allow orientation of the cylinder in the hole. The cylinder is grouted in place with a cement grout so that the cylinder becomes an integral part of the concrete. The hole can be grouted up to its collar to restore the integrity of the dam concrete.

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11.4.3 Foundation Stress Measurements

Concrete stress cells or "Gloetzl" cells, from the name of the earliest manufacturer, are described in Chapter 5. In concrete dams, they are used at rock–concrete interfaces at the foundation or abutments to measure the stress applied perpendicularly to these interfaces. Either rectangular or circular cells can be used to measure foundation stress. The cells incorporate a repressurization tube to restore contact between the pressure pad and the enclosing concrete once concrete has cured and shrunk, especially in situations where cells are used on vertical surfaces. The area of rock or concrete over which the cell is to be placed is prepared clean and flat ±10 mm (0.39 in.). The area is precoated with a 15 mm (0.59 in.) thick layer of nonshrink cement mortar prior to placing the cell on the uncured mortar pad.

Many existing concrete dams have post-tensioned anchors or tendons to improve stability. Long-term monitoring may be important to measure load retention or anchorage creep.

There are two basic opportunities to measure anchor or tendon performance. The first and easiest opportunity is to include a load cell in the anchor or tendon head as a part of the design, allowing direct measurement

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of load retention. Robust, simple, mechanical, vibrating-wire load cells such as those described in Chapter 5 are preferred.

Where the anchor or tendon head is accessible but no load cell is installed, the load can be verified with a lift-off test. Simply, the test involves lifting the head off the shims with a jack and verifying that the lock-off load is retained in the anchor or tendon. Where the tendon head is not accessible, load verification is not yet practical. However, several recent advancements in nondestructive techniques offer promise for confirming stress retention.

11.4.4 Earthquake Response

High-concrete dams in areas of moderate-to-high seismic activity are monitored for earthquake response. Agencies such as the U.S. Geological Survey and the California Division of Mines and Geology contract for the installation and maintenance of strong-motion instruments at dams. The advantages of this arrangement to dam owners are to avoid highly specialized installation, operation, and maintenance and to avoid processing the acceleration records obtained. Measuring earthquake response serves two purposes: (1) it provides design verification of expected versus actual performance during an earthquake and (2) it adds to the state of knowledge about how dams perform in earthquakes. Earthquake response is post facto and is not a performance indicator. What it does provide is a measurement of earthquake loading. Post-earthquake measuring gravity or arch dam response will reveal whether the dam is deformed.

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Seismographs are placed in arrays or nets to provide coverage of large regions and measure the size of an earthquake, as well as its location and time of occurrence. The output from a seismograph is referred to as a seismogram.

Ground motions are measured by accelerographs (strong-motion machines). Accelerographs are located on bedrock at the downstream toe or abutment, within a gallery in a dam, and on the crest of the dam.

Earthquake energy is carried away from the source of the earthquake through bedrock, transferring the energy to the dam foundation. By monitoring the bedrock at the dam foundation, the energy being transferred to the dam can be measured. Monitoring of the dynamic response of the dam typically is done by means of one or more accelerographs and at other strategic points on the dam. During an earthquake, the oscillatory motion of the earthquake causes a sympathetic oscillatory response in the dam. Concrete dams tend to amplify the peak acceleration of an earthquake. Amplification is exemplified by the fact that peak acceleration on the crest tends to be higher than the peak acceleration in the foundation bedrock. This magnification effect typically is greater for earthquakes with smaller peak accelerations, and the amplification fortunately tends to be less for larger earthquakes.

Strong shaking is likely to cause contraction joints to open and close, usually without requiring repair. Permanent earthquake concrete deformation

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Fig. 11-11. Accelerometer for arch dam base motion

may cause cracking or separation at the abutments. Otherwise, experience suggests that concrete dams are likely to survive strong shaking and retain the reservoir, provided that fault rupture displacement does not occur.

Measuring post-earthquake deformation is done by geodetic survey using the same techniques as those described in Chapter 6.

To measure the earthquake load on a dam, a three-component accelerometer can be placed on the ground surface near the dam, considered the free-field record. Another location might be in a drainage gallery that is near the base of the dam. An accelerometer may also be placed on an abutment near the crest if topographic or other amplification effects are of concern. Where an accelerometer is placed is important. What is desired is the response of the dam, not the response of the accelerometer housing. Accelerometers typically are bolted to a concrete slab and enclosed in lightweight fiberglass housing (Fig. 11-11).

Strong-motion accelerographs details are described in Chapter 5. Basic requirements for accelerometers on concrete dams include reliability and sensitivity. Because of the wide range in severity of shaking, accelerometers need to be capable of recording the peak accelerations exceeding 1.0 g (0.03 oz). For dams with multiple accelerometers, timing is important to correlate the accelerations measured at different locations on or within the dam. This can be accomplished by encoding an external time reference on the record. A desirable requirement is a record of the initial motions of an earthquake. Modern, digital accelerometers have a preevent memory. They continuously

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record accelerations, which are stored in memory. The memory is continuously updated to store the most recent data (e.g., the last 4 s). Then, when triggered by an earthquake, motions for 4 s before the earthquake also are obtained, providing a record of the entire event. With older film or cassette tape accelerometers, the initial accelerations before the threshold are not recorded.

Accelerometers measure the acceleration at a specific location at small time intervals (e.g., every 0.01 s) and usually in three dimensions. The records are used to determine the earthquake load and the dam's response.

Records from an earthquake can be used to estimate whether the design response was exceeded and, using dynamic analysis (finite element, finite difference, wave equation, or simplified methods), to calculate stresses, deformations, and other parameters from which to evaluate performance and safety.

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CHAPTER 12

OTHER DAMS AND APPURTENANT STRUCTURES

> The difficulty lies not so much in developing new ideas as in escaping from old ones.
>
> John Maynard Keynes

This chapter addresses instrumentation and monitoring needs for other dams and appurtenant dam structures such as spillways, intakes, outlet works and conveyances. "Other dams," cousins of conventional embankment and concrete dams, are slab-and-buttress, masonry, timber, inflatable, and concrete dams founded on piles or directly on soil.

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Other dams have the same vulnerabilities of overtopping, uncontrolled seepage, and structural inadequacy as embankment and concrete dams. Appurtenant structures exhibit distinctive vulnerabilities because of their unique designs. Performance indicators are selected to answer the question, "Is this other dam or appurtenant structure behaving as expected?" Instruments capable of measuring those performance indicators are described in detail in Chapter 5.

12.1 OTHER DAMS

This section addresses recommendations related to surveillance and monitoring of buttress dams, masonry dams, timber crib dams, and inflatable dams.

12.1.1 Buttress Dams

Slab-and-buttress and multiple-arch dams (sometimes referred to as "hollow dams") consist of concrete buttresses or piers that support a sloping

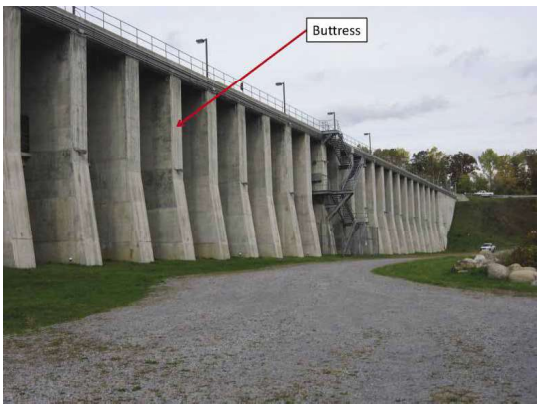

Fig. 12-1. Downstream view of slab and buttress dam

upstream concrete face to impound water. A photograph of a slab-and-buttress dam with a flat sloping upstream face is shown in Fig. 12-1, and a cross-sectional view is shown in Fig. 12-2. Several slab-and-buttress dams in the United States were constructed by the Ambursen Hydraulic Construction Company and other companies, mostly in the first half of the twentieth century in the United States, although more recent examples also exist. Many were also constructed in Europe after World War II. The name Ambursen Dam, however, is often used to generically refer to a slab-and-buttress-type dam, whether or not it was constructed by the Ambursen Company.

A variation on the slab-and-buttress dam is a multiple-arch dam, shown in Fig. 12-3, which consists of sloping arch structures instead of a flat sloping upstream face. The downstream buttresses support the arches that serve as the upstream water barrier.

Slab-and-buttress and multiple-arch dams differ from concrete gravity dams in that they are engineered to conserve materials. The members are sized and reinforced to carry the anticipated design loading condition. In contrast to concrete gravity dams, whose cross section is solid concrete, slab-and-buttress dams have hollow areas between their buttresses. Slab-and-buttress dams use significantly less concrete than is used for a concrete gravity dam of similar size. Rather than simply using the weight of concrete to resist sliding and overturning, the slab-and-buttress and multiple-arch dams elegantly make use of the downward force of hydrostatic pressure on the sloped upstream face to mobilize frictional resistance to sliding on the concrete–bedrock interface at the base of the supporting buttresses. The upstream face of the dam is tied into the bedrock foundation by a "plinth" or, alternatively, by structurally tying the slab to a foundation structural mat if the dam is founded on soil, as shown in Fig. 12-4.

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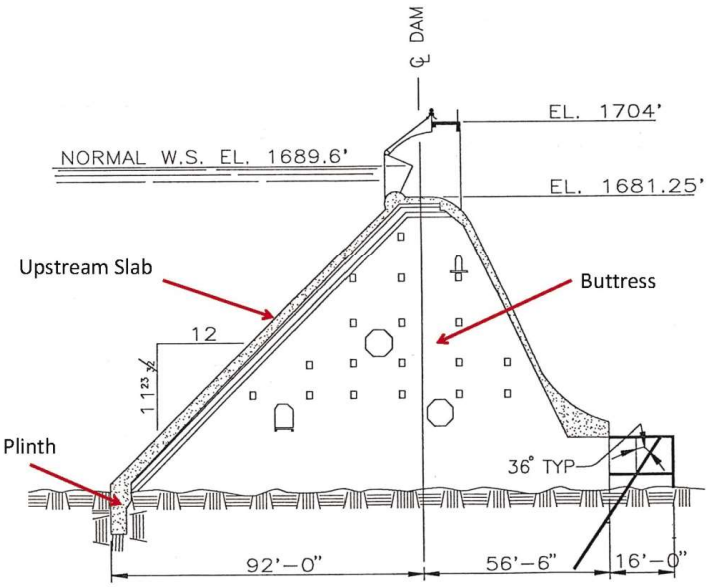

Fig. 12-2. Cross-sectional view of slab-and-buttress dam
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

Fig. 12-3. Multiple-arch dam
Source: Photographer Alex Stephens, U.S. Bureau of Reclamation.

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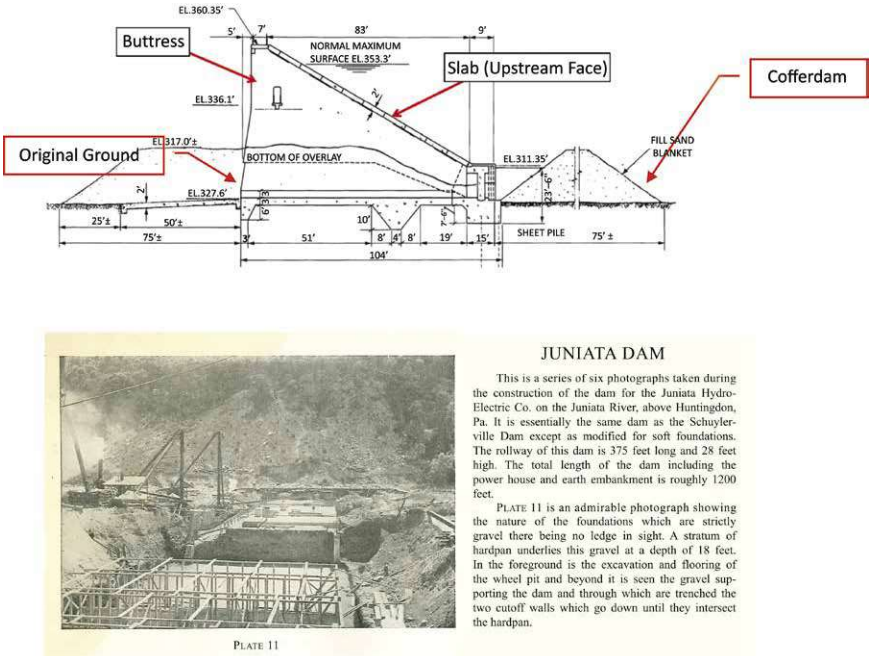

Fig. 12-4. Slab-and-buttress dams founded on soil
Source: Ambursen Hydraulic Construction Company (1907).

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Monitoring of buttress dams is similar to that of gravity dams; however, buttress dams have unique features that must remain stable for reservoir control.

The upstream face of a buttress dam (slab and plinth) serves as the water barrier for the dam. The buttress design allows foundation drainage into the hollow area of the dam between buttresses, and that drainage protects buttresses from uplift forces. Any foundation instability could result in possible movement of a slab and plinth, initiating a sequence of events that may lead to failure. The segmented configuration of slabs supported by spaced buttresses is sensitive to differential settlement and misalignment. Buttress movement could result in (1) redistribution of stress exceeding the strength of a face panel, creating a "domino" failure or (2) opening a privileged path of seepage creating an internal erosion path that robs a buttress of support, again leading to a "domino" failure.

Key performance indicators are (1) unexpected seepage or leakage, (2) crack growth, and (3) buttress movement.

Freeze–thaw cycles, ASR, or general aging can initiate cracks in the face slab or plinth. If leakage through the slab and plinth concentrates on the

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ground, it is typically channeled into trenches to convey flow to a weir or other flow-measuring device to detect increasing flow indicative of increased cracking or deterioration.

A buttress dam supported on a concrete structural mat on soil foundation is vulnerable to a lack of drainage, causing underlying soil to settle away from the mat locally or to be carried downstream by internal erosion flow. The concrete mat can form a "roof" spanning a developing void in the soil foundation, which is one of the primary causes of internal erosion. Slab-and-buttress dams founded on mat foundations on soil often have upstream and intermediary cutoffs (Fig. 12-4). Standpipes or piezometers are used to monitor changes in pore pressure indicative of internal erosion deterioration of a cutoff. Measuring water levels that decrease with time and slowly approach the tailwater levels could be a potential indication of the loss of flow resistance through a void between the instrument and tailwater. Leakage is measured with any flow-measuring device.

Cracks can be painted along their length to allow visual detection of any lengthening of cracking into unpainted regions of the dam face. Cracks may grow from seasonal temperature changes, foundation instability, or reservoir level variations. Measuring crack growth is performed using the same methods described in Chapter 10.

Buttress movement can be measured by geodetic techniques, tiltmeters, and extensometers.

For older buttress dams, it is unlikely that earthquake loads were considered in the original design, and their design may not meet current standards. Understanding of the seismic setting of the dam site itself may have changed since design. In earthquake-prone regions, loading in the cross-channel direction (direction of the axis of the dam) may be important because that probably will be the weak axis of a buttress dam. Because of their less-massive, segmented construction in comparison with concrete gravity dams, buttress dams are more susceptible to earthquake damage, and consequently, these structures are closely inspected following a reported or felt earthquake. The means and methods for measuring earthquakes and their effects on a dam are explained in Chapter 10.

12.1.2 Masonry Dams

Dams were commonly constructed using stone masonry in the late eighteenth and early nineteenth centuries because stone was a readily available construction material and labor skilled in stone cutting was plentiful (Figs. 12-5 and 12-6). Design and construction of masonry dams varied widely. In some cases, the entire structure was constructed of hand-laid cut stone (with or without mortar). In other instances, the upstream and

Fig. 12-5. Masonry-faced dam

Fig. 12-6. Masonry arch dam

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downstream faces of masonry dams were constructed using cut stones with mortared joints, and the internal zone of this containment was filled with rock rubble or cyclopean concrete that incorporated boulders to reduce the overall volume of concrete required.

Masonry dams may require little more than periodic movement surveys and visual surveillance. Where masonry is mortared, visual monitoring of the rate of deterioration may be sufficient to estimate when periodic maintenance, such as repointing, is necessary. If the interior interstitial spaces are not sealed with concrete or mortar or if the material has deteriorated over time, water pressure may rise between the upstream and downstream faces. Careful research of construction records and photographs may reveal a potential for water pressure rise.

Mortar used in masonry dams was proportioned to be durable at the time of construction, but it may now be weathered and susceptible to deterioration from acid rain or aggressive water chemistry. Visual surveillance is important to detect changes not amenable to instrumented monitoring. If monitoring of movement, instability, and seepage is required, measurements can be accomplished using techniques similar to those introduced in Chapters 6 and 10 for monitoring concrete dams.

A variation of the masonry dam theme is incorporation into the body of a taller concrete gravity dam, forming a composite dam with separate zones of masonry and concrete (Figs. 12-7–12-9). Typically, concrete forms the downstream portion of the composite section, and drains are required to collect the seepage along the joint between masonry and conventional concrete. Drains function to reduce the buildup of pore pressures within the dam. Monitoring seepage and pressure and leakage is important to understanding the effects on stability, if any. Appropriate instruments include various types of piezometers and strategically located seepage measuring weirs.

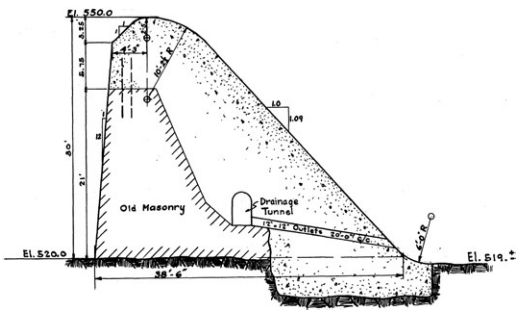

Fig. 12-7. Composite masonry and concrete dam
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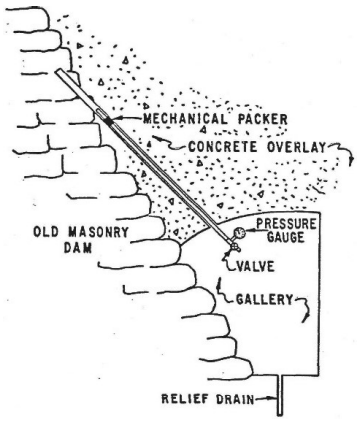

Fig. 12-8. Example of pressure monitoring in composite dam
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

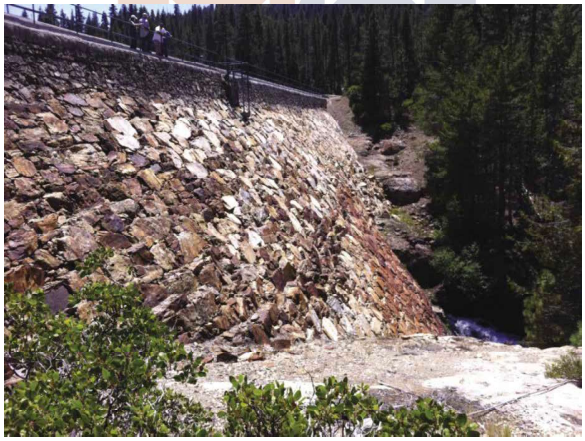

Fig. 12-9. Ashlar masonry composite dam

12.1.3 Timber Dams

Dams constructed of timber (Fig. 12-10) were widely used in the eighteenth century, mainly to divert water for mills.

Timber crib dams were typically constructed of compartments or "cribs" formed of individual logs or timbers. The cribs were filled with ballast, usually boulders to add weight to mobilize sliding and overturning resistance. An example is shown in Fig. 12-11.

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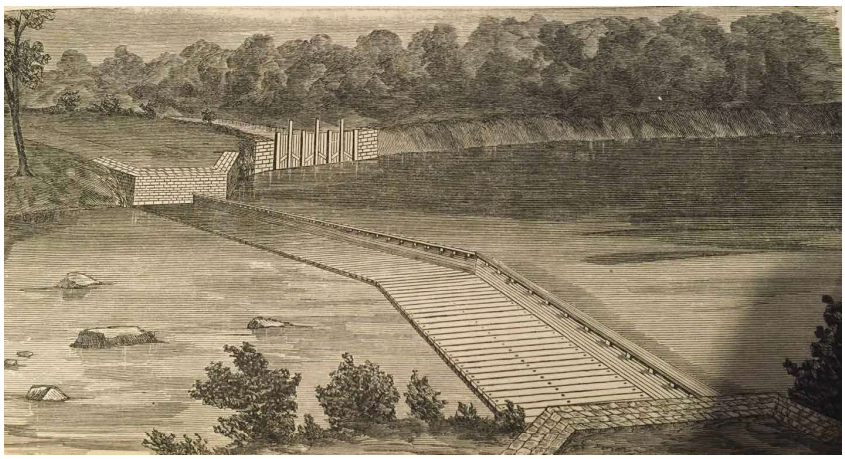

Fig. 12-10. Mad River Dam
Source: James Leffel & Co. (1874).

Fig. 12-11. Construction of log-crib (timber) dam at Postill Lake, east of Kelowna (c. 1911)
Source: Kelowna Museum photo, No. 10, 347.

Although wood timbers are most common, large timbers may be replaced by precast-concrete "sleepers." The upstream face of timber crib dams is typically constructed of tightly jointed or overlapped timbers, whether wood or concrete. Upstream faces constructed of wood timbers are not designed to be watertight but are constructed to allow some leakage so that the internal wooden timbers remain sufficiently saturated to minimize the rate of deterioration.

Timber crib dams can have varying cross sections. Some have been treated with shotcrete as reinforcement, encased in concrete, or integrated into larger composite dams. Similar to the composite masonry structures, composite timber crib dams (Fig. 12-12) can present some challenging internal pressure and uplift conditions and may require monitoring by piezometers.

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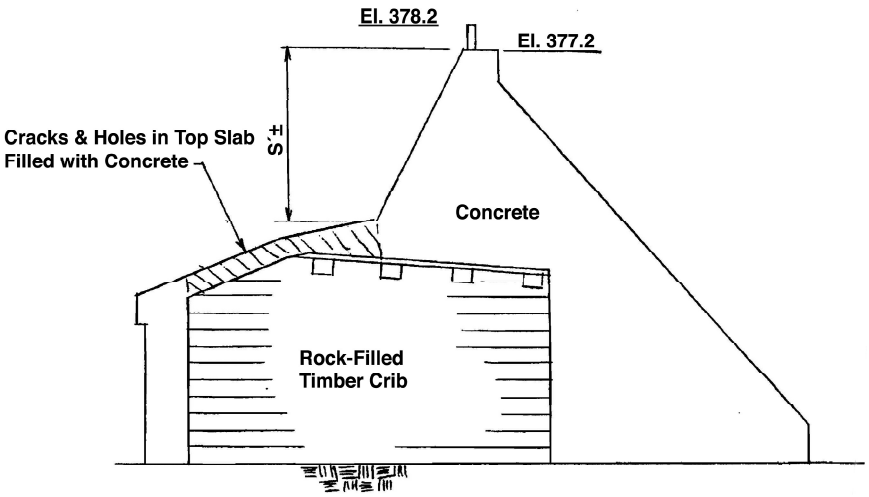

Fig. 12-12. Composite timber crib dam
Source: National Resources Committee (1938).

Analysis of the sliding stability of the cross section shown in Fig. 12-12 would benefit from a piezometer pressure measurement in the rock-filled timber crib to estimate the uplift profile on the base of the composite structure. Various composite cross sections exist, and a good understanding of their compositions is important to assess if and where instrumented monitoring could measure a performance indicator.

A few timber buttress dams (Fig. 12-13) are still in service, and their performance indicators are similar to those of concrete buttresses with added concern for timber strength.

Movement is a key performance indicator that is measured with the same geodetic techniques described for embankment or concrete dams. Survey measurements of crest and other monuments are used to detect vertical and lateral changes that may be indicative of structure movement caused by timber deterioration, settlement within the rockfill, or foundation settlement.

A potential initiator of a failure sequence is erosion of a soil foundation under the open rockfill contained within the timber cribs. Unfiltered, the foundation material may rise into the rockfill, resulting in settlement of the dam. Visual monitoring of foundation erosion or undercutting of timber crib dams may be accomplished with remotely operated vehicles or divers. Access is limited to the upstream and downstream edges of the dam unless voids under the cribs are accessible. Access may be difficult and potentially dangerous for divers owing to confinement in the case of undermined areas on the downstream side and the potential for high-velocity flow into and under the crib structure on the upstream side. For this reason, movement

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Fig. 12-13. Timber buttress dam

often is limited to regular surveys of monitoring points with visual surveillance (robot or manned) following large floods.

Deterioration of wood timbers themselves may be a problem with timber crib dams. Although wood members that remain submerged can last for many decades, timbers in or above water-level fluctuation zones may need periodic replacement because of rot from exposure to sunlight, temperature fluctuations, and the atmosphere (Fig. 12-14).

The upstream face of tightly jointed or overlapped wood forms the water-retaining envelope. The upstream face is best designed to allow some leakage to keep the internal wood as moist as possible while still retaining the reservoir. Fungi, insects, or rot may weaken wood. Regular, repeatable visual monitoring of the timbers is an important surveillance aspect for timber crib dams. Where visual surveillance suggests weakness, random core samples or removal of complete cross sections to inspect timbers may be helpful. Experienced timber and wood material specialists can provide opinions on longevity and strength through visual examination and possibly testing of the recovered core samples.

12.1.4 Inflatable Dams

Inflatable dams are tubes or bladders formed of a rubber-like material that is often reinforced by fabric or metal mesh. Inflatable dams are mounted

Fig. 12-14. Wood gates showing vegetation and leakage

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on concrete dam crests, in spillway bays, and on concrete sills placed directly across stream channels. They are water-control structures—storing water when inflated and releasing it when deflated. Air for inflation is typical for inflatable dams on the crest of concrete dams or in spillway bays. Water can be used for inflation if precise reservoir-level control is required. Freezing of water-filled tubes is avoided by adding antifreeze.

Loss of reservoir control is the only failure mode applicable to an inflatable dam. A sudden deflation could cause unanticipated reservoir discharge and, conversely, sudden inflation could cause unanticipated reservoir rise. Pressure maintenance is the performance indicator for inflatable dams, pressure being controlled by programmable logic.

Two types of inflatable dams are in common use. Both types are linked by piping to a compressor and accumulator that pump air or water into the bladder, causing the bladder to inflate. The first type is a rubber-like fiber or mesh-reinforced composite bladder, shown in Fig. 12-15. The dam is operated in the inflated condition to store water and deflated to release water.

The second type employs a similar bladder to raise and lower hinged flashboards, as shown in Fig. 12-16.

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Fig. 12-15. Rubber dam

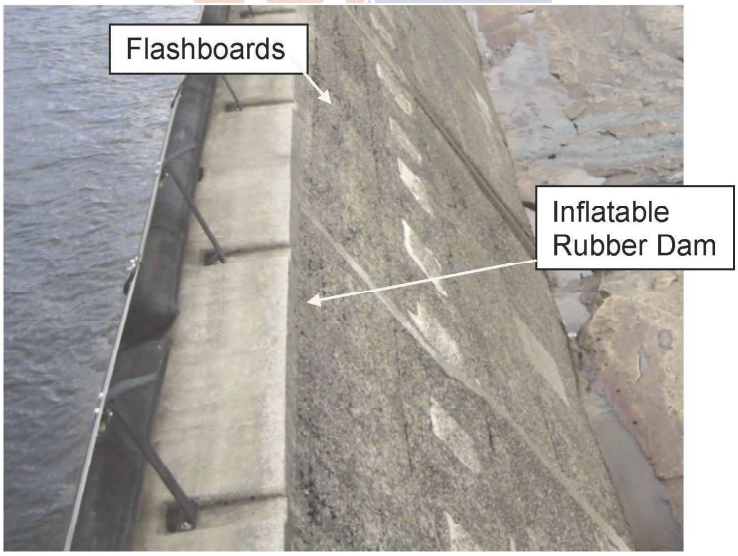

Fig. 12-16. Rubber dam with flashboards

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Instrumentation and monitoring of inflatable dams is done with prepackaged proprietary systems that do not require owner intervention. Rather, the supplier's technician usually works with the contractor on installation and startup. Vendor-supplied pressure gauges and pressure transducers are used to measure pressure in the bladder, compressor air accumulator, and reservoir surface elevation. Level control is achieved by a programmed logic controller (PLC). Periodic checks of the reservoir level sensed by the reservoir-level transducer are necessary by manually cross-checking with a staff gauge to calibrate the reservoir-level transducer.

12.1.5 Concrete Dams on Soil Foundations

Concrete dams founded on soil (Fig. 12-17) present the same vulnerabilities discussed under buttress dams founded on soil and require similar surveillance and instrumentation to address potential vulnerabilities associated with soil foundations. These dams include gravity dams and buttress dams supported on a mat or apron founded on soil. How these structures are designed and constructed vary in terms of the care provided to control seepage. As discussed previously in the section on buttress dams, seepage is a performance indicator to evaluate internal erosion potential through the foundation. Foundation settlement combined with internal erosion is the potential initiator of ground loss. Loss of ground under the concrete may cause the dam to move, adversely affecting stress distribution. If the concrete cannot successfully redistribute stresses, it will crack. Continued movement could lead to a loss of structural capacity. Movement and seepage measurements are the important performance indicators for

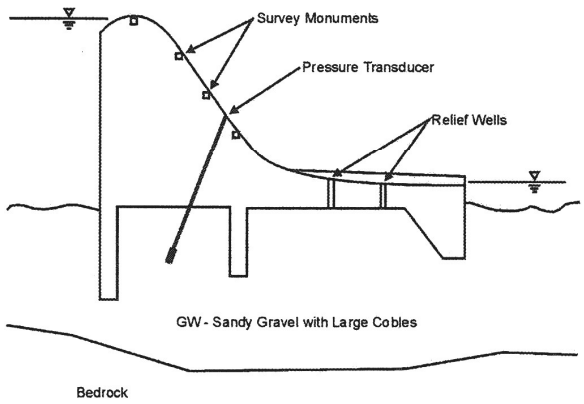

Fig. 12-17. Concrete dam on soil foundation
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

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concrete dams on soil foundations. Measurements can be obtained with piezometers and pressure transducers at the soil–concrete interface and by quantitatively measuring flow from drains or relief wells. Movement is measured by geodetic methods or with a movement-measuring instrument.

12.1.6 Concrete Dams on Pile Foundations

Concrete dams (gravity or slab-and-buttress dams) founded on piles (Fig. 12-18) are a subset of concrete dams founded on soil, with some distinctive vulnerabilities that need to be considered in the development of an instrumentation and monitoring program. Though not common, dams supported on piles do exist, particularly in the midwestern United States and in areas where competent bedrock is often quite deep.

Concrete dams on piles have similar vulnerabilities and performance indicators as dams on soil foundations. One distinct difference in design and construction is that the base of the dam is effectively supported on piles that restrain slab movement. The soil supporting the slab may provide a seepage path. Care must be taken to ensure that seepage does not exploit a path under the slab and initiate internal erosion.

Sealed piezometers, open standpipe piezometers, or pressure transducers can be used to measure uplift along the foundation interface. Foundation drains (relief wells or "weep holes" with reverse filters) within the dam can help control uplift pressures and safely collect and convey seepage flow without soil transport. Periodic measurements of flow from drains or wells, together with pressure readings from the piezometers, will allow an understanding of changes in the structural response to changes in reservoir levels

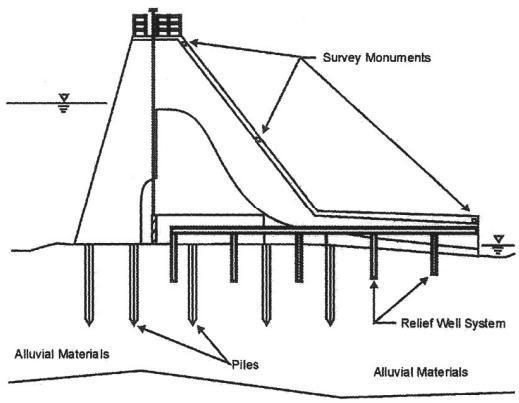

Fig. 12-18. Concrete dam on pile foundation
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

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and seasonal effects. Movement is measured by geodetic methods or other means for measuring movement.

A unique concern with dams supported on timber pile foundations is the potential for deterioration of the piles, leading to settlement and structural damage. There is no established practice to measure deterioration of timber or concrete piles supporting concrete dams.

12.1.7 Small Earth Dams

Thousands of small [less than 40 ft (12.2 m) high] earth dams exist around the world. Small dams often do not have established surveillance and monitoring programs. Many may not have any instrumentation at all. Owners of small dams may not be aware of their responsibilities for dam safety. Complete or partial failure of a small dam could cause loss of property or life, depending on reservoir capacity and downstream development. Small dams may not receive the attention that larger dams enjoy from state and federal regulators because they pose little or no hazard to the public. As is mentioned in Chapter 2, some small-dam owners may not realize their duty to retain the reservoir. A surveillance and monitoring plan for a small dam needs to be simple but effective. A visit by an experienced dam safety engineer to observe how a dam was designed, operated, and maintained may yield positive results by helping an owner develop the surveillance and monitoring plan. That plan could include a suggested frequency of owner observation of the dam, highlighting areas of vulnerability and performance indicators such as sink holes, new seepage, changes in seepage rate or clarity, and slope sloughing. Instrumentation such as headwater and tailwater staff gauges or, if seepage exists at the toe of the structure, a weir or set of piezometers, or other appropriate means of periodic measurement may be recommended to complement visual surveillance.

12.2 APPURTENANT STRUCTURES

Failure of an appurtenant structure may result in loss of reservoir control. Spillway gates, outlet works, intakes, and conveyances all have responsibilities to retain the reservoir. Instrumented monitoring is seldom employed to measure the functionality of these appurtenant structures. The means and methods for measuring movements and leakage of appurtenant structures are the same as those described in Chapter 10:

- Spillway gates are maintained to ensure that they pass a flood when needed, and plunge pools are measured to ensure that scour does not jeopardize dam stability.
- Outlet works are maintained to ensure that they are functional in the event that they are required to lower the reservoir.

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- Intakes are maintained with special attention to ensure that flow can be controlled in an emergency.
- Conveyances are maintained to ensure their ability to safely retain flow.
- Reservoir rims exhibiting instability are observed visually and occasionally instrumented to evaluate whether a landslide into the reservoir could initiate a failure sequence.

12.2.1 Spillways

Visual surveillance and regular maintenance are the most important duties to ensure that spillways are capable of safely discharging the inflow design flood. Instrumented monitoring is limited to periodic tests for trunnion friction, and strain measurements are occasionally performed to compute stress in gate members for comparison with design estimates.

Spillways discharging through a dam may deliver enough energy to scour the foundation progressively upstream to the point of undercutting the dam. Spillways discharging around a dam through a chute or tunnel may undercut a foundation as well. Routine scour measurements are important if foundations are erodible.

Stable transects are established across scour impact points. Geodetic techniques are useful for locating measurement points. Depths are measured at intervals along each transect to form a three-dimensional representation of scour, usually with a lead line or precision side-scan sonar. Figs. 12-19A and 12-19B illustrate the use of sonar and manual measurements, respectively, to check for scour from spillway discharge. Successive measurements are compared to estimate the rate of scour progression.

12.2.2 Outlet Works

Outlet works are located near a dam's foundation to permit discharge near the toe of accumulated reservoir sediment and to evacuate the reservoir in an emergency. Of interest is the depth of accumulated sediment, requiring a simple measurement.

The state of practice is for upstream control to leave conduits empty except when they are operated. Outlet conduits through concrete or bedrock typically are lined and are less problematic than conduits through the embankment fill. The primary areas of concern for monitoring outlet conduits through an embankment include development of seepage along the exterior of the conduit, which could initiate internal erosion, settlement, structural deformation, and collapse (Fig. 12-20).

Conduits may be instrumented, depending on identifying a performance indicator related to reservoir control. If pore-pressure measurements are required to evaluate stability, then hydraulic or electric piezometers can

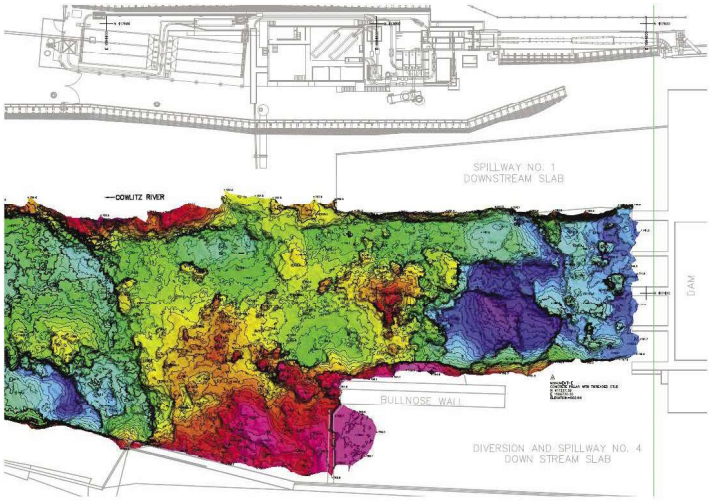

Fig. 12-19A. Multibeam side-scan sonar measurement
Source: Courtesy of City and Borough of Sitka, reproduced with permission.

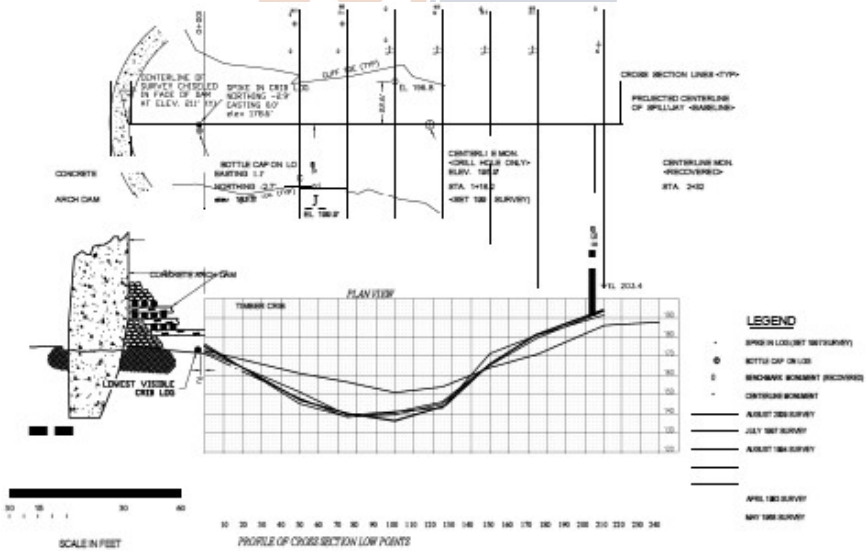

Fig. 12-19B. Manual measuring scour
Source: Courtesy of Lewis County PUD, reproduced with permission.

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Fig. 12-20. Internal erosion along outlet conduit
Source: ASDSO.

Fig. 12-21. Sediment from leaks into outlet works conduit

be placed alongside a conduit prior to backfill or during service by drilling through the wall of a conduit. Care must be taken that the tubing or cable does not form a preferential seepage path through the dam. When installed from the ground surface, piezometers should be situated at least one-conduit diameter away.

Outlet conduits are visually inspected to observe seepage at the downstream portal or at discrete locations (such as joints) along the length of the conduit. Periodic internal inspection includes checking alignment to judge whether movement is occurring, looking for sediment originating from the embankment, identifying structural cracking or corrosion that indicative of structural weakening.

Accumulation of sediment transported from an embankment into a conduit (Fig. 12-21) is a performance indicator that may require instrumented measurement because sediment transport into a conduit indicates a loss of ground outside that could initiate internal erosion in a failure sequence. Seepage at the outlet is measured using a weir or flume.

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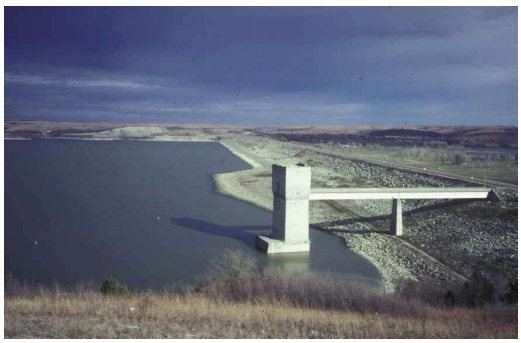

Fig. 12-22. Intake for outlet works

12.2.3 Intakes

Intakes may be free-standing for an embankment dam, integrated into a concrete dam, or located remotely from a dam (Fig. 12-22). Intakes supply flow directly to turbines or to a conveyance (e.g., penstock, flume, canal, or tunnel). If movement is a concern, then a typical measurement technique (e.g., geodetic survey, tiltmeter, or extensometer) is used.

12.2.4 Conveyances

Conveyances may be closed conduits such as penstocks or open channels such as flumes. Penstocks may be aboveground (supported on saddles) or belowground. Flumes and canals are aboveground. If leakage along a conveyance (Fig. 12-23) could be an initiator in a failure sequence resulting in a loss of reservoir control, then a typical measurement device (e.g., weir or flowmeter) is used. Similarly, if alignment of an aboveground conveyance (e.g., flume, penstock, or canal) requires checking, then instruments shown in Chapter 5 (such as strain gauges) may be required.

Scarps from active landslides and leaning trees are indicative of instability (Fig. 10-1).

Slope instability may initiate a landslide that blocks or damages a conveyance, causing a loss of capacity to deliver water. Geodetic techniques are used for surface measurements of movement, and inclinometers are used to measure movement in the subsurface. Heavy rainfall often causes soil to lose strength, but measuring rainfall may provide warning to suggest that enhanced visual surveillance is required (Fig. 12-24).

Rainfall is not the only initiator of slope instability above a conveyance.

Fig. 12-25 shows a section of a wooden flume that was destroyed by a rockfall that was triggered by an earthquake. Incipient landslide creep (Fig. 12-26) can also pose challenges to maintaining conveyance integrity.

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Fig. 12-23. Leaky penstock

Geodetic surveys measuring alignment allow for adjustments to be made to avoid penstock movement and potential rupture.

A piezometer in a canal (Fig. 12-27) can protect against overtopping by measuring a pressure equivalent to water surface elevation and transmit that elevation to the inlet and outlet controls to avoid overtopping.

Visual surveillance is the key to identifying a performance indicator that may lead to a loss of reservoir control through a canal appurtenant structure (Fig. 12-28).

Periodic inspection of tunnels will locate areas where reinforcement is required (Fig. 12-29). Instrumented measurements usually are not required except in cases where ground squeezing may require convergence measurements by tape extensometers.

12.2.4.1 Reservoir Rim. The Vajont disaster in Italy in 1963 (Fig. 12-30) took more than 2,000 lives and devastated the valley downstream from the

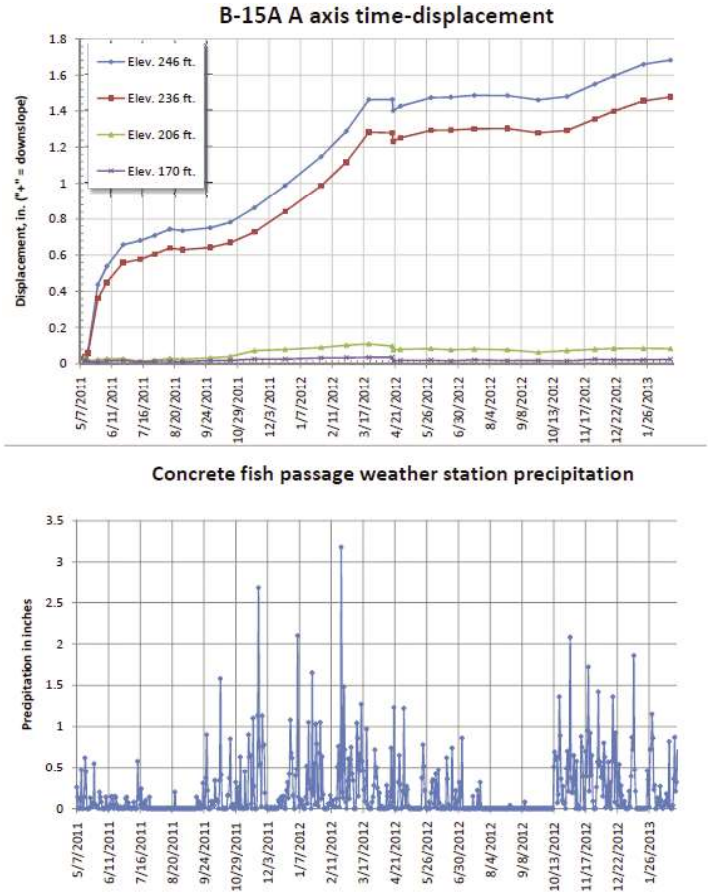

Fig. 12-24. Inclinometer response and rainfall
Source: Courtesy of PSE.

Fig. 12-25. Flume destroyed by rockfall initiated by an earthquake

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Fig. 12-26. Penstock ring girder shoe to adjust for creep in slope above

Fig. 12-27. Piezometer installation in power canal

Fig. 12-28. Canal buckle

Fig. 12-29. Partially lined tunnel in sandstone

Fig. 12-30. Vajont after landslide
Source: Wikimedia, https://en.wikipedia.org/wiki/Vajont_Dam (2018).

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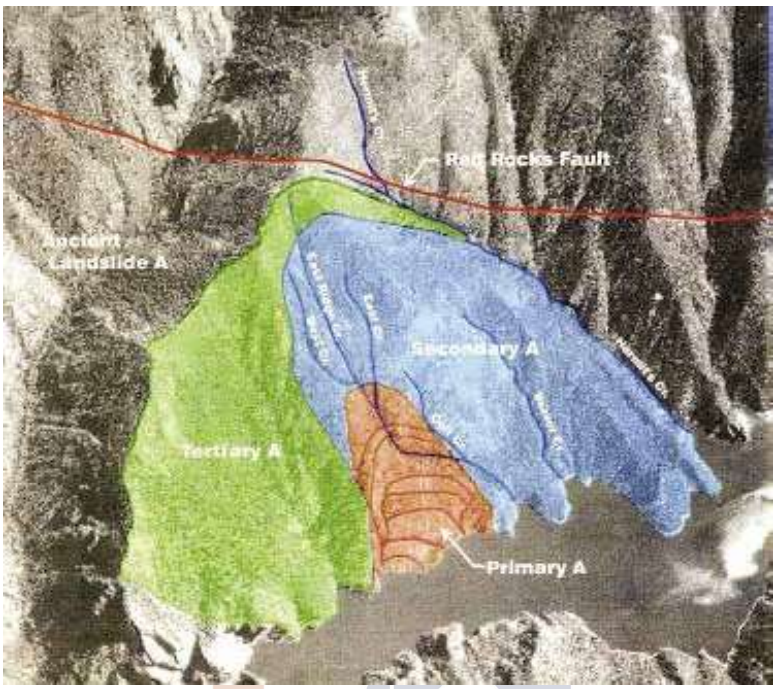

Fig. 12-31. Landslide moving toward reservoir
Source: U.S. Bureau of Reclamation.

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dam. An enormous block of rock on the left abutment slid into the reservoir, displacing the entire reservoir over the crest of the dam into the valley below.

The incident raised awareness of the potential for a landslide into a reservoir initiating a failure sequence by generating a wave that could overtop a dam. Although the Vajont disaster is the only known case of overtopping by a landslide, there are numerous cases of reservoir slope instability, especially during first filling and then during a reservoir's service life. Some landslides are large enough to merit monitoring because the conditions favor developing enough energy in a slope failure to generate a wave large enough to overtop a dam (Fig. 12-31).

The previously mentioned landslides have been monitored over time with geodetic techniques measuring positions of monuments and slope stakes. A steep valley, deep reservoir, adverse jointing, low strength of soil or rock, and a straight valley in front of a dam present the most likely candidates for instrumented monitoring. New techniques employing airborne LiDAR and InSAR also offer opportunities for measuring landslide movement.

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CHAPTER 13
SAMPLE DATA FORMS AND PLOTS

A picture is worth a thousand words.

English idiom

Chapter 8 discussed data recording and also presented some example formats for presenting seepage, pressure, and deformation data. This chapter presents sample reading recording forms, data tables, and sample data plots.

Many organizations use customized data forms and presentation formats that have been tailored to meet the needs of the organization. Many variations of data recording forms and data plots are suitable, but the use of standardized forms within an organization reduces recording errors and misinterpretation.

Best practice is to include the following elements on data recording forms:

- Instrument name,
- Date of the reading,
- Name of the person recording the data,
- Field reading, and
- Description of the field reading.

These elements provide the minimum information required to establish the value of the measured parameter at a specific point in space and time. The instrument name ties the field reading to a particular location and elevation when associated with the instrument attributes. The name of the person recording the data attests to the veracity of the reading and provides a contact point if there are any questions about the reading.

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The data recording form may include additional optional information. The following list shows several types of information and explains how each type could be useful:

Time (hour and minute) of the reading—This may not be relevant if readings are taken far apart in time, for example, quarterly. If the reading interval is short, including the time can help produce more accurate time plots or help correlate the reading with a particular loading condition or event. If multiple readings are taken in a single day, the time is necessary.

Current weather conditions—Wet, dry, hot, cold, or stormy conditions can affect some field instrument readings. The response of some instrument sensors may drift at temperature extremes. Freezing conditions can affect water-level measurements. Variations in barometric pressure can affect pressure sensors if they are not properly vented to the atmosphere.

Plan view of the instrument location—A small plan view or map of the instrument location can help field personnel locate the instrument. Although each instrument is clearly marked, a plan view can help avoid confusion among similar instruments, especially if they are located in a group.

Coordinates of the instrument location—GPS coordinates can help field personnel locate the instrument. Although each instrument is clearly marked, geographic coordinates can provide additional confirmation that the correct instrument is being read.

Cross section of the instrument location—A cross section of the instrument installation can be useful in some situations. For example, if several hydraulic piezometers are installed in a single borehole but at different depths, the cross section may help detect situations in which they have been confused. The cross section may also alert field personnel that the reading does not seem reasonable and lead to a repeat measurement to confirm or correct the reading.

Instrument attributes—The instrument attributes may provide information about the surface characteristics of the instrument such as the height of a piezometer standpipe above the ground surface. If the field situation does not correspond to the attributes, the instrument may have been modified or damaged. If the attributes include the maximum depth of a hydraulic piezometer, it can help field personnel determine whether the instrument is dry or obstructed at an elevation above the bottom.

Most recent previous readings—The readings of most field instruments do not change dramatically unless some loading condition has changed. Comparing the current reading to a previous reading can help identify changed conditions or erroneous readings.

Identification of the reading device—Most instruments are read with equipment that needs to be checked or calibrated periodically. Identifying the reading device links the reading to the calibration records, increasing confidence in the accuracy of the reading or identifying readings that may have been taken with a faulty reading device.

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Regardless of the details, well-designed forms and plots incorporate features that facilitate understanding of the instruments being read or evaluated. As such, best practices suggest that the forms and plots contain enough information about the instruments so that the user can understand the instruments and detect trends and anomalies. The following sections discuss a variety of forms and plots from actual projects.

13.1 READING RECORDING FORMS AND DATA REPORTS

13.1.1 Seepage

Table 13-1 shows a simple data report that tabulates weekly readings for several types of instruments. The form presents the reading dates, the instrument names, and the reading values. This form includes measurements of water level, flow rate, and pressure head from vibrating-wire piezometers (VWP in the table). Note that each data type includes the units of measurement.

13.1.2 Water Level

Table 13-2 presents a reading form for the manual collection of water-level data from a series of piezometers and wells. This was a standard form used for monthly data collection. Note that the preprinted form includes the list of instruments to read (column 1), the instruments' sensor elevations (column 3), and the elevations at the top of each instrument (column 4). The date in column 5 applies to all the readings. The field technician wrote the field reading, in this case the distance from the top of the pipe to the water level, in column 6 and then calculated the total head using the formula shown at the top of column 7. Any unusual aspects of the reading were noted as observations in column 8. Note, for example, that instrument L2-1 was contaminated and instrument P2-4 was dry.

In Table 13-2, the entries for the open hydraulic piezometers P2-4 and P2-5 should have indicated readings in column 6 for the depths measured even though no water was detected. This is considered a best practice and would have allowed a comparison of the elevation of the dry readings to the sensor elevations to distinguish between piezometers which were completely dry and ones that had obstructions in the standpipe at some elevation above the sensor.

13.1.3 Pressure

The data report in Table 13-3 presents pressure readings for VWPs installed in a standpipe. The readings were taken manually with a portable readout and then transferred to a spreadsheet to ease the conversion calculations

Table 13-1. Canal Flow and Pressure Record.

Table 1 - LMS Pipe Flows and Drain VWP Head									
	Canal	LMS 1		LMS 2		LMS 3		LMS 4	
DATE	(ft – el)	Pipe (gpm)	VWP (ft)	Pipe (gpm)	VWP (ft)	Pipe (gpm)	VWP (ft)	Pipe (gpm)	VWP (ft)
6/14	602.3	3.7	-1.04	13.3	-0.83	7.1	-0.28	10.7	-0.33
8/16	602.2	2.4	-0.92	4.1	-0.72	2.6	-0.13	5.9	-0.18
8/23	601.4	2.5	-1.0	6.2	-0.79	3.7	-0.20	7.4	-0.28
8/30	602.2	2.6	-0.98	5.0	-0.80	3.7	-0.19	7.3	-0.25
9/6	603.4	2.5	-1.06	4.9	-0.88	3.3	-0.31	7.2	-0.36
9/13	602.1	3.4	-1.11	5.6	-0.94	4.9	-0.33	8.7	-0.37
9/20	602.0	3.9	-1.05	8.2	-0.87	7.1	-0.23	11.6	-0.32
9/21	601.9	4.0		7.8		5.7		10.8	
9/22	602.2	5.6		8.4		6.1		11.3	
9/27	602.2	3.4	-1.09	7.5	-0.94	5.7	-0.32	10.8	-0.36
10/5	**602.2**	**4.8**	**-1.08**	**7.5**	**-0.94**	**5.7**	**-0.34**	**10.6**	**-0.37**
	Canal	LMS 5		LMS 6		LMS 7		LMS 8	
DATE	(ft – el)	Pipe (gpm)	VWP (ft)	Pipe (gpm)	VWP (ft)	Pipe (gpm)	VWP (ft)	Pipe (gpm)	VWP (ft)
6/14	602.3	7.9	-0.13	8.8	-0.31	6.3	-0.20	6.1	-0.54
8/16	602.2	3.6	-0.01	3.0	-0.27	3.1	-0.12	2.9	-0.41
8/23	601.4	4.8	-0.11	3.9	-0.37	4.4	-0.21	4.7	-0.47
8/30	602.2	5.6	-0.05	3.4	-0.34	3.0	-0.20	4.5	-0.47
9/6	603.4	4.5	-0.18	3.0	-0.47	3.5	-0.35	3.2	-0.60
9/13	602.1	7.3	-0.17	6.0	-0.44	5.4	-0.31	12.9	-0.58
9/20	602.0	8.3	-0.12	6.6	-0.31	7.2	-0.24	8.1	-0.54
9/21	601.9	7.8		6.4		6.3		6.1	
9/22	602.2	8.8		7.2		8.7		33.1	
9/27	602.2	9.3	-0.18	8.7	-0.38	11.5	-0.13	10.3	-0.58
10/5	**602.2**	**9.4**	**-0.15**	**7.6**	**-0.44**	**9.4**	**-0.28**	**9.4**	**-0.61**
	Canal	LMS 9		LMS 10		LMS 11		Total Flow	
DATE	(ft – el)	Pipe (gpm)	VWP (ft)	Pipe (gpm)	VWP (ft)	Pipe (gpm)	VWP (ft)	(gpm)	
6/14	602.3	6.4	0.29	2.1	-0.13	11.6	0.13	83.9	
8/16	602.2	4.2	0.47	3.1	0.05	13.3	0.29	48.1	
8/23	601.4	6.2	0.40	3.9	0	15.9	0.22	63.7	
8/30	602.2	4.7	0.40	2.7	-0.04	11.9	0.25	54.5	
9/6	603.4	3.6	0.25	1.9	-0.28	9.4	0.10	47.1	
9/13	602.1	3.3	0.28	6.1	-0.13	5.9	0.15	70.5	
9/20	602.0	7.7	0.34	4.8	-0.04	25.8	0.27	99.3	
9/21	601.9	6.2		3.8		19.8		84.8	
9/22	602.2	6.0		4.1		19.9		119	
9/27	602.2	7.9	0.30	5.1	-0.06	13.5	0.14	93.6	
10/5	**602.2**	**9.2**	**0.28**	**9.1**	**-0.11**	**19.7**	**0.18**	**102.4**	

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Table 13-2. Water-Level Record.

OIL LEVEL: 15'04"

AMUAY SURVEILLANCE

PERIOD: "July 79"

① INSTRUMENT	② LOCATION	③ SENSOR ELEVATION	④ TOP ELEVATION	⑤ DATE	⑥ READING	⑦=④-⑥ TOTAL HEAD	⑧ OBSERVATIONS
W2-1	FORS-2	- 0,594	+ 24,70	10-7-79	— o —	— o —	
W2-2		- 1,03	+ 20,30		17.29	+3.01	
W2-3		- 3,14	+ 6,50		4.76	+1.74	
L2-1		+ 6,52	+ 20,27		— o —	— o —	CONTAMINADO
P2-1		- 6,47	+ 20,10		17.61	+2.41	
P2-2		- 3.82	+ 25.36		21.82	+3.54	
P2-3		- 1,17	+ 24,68		21.31	+3.37	
P2-4		+ 10,68	+ 25,65		— o —	— o —	DRY.
P2-5		+ 12,70	+ 25,84		— o —	— o —	DRY
P2-6		- 6,05	+ 3,34		— • —	— • —	REVOSADO.
P2-7		- 15,70	+ 3,311		— • —	— • —	" "
P2-8		- 6,07	+ 3,395		2.13	+1.26	
P2-9		- 16,39	+ 3,395		4.43	-1.03	
P2-10		- 7,20	+ 3,352		5.96	-2.60	
P2-11		- 16,50	+ 3,352		5.98	-2.62	
P2-12		- 6,17	+ 3,62		2.74	+ 0.88	
P2-13		- 23,08	+ 3,62		6.98	+3.36	
P2-14		- 8,873	+ 6,927		5.67	+1.25	

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from the raw units (B units = Hertz^2/1,000) to head pressure in KPa to head pressure in meters of water head (m H_2O) to piezometric elevation. The conversion formula, which takes initial readings, temperature changes, and barometric changes into account, is shown in the table.

13.1.4 Displacement

The data report in Table 13-4 presents weekly readings for joint meters that are being used to measure the movement of the joints along a concrete-lined canal.

Note that in addition to the reading dates and instrument names, the report includes the raw field reading (frequency), the measured temperature, which is used in the strain calculation, the calculated displacement, and the displacement from the initial and most recent prior readings. At each location, the displacement is being measured in three dimensions.

13.1.5 Inclinometers

Tables 13-5 and 13-6 illustrate inclinometer raw data as collected by an inclinometer system. The table is actually a CSV file exported from the readout

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Table 13-3. VW Piezometer Record.

Piezo ID:	PZ- E	
Serial number:	VW22654	(2 MPA range)
Installation Date:	29-Nov-11	
Coordinates:		
Collar Elevation:	1583.76	MASL
Borehole Depth;	136	M
Elev. Borehole Bottom:	1447.76	MASL
	MASL: Meter ABOVE SEA LEVEL	

Base Reading (29-November-2011)

Reading (B units):	9203.00	(Prior to installation)
Temperature (°C):	14.90	(Prior to installation)
Sensor Elevation:	1479.738	(MASL)
Sensor Depth:	104.03	M

Pressure P(kPa)=C.F.(Li-Lc)-[Tk(Ti-Tc)]+[0.10(Bi-Bc)]
Li, Lc = Initial and Current Reading (B units)
Ti, Tc = Initial and Current Temperature (°C)
Bi, Bc = Initial and Current Barometric pressure (millibars)

Calibration Factors			
C.F. =	0.46396	kPa/B unit	
Tk =	0.89474	kPa/ºC rise	B units = Hz2 / 1000 ie: 2700Hz = 7290 B units

Barometric Pressure		
Initial (Bi):	1013.0	milibars

Date	Time	Collar Elevation (MASL)	Sensor Elevation (MASL)	Reading (B Units Hz)	Temperature (° C)	Barometric Pressure (millibars)	Calculated Head Pressure (kPa)	Calculated Head Pressure (m H2O)	Calculated Piezometric Elevation (MASL)	Observations
29/11/2011	17:00	1583.76	1479.74	7356.8	33.8	1013.0	974.77	99.365	1579.103	Initial reading after installation
30/11/2011	17:10	1583.76	1479.74	7327.2	34.0	1016.0	988.99	100.814	1580.552	
01/12/2011	16:25	1583.76	1479.74	7311.6	33.3	1021.0	996.10	101.539	1581.277	
02/12/2011	17:10	1583.76	1479.74	7367.0	32.7	1012.0	968.96	98.772	1578.510	
03/12/2011	9:20	1583.76	1479.74	7373.0	32.6	1011.0	965.98	98.469	1578.207	
05/12/2011	13:34	1583.76	1479.74	7395.6	32.4	1009.0	955.12	97.362	1577.100	
06/12/2011	16:30	1583.76	1479.74	7409.1	32.3	1015.0	949.37	96.775	1576.513	
07/12/2011	17:50	1583.76	1479.74	7420.8	32.3	1008.0	943.24	96.151	1575.889	
08/12/2011	17:20	1583.76	1479.74	7428.1	32.2	1012.0	940.16	95.837	1575.575	
09/12/2011	16:35	1583.76	1479.74	7436.9	32.2	1002.0	935.08	95.319	1575.057	
10/12/2011	10:10	1583.76	1479.74	7442.5	32.2	1005.0	932.78	95.085	1574.823	
12/12/2011	16:15	1583.76	1479.74	7459.4	32.1	1004.0	924.75	94.266	1574.004	
15/12/2011	15:50	1583.76	1479.74	7473.5	32.1	1015.0	919.31	93.711	1573.449	
19/12/2011	16:20	1583.76	1479.74	7497.4	32.0	1001.0	906.73	92.429	1572.167	
22/12/2011	10:00	1583.76	1479.74	7509.0	32.0	999.0	901.15	91.860	1571.598	
05/01/2012	11:12	1583.76	1479.74	7557.3	31.9	1014.0	880.15	89.720	1569.458	
09/01/2012	11:15	1583.76	1479.74	7569.7	31.9	1011.0	874.10	89.103	1568.841	
12/01/2012	17:55	1583.76	1479.74	7579.1	31.4	1005.0	868.69	88.551	1568.289	
16/01/2012	16:59	1583.76	1479.74	7588.9	31.8	1013.0	865.30	88.206	1567.944	
25/01/2012	17:58	1583.76	1479.74	7611.4	31.8	1011.0	854.66	87.121	1566.859	
01/02/2012	16:18	1583.76	1479.74	7626.1	31.8	1015.0	848.24	86.467	1566.205	
10/02/2012	10:21	1583.76	1479.74	7637.0	31.8	1016.0	843.28	85.962	1565.700	
15/02/2012	16:12	1583.76	1479.74	7641.5	31.8	1012.0	840.79	85.708	1565.446	
20/02/2012	15:51	1583.76	1479.74	7648.1	31.8	1013.0	837.83	85.406	1565.144	
23/02/2012	11:18	1583.76	1479.74	7651.8	31.8	996.0	834.42	85.058	1564.796	

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Table 13-4. Record of Joint Meters.

Table 2 Intake / Canal Liner Joint Meters
Baseline, 1st Reading, and Recent 2006 Data

	West Side Joint Meter														
Channel Axis	Ch. 1 West X					Ch. 2 West Y					Ch. 3 West Z				
DATE	Frequency (Hz)	Temp. °C	Distance (mm)	Change from Prev. (mm)	Change from Base (mm)	Frequency (Hz)	Temp °C.	Distance (mm)	Change from Prev. (mm)	Change from Base (mm)	Frequency (Hz)	Temp °C.	Distance (mm)	Change from Prev. (mm)	Change from Base (mm)
Base Line Installation (6/21/06)	**2860.9**	**N/A**	**40.81**			**2629**	**N/A**	**29.45**			**2580.6**	**N/A**	**28.68**		
6/28/06	2857.1	10.9	40.65		0.16	2615.4	10.9	28.95		0.51	2565	11	28.11		0.57
9/6/06	2851.10	12.3	40.40	0.01	0.41	2605.60	12.3	28.59	0.03	0.87	2575	12.30	28.47	-0.16	0.21
9/13/06	2850.10	12	40.36	0.04	0.45	2604.70	12	28.55	0.03	0.90	2575.7	12.00	28.50	-0.03	0.18
9/20/06	2850.80	11.7	40.39	-0.03	0.43	2603.50	11.7	28.51	0.04	0.95	2577.3	11.80	28.56	-0.06	0.12
9/27/06	2851.60	11.2	40.42	-0.03	0.39	2604.00	11.2	28.53	-0.02	0.93	2578	11.20	28.58	-0.03	0.10
10/5/06	2849.00	13.4	40.31	0.11	0.50	2601.50	13.5	28.44	0.09	1.02	2578.2	13.50	28.59	-0.01	0.09
	East Side Joint Meter														
Channel Axis	Ch 1 East X					Ch. 2 East Y					Ch. 3 East Z				
DATE	Frequency (Hz)	Temp °C.	Distance (mm)	Change from Prev. (mm)	Change from Base (mm)	Frequency (Hz)	Temp °C	Distance (mm)	Change from Prev. (mm)	Change from Base (mm)	Frequency (Hz)	Temp °C	Distance (mm)	Change from Prev. (mm)	Change from Base (mm)
Base Line Installation	**2599.6**	**N/A**	**30.55**			**2820**	**N/A**	**39.27**			**2666.6**	**N/A**	**32.27**		
6/28/06	2604.7	10.8	30.75		-0.20	2818.5	10.8	39.21		0.06	2615.3	10.8	30.28		1.99
9/6/06	2603.20	12.20	30.69	-0.02	-0.14	2813.10	12.2	38.98	0.01	0.29	2623.60	12.3	30.60	-0.14	1.67
9/13/06	2602.90	11.80	30.68	0.01	-0.13	2813.10	11.7	38.98	0.00	0.29	2624.10	11.8	30.62	-0.02	1.65
9/20/06	2603.78	11.60	30.72	-0.03	-0.16	2813.00	11.7	38.98	0.00	0.29	2625.20	11.8	30.66	-0.04	1.61
9/27/06	2604.70	11.20	30.75	-0.04	-0.20	2813.30	11.1	38.99	-0.01	0.28	2626.20	11.1	30.70	-0.04	1.57
10/5/06	2602.00	13.40	30.65	0.10	-0.09	2811.70	13.4	38.92	0.07	0.34	2626.00	13.4	30.69	0.01	1.58

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Table 13-5. Inclinometer Data Record.

Inclinometers	Location	Depth [Elevation] of Movement (feet)		Movements Recorded by Inclinometers: Prior to May 2000	May 2000 to May 2001	May 2001 to April 2002	April 2002 to April 2003
Forebay							
B-17	Mid-slope	13	[763]	1.0"(a)	--	--	--
F-1	Upper slope - west side	35	[744]	0.8"	0.0"	0.3"	0.2"
F-2	West side on canal road	(b)	--	0.0"	0.0"	0.0"	0.0"
F-3	Upper slope - east side	25	[767]	1.0"	0.0"	-- (a)	--
F-4	East side on canal road	(b)	--	0.0"	0.0"	0.0"	0.0"
F-5	West side above cut slope	53	[747]	0.3"(c)	0.0"	0.3"	0.2"
F-6	East side above cut slope	69	[756]	0.2"(c)	0.0"	0.4"	0.3"
East Percy							
B-9	Top of slide	19	[850]	0.2"	0.0"	0.0"	0.0"
Swafford							
B-14	Upper slide area (west side)	58	[769}	0.3"	0.0"	0.0"	0.0"
B-15	Top of steep slope (west side)	64	[762]	0.4"	0.0"	0.0"	0.0"
SF-1	Upper slide area (center)	80	[750]	0.3"	0.0"	0.0"	0.0"
SF-2	Upper slide area (east side)	28	[794]	0.5"	0.0"	0.0"	0.0"
"	" " " "	72	[750]	0.2"	0.0"	0.0"	0.0"
SF-3	Lower slide area (east side)	(b)	--	0.0"	0.0"	0.0"	0.0"
McKenzie River							
B-19	Top of embankment (east)	21	[719]	0.2"	0.0"	0.0"	0.0"
B-21A	Top of embankment (center)	5	[735]	0.3"	0.0"	0.1"	0.0"
B-23	Top of embankment (west)	27	[713]	0.5"	0.1"	0.0"	0.0"
Canal Embankment							
C-1	Top of embankment (Sta. 246+00)	(b)	--	--	--	0.0" (d)	0.0"
C-2	Top of embankment (Sta. 231+50)	(b)	--	--	--	0.0" (d)	0.0"
C-3	Top of embankment (Sta. 220+00)	(b)	--	--	--	0.0" (d)	0.0"

(a) Unable to read; slide movement has distorted casing too much for monitoring probe to pass through shear zone; (b) No movement to date; (c) Installed October 1999; and (d) Installed January 2002.

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Table 13-6. Inclinometer Data Record.

File Version	2.1	
File Type	Digital Inclinometer	
Site	Left Abutment	
Borehole	Borehole_1	
Probe Serial#	DP03530000	
Reel Serial#	DR08380000	
Reading Date(m/d/y)	22/06/2008	12:16:32
Depth	-41	
Interval	0.5	
Depth Units	meters	
Reading Units	meters	
Operator	JJR	
Comment: Initial Reading Inclo I-1		
Offset Correction	0	
ELEV. 448.176		

Depth		Face A+	Face A-	Face B+	Face B-
-0.5	447.676	-0.000064	0.001299	-0.005706	0.006658
-1	447.176	-0.000515	0.001569	-0.006643	0.007637
-1.5	446.676	0.000611	0.000807	-0.006418	0.032546
-2	446.176	0.001157	0.000182	-0.006276	0.017232
-2.5	445.676	0.001795	-0.000153	-0.005835	0.042591
-3	445.176	0.000965	0.000484	-0.004605	0.029980
-3.5	444.676	0.000818	0.000416	-0.002506	0.007832
-4	444.176	-0.001470	0.002610	-0.001659	0.002635
-4.5	443.676	-0.001460	0.002549	-0.001119	0.002095
-5	443.176	-0.000250	0.001938	-0.000117	0.036616
-5.5	442.676	0.001478	-0.000355	0.001752	-0.000654
-6	442.176	0.004008	-0.002768	0.003622	0.001911
-6.5	441.676	0.005700	-0.004566	0.004526	-0.003575
-7	441.176	0.003817	-0.002654	0.003742	-0.002775
-7.5	440.676	0.003161	-0.002037	0.002923	-0.001932
-8	440.176	0.003267	-0.002053	0.003428	-0.002457
-8.5	439.676	0.002102	-0.000978	0.003078	-0.002118
-9	439.176	0.002342	-0.001146	0.003032	-0.002043
-9.5	438.676	0.003758	-0.002581	0.003806	-0.002661
-10	438.176	0.003635	-0.002448	0.004285	-0.002135

Source: Courtesy of Puget Sound Energy.

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unit of the inclinometer system and shows readings in two perpendicular directions (A and B) that correspond to the orientation of the grooves in the inclinometer casing in which the inclinometer probe is run. In most systems, a first pass is done from bottom to top and readings are taken every 0.5 m (2 ft), followed by a second pass where the probe is turned 180° to eliminate systematic error bias that may be present in the probe. By convention, the readings of the first pass are labeled A+ and B+, and the readings of the second pass are labeled A– and B–. The actual reading that will be used in subsequent calculations will be 0.5 × [(A+) + (A–)] and 0.5 × [(B+) + (B–)], thereby eliminating the error bias. At the same time, calculating and plotting 0.5 × [(A+) – (A–)] and 0.5 × [(B+) – (B–)] display the error bias, usually called "checksums," and the checksums should be about constant for all readings in order to consider the set of readings as acceptable.

Additional calculations then need to be done in order to determine the profile of horizontal displacements in directions A and B, which is the objective of an inclinometer survey. These calculations are now always done by software, using either a spreadsheet or dedicated inclinometer software.

13.1.6 Temperature

Temperature per se is seldom measured; however, it is routinely measured if temperature can affect a measurement. As shown in Table 13-7, three separate temperatures are displayed (air, water, and gallery) to aid in the interpretation of the pressures in gravity dam monolith drains. The drain pressures respond to changes in the difference between air and water temperature and temperature in the gallery. Pressures rise during cold weather and decline in hot weather.

13.2 DATA PLOTS

This section presents sample data plots for various instrument types and highlights some of the key features and best practices of the plots.

Fig. 13-1 shows an instrument layout through a gravity dam, identifying the locations of the crest monuments, piezometers, and drains. The legend and the symbols identify the type of each instrument. This type of plan view is useful both to the field technician who needs to locate and read the instruments and to the engineer who needs to evaluate the significance of the readings.

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Table 13-7. Temperature Relative to Drain Pressures.

		HEADWATER	561.2	560.5	560.1	560.4	561.1	560.3
		TAILWATER	492.3	493.2	490.5	490.7	492.8	492.6
		HEAD	68.9	67.3	69.6	69.7	68.3	67.7
		AIR TEMP	33	38	36	35	36	37
		WATER TEMP	40	40	40	40	40	40
		GALLERY TEMP	52	52	51	50	50	50
DRAIN HOLE DATA:			2:00 PM	9:00 AM	8:45 AM	9:30 AM	6:45 AM	8:00 AM
STATION		DATE	1/5/2015	1/6/2015	1/7/2015	1/8/2015	1/9/2015	1/12/2015
==========	====	==========	==========	==========	==========	==========	==========	==========
Mono 4								
4-1.75		PRESSURE (PSI)	8.5	8.0	9.0	7.0	8.0	7.5
		STATIC WATER ELEV. (FT)	487.6	486.5	488.8	484.2	486.5	485.3
4-2.5		PRESSURE (PSI)	17.5	17.0	16.5	16.5	15.5	16.0
		STATIC WATER ELEV. (FT)	508.4	507.3	506.1	506.1	503.8	505.0
4-5.5		PRESSURE (PSI)	31.0	31.0	30.0	31.0	30.0	32.0
		STATIC WATER ELEV. (FT)	539.6	539.6	537.3	539.6	537.3	541.9
4-6.25		PRESSURE (PSI)	20.2	22.0	19.0	25.0	28.0	25.0
		STATIC WATER ELEV. (FT)	514.7	518.8	511.9	525.8	532.7	525.8

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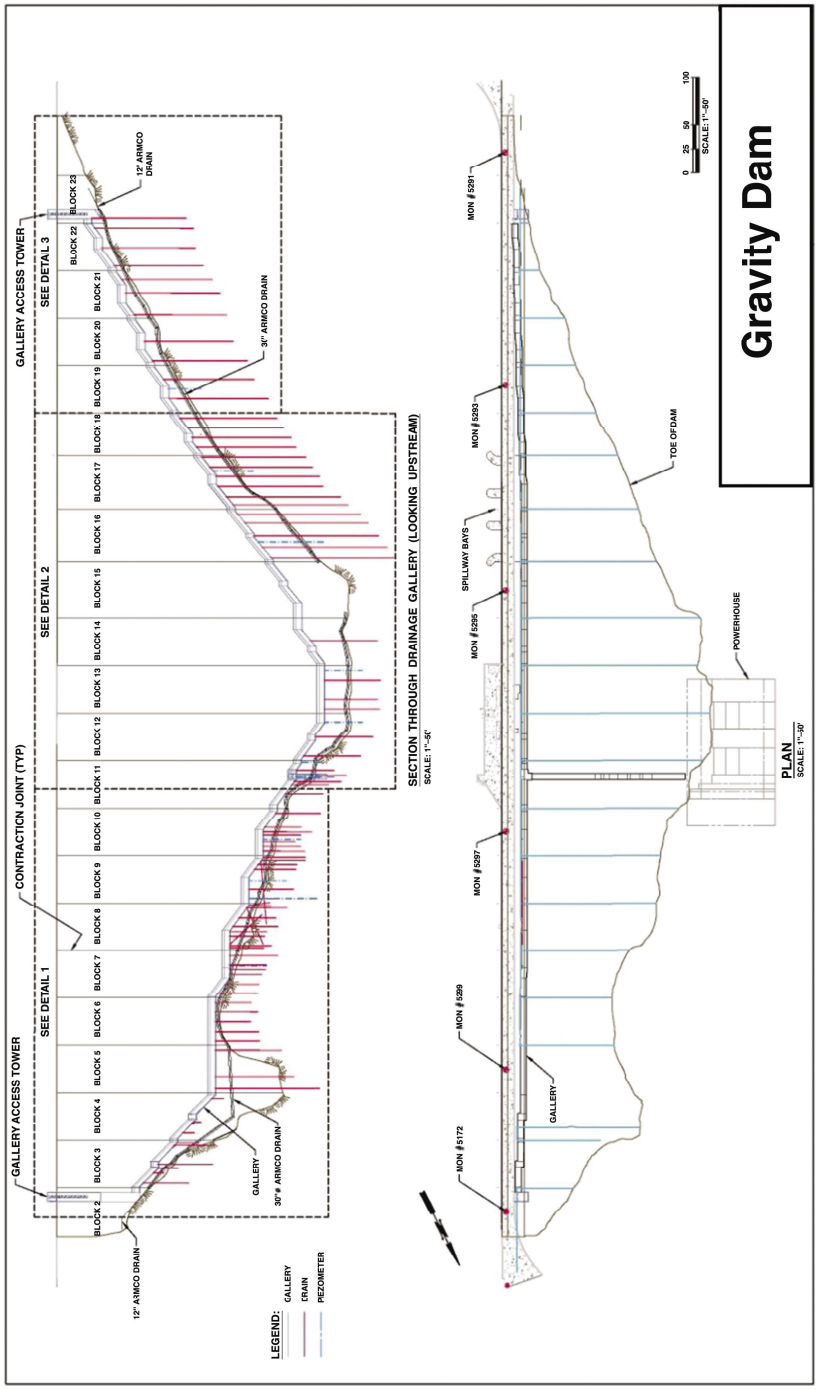

Fig. 13-1. Gravity dam instrument layout
Source: Courtesy of PSE.

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13.2.1 Seepage

Fig. 13-2 presents a historical trend data plot of the flow rates measured in four toe drains associated with a hydroelectric dam. The data plot includes a separate graph of the reservoir and tailwater elevations to help evaluate whether changes in the toe drain flows correlate with changes in the water elevations. The data are continuous, indicating that they were obtained with a data-acquisition system, not manual readings. Several of the lines show dramatic variations in the flow rates over short time periods, which should be explained in the data evaluation report, confirms the high degree of scatter in the data ($R^2 = 0.59$), and quantifies the average long-term decline in the flow rate (decreasing by 0.12 gpm/day).

Fig. 13-3 summarizes the flow data from all the toe drains at a hydroelectric dam referenced in Fig. 13-1 over the same time period and the equation associated with the trend line.

13.2.2 Foundation Pressure

Fig. 13-4 shows a typical plot of piezometer response with time and headwater.

13.2.3 Pressure

Piezometer levels can indicate drain pressures and may be expressed as shown on Fig. 13-5.

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13.2.4 Displacement

Fig. 13-6 illustrates a seasonal data plot of the horizontal movement of the center point in a concrete arch dam. Note the cyclical nature of the movements and their correlation to changes in temperature. Note also that the legend indicates the direction of the movements.

Fig. 13-7 presents another way of showing displacement. The vertical axes indicate the horizontal and vertical movements across a crack gauge across two monoliths and the reservoir elevation.

13.2.5 Temperature

Fig. 13-8 presents reservoir temperature versus time and concrete temperature at various depths versus time. From these two plots, the temperature differential can be extracted, allowing input to a thermal model for estimating stress in concrete.

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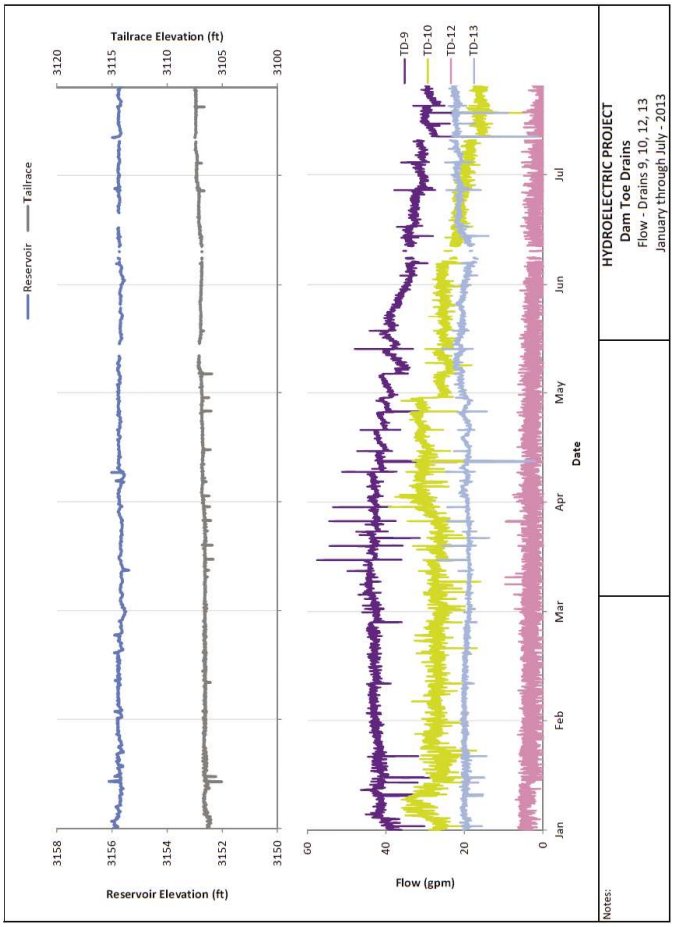

Fig. 13-2. Drain flows
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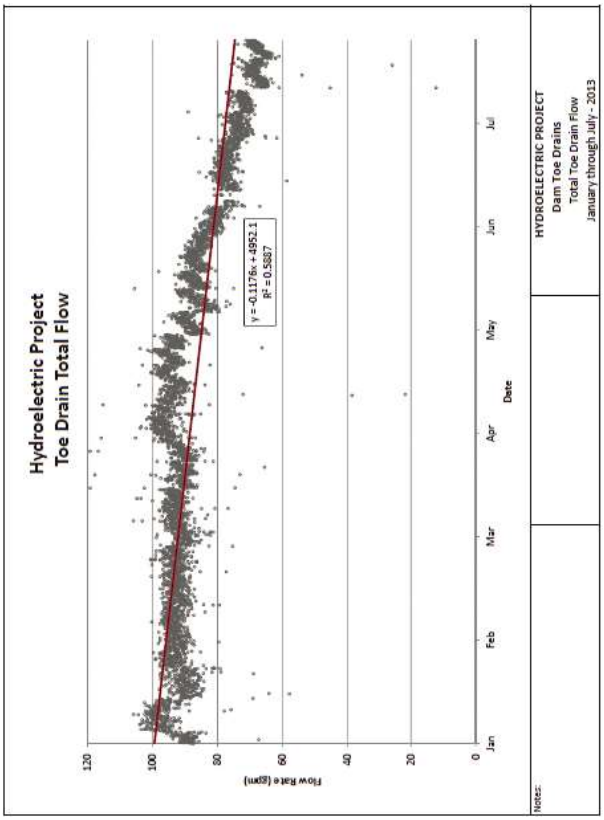

Fig. 13-3. Decreasing trend of drain flows
Source: Courtesy of Portland General Electric, reproduced with permission.

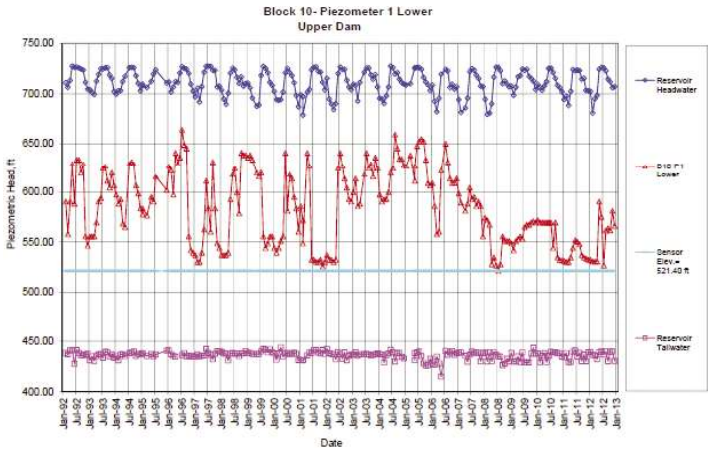

Fig. 13-4. Piezometer response with time and headwater
Source: Courtesy of PSE.

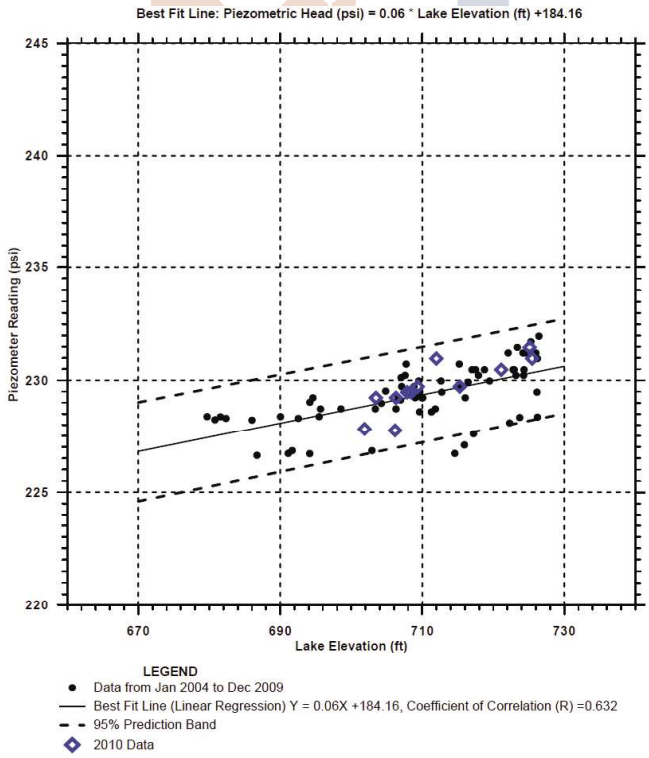

Fig. 13-5. Pressure versus headwater
Source: Courtesy of PSE.

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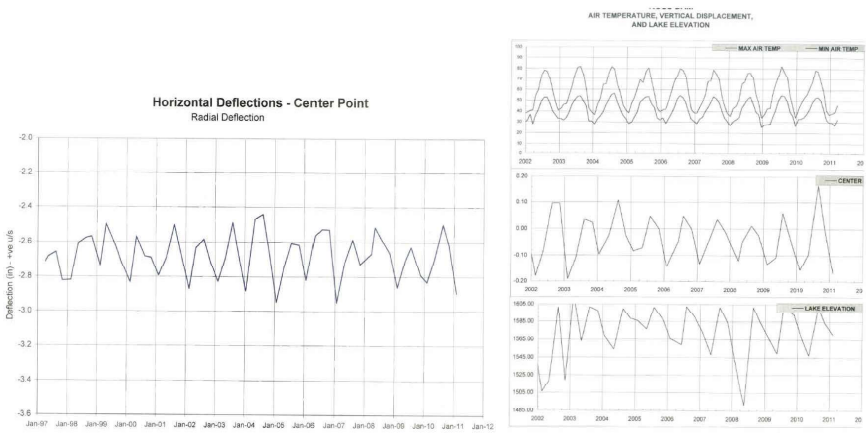

Fig. 13-6. Displacement versus time and temperature
Source: Courtesy of Seattle City Light, reproduced with permission.

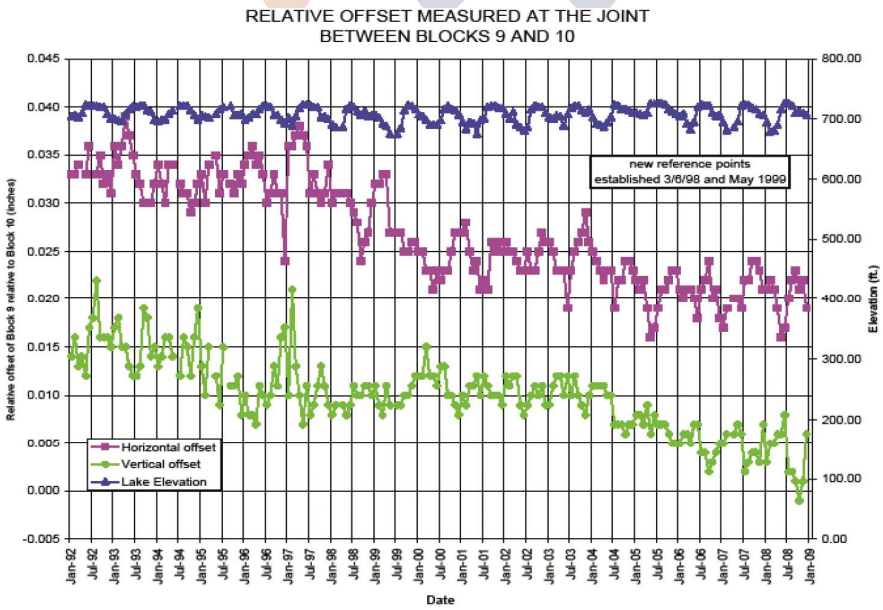

Fig. 13-7. Movement across a crack gauge
Source: Courtesy of PSE.

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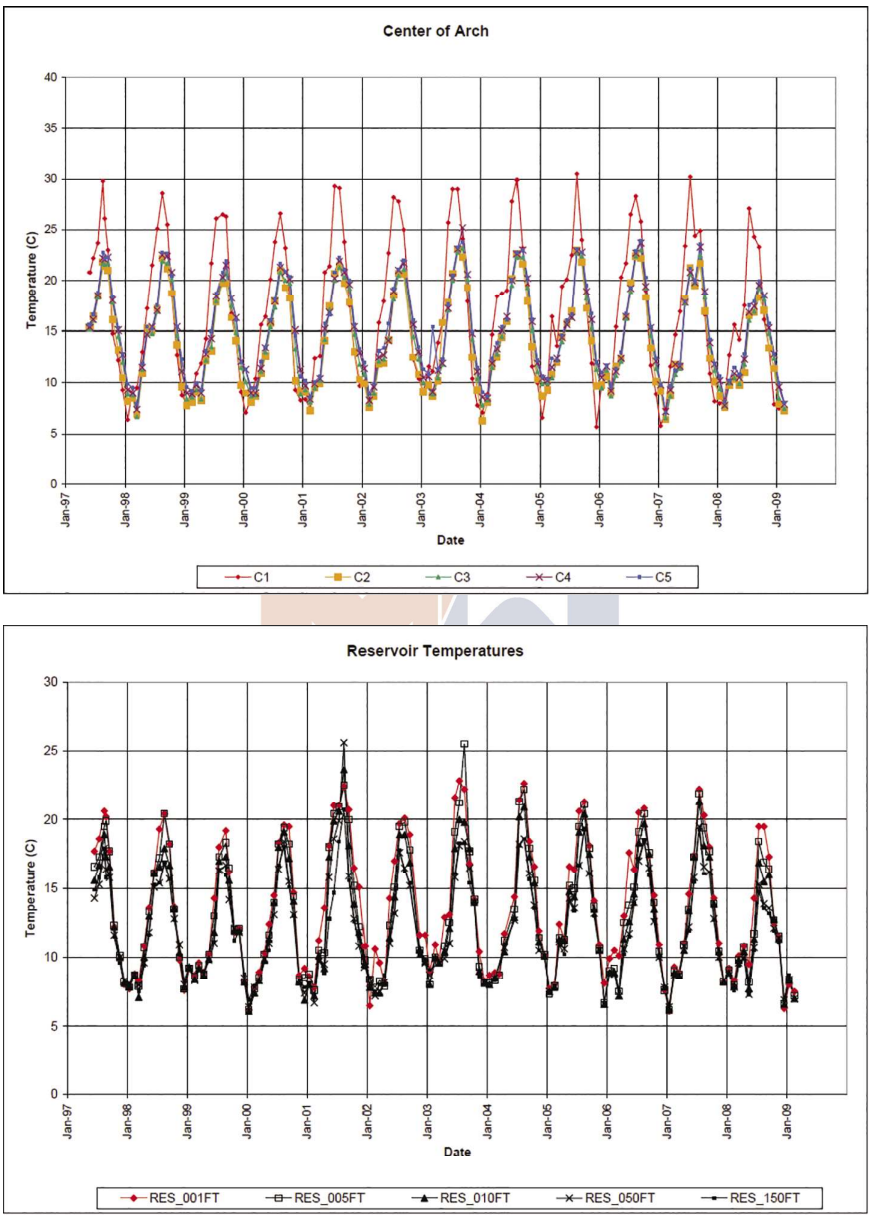

Fig. 13-8. Concrete and reservoir temperatures versus time
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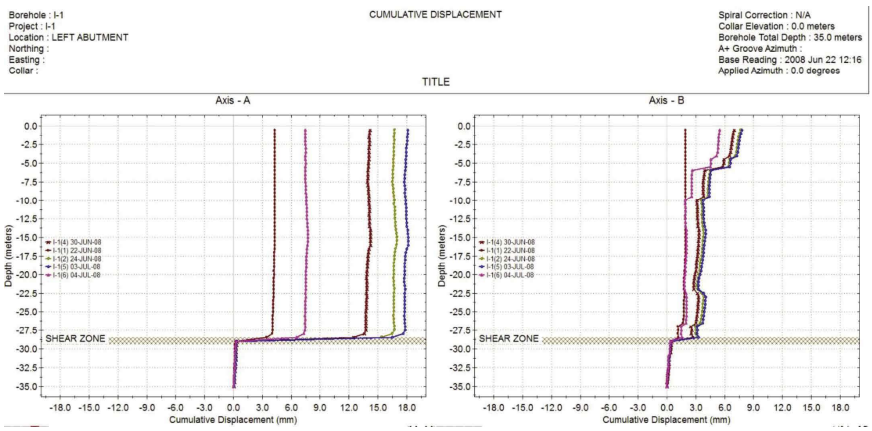

Fig. 13-9. Inclinometer profile showing horizontal displacement versus depth
Source: Courtesy of PSE.

13.2.6 Inclinometer Profile

Fig. 13-9 illustrates a typical inclinometer profile. This example is from a dam abutment in which excavation of the portal of a diversion tunnel triggered movement along an existing shear zone delimiting a large-size block. The movement was eventually stabilized by adding a massive concrete abutment to the tunnel portal. The profile shows the cumulative horizontal displacement in two perpendicular directions (A and B). The inclinometer casing is usually anchored at a depth where the ground is known to be stable, and displacements are calculated from the stable casing bottom.

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CHAPTER 14

HISTORY OF INSTRUMENTATION AND MONITORING

> A people without the knowledge of their past history, origin and culture is like a tree without roots.
>
> Marcus Garvey

The oldest surviving evidence of dam building is in Egypt. Sadd El Khafara was built across the Wadi El Garawi approximately 5,000 years ago. Constructed of rubble masonry in the form of an embankment dam, it was 37 ft (11.2 m) high, 265 ft (80.7 m) wide at the base, and 348 ft (106 m) long at the crest. The upstream face was lined with tightly fitting blocks of limestone. Apparently, it had an inadequate spillway, and it probably failed by overtopping.

Surviving evidence of achievements in the next two millennia B.C.E. is scarce. North of Baghdad, a dam known as Nimrod's Dam was built, and it diverted the entire flow of the Tigris into another channel of the river. Other projects followed not only along the Tigris but also along the Euphrates, Khost, and other rivers near Babylon. Alexander the Great is known to have commissioned a large canal joining the Tigris and Euphrates, probably in the fourth century B.C.E.

Romans are credited with the first use of curved dams in the south of France (Glanum) and in Tunisia (Kasserine) near where the Allies defeated Rommel in an epic World War II battle. These were curved gravity dams, and their shape is attributed to lengthening the crest to accommodate uncontrolled overflow. Two splendid examples of other Roman dams were constructed sometime in the first 500 years C.E. near Merida in Spain. Cornalvo (Fig. 14-1) and Proserpina Dams survive, and their outlet works still deliver water for beneficial use.

The Mongols invaded and defeated the Persians, and early in the second millennium, they built dams in Iran. Two of them, Kurit and Kebar (circa

Fig. 14-1. Roman masonry at Cornalvo Dam after 1,000 years of service
Source: W. D. Schram (2018), reproduced with permission.

Fig. 14-2. Kebar Dam
Source: N. J. Schnitter (1994).

1280 C.E.), are worthy of mention because they were among the first true arch dams known to have been constructed; the Kebar Dam is shown in Fig. 14-2. The arch had nearly vertical upstream and downstream faces, with a constant radius of 125 ft (38.1 m) and it was 85 ft (25.9 m) high, 15 ft (4.57 m) wide at the crest, and 180 ft (54.8) long. They were keyed into trenches cut into rock on both abutments and built with a cemented (lime and ash mortar) core wall of rubble masonry faced both upstream and downstream with cemented dimension stones. Both had several levels of

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outlets presumed to be temporary diversion conduits during construction. Kurit has been raised several times, and a road across the crest still provides for vehicular traffic (de Rubertis 2011, Yang et al. 1999, Schnitter 1994, Smith 1971).

Early American dam building concentrated on providing a water supply and water power for mills, and timber was a common building material. Masonry construction was favored, and many of the structures built in the 1800s, such as Croton Dam, still are in use in New England. Embankment dams then began to make an appearance. The oldest surviving dam in the United States is Mill Pond Dam near Newington, Connecticut. It was constructed in 1676. The first recorded use of concrete was in 1890 at Lower Crystal Springs Dam, when interlocking concrete blocks were employed. The first concrete arch dam, Upper Otay, was built in 1888. Construction of Cheesman Dam on the South Fork of the Platte began in 1890, a beautiful masonry dam that still supplies water to the Denver metropolitan area. Hydraulic fill dams appeared to offer advantages in ease of construction and resistance to seepage in the fine-grained glacial soils of the East. Homogeneous embankments gradually gave way to zoned earthfill and rockfill dams (de Rubertis 2011).

Physical and visual inspections were part of these early projects from construction through initial filling and then through years of project operation. Formal instrumentation and monitoring systems came later. For example, surveyed elevation readings were known to be taken on the crest of Grosbois masonry dam in France in 1853. The dam was built between 1830 and 1838, and the readings were to measure the displacement of the dam's crest. In later years, surveyed elevation measurements became common practice for monitoring masonry dams.

ME SOLUTIONS 40 - 50

In the late nineteenth century, open piezometers were used in India to study seepage flows under irrigation dams built on alluvium. In 1907, this type of instrument was employed by English engineers to determine the free surface of the nappe over a homogeneous earth dam.

In the United States, observation wells, water-level indicators, and hydrostatic-level indicators were some of the earliest measuring devices to record the water level or the pore water pressure on U. S. Bureau of Reclamation embankment dams. In 1911, observation wells, sometimes called saturation pipes, were installed at the Belle Fourche Dam in South Dakota. The wells were installed for the purpose of delineating the phreatic line and to help determine flow patterns through earth dams, abutments, and foundations. In 1935, water-level indicators were developed and installed at Hyrum Dam in Utah (Fig. 14-3) and Agency Valley Dam in Oregon. The water-level indicator apparatus was not very accurate because all the water in the system could not be displaced simultaneously. Moreover, the column of water in the system did not represent true pressure conditions because air

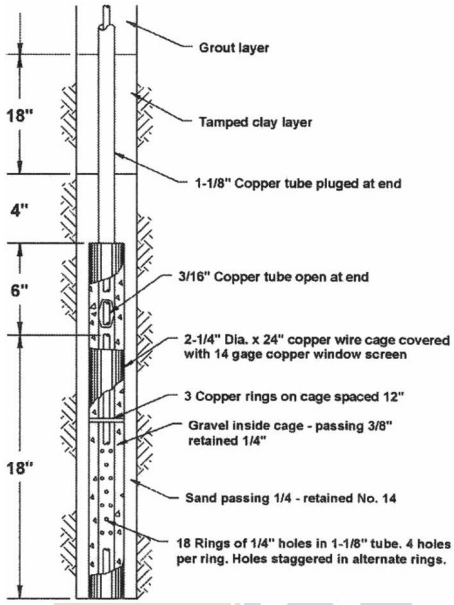

Fig. 14-3. Water-level indicator used at Hyrum Dam (Utah) in 1935
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

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was compressed in the soil voids and escaped into the larger tube. Extreme care was necessary during installation to prevent damage to the pressure point. Construction pore pressures could not be recorded because the water-level indicators could not be installed until after dam construction was completed. The water-level indicators were eventually abandoned. Concurrent with the development of the water-level indicators, hydrostatic pressure indicators were designed and installed in several dams. The 1938 and 1939 installation of hydrostatic pressure indicators at the Caballo Dam in New Mexico was typical for this period (Bartholomew 1987).

In 1922, the Engineering Foundation, an affiliation of the major engineering societies of the time, in cooperation with the Southern California Edison Company, constructed an extensively instrumented test arch dam to better understand the performance of these structures (Redinger 1986). The dam was constructed in a narrow canyon on a tributary of the San Joaquin River in the Sierra Mountains of California. Instrumentation included 140 "electric telemeters" buried in the concrete of the dam. These instruments consisted of a stack of carbon disks whose resistance changed with stress applied axially to the stack. Stress changes in the dam at different reservoir levels and temperature changes were measured by the telemeters and greatly added to the understanding of the behavior of arch dams.

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In the United States, Roy Carlson, shown in Fig. 14-4, designed an apparatus that would measure pressure in concrete and earth dams by means of a sensor consisting of a stretched wire.

Carlson measured the variations in electrical resistance of the wire relative to the movement of two anchoring points using a Wheatstone bridge. This unbonded resistance-wire strain gauge was first produced commercially in 1932 and was later supplemented by piezometers, concrete stress cells, earth pressure cells, and joint meters.

Also in the United States in the 1930s, the U.S. Bureau of Reclamation began the use of controlled measurements to correlate the performance of completed dam structures with design assumptions and construction practices. This effort led to the design of water-level indicators and hydraulic twin-tube piezometers to measure pore pressure, as shown in Fig. 14-5 (Walker 1948, Daehn 1962).

An important milestone in the field was marked in 1931 in France when André Coyne (Fig. 14-6), founder of the French consulting engineering firm Coyne et Bellier and designer of more than 55 arch dams, obtained a patent for the vibrating-wire sensor (Fig. 14-7), then called an acoustic indicator (Coyne 1938).

With the major focus on the construction of new dams from the 1930s through the 1970s came the extensive utilization of dam instrumentation and monitoring programs. Many of the dam instrumentation and monitoring

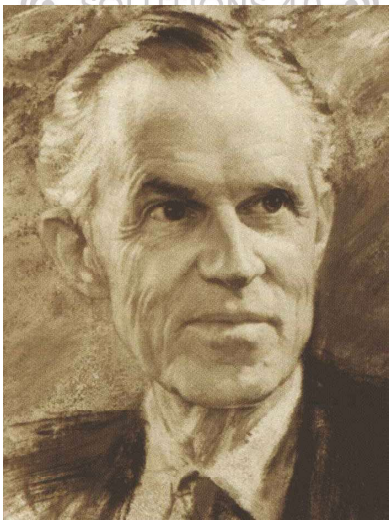

Fig. 14-4. Dr. Roy Carlson (1900–1990)
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

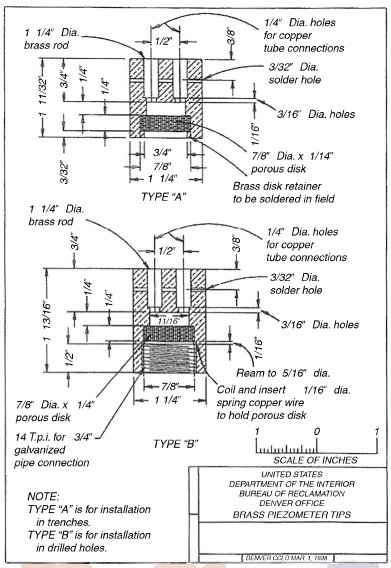

Fig. 14-5. Early twin-tube hydraulic piezometer tips developed by USBR in 1940
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

Fig. 14-6. André Coyne (1891–1960)
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

programs that were initiated during that era are still in existence today, although there have been many developments and improvements through the years.

Dam instrumentation and monitoring programs have undergone great changes in recent years because of progress in microcomputer technology. Developments in this field permit the acquisition and processing of data

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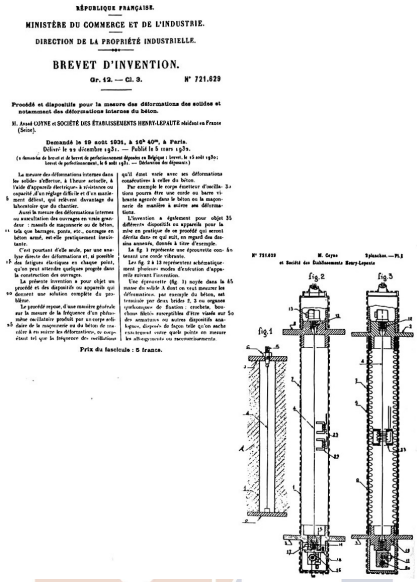

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Procédé et dispositifs pour la mesure des déformations des solides et notamment des déformations internes du béton.

M. André COYNE et SOCIÉTÉ DES ÉTABLISSEMENTS HENRY-LEPAUTE résidant en France (Seine).

Demandé le 19 août 1931, à 10h 40m, à Paris.
Délivré le 22 décembre 1931. — Publié le 5 mars 1932.

(2 demandes de brevet et de brevet de perfectionnement déposés en Belgique : brevet, le 18 août 1930; brevet de perfectionnement, le 8 août 1931. — Détention du dépositaire.)

Prix du fascicule : 5 francs.

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Fig. 14-7. André Coyne's patent for the vibrating-wire sensor registered in French as a témoin sonore (acoustic indicator)
Source: ASCE Task Committee on Instrumentation and Monitoring Dam Performance (2000).

that have changed the very nature of the results yielded by instrumentation. Instrumentation on dams in very remote areas of the world can be accessed via satellite, transmitting the data (GNSS) to offices on other continents. The data can instantly be reduced by a personal computer and plotted in a format that immediately allows engineers to identify deviations from the expected performance of the structure.

It is interesting to note that much of the design and manufacture of dam instruments has been conducted by practicing engineers and manufacturing companies specializing in the field. In some cases, industrial sensors have been adapted for use, yet a great many of the sensor designs used on dams today are unique to the field.

Equipment used in dam monitoring systems has to satisfy two challenging requirements. First, it must have a life span comparable to the design life of the dam. Second, it must be extremely stable with virtually no zero drift.

Scientific organizations, particularly the International Committee on Large Dams (ICOLD), have always given the utmost importance to collecting dam-related data from actual experience. The large number of reports on the subject, as well as the special ICOLD publications on dam instrumentation

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(ICOLD 1988, 1989, 1992, 1994, 1999, 2000, 2005, 2009) clearly show the interest generated by changing instrumentation components and measuring techniques.

Agencies with global expertise and jurisdiction have also emphasized instrumentation and measurements with the publication of reference manuals and guidelines.

Although there have been many advances in data acquisition, processing, instrumentation components, and measuring techniques, neither the factors affecting dam performance nor the performance indicators have changed.

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APPENDIX A
REFERENCES

American Congress on Surveying and Mapping (ACSM). (1989). "Definitions of surveying and associated terms." ASCE/ACSM, Gaithersburg, MD.

ASCE Task Committee on Instrumentation and Monitoring Dam Performance. (2000). *Guidelines for Instrumentation and measurements for monitoring dam performance*, ASCE, Reston, VA.

ASTM International. (2016). "Standard test method for low strain impact integrity testing of deep foundations." *ASTM D5882-16*, Gaithersburg, MD.

Balouin, T., Lahoz, A., Bolvin, C., and Flauw, Y. (2012). "Risk assessment required in the framework of new French regulation on dams. Methodology developed by INERIS." *Proc., 24th Congress and Int. Symp. on Dams for a Changing World*, International Commission on Large Dams (ICOLD), Paris.

Barker, M. (1998). *Australian risk approach for assessment of dams*, Australian National Committee on Large Dams, Hobart, Tasmania.

Bartholomew, C. L., and Haveland, M. L. (1987). *Concrete dam instrumentation manual: A water resources technical manual*, U.S. Department of the Interior, Bureau of Reclamation, Washington, DC.

Bartholomew, C. L., Murray, B. C., and Coins, D. L., eds. (1987). *Embankment dam instrumentation manual*, U.S. Department of Interior, Bureau of Reclamation, Washington, DC, 17.

Bartsch, M. (2004). *FMECA of the Ajaure Dam—A methodology study*, Thomas Telford Publishing, London, 187–196.

Bennett, V., Abdoun, T., Danisch, L., and Shantz, T. (2007). "Unstable slope monitoring with a wireless shape acceleration array system." *Proc., 7th Int. Symp. on Field Measurements in Geometrics*, September 24–27, Boston, FMGM.

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Bordes, J. L., and Debreuille, P. J. (1985). "Some facts about long-term reliability of vibrating-wire instruments." *Transport. Res. Rec. No. 1004*: 20–26.

Central Water Commission, Dam Safety Organisation. (1986). "Report on dam safety procedures." Ministry of Water Resources, Government of India.

Choquet, P., Juneau, F., Debreuille, P. J., and Bessette, J. (1999). "Reliability, long-term stability and gate performance of vibrating wire sensors with reference to case histories." *Proc. 5th Int. Symp. Field Measurements in Geomechanics-FMGM99*, C. F. Leung, S. A. Tan, and K. K. Phoon eds., Singapore, December 1–3.

Choquet, P., Quirion, M., and Juneau, F. (2000). "Advances in fabry-perot fiber optic sensors and instruments for geotechnical monitoring." *Geotechnic. News*, March.

Chrzanowski, A., Avella, S., Chen, Y. Q., and Secord, J. M. (1992). "Existing resources, standards, and procedures for precise monitoring and analysis of structural deformations, Vol. 1." *TEC0025*, U.S. Army Corps of Engineers, Washington, DC.

Contreras, I. A., Grosser, A. T., and VerStrate, R. H. (2012). "Update of the fully grouted method for piezometer installation." J. Dunnicliff, ed. *Geotechnic. News* 30(2), 20–25.

Coyne, A. (1938). "Some acoustic monitoring results on concrete, reinforced concrete and metallic works." *Annales de I'Institut Technique des Batiments et des Travaux publics*, March (in French).

Daehn, W. W. (1962). "Development and installation of piezometers for the measurements of pore-fluid pressures in earth dams." *Proc., Symp. Soils and Foundation Engineering*, American Society for Testing and Materials, West Conshohocken, PA.

de Rubertis, K. (2011). "ASDSO. Dam engineering through the ages." *J. Dam Safety* 9(2), 31–39.

Dienum, P. J. (1987). "The use of tiltmeters for measuring arch dam displacements." *Water Power Dam Const.*, June.

Duffy, M. A., and Whitaker, C. (1998). "Design of a robotic monitoring system for the Eastside Reservoir in California." *Proc.*, American Congress on Surveying and mapping, Vol. 1, Baltimore, 34–44.

Dunnicliff, J. (1988). *Geotechnical instrumentation for monitoring field performance*, Wiley, New York.

Dunnicliff, J. (1993). *Geotechnical instrumentation for monitoring field performance*, Wiley, New York.

FEMA. (2004). "Federal guidelines for dam safety, selecting and accommodating inflow design flood for dams." Interagency Committee on Dam Safety, Washington, DC.

FEMA. (2013). *Federal guidelines for inundation mapping of flood risks associated with dam incidents and failures, P-946*, Washington, DC.

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FEMA. (2015). *Federal guidelines for dam safety risk management, P-1032*, Washington, DC.

FERC (U.S. Federal Energy Regulatory Commission). (2005). "Engineering guidelines for the evaluation of hydropower projects." In *Dam safety performance monitoring program*, Rev. 1, U.S. Federal Energy Regulatory Commission, Washington, DC.

FERC. (2007). "Dam safety performance monitoring program." In *Engineering guidelines for the evaluation of hydropower projects*, Rev. 1, U.S. Federal Energy Regulatory Commission, Washington, DC.

FERC. (2008a). "Dam safety surveillance and monitoring plan outline." In *Engineering guidelines for the evaluation of hydropower projects*, Rev. 2, Jan. 15, U.S. Federal Energy Regulatory Commission, Washington, DC.

FERC. (2008b). "Dam safety performance monitoring program." In *Engineering guidelines for the evaluation of hydropower projects*, Rev. 2, Jan. 15, U.S. Federal Energy Regulatory Commission, Washington, DC.

Holt, J., Poiroux, G., Lindyberg, R., and Cesare, M. (2013). "Detection without danger." *Civil Eng.* 83(1), 68–77.

Hydrometrics, Inc. (2011). *Guidelines for conducting a simplified failure mode analysis for Montana dams*, Montana Department of Natural Resources and Conservation, Helena, MT.

ICOLD (International Committee on Large Dams). (1972). "Dam monitoring—General consideration." *Bull. 60* (1988), revised and edited version of Bulletins 21 (1969) and 23, Paris.

ICOLD. (1988). "Dam monitoring general considerations." *Bull. 60*, Paris.

ICOLD. (1989). "Monitoring of dams and their foundations—State of the art." *Bull. 68*, Paris.

ICOLD. (1992). "Dam monitoring improvements." *Bull. 87*, Paris.

ICOLD. (1994). "Ageing of dams and appurtenant works." *Bull. 93*, Paris.

ICOLD. (1999). "Seismic observation of dams—Guidelines and case studies." *Bull. 113*, Paris.

ICOLD. (2000). "Automated dam monitoring systems—Guidelines and case histories." *Bull. 118*, Paris.

ICOLD. (2005). "Dam foundations, geologic considerations, investigation methods, treatment, monitoring." *Bull. 129*, Paris.

ICOLD. (2009). "General approach to dam surveillance." *Bull. 138*, Paris.

Inaudi, D., and Branko, G. (2006). "Reliability and field testing of distributed strain and temperature sensors." *Proc., SPIE Smart Structures and Materials Conf.*, International Society for Optics and Photonics (SPIE), Bellingham, WA.

ISRM (International Society for Rock Mechanics and Rock Engineering). (1981). "Suggested methods for pressure monitoring using hydraulic cells." In *The ISRM suggested methods for rock characterization, testing and monitoring*, Pergamon Press, Oxford, UK, 201–211.

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Jacobs UK, Ltd. (2007). *Engineering guide to early detection of internal erosion, Jacobs Ref B2220300*, Department for Environment, Food and Rural Affairs, London.

Jansen, R. B. (1988). *Advanced dam engineering for design, construction and rehabilitation*, Van Nostrand Reinhold, New York.

Kane, W. F. (1998). "Embankment monitoring time domain reflectometry." *Proc., 5th Int. Conf. on Tailings and Mine Waste*, Colorado State University, Fort Collins, CO, 223–230.

Mazzanti P., Perissin D., and Rocca, A. (2015). "Structural health monitoring of dams by advanced satellite Sar interferometry: Investigation of past processes and future monitoring perspectives." *Proc., 7th Int. Conf. on Structural Health Monitoring of Intelligent Infrastructure*, Nhazca S.r.I., Rome.

McRae, J.-B., and Simmonds, T. (1991). "Long-term stability of vibrating-wire instruments: one manufacturer's perspective." *Proc., 3rd Int. Symp. on Field Measurements in Geomechanics*, Vol. 1, Rotterdam, Balkema, 283–293.

Mikkelsen, P. E., and Green, E. G. (2003). "Piezometers in fully grouted boreholes." *Proc., 6th Int. Symp. on Field Measurements in Geomechanics*, Oslo, Norway, September, 545–553.

Peltzer, G., Hudnut, K. W., and Feigl, K. L. (1994). "Analysis of coseismic surface displacement gradients using radar interferometry: New insights into the Landers earthquake." *J. Geophys. Res*. 99(19), 971–981.

Professional Association of Civil Engineers, Spanish National Committee on Large Dams. (2006). *Risk analysis applied to management of dam safety*, Madrid.

Redinger, D. H. (1986). *The story of Big Creek*, Trans-Anglo Books, Glendale, CA.

Regan, P. (2009). "An examination of dam failures vs. the age of dams." FERC, Washington, DC.

Schnitter, N. J. (1994). *A history of dams*, Balkema, Leiden, The Netherlands.

Schram, W. D. (2018). "Roman aqueducts." http://www.romanaqueducts.info/.

Sellers, J. B., and Taylor, R. (2008). "MEMS Basics." *Geotechnic. News*, March, 32–33.

Shakal, A. F., and Huang, M. J. (1996). "Strong motion instrumentation and recent data collected at dams." *Western Regional Technical Seminar*, Association of State Dam Safety Officials, Lexington, KY, 111–128.

Sherard, J. L., Woodward, R. J., and Gizienski, S. F. (1983). *Earth and earth–rock dams: Engineering problems of design and construction*, Wiley, New York.

Smith, N. (1971). *A history of dams*, Citadel Press, Secaucus, NJ.

Solinst. (1997). *Waterloo system*, Georgetown, Ontario, Canada.

Teixeira da Cruz, P., Materon, B., and Freitas, M. D., Jr. (2010). *Concrete face rockfill dams*, CRC Press, Boca Raton, FL.

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