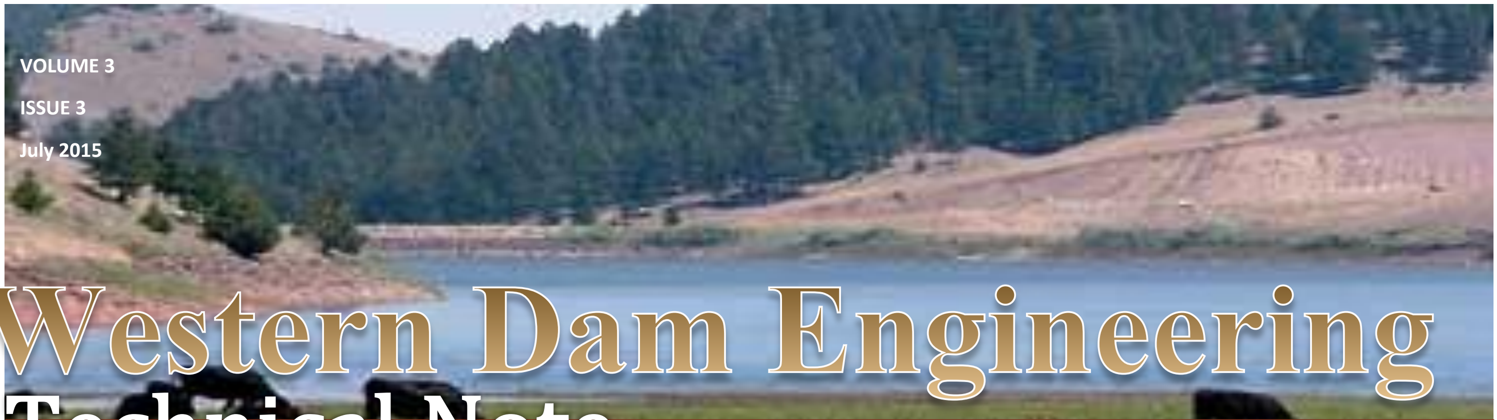

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A QUARTERLY PUBLICATION FOR WESTERN DAM ENGINEERS

In this issue of the ***Western Dam Engineering Technical Note***, we present articles on **spillway erosion**, **instrumentation**, and design considerations for **outlet controls**. This newsletter is meant as an educational resource for civil engineers who practice primarily in rural areas of the western United States. This publication focuses on technical articles specific to small and medium dams. It provides general information. The reader is encouraged to use the references cited and engage other technical experts as appropriate.

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This Technical Note was compiled, written, and edited by AECOM in Denver, Colorado.

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Article Contributors:

AECOM: Karen Aguillard, PE; Jessie Drayton, PE; Guirec le Bouteiller; Jenn Williams, PE; Jason Boomer, PE; Frank Lan, Ph.D., PE; Rick Dulin, PE

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In this Issue

Spillways: Spilling the Right Way

Introduction

Unlined earthen spillways are common features of small earthen dams. They can be principle, emergency or auxiliary spillways made to pass rare flood flows around the embankment. While many spillways may never pass a significant flow or volume of water, one large event could result in significant erosional consequences.

Historically there was little engineering design of the spillways for small dams. The spillway was often situated in the dam borrow area or incorporated as part of an auxiliary dam. Today there has been much more research conducted on spillway performance and there are many tools available for spillway design and evaluation. Some of the most popular tools are discussed in this article. Spillways should be designed to experience flow from a known recurrence interval storm and earthen spillways can expect to suffer some erosion damage during events that cause them to flow. However, where erosion enlarges enough to destabilize the structure or cause an uncontrolled release, it could lead to a dam failure with downstream consequences.

Inadequate spillway capacity is one of the most common safety deficiencies in small dams and can occur due to original design deficiencies or changes in conditions. The necessary capacity may have changed due to a change in the watershed, downstream channel, design flood, hazard class, or spillway condition. Earthen spillways are common at small dam sites and present additional deficiencies regarding erosion and stability as compared to concrete lined spillways. Spillway erosion occurs when a precipitation event increases the reservoir elevation above the spillway crest, resulting in water flowing down the spillway channel. Due to the force of the flowing water, erosion of the vegetation on the surface of the spillway will begin to occur. After the water has removed the vegetation, erosion of the soil will enlarge and deepen the eroded area. As the flow area increases in size and depth, the flow becomes more turbulent, increasing the rate of erosion. With continued flow, headcutting begins as the eroded area continues to grow and progress upstream. Depending on the configuration of the dam, the headcutting could proceed to the spillway crest, eventually reaching the reservoir and creating an uncontrolled release as the embankment erodes away or concrete structures destabilize, allowing them to slide or overturn. A typical event tree of a spillway erosion or headcutting failure mode is described below

Spillway Erosion Failure Mode

- Reservoir level reaches spillway crest and begins to flow
  - Vegetation (if present) is removed or eroded
    - Concentrated flow erosion begins (downcutting forms headcut) and worsens
      - Headcut advancement begins (deepens and advances towards spillway crest/control section)
        - Intervention is attempted and unsuccessful (more likely to be successful if attempted early)
          - Headcut advances through crest of spillway or headcut undermines control structure/section and flow control is lost
            - Headcut advances into reservoir pool and breaching occurs

The erodibility of a spillway is a function of the geology, channel geometry, and the expected volume, velocity and duration of the flood flow. Fine granular materials such as silts and sands are more likely to erode as compared to cohesive clayey materials. Soils with cohesion have a plasticity or inherent ‘stickiness’ that holds the particles together. The performance of rock is more complex to predict due to weathering, fracturing, joints, process of formation, and strength. In general, vegetation improves the performance of spillways to a certain point, if it is uniform grass or ground cover. Discontinuities and obstacles such as trees, shrubs, groins, roads, paths, ditches, and changes in slope will concentrate flow and create areas prone to turbulent flow, leading to erosion.

Areas that experience hydraulic jumps are particularly susceptible to erosion due to pressures created by the energy change. This occurs most commonly at the end of the spillway where the flow meets the downstream river channel, stilling basin or areas of change in slope of the spillway. Narrow steep channels will increase the depth and speed of flow, increasing the likelihood of turbulent flow and erosion. Movement of larger particles requires higher velocities and or steeper slopes. There are several methods available to assess the risks of spillway erosion. This article will present five of the most common. Any spillway analysis for design or modification should be approached in more detail than discussed below. The references at the end of this article provide more information for each method.

1. USACE Repair, Evaluation, Maintenance and Rehabilitation (REMR)
2. Cohesionless Soil
3. Annandale
4. Water Resources Site Analysis Program (SITES)
5. Computation Fluid Dynamics (CFD)

USACE REMR

The method developed by the USACE REMR is a qualitative classification of erodibility based on spillway characteristics. The method predicts whether erosion is likely to occur but does not provide information regarding the extent or severity of erosion. The method involves assessing the erosion 'risk' as a function of spillway channel slope, flow velocity and the effect of anomalies in the spillway geometry. Based on these characteristics, the spillway is assigned an "A" rating as outlined below.

Soil or rock classification:
AAAA = Erosion-resistant rock
AAA = Moderately erosion-resistant rock
AA = Moderately erodible material
A = Erodible soil (nonvegetated)

Table 1: Erosion Risk Class

Spillway Characteristic	Erosion Risk Class			
	AAAA	**AAA**	**AA**	**A**
Slope (percent)	30-45	15-30	4-15	<4
Flow velocity (ft/s)	10-15	7-10	4-7	<4
Anomaly Effect	Minor	Moderate	Major	Severe

The erosion 'risk' is compared to the erosion 'potential,' which is based on geologic material behavior factors as shown in Table 2.

Table 2: Erosion Potential Class

Spillway Characteristic	Erosion Potential Class			
	AAAA	**AAA**	**AA**	**A**
Lithology				
Sandstone				
Shale & Limestone				
Limestone				
Granular Soil (Low PI)				
Cohesive Soil (High PI)				
Intrusive Igneous				
Extrusive Igneous				
Massive Metamorphic				
Foliated Metamorphic				
Substance				
Density (pcf)	>140	140-125	125-116	<116
Uniaxial Strength (psi)	>6000	6000-2000	2000-150	<150
Genesis				

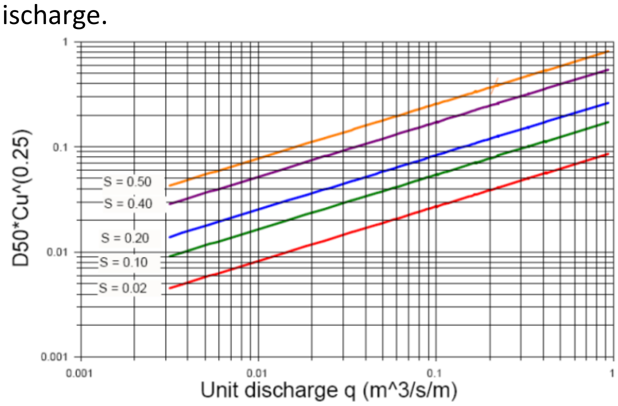
Table 2: Erosion Potential Class

Spillway Characteristic	Erosion Potential Class			
	AAAA	AAA	AA	A
Vertical Consistency (ft)	>6	6-2	2-0.25	<0.25
Lateral Consistency (#)	1	2	>2	>2
Tectonics: Unit Orientation Related to Flow Direction	Flat	Dip Toward	Dip Parallel	Dip Away
Rock Mass				
Fracture Spacing (ft)	>3	3-1	1-0.5	<0.5
Particle Diameter (ft)	3-5	1-3	1-0.5	<0.5
Fracture Size/Opening (in)	<1/8	1/8-1/2	>1/2	Open/clean
Fracture Sets (No.)	2	2-3	>3	shattered

For each factor, an A rating is assigned and the number of A's (between 1 and 4) is averaged for the erosion risk and the erosion potential. If the erosion risk average is higher than the erosion potential average, it is estimated that the spillway is likely to erode. If there are multiple distinctly identifiable geologic units within the spillway this process should be repeated for each unit to identify the most critical. Refer to USACE Technical Report REMR-GT-3 Supplement (1998) for a more detailed description of the method. It is important to note this analysis method is empirical and engineering judgment is required to make any decisions regarding the safety of a spillway.

Cohesionless Soil

If the spillway includes cohesionless materials with a D_{50} larger than 4 inches, the curves developed by Frizell et al. (1998) can be used to estimate the flow at which erosion could occur. The data is based on the slope (S) of the spillway, the D_{50} grain size, the coefficient of uniformity ($C_u = D_{60}/D_{10}$) and the unit discharge.



Unit Conversion 1 m^3 = 35.3 ft^3

Figure 1 – Erosion Potential of Cohesionless Soil

Source: *Best Practices Dam and Levee Safety Risk Analysis*, Chapter 15

Plotting on the line represents a 20 percent chance of erosion beginning and below the line means an increase in probability. These curves do not represent the probability of breach, only of erosion, and the data are based on testing with uniformly sized angular riprap in ideal conditions.

Annandale

The analysis method developed by Annandale (1995 and 2006) quantifies two properties: the erodibility index (K_h) and stream power (P). The erodibility index K_h represents the susceptibility of a material to erode and is computed as follows:

$K_h = M_s K_b K_d J_s$

M_s = Material strength number, relates to unconfined compressive strength

K_b = Block or particle size, based on RQD/J_n where J_n is the joint set number

K_d = Inter-particle bond shear strength, taken as J_r/J_a (joint roughness/joint alteration)

J_s = Relative shape and orientation of blocks, ease with which water can penetrate discontinuities and dislodge blocks (is equal to 1 for soils)

Values associated with the J variables can be obtained from tables developed by Annandale and are available in the USBR and USACE *Best Practices Dam and Levee Safety Risk Analysis* (Chapter 15, tables 15-1 through 15-4). The stream power P represents the rate of energy dissipation per unit of surface area and is computed as follows:

$P = \gamma U h S$

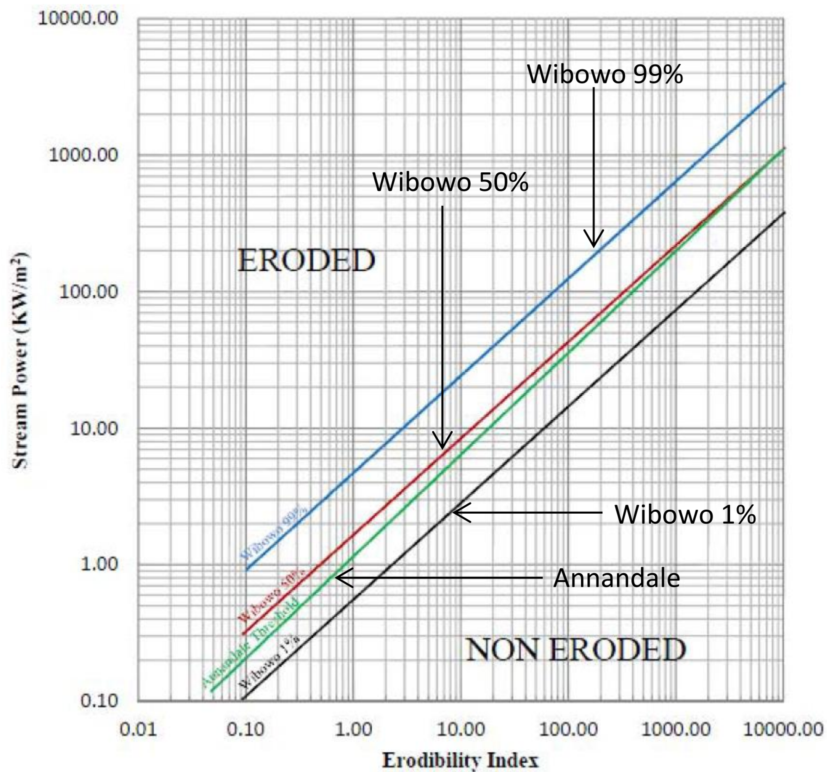
γ = Unit weight of water

U = Flow velocity

h = Flow depth

S = Slope

Annandale developed an erosion threshold curve based on approximately 150 field observations from spillways and plunge pools. The curve is shown on Figure 2, along with confidence curves developed by Wibowo et al. (2005). As with any simplified analysis method, engineering judgment is required in using the curves and multiple conditions and assumptions should be considered. The data are particularly sensitive to the K_b value. However, the curves are helpful in providing a range of likelihood for erosion and progression of headcutting.



Unit Conversion 1 kW/m^2 = 0.093 kW/ft^2

Figure 2 – Annandale Likelihood of Erosion

Source: *Best Practices Dam and Levee Safety Risk Analysis*, Chapter 15

SITES

The SITES spillway erosion analysis software was developed by the National Resource Conservation Service (NRCS), the Agricultural Research Service (ARS) and Kansas State University. It is a one-dimensional hydraulic simulation of flow through the spillway channel. It was developed based on lab testing and field data of headcutting in soil- and grass-lined spillways, but has also been applied to rock channels. The analysis estimates whether headcutting will occur and whether flow duration is long enough to deepen the headcut and advance upstream. The model assumes failure when the erosion reaches the spillway sill. The model represents a three-phase analysis as follows:

- Phase 1: Surface Erosion
  - Flow persists long enough to initiate erosion and the flow concentrates at a location and removes vegetation.
  - The model can account for surface discontinuities.
  - Erosion is estimated based on effective stresses and bond strength of underlying soil.
  - If no vegetation exists, Phase 1 is negligible.
- Phase 2: Concentrated Flow Erosion
  - Flow enlarges and deepens erosion

- ○ Assumes that flow continues to be somewhat uniform.
- ○ Erosion is estimated based on effective and critical stresses, and bond strength of underlying soils.
- ○ Critical stress is a function of clay and density properties for soils and particle size for rock.

- Phase 3: Headcut Advance and Deepening
  - ○ Flow is turbulent.
  - ○ Headcutting is considered in two parts: downward movement and headward movement
  - ○ Rate of headcut migration is a function of the material strength (erodibility index K_h) and hydraulic power being dissipated.
  - ○ Numerous layers of material require determination of a representative value using a depth-weighted log averaging scheme.

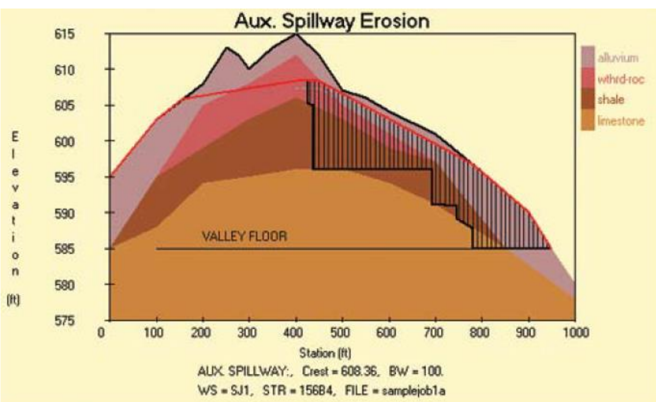


Figure 3 – SITES Output of Predicted Erosion
Source: *SITES 2005 Water Resources Site Analysis Computer Program User Guide*

The SITES software is available for public use. NRCS, ARS and Kansas State University have developed WinDAM B which can incorporate data from SITES into analysis of full breach development.

Hydraulic Models

There are a number of hydraulic models that could be used to help assess the spillway erosion potential, from the popular one-dimensional model HEC-RAS (USACE HEC), to the two-dimensional models such as MIKE-21 (Delft Hydraulics Institute) and RiverFlow2D (Hydronia), to the more complex three dimensional Computational Fluid Dynamics (CFD) models. CFD is becoming a popular and powerful tool for evaluating spillway erosion potential in recent years because of the increasing accuracy of CFD against prototype measurements, ease of use, and dramatic increase in computer processing speed. CFD programs, such as FLOW-3D, FLUENT, CFX, OpenFOAM, STAR-CD, and others, simulate hydrodynamic characteristics of flow, such as velocity, pressure, shear stress, etc., over the spillway and further downstream in three-dimensions and thus provide more detailed information that can be used to assist the evaluation of spillway erosion potential. Some CFD programs, such as FLOW-3D, also have sediment scour modules that could be used to evaluate sediment erosion.

Typical outputs from CFD model include flow velocity, dynamic pressure, bed shear stress, shear velocity, turbulence energy, etc. The stream power used in the Annandale Method could then be easily calculated based on the results from the CFD, using equations such as the following (Annandale, 2010):

$$P = 7.853\,\rho\left(\sqrt{\frac{\tau_o}{\rho}}\right)^3$$

where P = stream power in w/m^2
ρ = fluid macro density in kg/m^3, 1000 kg/m^3 for clear water
τ_o = bed shear stress in N/m^2.

CFD is widely used in spillway design to help identify alternatives that could minimize adverse hydraulic conditions leading to potential erosion or other undesirable hydraulic conditions. However, the current cost of the analysis may be prohibitive for small projects.

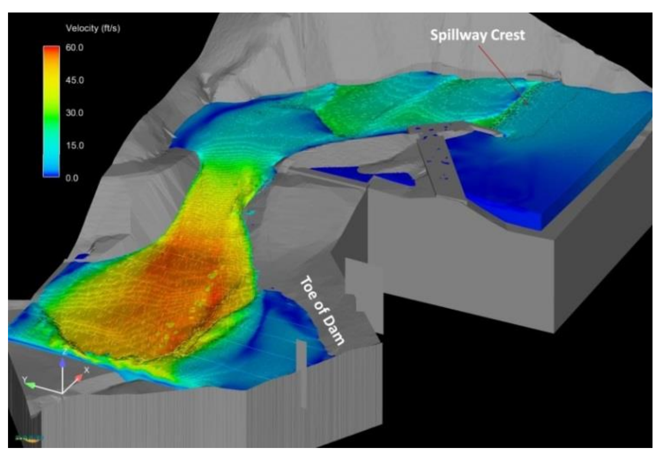


Figure 4 – CFD Model Estimated Velocities

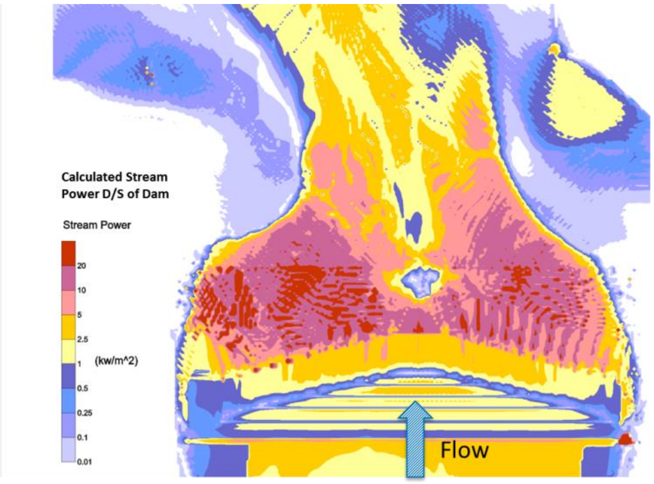


Figure 5 – CFD Model Estimated Stream Power

Case Study: Sugar Creek L-44

Sugar Creek L-44 is a 550-foot long embankment dam constructed in Southwest Oklahoma in 1971. The dam is 64 feet high with an auxiliary spillway width of 40 feet. In August 2007, the area around the dam experienced 8 to 10 inches of rainfall in 3 to 4 hours, exceeding the 100-year, 6, 12, and 24-hour event as well as the 500-year, 3-hour event. The flow through the spillway during the rain event was estimated by SITE analysis afterwards to be 740 cubic feet per second. The flow resulted in erosion of the inside training dike, the spillway down to the underlying bedrock, and the downstream toe of the dam near the spillway outlet (causing embankment instability). The event also washed out a county road 300 feet downstream, inundated a house, and caused activation of the Emergency Action Plan.

Figure 6 – Sugar Creek L-44 Spillway and Dam Erosion

A total of 38 auxiliary spillways flowed in the region due to the rain event but Sugar Creek L-44 was the only site in the area with damage to the embankment. One other site only incurred damage to the spillway. Numerous factors led to the erosion at L-44. The spillway had vegetated silty sands at the surface underlain by sandstone bedrock that dipped towards the embankment. The downstream road had an 18-inch pipe and riser to pass flows that had been reported difficult to keep clear of debris. The flow was observed to be 3 feet over the roadway during the event and led to backing up of water onto the lower 27 feet of the auxiliary spillway. This increase in tailwater at the bottom of the spillway and toe of the dam was determined to have decreased the stability of the soils and increased erosion. However, erosion would have occurred without the increase in tailwater due to the orientation of the spillway. Prior to construction, the design centerline of the spillway was shifted 95 feet towards the embankment, the channel was rotated 9.5 degrees toward the embankment and the exit channel slope was increased from 7.5 degrees to 9.75 degrees. All of the design changes increased the flow velocities on the spillway and at the toe of the dam. Had the tools described above been available to analyze the erosion capabilities of the increased flow on the silty sand material, the design changes may not have been made.

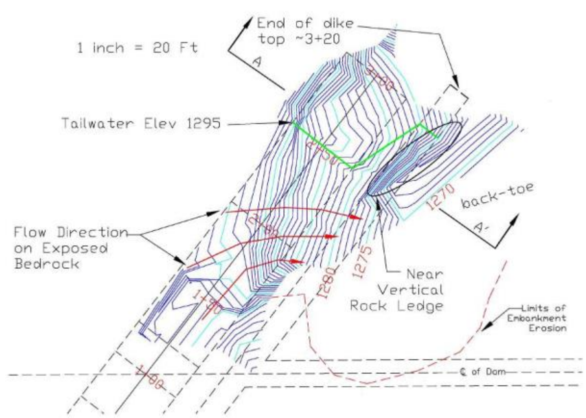


Figure 7 – Sugar Creek L-44 Plan of Erosion Damage

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Water on Tap: Design Considerations for Outlet Controls

Introduction

Outlet works provide for the controlled release of water from reservoirs and may consist of channels, canals, or conduits. Outlet controls typically consist of gates or valves that control the rate of flow through the outlet works. The controls allow the owner/operator to regulate flow in order to accommodate downstream water needs, control reservoir pool levels or drawdown the reservoir at a controlled rate. Failure or improper operation of the outlet controls may induce failure of not only the outlet structure, but of the dam itself due to increased pressures through embankments or overtopping. Appropriate design and maintenance of outlet controls are important to ensure long-term operability and support dam safety.

There are various types of outlet control structures and each serves a different purpose. Some systems are used almost continuously, with frequent adjustment to the outlet controls, while others are seldom used and serve only as backup to another primary outlet, such as an overflow spillway for emergency or lower-level releases. This article presents an overview of the commonly used systems, the key parameters to consider in the design and maintenance of the equipment, and the hydraulic regulation those systems typically provide. Since dam operations, obligations, and impacts are specific to individual circumstances, any decisions for outlet control design or rehabilitation should include appropriate engineering analysis.

Previous *Western Dam Engineering* articles that complement this topic include: Outlet Works Air Vent Design; How Low Can You Go? The Needs and Considerations for Outlets; Letting It All Out: Hydraulic Design of Outlet Works; and Spillways on Small Dams. Structural and hydraulic analyses are an important component of outlet structure design but they are not addressed in this article.

Design Considerations

Whether a channel, canal, or conduit outlet is used, a variety of equipment and design characteristics can be considered. Among many factors that will impact the design, the most important are listed below:

- Discharge capacity
- Frequency of operation
- Access for maintenance
- Operational life
- Rate of opening
- Reliability
- Flow control accuracy
- Cost

A best estimate of these parameters must be determined to select the most appropriate solution and design for the outlet.

Power Supply for Control Systems

Frequently used outlet systems usually rely on electric motors to operate the control system. Redundant power sources are often warranted for dam safety concerns. Redundant sources can be different independent power sources, generators, or manual operation. For smaller systems, or those with low frequency of operation, manual operation may be sufficient and can be accomplished with a wheel, lever, or hand pump connected to hydraulic cylinder.

Control Operators

Gates and valves can be operated by different mechanisms, depending on the physical configuration of the system. Historically, most gates were designed with an operator consisting of a hoist with chains, vertical screws or gear mechanisms such as worm gears or linear gears. Recent designs tend to favor wire rope hoists that offer a good combination of cost, longevity, and ease of maintenance.

Photo 1: Wire Rope Hoisting Systems for Fixed Wheel Gates

Most valves are provided with an actuator that can be either rotary or linear, depending on the type of valve. There will often be a back-up manual operation with a wheel or lever. Small valves or valves with low frequency of operation may only have a manual operation system.

Hydraulic cylinders do not require large space and can be installed on top of the gates. They can also easily be submerged and connected to an accessible location such as the side of the dam. Rising concern about oil leakage is driving the use of biodegradable oil or oil free electric linear actuators for submerged applications.

Screening

Screening at the inlet is used to prevent debris from clogging, impinging, or otherwise blocking the conduit or preventing the control from closing. A trashrack of appropriate size should be designed, that can be permanently fixed or removable for cleaning. A good rule of thumb for trashrack bar spacing is half the diameter of the pipe so small debris washes through and larger debris is arrested on the rack. Motorized trashrack cleaners are typically used in areas exposed to heavy organic loading and frequent debris build up that require regular cleaning.

'Head Killer' versus Head Loss Reduction

An outlet system can serve different purposes. In the case of the outlet discharging to a channel or river, high velocity flow is usually undesirable due to its potential for channel erosion. Energy dissipation structures, such as impact basins, plunge pools, or baffles may be used to reduce the erosion potential of the discharging flows. Riprap or other armoring can also be used to protect channels from erosion.

If the flow outlet is a pressurized pipe, a cone valve or a pressure reduction valve may be used. Pressure reduction valves keep turbulence inside the piping and release flow at the desired pressure and velocity. However, when the pressure head needs to be conserved for downstream use, minimizing friction losses is desirable and can be achieved by:

- Smooth pipe entrance and smooth angles
- Low friction factor associated with gates or valves along the circuit
- Large diameter pipe to reduce velocity
- Low friction pipe lining.

Valve or Gate?

There are several considerations that affect the choice of using a gate versus a valve. Valves may be preferable for the following reasons:

- Valves tend to be chosen for smaller flows and pipes.
- Valves are easy to operate and require relatively small space for installation and operation.
- Valves can be installed anywhere along the piping system where access is easiest for operation and maintenance.
- A variety of standard valves are readily available that can be chosen to match the conditions required for flow control, pressure, and reliability.

Gates may be preferable over valves for the following reasons:

- Valves are not feasible for open channel conditions
- Gates are less expensive for control of large flows
- Access constraints may make valve operation and maintenance difficult, (e.g., on an inclined upstream face with limited access)

Location of the outlet control can be on the upstream face, downstream face or somewhere within the dam. It can also be located downstream of the dam if the water is conveyed through a pipe, in a vault, or a valve chamber. Embankment and earth filled dams tend to favor upstream control locations to avoid constant pressure conduits through the dam with the risk of leakage damaging the dam itself. Often times an upstream guard gate is accompanied by an in line control valve for redundancy and improved operation capabilities.

Types of Control: Gates

Tainter Gate: Also commonly called a segment or radial gate, these gates offer suitable control of large spillways due to their efficient structural designs that are generally less massive than other large gates. They are better suited for surface water control, although they can be used for pressurized conduits. They require relatively small motors to operate and can, in some cases, be hydraulically regulated by means of floats and counterweights, with no hoist needed. However, they are generally not cost-effective for small applications due to the level of design required.

Tainter gates usually require regular maintenance to the gate guides, lifts, and trunnion rotation anchor points to limit the risk of failure. (See the 1995 tainter gate failure on Folsom dam).

Photo 2: Tainter Gates

Fixed Wheel Gate: These gates employ a more robust system suitable for operating in smaller spaces (e.g., conduits) and in higher head conditions. They can easily be used under high pressure and are preferred for submerged applications. In cold climates, they are also chosen over tainter gates for surface applications for the good resistance they offer to ice loads and floating debris, as well as operability under high stress. Relatively large motors and hoist systems are required to overcome the weight and friction of a fixed wheel gate. The presence of wheels requires regular testing and operation to ensure proper operation. Fixed wheel gates can be installed with a stoplog system immediately upstream to facilitate maintenance.

Photo 3: Fixed Wheel Gate

Sluice Gate: Similar to fixed wheel gates, sluice gates (also called slide or knife gates) consist of a vertical plate installed through the water passage; however, the contact with the structure is a frictional interface rather than rolling wheels. Vertical contact is made by a steel or bronze plate sliding against the guide. Bronze is more expensive than steel but also more durable and reduces the friction during operation. The main advantages of sluice gates are their low cost and simplicity. Maintenance due to the wheeled system is eliminated. The induced friction restricts usage to small open areas or low head applications. A sluice gate is a good choice in situations where it is operated infrequently or for small systems. See reference [12] for more information

Photo 4: Angled Intake Sluice Gate Parallel with Upstream Face of Dam

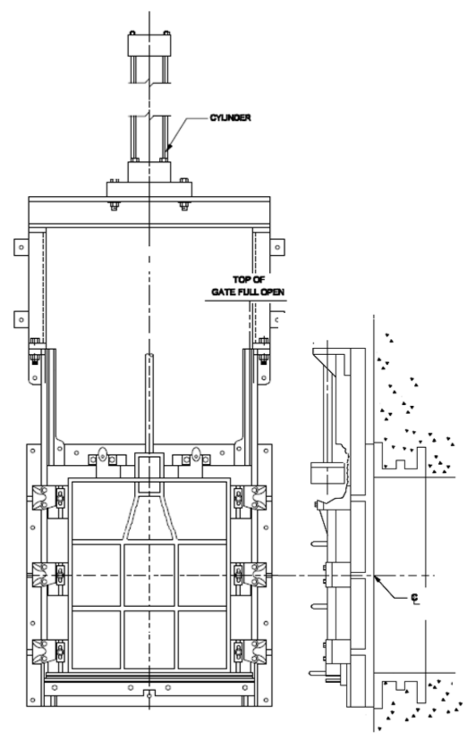

Figure 1: Vertical sluice gate, Cylinder operated

Types of Control: Valves

Valves are used to control the flow in pipes and can be located anywhere along the pipe alignment. They are particularly applicable for pressurized conduits. There are many valve options available that can be used for flow regulation and the most commonly used are described below.

Butterfly Valve: These valves were mainly used as on/off valves from their first use in the 1930s until the late 1970s, when design advancements made throttling more applicable (caution should still be used when butterfly valves are used as throttle valves). Since then, they have become the preferred system for most pipe applications because of their simplicity and reliability. Butterfly valves are used for a large range of conduit diameters and can be automatically or manually operated. They operate well for various pressure and flow conditions.

When the valve is located near another component such as a pump, turbine, bifurcation or angle, a distance of six pipe diameters upstream and four pipe diameters downstream should be maintained between components to minimize turbulence effects on the operation of the valve. Butterfly valve shafts are typically oriented vertically with the actuator on top but the following applications require horizontally mounted shafts:

- Water carrying heavy organic load or sedimentation (enhances flush effect)
- When installed downstream of a centrifugal pump or any component inducing flow rotation around the vertical axis
- When space limitations require the actuator be located on the side of the pipe

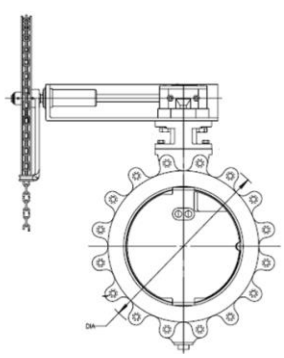

Figure 2: Butterfly Valve, with chainwheel actuator for distant operation

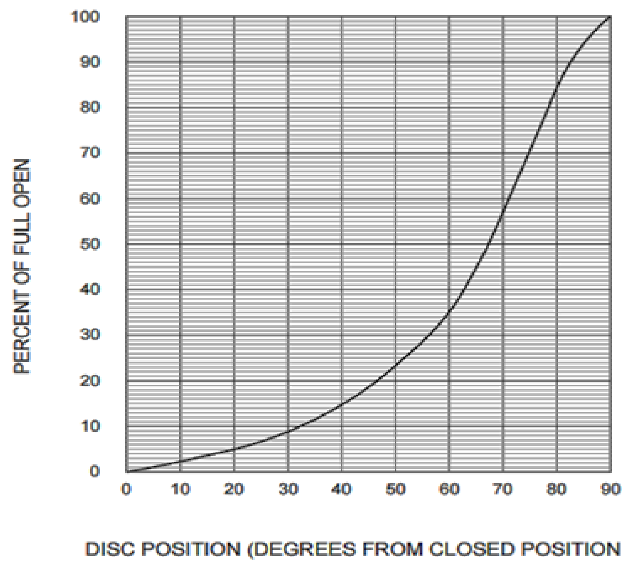

Figure 3: Typical relationship between percentage opening and disc position for a butterfly valve

Figure 3 shows that disc position versus percent opening curve is not linear for butterfly valves. For example a valve disc open at 45 degrees and would result in a nearly 20% opening. Caution should be used when using them as throttling valves.

Photo 5: Interior view of Butterfly Valve

Ball Valve: When dealing with high pressure, ball valves are usually the best choice. Ball valves minimize vibration for highly pressurized conduits and provide relatively accurate flow control. They are often used as guard valves for downstream systems such as pressure reducing valves, turbines, or piping bifurcation. Ball valves have a low friction value when fully open and they can easily be found for diameters up to 48 inches.

It is not recommended that ball valves be used for throttling as demonstrated on Figure 4 below, showing the exponential curve relating the ball position and percent valve opening.

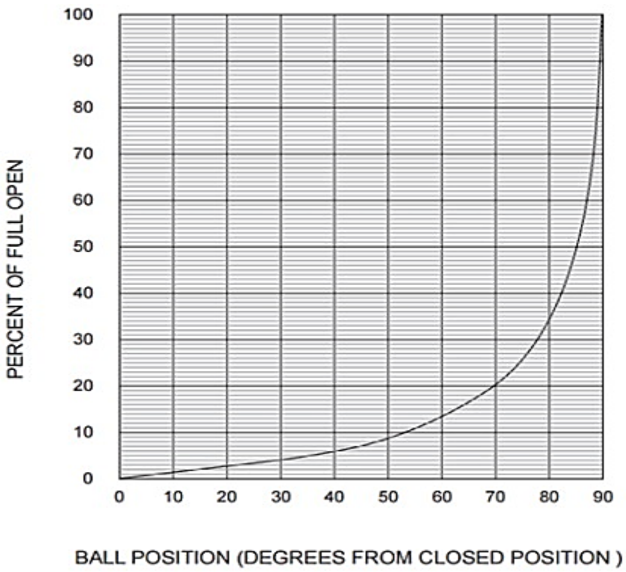

Figure 4: Typical relationship between percentage opening and ball position for a ball valve

Photo 6: Ball Valve with Actuator

Howell Bunger Valve (fixed cone or jet flow valves): These are generally used for discharge of pressurized flow from a closed conduit to free atmosphere. Fixed cone valves provide accurate control of the flow rate during discharge and their symmetric geometry minimizes the vibration associated with the release of the water. The discharge pool should be secured from the public around the jet flow area because of the danger of high pressure flow release. A hood can also be added to limit the spray area. Cold weather discharge from these valves can cause ice to form on downstream structures. This can create safety concerns if access in this area is necessary.

Photo 7: Howell Bunger Valve

In Line Gate Valve: These valves have a low friction coefficient when fully open. They are not recommended for flow regulation or throttling. Manual operation is usually slow, minimizing the risk of water hammer effect. They are sometimes divided into seating-head gates upstream and unseating-head gates downstream.

Table 1: Comparison Table for Control Valves in Pressure Conduits

	Butterfly Valve	Ball Valve	Gate Valve
External space required	Low	Low	High
Friction (open position)	High	Low	Low
Vibration during opening under high pressure	High	Average	Average
Vibration while open under high pressure	Average	Low	Low

Hydraulic Control

Valve Discharge

Valves are characterized by a coefficient, C_V, which varies with the degree of opening. This coefficient, usually given by the manufacturer, can be used to calculate the flow as follows:

$$Q = C_v \sqrt{\frac{\mathrm{P_1} - \mathrm{P_2}}{\mathrm{S_G}}}$$

With:
Q = Flow rate
C_v = Valve coefficient
S_G= Specific gravity of water
P_1= Upstream pressure
P_2= Downstream pressure

The lower the friction, the lower the C_V coefficient, and thus the higher the flow rate through the valve. If the system includes other equipment and a long pipe length driving flow at high velocity, those elements should be included in the flow rate calculations by using the Bernoulli equation.

Gate Discharge

Discharge over a gate-controlled ogee crest:

$$\mathrm{Q} = \frac{2}{3}\sqrt{2\mathrm{g}}\, C_d L(H_1^{3/2} - H_2^{3/2})$$

With:
Q = Flow rate (ft^3/sec)
g= Acceleration due to gravity (ft^2/sec)
C_d= Coefficient of discharge (see graph below)
L= Gate crest length (ft)
H_1= Vertical distance between sill and reservoir level (ft)
H_2= Vertical distance between bottom of gate and reservoir level (ft)

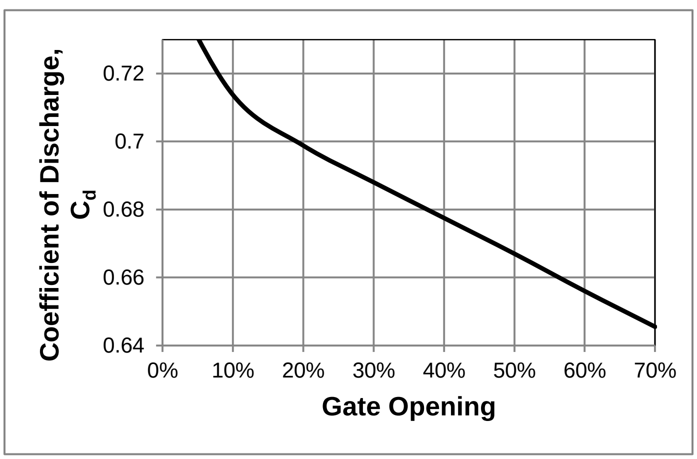

Figure 5: Coefficient of discharge for flow under gates – Reclamation, *Design of Small Dams*, 1973

See references [3] and [4]for more information

Cavitation

After fluid passes the narrowest point of the system, pressure decreases inversely as velocity increases. If the pressure drops below the vapor pressure of water at that particular condition, vapor bubbles start to form. As the fluid moves into a larger area of the vessel or downstream piping, the pressure stops dropping and increases over the vapor pressure, causing the vapor bubbles to collapse or implode. This two-step process is called cavitation and is a major factor in causing surface damage inside the pipe and valves and causing erosion on the pipe surfaces.

The cavitation index "σ", that was approved in 1995 by the Instrument Society of America, is a ratio of forces that resist cavitation to forces that promote cavitation and is written as:

$$\sigma = \frac{\mathrm{P_2} - \mathrm{P}_V}{\mathrm{P_1} - \mathrm{P_2}}$$

With:
σ = Cavitation index
P_1= Upstream pressure
P_2= Downstream pressure
P_V= Liquid vapor pressure

The cavitation potential is inversely proportional to the cavitation index; the lower the cavitation index, the higher the cavitation potential. It is typically recommended to keep σ above 2.5 to eliminate potential for cavitation.

The following adaptations can reduce or suppress the risk of cavitation:

- Venting (See references [2], and [8] and our previous article, Design Considerations for Outlet Works Air Vents (Vol. 1 Issue 2) for more information.)
- Use of additional valves to reduce the pressure differential
- Use of a bypass system.

See reference [7] for more information.

Water Hammer Effect

Water hammer is generated when the flow is suddenly stopped in the hydraulic conduit, and a large shock wave is generated. This situation can be produced by a sudden turbine or pump shutoff or a valve slamming shut. Pressure induced by the water hammer effect can be calculated from the Joukowsky equation:

$$\Delta P = \rho a_0 \Delta v$$

With:
ΔP= Magnitude of the pressure wave
ρ= Density of the water
a_0= Speed of sound in water
Δv= Change in the fluid's velocity

High pressure is induced into the system, creating noise and vibrations. It can result in severe damage to valves, gaskets, and any equipment exposed to water hammer.

Systems involving a long length of pipe with high pressure are more exposed to water hammer effects. The valve must close slowly to minimize water hammer, and the safe closure rate can be calculated. If the fluid is assumed to be incompressible and the water column deceleration after closure is assumed constant, the resultant pressure can be calculated as follows:

$$P = \rho v . \frac{L}{t}$$

With:
P = Pressure (lb/ft^2)
ρ = Fluid density (lb/ft^3)
L = Pipe length (ft)
v = Fluid velocity (ft/sec)
t = Valve closure time (sec)

To mitigate the pressure induced by the shock wave in the system, a surge tank can be added. Surge tanks will act as a "bumper" to provide a release for overpressure and level the overall pressure in the pipe.

Summary

Outlet control is a critical component of dam operation. Important parameters need to be considered during the design of the outlet, such as the flow, pressure, and frequency of use. Also, the type of power source and system will be influential in choosing the appropriate equipment.

A variety of valves or gates can be used for outlet controls, each with particular characteristics. Although valves can accommodate any conduit size, they are used most often for conduits under 36 inches in diameter, with the butterfly valve suitable for most applications. The ball valve is recommended for high pressure applications and the Howell Bunger valve is used for discharge into free atmosphere.

Gates are more suitable to control outlet conduits difficult to access, and any open channel outlet or spillway application. Tainter gates involve more specialized design, but are lightweight and widely used for surface water control applications. Fixed wheel gates are more robust and heavier; they can resist high pressure and rough conditions. Finally, sluice gates are also pressure resistant and very simple, but have high friction forces, requiring more power to operate. Sluice gates are typically used on low to medium flow rate outlets with low frequency of operation.

Flow rate can be calculated with variable accuracy depending on the equipment used. Flow through valves or gates can easily be calculated using the associated coefficient. However, more complex outlet systems with several controls and long piping will require more developed calculations based on the Bernoulli equation. Gates and valves are specialized equipment that can have long lead times. Manufacturers should be consulted early in the design process.

Useful References

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Does Your Dam Measure Up? – Developing an Effective Instrumentation Program for Small Earth Dams

Introduction

An effective dam surveillance and monitoring program is an important component to any dam safety program. The goal of a dam surveillance and monitoring program is to detect if the dam is not performing per the design or if the dam is developing a condition that could lead to adverse performance of the dam. These conditions, if left undetected, could culminate in a dam safety incident or failure and may present a risk to the public, property, or environment. There are two necessary elements of an effective program: surveillance (visual observations) and monitoring (instrumentation measurements). This article focuses on developing an effective instrumentation program for small earth dams based on a identifying a clear set of objectives and selecting the right instrument for the job.

Surveillance

The article Dam Safety Inspections…A Closer Look (Volume 3 Issue 2 Western Dam Engineering Technical Note) presented the aspects of visual inspections. Surveillance is the most important component in a dam safety program and consists of the routine visual inspection of the dam. However, not every potential deficiency can be detected or understood by visual surveillance alone. The need for both instrumented and visual monitoring exists for nearly every dam.

For example, while visual inspections for slides, slumps, cracking, and bulging are important visual indications of a potentially unstable slope, additional water pressure data from piezometers and sediment traps will help the owner and engineer understand the extent and possible cause of any observed abnormality.

Instrumentation provides the necessary quantitative data to support a safety evaluation on a dam's performance. These data are used to detect changes in dam behavior and potential slowly developing problems.

Monitoring

The main objectives of a monitoring program are to verify the performance of the project structures with respect to the design parameters, quickly identify any change in conditions that has a potential for safety concerns, and develop data for analytical assessment and prediction of future performance. Detecting a developing problem early through active monitoring can allow for successful intervention, thereby reducing the risks of economic loss and downstream consequences.

A good monitoring program should include monitoring for potential failure modes (PFMs) of the dam as well as monitoring for general health of the structure. General health monitoring refers to implementing best-practices to gather data vigilant dam owners should know about their structure, regardless of specific failure modes, such as reservoir level and periodic measurement of crest elevation. Monitoring for identified PFMs requires that specific defects or conditions that could lead to failure be identified by a thorough review of the design, construction, operation, and performance history of the dam. This review helps establish the objectives of any measurement device and identify the best instrument and location to obtain the targeted measurement. This will be described throughout this article.

Typical Instruments for Small Earth Dams

The information below focuses on common instrumentation that may be appropriate for small earth dams. Information on all the possible types of instrumentation can, and has, filled books (see References [2] and [3]). This section provides a basic overview of the most commonly used instruments. Typical instrumentation appropriate for small earth dams includes means for measuring water levels; weirs to measure seepage flow and turbidity; observation wells/piezometers to measure pore water pressure, and survey monuments to measure settlement and crest elevation. The reader is encouraged to review the references at the end of this article for more details regarding the proper selection, design, installation, and readings for each instrument they plan to implement into their monitoring program.

Reservoir and Tailwater Levels

For most dams, it is important to know the water level in the reservoir and downstream pool or channel. This information can help correlate other monitoring data for a better understanding of the response of the structure at varying pool levels as well as provide useful information for documenting typical operating conditions for each dam. Typical instruments and tools used to determine reservoir levels include staff gauges, slope stakes, or a tape measure. A graduated staff gauge can provide accurate and repeatable water level measurements for reservoir pools, tailwater, and within weirs or flumes. They can be permanently mounted on any flat surface and bolted directly to a structure. The gauge should be made with indelible graduations and markings, so it is resistant to sun bleaching, rusting or other forms of deterioration.

Staff gauges can be mounted on a vertical surface such as a pier or post or on an inclined surface such as the embankment slope. The gauge should be carefully surveyed to accurately mark the elevation graduations.

A typical staff gauge is shown in Photograph 1.

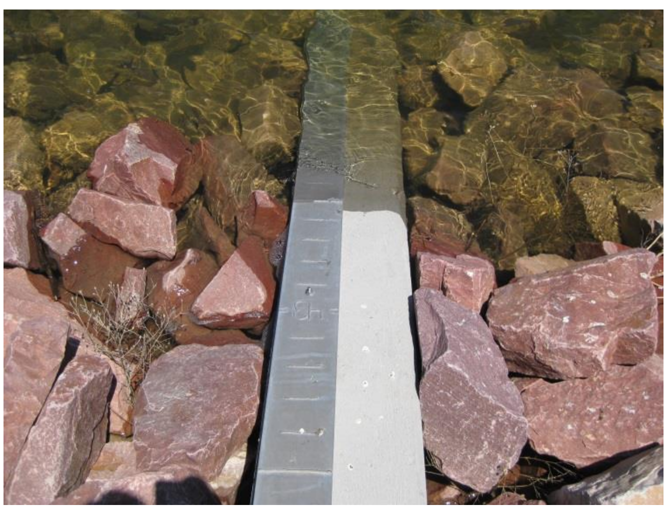

Photo 1: Staff Gauge

Slope stakes are another common method of measuring reservoir level. These consist of stakes that are installed in a line along a slope with a consistent grade and can be used to interpolate reservoir level. Typically, the slope stakes cannot be installed on the embankment, as wave protection is often present. The water level measurement is recorded relative to the known position of the stakes and using a correlation table to interpret the reading. If the potential for movement is detected or suspected, regular resurvey of the stakes is required to maintain an accurate water level reading. An example of a typical slope stake system table is shown in Figure 1.

The 0+00 pin is located at the upper left (north west) corner of the west boat ramp slab.
The alignment is down the left side of the slab.
The slab surface is the datum for the slope distance elevations.
Pins installed and elevations established by GPS on October 19, 2000.
A top of cap elevation is listed for the 50 foot pins.
Elevations listed for top left end center of the boat ramp sections below the solid slab.
Storage volumes from the March, 1965 active storage table; storage volumes above 4315.9 ft were extrapolated by HKM Engineering (2007)

DISTANCE feet	ELEVATION feet	STORAGE acre-feet	DISTANCE feet	ELEVATION feet	STORAGE acre-feet
0+00 CAP	4319.61	6,124	34	4315.32	5,052
1	4319.49	6,094	35	4315.20	5,022
2	4319.38	6,066	36	4315.07	4,990
3	4319.26	6,036	37	4314.94	4,957
4	4319.14	6,006	38	4314.82	4,927
5	4319.02	5,976	39	4314.69	4,895
6	4318.90	5,946	40	4314.56	4,862
7	4318.78	5,916	41	4314.44	4,833
8	4318.66	5,886	42	4314.31	4,800
9	4318.54	5,856	43	4314.18	4,768
10	4318.42	5,826	44	4314.06	4,738
11	4318.29	5,794	45	4313.93	4,706
12	4318.16	5,761	46	4313.81	4,676
13	4318.03	5729	47	4313.68	4,643
Spillway	4318.00	5,721	48	4313.55	4,611
14	4317.90	5,696	49	4313.43	4,582
15	4317.77	5,664	50	4313.30	4,549
16	4317.64	5,631	51	4313.17	4,517
17	4317.51	5,599	52	4313.04	4,484
18	4317.38	5,566	0+50 CAP	4313.01	4,477
19	4317.25	5,534	53	4312.91	4,454
20	4317.12	5,501	54	4312.79	4,427
21	4316.99	5,469	55	4312.66	4,398
22	4316.86	5,436	56	4312.53	4,370
23	4316.73	5,404	57	4312.40	4,340
24	4316.61	5,374	58	4312.27	4,311
25	4316.48	5,341	59	4312.14	4,282
26	4316.35	5,309	60	4312.01	4,253
27	4316.22	5,276	61	4311.89	4,227
28	4316.09	5,244	62	4311.76	4,197
29	4315.96	5,212	63	4311.64	4,171
30	4315.83	5,179	64	4311.51	4,142
31	4315.70	5,146	65	4311.39	4,115
32	4315.58	5,116	66	4311.26	4,086
33	4315.45	5,084	67	4311.14	4,059
DISTANCE	ELEVATION	STORAGE	DISTANCE	ELEVATION	STORAGE

A2 2011

Figure 1: Slope Elevation Storage Table

The third and obviously simplest means of measuring reservoir and tailwater level, is by dropping a weighted tape measure from a reference mark with a known elevation. Similar to the consideration for locating a permanent staff gage, the measurement location should consider effects of drawdown caused by entrance flow into a gate, structure, or weir. For this reason measurements should not be performed into a drop structure or near a spillway with an open gate as the head losses at the inlet can cause significant error.

Seepage Flow Measurements

Weirs are frequently chosen to measure seepage rates, monitor turbidity, and monitor sediment transport. Common weir shapes are square, trapezoidal, and V-notch. The appropriate shape and size of a weir depends mainly on the volume of flow to be measured. Some weirs are more capable and accurate at measuring small, or large, flows than others. A weir can be installed to measure seepage flow and sediment transport out of a toe drain system or known surface seep that can be concentrated into a channelized path. A V-notch weir is shown on Figure 2 and Photograph 2. V-notch weirs are efficient at measuring low flows (less than about 450 gallons per minute (1 cubic foot per second)). The depth of water is typically measured with a staff gauge installed within the upstream pool, away from velocity drawdown effects as shown in Figure 2.

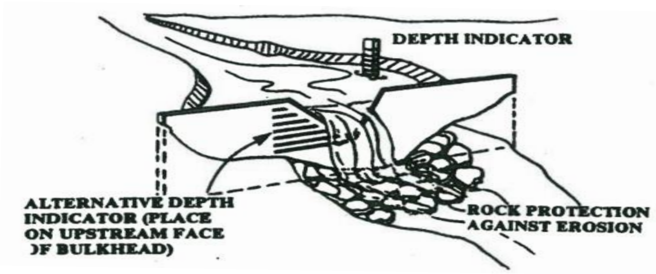

Figure 2: V-Notch Weir and Staff Gauge

Reference: FEMA, Dam Safety: An Owner's Guidance Manual, 1987

Photo 2: V-Notch Weir

Other seepage monitoring devices and methods include parshall flumes, pipes (toe drains), timed-bucket methods, and flow meters. Internal erosion can be detected by observing the water for increased turbidity and sediment. The Bureau of Reclamation provides a useful reference for details on methods to measure seepage in the Water Measurement Manual. Reclamation also provides download links for obtaining the USBRWeir.xls spreadsheet, used to determine rating tables and equations for sharp-crested weirs.

Piezometers

Piezometers installed in the abutments, foundations, and embankments of a dam are used to monitor phreatic surface levels, uplift pressures, and seepage gradients (through the use of piezometer groups). A line of piezometers installed at the dam crest, mid-slope and toe provides information to create a profile of the phreatic surface through the dam. A set of piezometers installed at different elevations in one location (nested piezometers) provides pore pressures in different soil strata and can be used to estimate vertical seepage gradients.

Some common types of piezometers include open standpipe, fiber optic, and vibrating wire piezometers. The open standpipe, or observation well, installation offers the added benefit of being able to manually confirm the water level reading versus sole reliance on a digital instrument. A standpipe piezometer is shown in Photograph 3. Fiber optic piezometers are generally contained within a stainless steel tube and separated from the environment by a porous filter material. Some advantages include their immunity to vibration, lightning damage, and radio and electromagnetic noise interference.

Vibrating wire piezometers contain electrical pressure transducers that read and record the pore-water pressure automatically, allowing for easier and more frequent data acquisition. Each of these types of piezometers has specific design specifications and a qualified engineer should be involved in specifying the appropriate piezometer type, location, and installation procedures.

Piezometers, when strategically placed, can provide useful information on the overall seepage regime within and below the dam. However, since they are a single point measurement, they are not often effective at detecting or characterizing potential concentrated seepage paths unless they are installed to monitor an already known location of a defect.

Photo 3: Standpipe Piezometer

Settlement Monitoring

Typically, movement in dams is monitored to detect settlement or deformations in the dam. These conditions may be due to consolidation, creep, or subsidence or other factor. Settlement within an embankment dam may lead to loss of freeboard or cracking due to differential settlement. Depressions, sinkholes, scarps, sloughs or bulges, which may be indicative of slope instability or internal erosion, are often localized features that are more effectively detected through frequent visual surveillance rather than relying on periodic reading of instrumentation. However, once identified, the known condition can be more quantitatively monitored with instrumentation. Instruments commonly used to monitor settlement include survey monuments, settlement plates/sensors, extensometers, piezometers, and inclinometers.

More Sophisticated Instruments

A large selection of instrumentation is available to measure ground water and pore pressures; seepage, flow, and turbidity; stress and strain; load; temperature; precipitation and wind; reservoir and tailwater levels; water quality; seismic measurements; and deformation. Details of these types of instruments are included in the ASCE Task Committee Guidelines for Instrumentation and Measurements for Monitoring Dam Performance (2000).

Planning an Effective Monitoring Program

Identifying the Need for Instrumentation

Planning a monitoring program should begin by identifying the need for instrumentation. The need may be based on an observed condition or a known vulnerability. Site conditions such as underground mine workings, deep groundwater pumping, soft foundation, or abrupt changes in the subsurface profile could be known vulnerabilities that may lead to subsidence or differential settlement. Observed seepage, sediment deposition, or depressions, may all be observed conditions that would warrant seepage monitoring through collection and measurement, or piezometers to evaluate seepage gradients. Signs of slope instability (cracking, scarps, bulges) may be justification to install piezometers to estimate internal pore pressures for stability analyses. The need for instrumentation is based on a comprehensive understanding of the design and performance of the dam. If an observed condition or known vulnerability is identified, then one must assess whether it can be reliably and efficiently monitored with instruments. If so, which instrument is best suited for the specific objective?

Selecting the Right Instrument

Once the need and specific monitoring objective is identified, selecting the proper instrument is the next step. Instrumentation should be selected based on the answers to several pertinent questions:

- What is the observed condition, known vulnerability, or PFM?
- What parameter would best monitor the condition (flow, pore pressure, gradient, deformation, etc)?
- Where would the instrument need to be located to monitor the condition?
- How would the instrument need to be installed?

- What method should be used for collecting data?
- What threshold level should be established for each instrument as a guideline for recognizing an usual reading?

Answers to these questions will guide the decision to which instrument is best suited for the job. Selecting the right instrument includes having an idea on the known parameter. Do I need a 90-degree v-notch weir or a parshall flume? Should my piezometer be an open standpipe or a nest of fully grouted vibrating wires?

The following table is one example that can be used in selecting the right instrument. This assessment is recommended to insure selection of the correct instrument to serve the required purpose. Note that the examples shown in the table are for illustration purposes only; actual information and recommendations would be very site specific depending on the potential deficiency and site specific conditions of the dam.

Table 1: Instrument Planning Table

Problem / Deficiency / PFM	What information/parameter does the Instrument need to provide?	At what location is the parameter best measured? (e.g. where is the source of water?)	Construction Recommendation	Potential Problems with Installation	Reading Method and Frequency[1]
Example: Saturated foundation, dry embankment	*Are foundation uplift pressures present? Is a confining layer present? If so, how extensive?*	*Fractured bedrock Water present at approximately 20 ft deep in boreholes*	*2 piezometers in same 6" borehole 1 in fractured bedrock 1 in presumed cap layer*	*Isolating strata, can bedrock be augered through?*	*Manually read with water probe, monthly*
Example: Seepage observed at downstream toe	*Flow rate of seepage Is sediment present within the flow? What is the relationship or time lag with pool level – does it stop or increase at different pool levels?*	*Collect seepage at a location near downstream toe*	*90^{o} v-notch weir Collect into an upstream basin to monitor for sediment Install water level gauge*	*Collect and convey seepage to single point, avoid seepage bypassing weir. limit adjacent runon if possible*	*Manually read water level and convert to flow (gpm), monthly*

1. Record reservoir level and recent precipitation at every reading

Implementation (Overview of Design Considerations)

Key design considerations when implementing a monitoring program include:

- Instrument Location
- Instrument Design
- Installation and Protection

The following are some considerations for picking the best instrument location. Limiting impacts of the existing structure should be considered when selecting the preferred location. For example, in order to monitor the phreatic level within the foundation, is it necessary to drill the piezometer through the embankment or can it be installed at the toe?

The location should consider both the existing conditions as well as the post-installation conditions. For example, toe drain outlets that look great during installation, may become inundated over time if installed in a location that does not allow positive drainage.

The ability to measure the instrument should be considered when selecting the installation location. Can you get a bucket underneath the outlet pipe to measure outflow using the timed-bucket method? Staff gauges need to be easily visible from the shore, and within calm water, far enough away from any water intake gate, channel, weir or flume to avoid inaccurate readings due to drawdown effects. A staff gauge should be placed in a sheltered area that is not influenced by wave action or susceptible to ice damage.

The location should also be chosen such that the instrument does not become a target of vandalism. Instruments in areas accessible to the public are better placed in discreet locations. Piezometers and weirs are common targets of vandalism.

Installation methods of intrusive instruments such as piezometers should be carefully considered, as they can be costly and may impose a risk of damaging the structure in the process. Shallow piezometers (less than about 10 feet) in fine grained sands, silts and clays can be installed with a hand auger as shown in Photograph 4. A video describing the method for installing a piezometer by hand auger can be viewed here: Installing Piezometers in Your Dam. Deeper piezometers, or those to be installed in dense or coarse grained material usually require an auger drill rig as shown in Photograph 5.

Photo 4: Installing Piezometer by Hand Auger

Photo 5: Auger Drill Rig

Reservoir level and seepage conditions should also be considered prior to installing intrusive instrumentation. Installing these instruments can impose conditions that may increase seepage risks for the dam if not accounted for properly. The Bureau of Reclamation provides a guidance document on the installation of piezometers in their Embankment Dam Instrumentation Manual.

Design and installation considerations for all instruments are beyond the feasible scope of this article. However, the effectiveness of the monitoring system is dependent on, and very sensitive to, the correct selection, design, and installation of the instrument. Therefore, an experienced engineer, manufacturer, and/or regulator should be consulted.

Data Collection

Staff should be trained to collect and field-evaluate data and maintain the instrumentation. The collection method and frequency will depend on the parameter being measured and the expected variation. The monitoring frequency is usually based on the expected rate of change, reservoir level variations, time of year, observed variability in reading, and collection method.

The individual reading the instrument should have the historical data or threshold levels accessible in the field. Any spurious reading that deviates from the historical trend or threshold level should be immediately checked by an additional field measurement to confirm the reading. Instrument data are influenced by external factors such as reservoir level, temperature, and recent precipitation. Therefore, this information should be documented along with the instrument reading.

The individual reading the instruments (and performing visual observations) should be trained in detecting unusual or adverse conditions and how to respond. Response may be to call a pre-identified dam safety engineer or regulator, and in severe conditions, such as observing an active sand boil, be prepared to implement immediate risk reduction measures such as lowering the reservoir, placing sand bags, or installing a reverse filter, as well as, when required, activating the Emergency Action Plan.

Manual data collection is most common for small dams. In some rare cases data collection using automated data acquisition systems (ADAS) is warranted on small dams if there is a high risk associated with changes in the measured parameter and the dam is not observed frequently. On larger dams, the trend is away from manual measurements and mechanical recorders and toward electronic measurements and ADAS. Potential benefits of automated data acquisition include:

- A current and continuous data record
- Repeatability of data acquisition
- A variety of data processing options to improve accuracy
- Potentially lower costs to collect a large volume of data
- Reallocation of labor resources to the more valuable functions of analysis and decision-making
- The ability to assess real-time data remotely
- The ability to automatically initiate alarms and other actions if critical thresholds are exceeded

Data Interpretation

The purpose of data interpretation is to evaluate what the data indicate about the performance of the dam. In order for instrumentation and monitoring to be effective, data needs to be evaluated in a timely manner and with respect to project conditions including reservoir elevation, temperature, precipitation, operational changes, and loading conditions. All data should be compared with expected behavior based on engineering concepts of dam behavior. Expected behavior may be based on analyses, historic readings, or educated judgment. Expected behavior of the data may follow trends, such as decreasing or increasing with time or depth, seasonal fluctuation, variation with reservoir or tailwater level, or a combination of such trends. Trends are best identified through plotting of the data.

Variations from expected behavior may suggest development of conditions that should be evaluated further. If no unusual behavior or evidence of problems is detected, the data should be filed for future reference. If data deviate from expected behavior or design assumptions, action should be taken. The action to be taken depends on the nature of the problem, and should be determined on a case-by-case basis. Possible actions may include more frequent readings, detailed visual inspection, analysis using the new data, or risk reduction measures such as lowering the reservoir, or designing remedial measures. ASCE Task Committee Guidelines for Instrumentation and Measurements for Monitoring Dam Performance (2000) provides more detail on instrumentation interpretation for dam safety.

Conclusions

A robust and effective surveillance and monitoring program is well designed, proactive, and well understood. Every dam is unique and there is not a "one size fits all" approach to determining the appropriate level of instrumentation. Not every dam warrants a robust instrumentation program. The required monitoring is dependent on the size and type of the dam, the hazard potential classification of the dam, the site conditions including the foundation conditions, existing deficiencies or problems, design of the dam, and any identified PFMs. Table 2 presents a list of common potential failure modes and their associated monitoring instruments. This list can be used as a guide to get started on planning and implementing a monitoring program appropriate for each dam. The monitoring program should be determined by the dam owner/operator, designer, and regulator, should be related to the design criterion and reflect the needs based on the observed behavior, known vulnerabilities, and identified PFMs. Every instrument should address a specific need. While the right instrumentation can be essential in identifying potential problems, in the words of Dr. Ralph Peck, "An instrument too often overlooked in our technical world is a human eye connected to the brain of an intelligent human being." Routine visual inspection of the dam is of the upmost importance and should always be the primary means of monitoring the dam. It is important to know how to recognize and respond to adverse conditions. Proactive, effectively designed and implemented programs have been successful in detecting a developing adverse condition in sufficient time to allow for successful intervention preventing a dam safety incident or failure.

Table 2: Typical Instrumentation and Monitoring Used in Evaluating Causes of Common Problems/Concerns
[Reference taken from Table 9-4c in FERC Chapter 9 (FERC, 2005)]

Problem/Concern	**Typical Instrumentation**
Seepage or leakage	Visual observation, weirs, flow meters, flumes, calibrated containers, observation wells, piezometers
Boils or piping	Visual observation, piezometers, weirs
Uplift pressure, pore pressure, or phreatic surface	Visual observation, observation wells, piezometers
Drain function or adequacy	Visual observation, pressure and flow measurements, piezometers
Erosion, scour, or sedimentation	Visual observation, sounding, underwater inspection, photogrammetric survey
Dissolution of foundation strata	Water quality tests
Total or surface movement (translation, rotation)	Visual observation, precise position and level surveys, plumb measurements, tilt meters
Internal movement or deformation in embankments	Settlement plates, cross-arm devices, fluid leveling devices, pneumatic settlement sensors, vibrating wire settlement sensor, mechanical and electrical sounding devices, inclinometers, extensometers, shear strips
Internal movement or deformation in concrete structures	Plumb lines, tilt meters, inclinometers, extensometers, joint meters, calibrated tapes
Foundation or abutment movement	Visual observation, precise surveys, inclinometers, extensometers, piezometers
Poor quality rock foundation or abutment	Visual observation, pressure and flow measurements, precise surveys, extensometers, inclinometers
Slope stability	Visual observation, precise surveys, inclinometers, extensometers, observation wells, piezometers, shear strips
Joint or crack movement	Crack meters, reference points, plaster or grout patches
Stresses or strains	Earth pressure cells, stress meters, strain meters, over coring
Seismic loading	Accelerographs
Relaxation of post-tension anchors	Jacking tests, load cells, extensometers, fiber-optic cables
Concrete deterioration	Visual observation, loss of section survey, laboratory and petrographic analyses
Concrete growth	Visual observation, precise position and level surveys, plumb measurements, tilt meters, plumb lines, inclinometers, extensometers, joint meters, calibrated tapes, petrographic analyses
Steel deterioration	Visual observation, sonic thickness measurements, test coupons

[1] Appropriate remedial measures should be taken for all problems and concerns. Possible remedial measures for a wide variety of problems and concerns are discussed in EPRI (1986), National Research Council (1983), ASCE (1975 and 1988) and USACE (1986a).

Useful References

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