Technical Manual: Conduits through Embankment Dams

Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair

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Conduits through Embankment Dams

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Preface

Tens of thousands of conduits through embankment dams in the United States are aging and deteriorating. These conduits often were poorly constructed and are not frequently inspected, if at all. Deteriorating conduits pose an increasingly greater risk for developing defects that can lead to embankment dam failure with each passing year. In an effort to deal with this problem, this document has been prepared to collect and disseminate information and experience that is current and has a technical consensus.

This document provides procedures and guidance for "best practices" concerning design, construction, problem identification and evaluation, inspection, maintenance, renovation, and repair associated with conduits through embankment dams. Most of the available information on these topics was reviewed in preparing this document. Where detailed documentation existed, it was cited to avoid duplicating available materials. The authors have strived not to reproduce information that is readily accessible in the public domain. This document attempts to condense and summarize the vast body of existing information, provide a clear and concise synopsis of this information, and present a recommended course of action. This document is intended for use by personnel familiar with embankment dams and conduits, such as designers, inspectors, construction oversight personnel, and dam safety engineers.

In preparation of this document, the authors frequently found conflicting procedures and standards in the many references they reviewed. Where conflicts were apparent, the authors focused on what they judged to be the "best practice" and included that judgment in this document. Therefore, this document may be different than some of the various participating agencies' own policies.

Embankment dams, regardless of their size, create a hazard potential from the stored energy of the water they impound. Examples, such as Kelley Barnes Dam, which failed suddenly in 1977, show the destructive power of water when it is released suddenly from behind even a small embankment dam. This embankment dam was less than about 40 feet high and about 400 feet long, but when it failed, it released water downstream at an estimated flow rate of over 24,000 ft^3/s, killing 39 people. The hazard potential of an embankment dam is based on the consequences of failure, rather than its structural integrity.

Embankment dams can be classified according to their hazard potential for causing damages downstream should they fail. Various State and federal agencies have different systems for rating the hazard classes of embankment dams. A single, universally accepted hazard classification system does not exist. All of the hazard classification systems group embankment dams into categories based on the potential impacts of a theoretical release of the stored water during a dam failure. However, the most common problem with all of these classification systems is the lack of clear, concise, and consistent terminology. The Federal Emergency Management Agency (FEMA) has a hazard classification system that is clear and succinct, and this system was adopted for the purposes of this document. The reader is directed to FEMA 333, *Federal Guidelines for Dam Safety: Hazard Potential Classification Systems for Dams* (1998), for a complete version of their system. The FEMA document uses three hazard potential levels to classify embankment dams. These levels are summarized as follows:

- *Low hazard potential.*—Embankment dams assigned the low hazard classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

- *Significant hazard potential.*—Embankment dams assigned the significant hazard classification are dams where failure or misoperation results in no probable loss of human life, but can cause economic loss, environmental damage, or disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas, but could be located in areas with population and significant infrastructure.

- *High hazard potential.*—Embankment dams assigned the high hazard classification are those where failure or misoperation will probably cause loss of human life.

Hazard potential classification	Loss of human life	Economic, environmental, lifeline losses
Low	None expected	Low and generally limited to owner
Significant	None expected	Yes
High	Probable—one or more expected	Yes (but not necessary for this classification)

Embankment dam hazard classifications are assigned based on their potential for causing downstream damage, but these classifications do not reflect in any way on the likelihood that the dam may fail. An embankment dam might be classified as having a low hazard potential based on the impacts a failure would have on the downstream area, but have a high probability of failure if it were in very poor condition. The hazard classification says nothing about the safety or condition of the structure.

The guidance in this document is considered valid technically without regard to the hazard potential classification of a particular embankment dam. However, some design measures that are commonly used for design of high and significant hazard embankment dams may be considered overly robust for use in low hazard dams. As an example, chimney filters that extend across the entire width of the embankment fill section are recommended for most high hazard embankment dams. Many smaller, low hazard embankment dams are constructed without this feature. This document recommends that even low hazard embankment dams should contain other currently accepted design measures that address seepage and internal erosion along the conduit. Specifically, a filter diaphragm or filter collar around the conduit is recommended for all embankment dams penetrated by a conduit.

Often, low hazard embankment dams are small structures (height or reservoir volume). The term "small embankment dam" does not have a single widely accepted definition. Some designers may consider a 25-foot high embankment dam to be the largest dam in the small dams category, and others may consider this to be the smallest dam in the large dam category. The International Commission on Large Dams defines large embankment dams as being more than about 50 feet high. For this reason, this document will consider only the hazard potential of the embankment dam. The focus of this document is on significant and high hazard embankment dams due to the concern for loss of life and property damage. However, where appropriate, deviation from the guidance is noted for low hazard embankment dams. This deviation is not all inclusive, and the designer may find additional guidance on the design and construction of conduits within low hazard embankment dams in Natural Resources Conservation Service (NRCS) *National Handbook of Conservation Practice Standard Code 378 Pond* (2002). The designer should be aware that future downstream development could require revising the hazard potential classification from low to significant or high. Pressurized conduits are not recommended at low hazard embankment dams, since these structures often lack regular inspections and may not contain the appropriate safety features as discussed in this document.

FEMA's National Dam Safety Program sponsored development of this document in conjunction with the Association of State Dam Safety Officials, Bureau of Reclamation, Federal Energy Regulatory Commission, Natural Resources Conservation Service, and U.S. Army Corps of Engineers.

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The National Dam Safety Review Board (NDSRB) reviewed this document prior to issuance. The NDSRB has responsibility for monitoring the safety and security of dams in the United States, advising the Director of FEMA on national dam safety policy, consulting with the Director of FEMA for the purpose of establishing and maintaining a coordinated National Dam Safety Program, and monitoring of State implementation of the assistance program.

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If conduits are not designed and constructed correctly, embankment dams will have an increased probability of failure, which endangers the public. The particular design requirements and site conditions of each embankment dam and conduit are unique. No single publication can cover all of the requirements and conditions that can be encountered during design and construction. Therefore, it is critically important that conduits through embankment dams be designed by engineers experienced with all aspects of the design and construction of these structures.

The users of this document are cautioned that sound engineering judgment should always be applied when using references. The authors have strived to avoid referencing any material that is considered outdated for use in modern designs. However, the user should be aware that certain portions of references cited in this document may have become outdated in regards to design and construction aspects and/or philosophies. While these references still may contain valuable information, users should not automatically assume that the entire reference is suitable for design and construction purposes.

Many sources of information were utilized in the development of this document, including:

- Published design standards and technical publications of the various federal and State agencies involved with the preparation of this document.

- Published professional papers and articles from selected authors, technical journals and publications, and organizations.

- Experience of the individuals and the federal and State agencies involved in the preparation of this document.

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Common Abbreviations

AAR, alkali-aggregate reaction
AASHTO, American Association of State Highway and Transportation Officials
ACI, American Concrete Institute
ADCI, Association of Diving Contractors International
AISI, American Iron and Steel Institute
ASCE, American Society of Civil Engineers
ASDSO, Association of State Dam Safety Officials
ASTM, ASTM International
ATV, all-terrain vehicle
AWS, American Welding Society
AWWA, American Water Works Association
BIA, Bureau of Indian Affairs
CAT, computerized axial tomography
CCTV, closed circuit television
CD-ROM, compact disc—read-only memory
CFR, comprehensive facility review
CIPP, cured-in-place pipe
CMP, corrugated metal pipe
CPR, cardiopulmonary resuscitation
CPS, cathodic protection system
CSP, corrugated steel pipe
DH, drill hole
DOS, disc operating system
DVD, digital versatile disc
EAP, Emergency Action Plan
FEMA, Federal Emergency Management Agency
FERC, Federal Energy Regulatory Commission
FFP, Fold-and-Formed Pipe
FHWA, Federal Highway Administration
FLAC, Fast Lagrangian Analysis of Continua
GPR, ground penetrating radar
GPS, global positioning system
HDD, horizontal directional drilling
HDPE, high density polyethylene
ICODS, Interagency Committee on Dam Safety
ICOLD, International Commission on Large Dams
IRCI, International Concrete Repair Institute
JHA, job hazard analysis
LL, liquid limit
LMG, limited mobility grout
LOTO, lockout tag-out

MASW, multichannel analysis of surface waves
MCE, maximum credible earthquake
MRI, magnetic resonance imaging
NDL, no decompression limit
NDSRB, National Dam Safety Review Board
NDT, nondestructive testing
NPSHA, net positive suction head available
NPSHR, net positive suction head required
NRCS, Natural Resources Conservation Service
O&M, operation and maintenance
OSHA, Occupational Safety and Health Administration
PCCP, prestressed concrete cylinder pipe
P.E., professional engineer
PE, polyethylene
PDF, portable document format
PFR, Periodic Facility Review
PI, plasticity index
PPI, Plastic Pipe Institute
PVC, polyvinyl chloride
RCCP, reinforced concrete cylinder pipe
RCP, reinforced concrete pipe
Reclamation, Bureau of Reclamation
ROF, report of findings
ROV, remotely operated vehicle
SASW, Spectral Analysis of Surface Waves
SCS, Soil Conservation Service
SD, strength design
SDR, standardized dimension ratio
SNTC, South National Technical Center
SP, self potential
TADS, Training Aids for Dam Safety
UNEP, United Nations Environment Programme
USACE, U.S. Army Corps of Engineers
USDA, United States Department of Agriculture
UV, ultraviolet
VHS, Video Home System
WSD, working stress design

Conversion Factors
To the International System of Units (SI) (Metric)

Pound-foot measurements in this document can be converted to SI measurements by multiplying by the following factors:

Multiply	By	To obtain
acre-feet	1233.489000	cubic meters
cubic feet	0.028317	cubic meters
cubic feet per second	0.028317	cubic meters per second
cubic yards	0.764555	cubic meters
degrees Fahrenheit	(°F-32)/1.8	degrees Celsius
feet	0.304800	meters
feet per second	0.304800	meters per second
gallons	0.003785	cubic meters
gallons	3.785412	liters
gallons per minute	0.000063	cubic meters per second
gallons per minute	0.063090	liters per second
inches	2.540000	centimeters
mils	0.000025	meters
mils	0.025400	millimeters
pounds	0.453592	kilograms
pounds per cubic foot	16.018460	kilograms per cubic meter
pounds per square foot	4.882428	kilograms per square meter
pounds per square inch	6.894757	kilopascals
pounds per square inch	6894.757000	pascals
square miles	2.589988	square kilometers
tons	907.184700	kilograms

Symbols

c, cohesion
E', modulus of soil reaction
f'c, compressive strength of concrete
p, pore pressure
s, shear strength
θ, angle of internal friction
P-, primary. A P-wave is a seismic compression wave.
S-, secondary. An S-wave is a seismic shear wave.

ASTM Standards

ASTM Standard	Title
A 36	Standard Specification for Carbon Structural Steel
A 53	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless
A 796	Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe-Arches, and Arches for Storm and Sanitary Sewers and Other Buried Applications
C 33	Standard Specification for Concrete Aggregates
C 94	Standard Specification for Ready-Mixed Concrete
C 150	Standard Specification for Portland Cement
C 361	Standard Specification for Reinforced Concrete Low-Head Pressure Pipe
C 397	Standard Practice for Use of Chemically Setting Chemical-Resistant Silicate and Silica Mortars
C 497	Standard Test Methods for Concrete Pipe, Manhole Sections, or Tile
C 822	Standard Terminology Relating to Concrete Pipe and Related Products
C 939	Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method)
D 638	Standard Test Method for Tensile Properties of Plastics
D 653	Standard Terminology Relating to Soil, Rock, and Contained Fluids
D 698	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 [600 kN-m/m^3])
D 790	Standard Test Methods for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials
D 883	Standard Terminology Relating to Plastics
D 1556	Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method
D 2216	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D 2447	Standard Specification for Polyethylene (PE) Plastic Pipe, Schedules 40 and 80, Based on Outside Diameter
D 2657	Standard Practice for Heat Fusion Joining of Polyolefin Pipe and Fittings
D 2922	Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)
D 3017	Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)
D 3035	Standard Specification for Polyethylene (PE) Plastic Pipe (DR-PR) Based on Controlled Outside Diameter
D 3261	Standard Specification for Butt Heat Fusion Polyethylene (PE) Plastic Fittings for Polyethylene (PE) Plastic Pipe and Tubing

D 3350	Standard Specification for Polyethylene Plastics Pipe and Fittings Materials
D 4221	Standard Test Method for Dispersive Characteristics of Clay Soil by Double Hydrometer
D 4253	Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table
D 4254	Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density
D 4647	Standard Test Method for Identification and Classification of Dispersive Clay Soils by the Pinhole Test
D 5813	Standard Specification for Cured-In-Place Thermosetting Resin Sewer Piping Systems
D 6572	Standard Test Methods for Determining Dispersive Characteristics of Clayey Soils by the Crumb Test
E 165	Standard Test Method for Liquid Penetrant Examination
F 412	Standard Terminology Relating to Plastic Piping Systems
F 585	Standard Practice for Insertion of Flexible Polyethylene Pipe Into Existing Sewers
F 714	Standard Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Outside Diameter
F 905	Standard Practice for Qualification of Polyethylene Saddle-Fused Joints
F 1216	Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube
F 1743	Standard Practice for Rehabilitation of Existing Pipelines and Conduits by Pulled-in-Place Installation of Cured-in-Place Thermosetting Resin Pipe (CIPP)

Websites

The following websites can provide additional information and publications related to conduits and embankment dams:

American Society of Civil Engineers: www.asce.org
Association of State Dam Safety Officials: www.damsafety.org
Bureau of Reclamation: www.usbr.gov
Bureau of Reclamation Publications:
www.usbr.gov/pmts/hydraulics_lab/pubs/index.cfm
Canadian Dam Association: www.cda.ca
Federal Emergency Management Agency: www.fema.gov/fima/damsafe/resources
Federal Energy Regulatory Commission: www.ferc.gov/industries/hydropower.asp
International Commission on Large Dams: www.icold-cigb.net
Mine Safety and Health Administration: www.msha.gov
National Performance of Dams Program: npdp.stanford.edu
Natural Resources Conservation Service: www.nrcs.usda.gov/technical/eng
Natural Resources Conservation Service Publications: www.info.usda.gov/ced
U.S. Army Corps of Engineers: www.usace.army.mil
U.S. Army Corps of Engineers Publications:
www.usace.army.mil/inet/usace-docs/eng-manuals
United States Society on Dams: www.ussdams.org

Introduction

Conduits convey water from a reservoir through, under, or around an embankment dam in a controlled manner. Conduits through embankment dams serve a variety of purposes. Conduits typically convey releases for:

- Releasing stored waters to meet downstream requirements
- Providing emergency reservoir evacuation
- Flood control regulation to release waters temporarily stored in flood control space
- Diverting flow into canals or pipelines
- Providing flows for power generation
- Satisfying a combination of multipurpose requirements
- Stream diversion during construction

Most conduits through embankment dams are part of outlet works systems. However, some conduits act as either primary or service spillways; auxiliary or secondary spillways to assist the primary spillway structure in passing floods; or power conduits (penstocks) used for the generation of power. Conduits can be classified as either:

- *Nonpressurized flow*.—Open channel flow at atmospheric pressure for part or all of the conduit length (figure 1). This type of flow is also referred to as "free flow."
- *Pressure flow*.—Pressurized flow throughout the conduit length to the point of regulation or control or terminal structure (figure 2)

Many types of materials have been used for conduits over the years, such as reinforced cast-in-place and precast concrete, thermoplastic and thermoset plastic, cast and ductile iron, welded steel, corrugated metal, and aluminum. Some early builders of conduits used whatever materials were readily available, such as wood (figure 3) and hand-placed rubble masonry (figure 4). Regardless of the material use, a conduit represents a discontinuity through an embankment dam and its foundation.

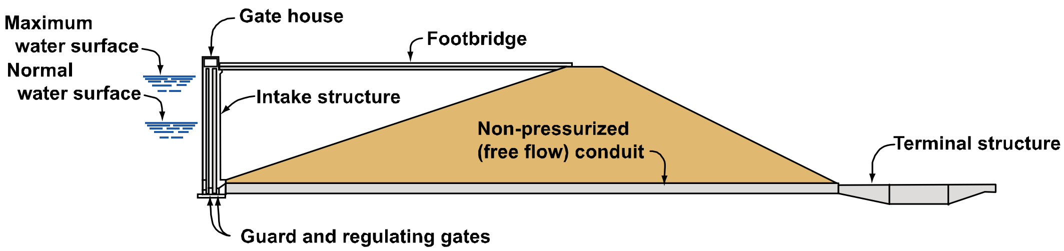


Figure 1.—Nonpressurized outlet works.

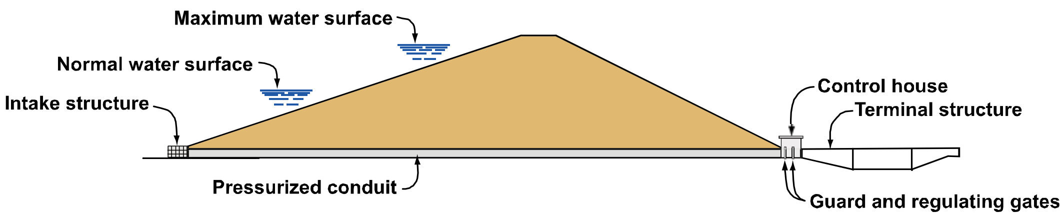


Figure 2.—Pressurized outlet works.

This discontinuity can cause settlement to be different adjacent to the conduit than it is in the rest of the embankment dam. Earthfill may also be compacted differently around a conduit than for the rest of the embankment dam. These factors can cause cracking of the earthfill and lead to other consequences. Failures of embankment dams caused by the uncontrolled flow of water through the dam or foundation are a common problem. A conduit can develop defects from deterioration, cracking from foundation compressibility, or joint separation due to poor design and construction. Water leaking from defects in conduits can contribute to seepage pressures exceeding those that occur solely from flow through soils in the embankment dam from the reservoir. When preferential flow paths develop in the earthfill through which water can flow and erode the fill, severe problems or breaching type failures often result. The reasons that conduits contribute to these failures are discussed more extensively in several sections of this document.

Historically, a single term, "piping," has been commonly used in literature to describe all erosional processes involved in embankment dam failures. The reason for this is that frequently after a failure, a tunnel-shaped feature resembling a pipe is observed. See figure 5 for an example. In this document, two terms will be used to describe failures of embankment dams associated with uncontrolled flow of water, rather than using a single generic term. The two terms that will be used in this document are:

- Backward erosion piping and

- Internal erosion

Figure 3.—A 15-inch diameter, wire-wrapped wood stave pipe used as an outlet works conduit within a 75-year old embankment dam. The outlet works conduit was removed and replaced due to deterioration and backward erosion piping concerns.

Figure 4.—A 100-year-old, mortar-lined, rubble masonry outlet works conduit.

Figure 5.—Embankment dam failure caused by internal erosion of earthfill near the conduit. Flow was not directly along the contact between earthfill and conduit, but in the earthfill away from conduit. Hydraulic fracture in highly dispersive clay embankment soils caused the failure. The embankment design included antiseep collars, but not a filter diaphragm.

These two terms are more descriptive of the distinctly different mechanisms by which water can damage embankment dams. In this document, the term "backward erosion piping" will be reserved to describe conditions where water flows not through preferential flow paths, such as cracks in the soil, but through the pores of a soil. The flow causing the mechanism of failure termed "backward erosion piping" is solely that from intergranular flow causing excessive seepage forces at an exit face. These seepage forces cause a boil condition or particle detachment at an exit face, if it is not protected by a properly designed filter. The term "backward erosion piping" is used in an attempt to define this precise condition of failure. The term "internal erosion," discussed in the following paragraph, describes the more common way that water can damage embankment dams, as it flows through cracks, discontinuities at the interfaces between conduits and earthen embankments or their foundations, or other preferential flow paths. Seepage flow for internal erosion is typically concentrated.

The term "internal erosion" will be used in this document to describe all conditions other than "backward erosion piping" by which water flowing through embankment dams or foundations erodes the soils and causes a failure. Internal erosion occurs where water flows through a discontinuity in the embankment dam and/or foundation, and erodes the sides of the crack to enlarge it and cause a failure. The term "internal erosion" is used in lieu of a number of terms that have historically been used to describe variations of this generic process including scour, concentrated leak piping, and others. This term will also be used for another type of condition called suffosion. Suffosion is the type of erosion where the matrix of the soil mass is unstable. When seepage occurs, the finer part of the soil matrix is eroded out, leaving behind a much coarser fraction.

"Backward erosion piping" can develop only in a category of soils susceptible to this mechanism of failures. Certain conditions are required for backward erosion piping to occur, as described by Von Thun (ASDSO, 1996, p. 5), with added conditions suggested by McCook (ASDSO, 2004, p. 8), summarized as follows:

- A flow path/source of water.
- An unprotected exit (open, unfiltered) from which material can escape.
- Erodible material within the flow path that can be carried to the exit.
- The material being piped or the material directly above it must be able to form and support a "roof" or "pipe."
- Water initially flows exclusively within the pore space of soils. This is often termed intergranular seepage. If flow is through macro-features or cracks in the soil or along an interface between the soil and another structure, the term internal erosion is more correct for describing problems that occur.
- The soil through which water is seeping is susceptible to backward erosion piping. The most susceptible soil types are fine sands and silts with little clay and no plasticity. Clays and clayey coarse-grained soils are highly resistant to backward erosion piping. The resistance of clays and clayey coarse-grained soils results from the high interparticle attraction caused by electrochemical forces. Internal erosion mechanisms are responsible for most failures where clayey soils are in the flow path.

Internal erosion may develop any time a discontinuity occurs within an embankment dam that is accessible to water in the reservoir or to water flowing in conduits. Cracks caused by hydraulic fracture of the earthfill, cracks in bedrock that the embankment is in contact with, and other preferential flow paths provide a way that water can erode soils in contact with the feature. Internal erosion is extremely dangerous because of the rapidity in which flow paths can erode, particularly for highly erosive soils, such as low plasticity silts or dispersive clays. Figures 5 and 6 show examples of failure due to internal erosion associated with a conduit through the embankment dam. The terms backward erosion piping and internal erosion are defined in the glossary in this document. To further assist readers in understanding the definition of these two terms in the context of this document, the following illustrations are provided:

Figure 6.—Embankment dam failure caused by internal erosion of earthfill near the conduit. The initial failure was tunnel shaped, but the collapse of the roof of the tunnel resulted in the observed shape of breach. Hydraulic fracture in highly dispersive clay embankment soils caused the failure. The embankment dam design included antiseep collars, but no filter diaphragm.

- Figure 7 illustrates the internal erosion process as a result of a hydraulic fracture through the embankment dam.

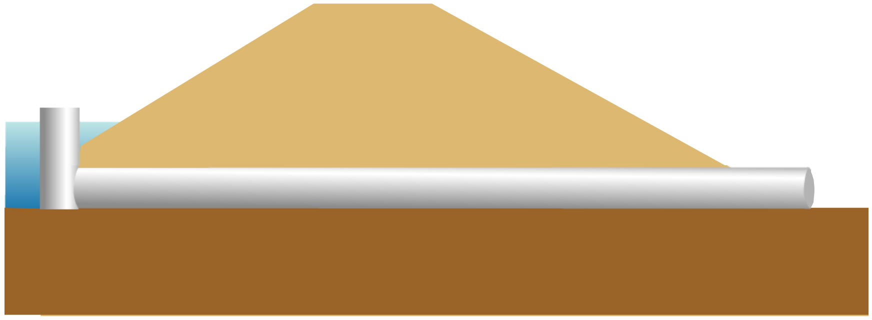
- Figure 8 illustrates the internal erosion process as a result of low density embankment materials under the haunches of a pipe due to poor compaction.

- Figure 9 illustrates the internal erosion process associated with the creation of a void caused by excessive compactive energy used to compact embankment materials against the conduit.

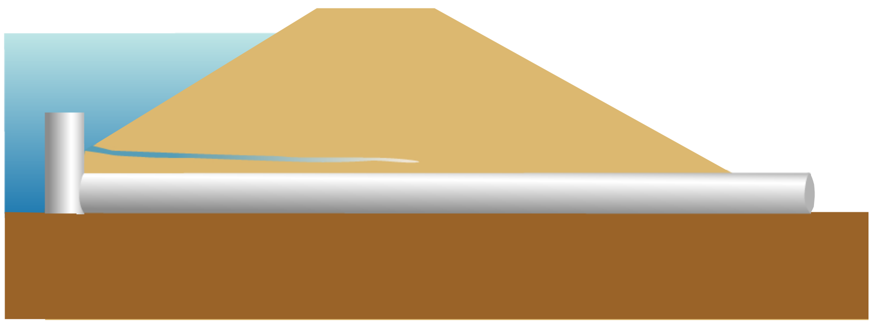
- Figure 10 shows the backward erosion piping process associated with intergranular seepage and the subsequent backward erosion of soil particles.

All four of these mechanisms can lead to partial or full failures of the embankment dam.

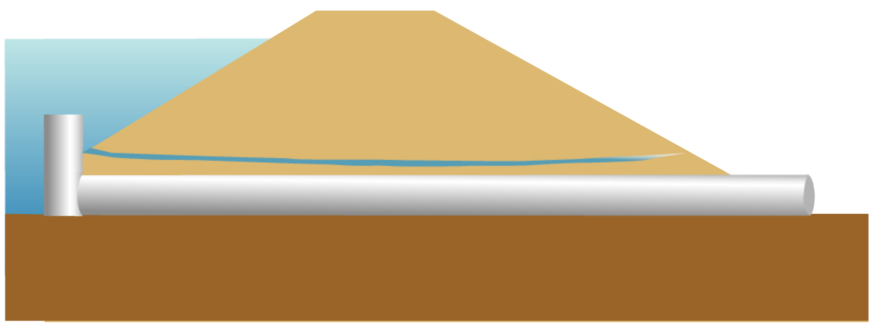
Internal erosion and backward erosion piping can occur suddenly and with little warning. In these cases, little may be done to address the problem quickly enough to avert a failure. Recognizing conditions likely to result in these failure mechanisms is essential to design of conduits and embankment dams that are resistant to failures. In other cases, the failure mechanisms may develop slowly and go unrecognized until the subterranean erosion develops cavities in the embankment dam large enough to



An embankment dam with a low reservoir water level.

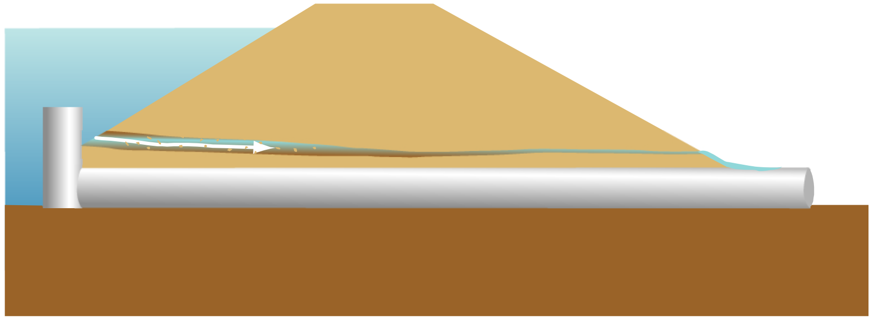


The reservoir water level rises, inducing a hydraulic fracture within the embankment dam due to poor construction or defective soils.

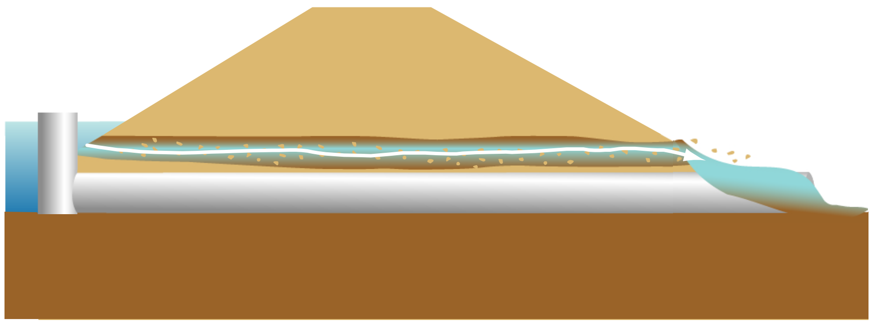


The hydraulic fracture extends through the embankment dam as a result of arching of the overlying embankment, resulting in low stress concentrations in the soil and a reservoir level high enough to cause the fracture. Conduits often create differential settlement and arching of the earthfill, because settlement of the embankment dam is less above the conduit than on either side of it.

Figure 7.—The internal erosion process as a result of a hydraulic fracture.

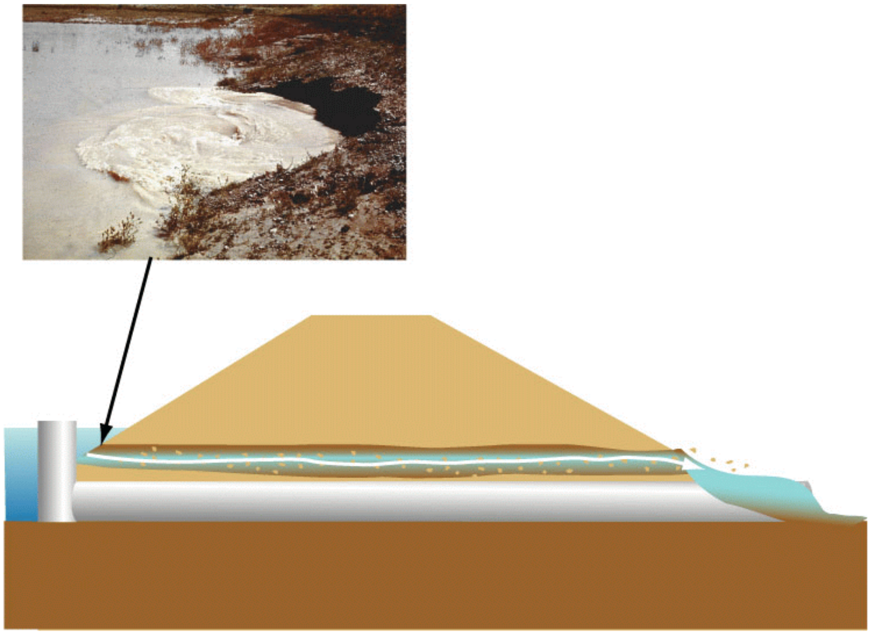


Water from the reservoir penetrates the hydraulic fracture, initiating internal erosion of the side walls of the fracture.

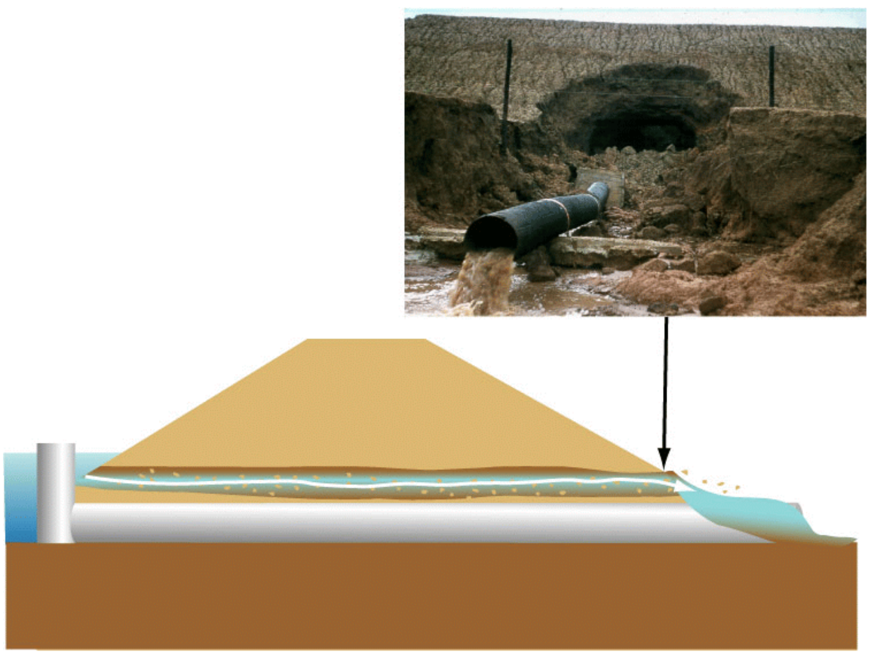


The internal erosion process continues as water flowing in the hydraulic fracture widens the walls of the fracture. Intergranular seepage is not involved in the process, and the soils surrounding the fracture are unsaturated. Intergranular seepage rarely has time to develop, since this type of failure occurs most frequently on first filling of the reservoir.

Figure 7 (cont'd).—The internal erosion process as a result of a hydraulic fracture.

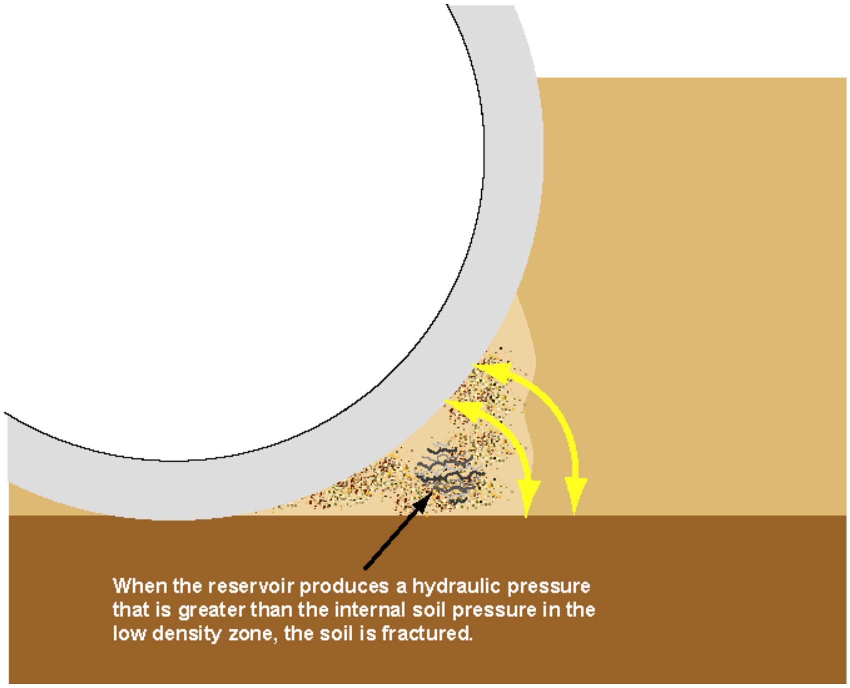


A vortex may form at the location where water in the reservoir enters the upstream end of the hydraulic fracture.

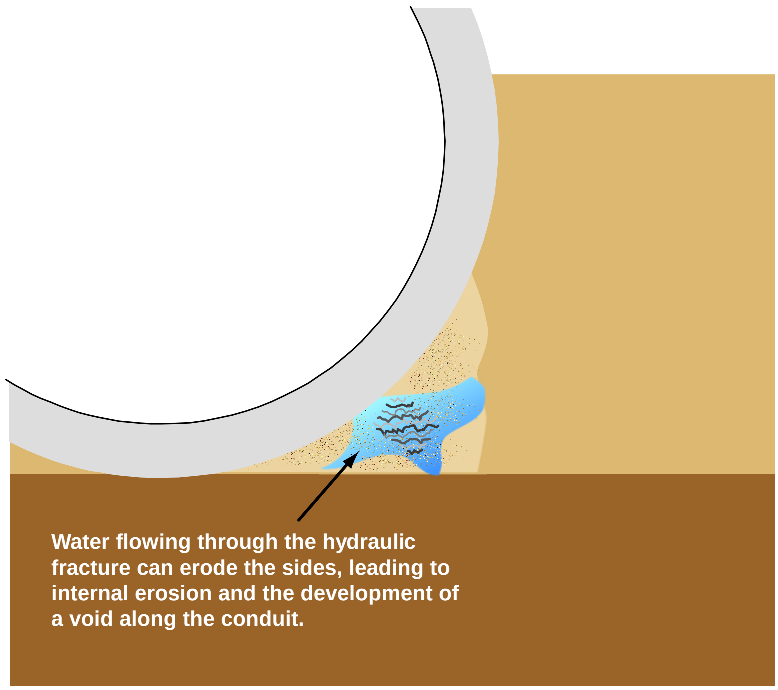


Often, the end result of internal erosion along the hydraulic fracture is a tunnel-shaped void (see figure 5). Loss of the reservoir contents can occur by water flowing through the tunnel-shaped void. Where the tunnel-shaped void enlarges sufficiently (see figure 5), the roof of the tunnel collapses, leaving a v-shaped notch in the embankment dam like that shown in figure 6. If the reservoir had contained a little more storage and the flow had continued a little longer through the tunnel-shaped void, the embankment dam shown in figure 5 would have collapsed and looked similar to the embankment dam in figure 6.

Figure 7 (cont'd).—The internal erosion process as a result of a hydraulic fracture.

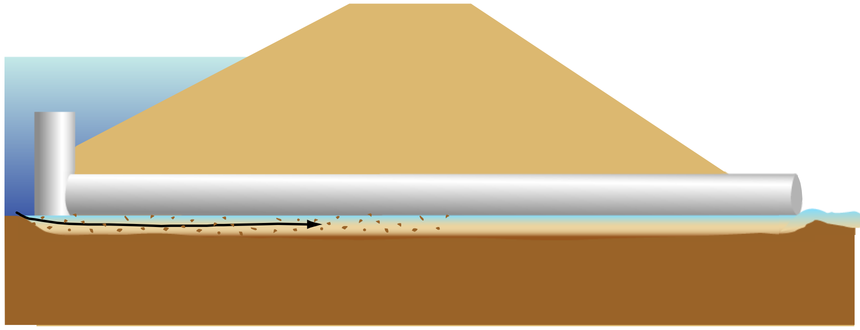


Poor compaction causes arching to occur in the area of the conduit's haunches. This creates a low density zone subject to hydraulic fracture.

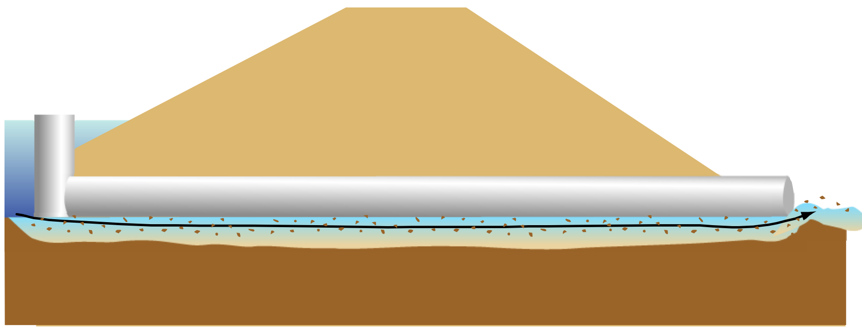


The hydraulic fracture can propagate in a downstream direction and initiate flowing water from the reservoir.

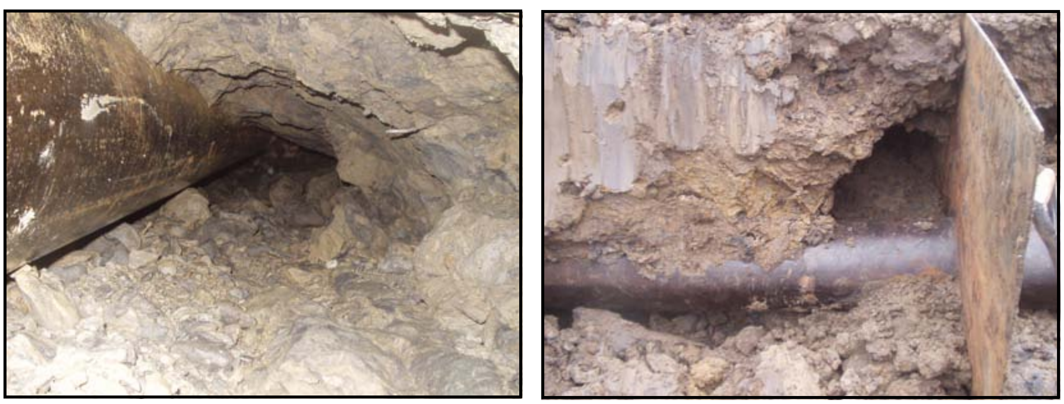
Figure 8.—The internal erosion process as a result of low density embankment materials under the haunches of a pipe due to poor compaction.



Water from the reservoir penetrates the hydraulic fracture, initiating internal erosion of the side walls.

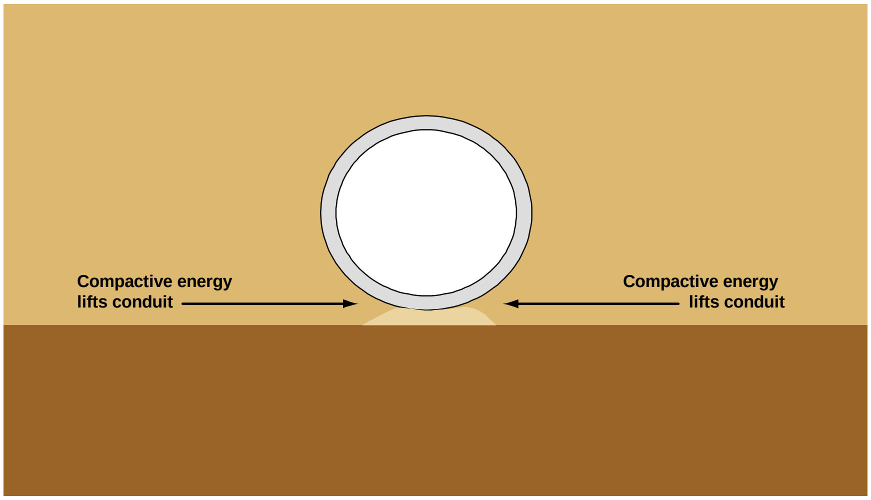


Flowing water from the reservoir continues the internal erosion process within the hydraulic fracture. Intergranular seepage is not necessarily involved in this process, and the embankment materials surrounding the void may be unsaturated.

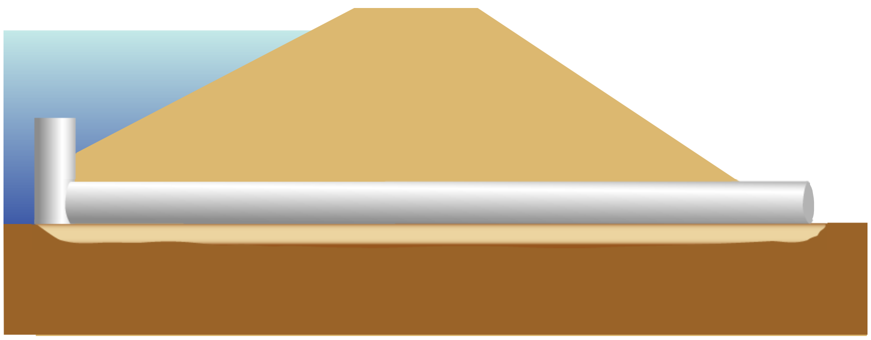


A void was formed along the conduit due to water flowing through a hydraulic fracture.

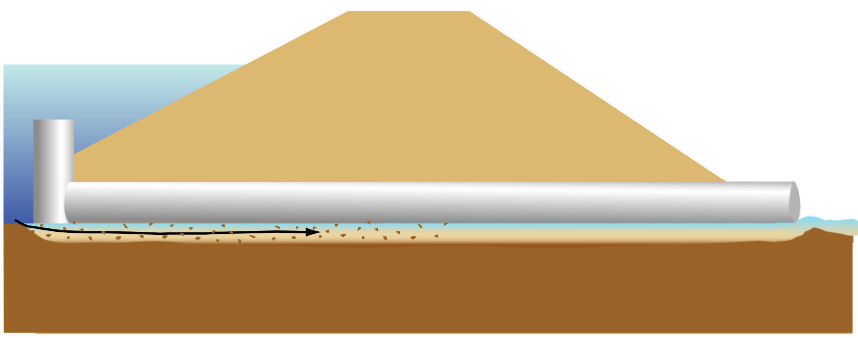
Figure 8 (cont'd).—The internal erosion process as a result of low density embankment materials under the haunches of a pipe due to poor compaction.



If too much compactive energy is applied while attempting to compact the embankment materials under the haunches of the conduit, a void can occur beneath the conduit.

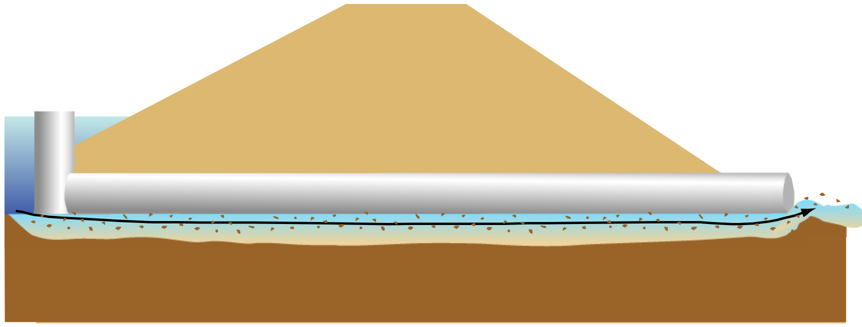


The void can extend beneath the entire length of the conduit.



Water from the reservoir can penetrate the void, initiating internal erosion of the side walls.

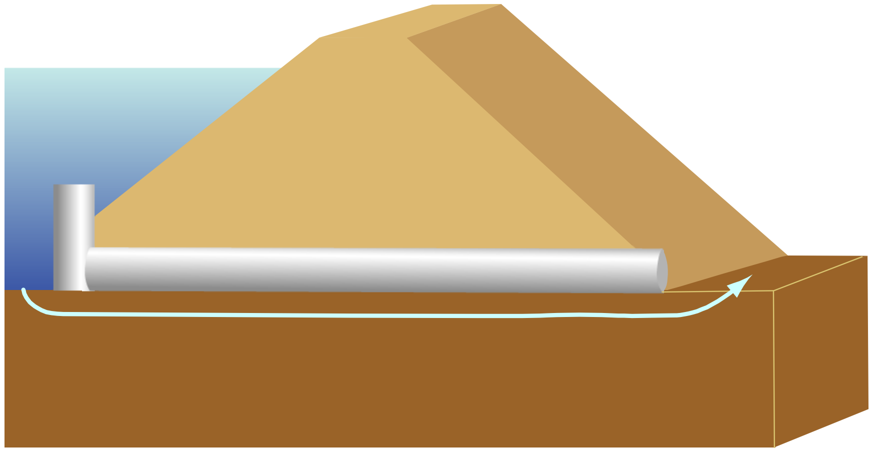
Figure 9.—The internal erosion process associated with the creation of a void caused by excessive compactive energy used to compact embankment materials against the conduit.



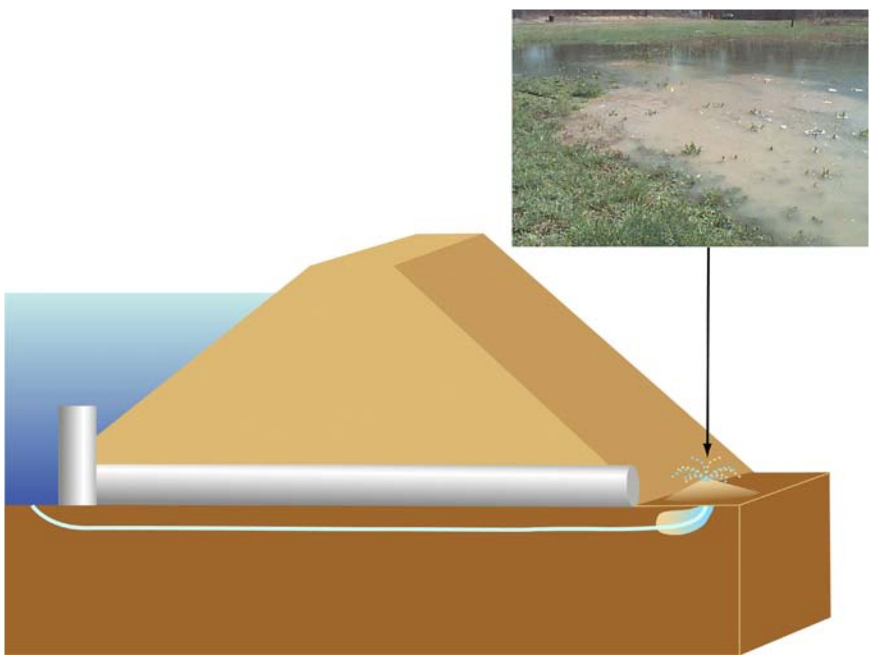
Flowing water from the reservoir continues the internal erosion process within the void. Intergranular seepage is not necessarily involved in this process, and the soils surrounding the void may be unsaturated.

The resulting failure often leaves a tunnel-shaped void along the conduit.

Figure 9 (cont'd).—The internal erosion process associated with the creation of a void caused by excessive compactive energy used to compact embankment materials against the conduit.

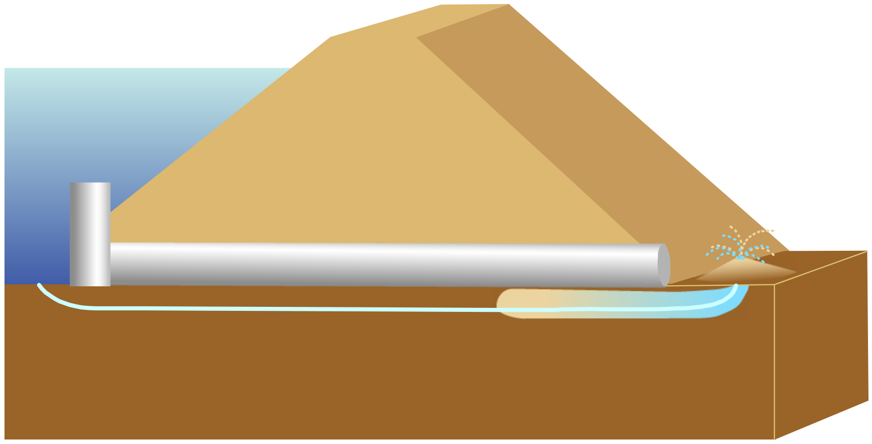


Backward erosion piping begins in fine sand foundation materials at the toe of an embankment dam. The foundation is assumed to be a soil susceptible to piping such as fine, poorly graded sand. A filter zone is not provided at the seepage discharge face (or the filter has been improperly designed), allowing backward erosion piping to begin.

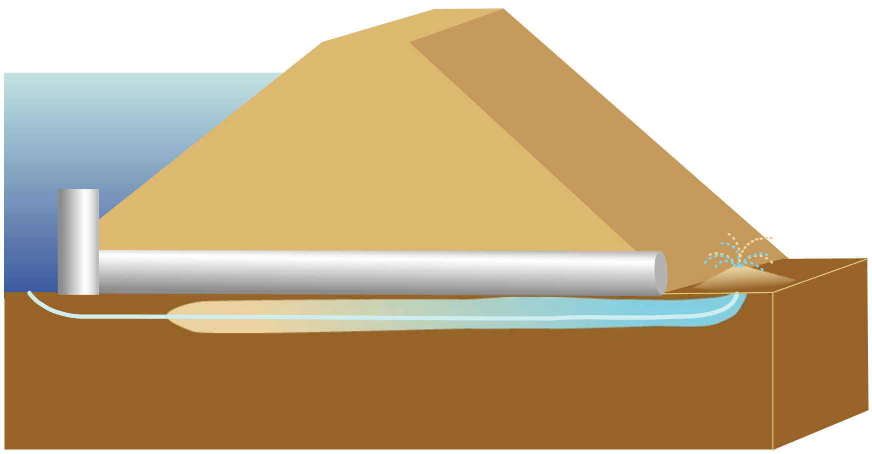


Intergranular seepage flow conditions exist within the foundation under the embankment dam, and soil particles are removed. The particles are deposited (sand boils) on the ground surface at the downstream toe, or washed away if flow is higher.

Figure 10.—The backward erosion piping process associated with intergranular seepage and the subsequent backward erosion of soil particles.

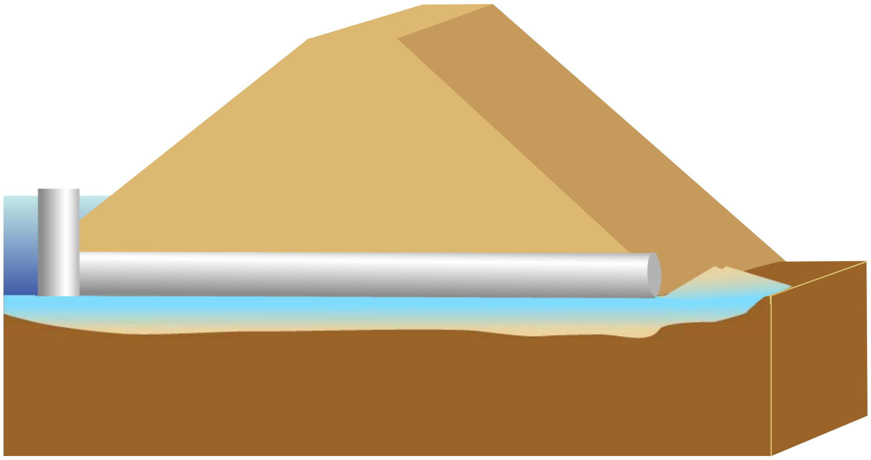


The process of dislodging soil particles continues at an escalating rate because of the hydraulic gradient increase. As the backward erosion piping gets closer to the reservoir, seepage quantity also increases.



A tunnel develops due to the continued erosion of the soils in the foundation. This assumes the overlying embankment dam, foundation layer, or conduit is able to support the tunnel that is forming. The soil exposed to flow in the developing tunnel is erodible, and the walls of the tunnel can grow larger at the same time that the discharge face moves upstream.

Figure 10 (cont'd).—The backward erosion piping process associated with intergranular seepage and the subsequent backward erosion of soil particles.



Eventually, the tunnel erosion feature reaches the reservoir. Outflow will then increase substantially, leading to direct erosion of the embankment dam and complete breach or draining of the reservoir through the tunnel that develops.

The resulting failure often completely destroys the embankment dam, leaving few traces of the original piping tunnel. The failure of this embankment dam was attributed to piping of foundation sands. Photo courtesy of National Oceanic and Atmospheric Administration.

Figure 10 (cont'd).—The backward erosion piping process associated with intergranular seepage and the subsequent backward erosion of soil particles.

be observable at the surface of the embankment or foundation. Visual inspection, seepage and turbidity measurements, and pore pressure readings are useful in detecting whether problems like these may be developing in an embankment dam. Chapter 9 discusses inspection techniques in detail.

Design and construction inadequacies are often to blame for internal erosion and backward erosion piping incidents. Designers must understand which design measures are effective in preventing these mechanisms of failure.

In nonpressurized conduits, water seeping through the embankment dam can enter the conduit through defects. If the surrounding soils are susceptible to backward erosion piping, cavities can develop in the embankment and foundation of the conduit. This problem is discussed in more detail in section 7.1.

In pressure flow conduits, water under pressure can escape through defects and damage the surrounding embankment and foundation. This problem is discussed in more detail in section 7.2.

In nonpressurized or pressurized conduits, water seeping along the interface between the conduit and surrounding soil may be concentrated enough to result in backward erosion piping, if the soils are susceptible. If the soils are resistant to backward erosion piping, but a crack or potential flow path develops near the conduit, internal erosion can result. This problem is discussed in more detail in section 7.3.

If the soils surrounding the conduit are resistant to backward erosion piping, hydraulic fracture may occur. The hydraulic fracture created can then erode and lead to a failure tunnel that is similar to that which develops in soils that are susceptible to backward erosion piping. This problem is discussed in more detail in section 7.4.

Internal erosion and backward erosion piping incidents are often associated with conduits through embankment dams. The following factors increase the likelihood of these problems developing at a given site:

- Conduits constructed across abruptly changing foundation conditions (i.e., a concrete core wall or bedrock with a quickly changing profile) are more likely to experience differential settlement. See section 1.2 for more discussion on factors in locating conduits in the most favorable conditions.

- Circular conduits constructed without concrete bedding or cradles are more likely to experience problems than conduits in more favorable shapes (i.e., horseshoe). See section 4.1 for more discussion on conduit shapes.

- Conduits with an excessive number of joints are more likely to develop defects that can lead to problems. See section 4.3 for discussion on joints in conduits.

- Excavations made to replace unsuitable foundation materials for conduits increase the potential for differential settlement problems. Section 5.1 also discusses this factor.

- Conduits with compressible foundations are more likely to deform excessively, which may damage the conduit. Compressible foundations may also contribute to differential settlement that can result in hydraulic fracture of the earthfill surrounding the conduit. Section 5.1.2 discusses soil foundations for conduits. Locating conduits on bedrock is desirable, but not always practical. See section 1.2 for more discussion on factors in locating conduits in the most favorable conditions.

- Conduits located in closure sections in embankment dams contribute to differential settlement problems. Section 5.2 discusses this factor in detail.

- Embankment dams constructed with materials susceptible to internal erosion or backward erosion piping. Sections 5.2 and 5.3 discuss this factor in detail.

- Conduits constructed without adequate compaction around the conduit are more likely to experience erosional problems. Section 5.3 discusses this factor in detail.

- Embankment dams constructed without a chimney filter or conduits constructed without a filter collar or filter diaphragm. See chapter 6 for more discussion on the design and construction of filters.

- Conduits constructed of materials susceptible to deterioration, such as corrugated metal pipe. See chapter 8 for discussion of defects in conduits.

Understanding the steps involved in a failure mode as the result of internal erosion or backward piping erosion is important in designing defensive measures to prevent these failures. An event tree can be used to understand the series of events that can lead to embankment dam failure by internal erosion or backward erosion piping. An event tree used by the Bureau of Reclamation (Reclamation, 2000, p. 15) for internal erosion of an existing embankment dam is shown in figure 11. The steps or "nodes" of the event tree shown on figure 11 are generally described as follows:

1. The reservoir rises, causing a water load on the embankment dam. The information is generally derived from the statistical historic record of reservoir operations. Normally, it is the probability of a reservoir to rise onto a portion of the embankment dam that might contain a flaw and not usually the time that the reservoir exists at a specific elevation.

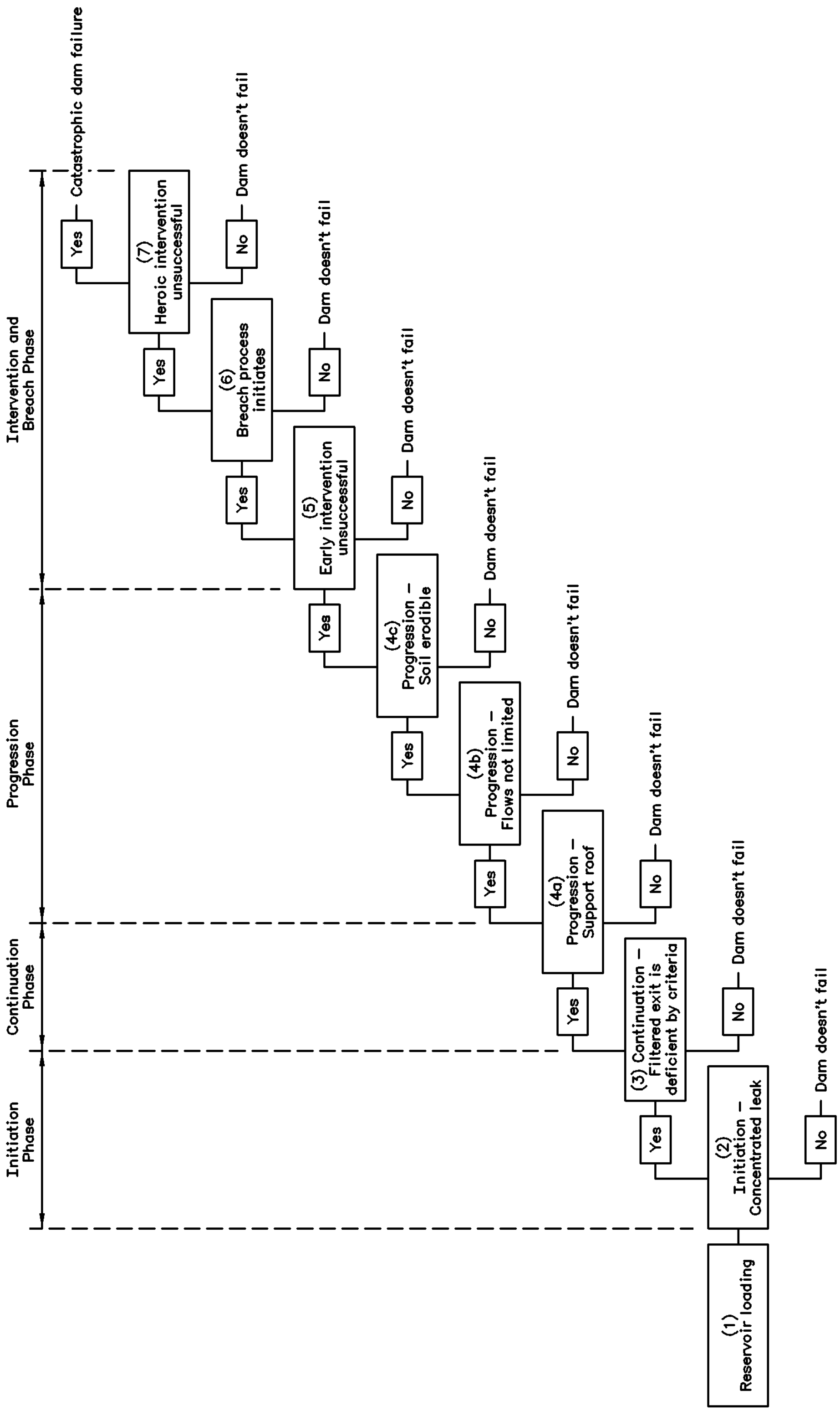


Figure 11.—Example of an event tree used to illustrate the internal erosion process.

2. The next node of the event tree considers the potential for a concentrated leak to exist or newly occur. The leak must be of sufficient size to reasonably expect the soil erosion process to begin.

3. The next node then considers if the erosion process continues. This is usually done by assessing the potential for an adequate filter to exist at the downstream end of the leak.

4. If there is no reasonable expectation of a filter, the potential for the erosion process to progress is examined in the next three nodes by considering (a) the potential for a roof to form over the pipe channel, (b) the potential for an element at the upstream end of the leak to limit flow, and (c) the erosion characteristics of the embankment material.

5. If the erosion process will fully reach the progression stage, the potential to successfully intervene to prevent failure soon after detection of the erosion is considered.

6. If such early intervention will not likely be successful, the potential for the embankment dam to fully breach is considered.

7. If the embankment dam is of a type that can actually breach, the potential to heroically intervene to save the dam is examined (e.g., the potential to quickly lower the reservoir). The culmination of a negative outcome of all the events in the event tree is the catastrophic release of the reservoir.

This event tree is usually used by Reclamation to assess the potential for the failure of an existing embankment dam in a risk context. In a risk assessment of an internal erosion failure mode, a probability of the event tree would be estimated and this would be multiplied by some consequence of the embankment dam failure, usually life loss. The event tree was developed for the internal erosion failure mode only. An event tree for a failure mode of backward erosion piping might be slightly different than this one. For instance, instead of the potential for a concentrated leak, the initiation node might evaluate the potential for a high exit gradient to begin the erosion process.

For a new embankment dam being designed, understanding the events that can lead to failure by internal erosion or backward erosion piping can lead to improvements in the design. As most of the steps of the process are considered, opportunities for multiple lines of defense in the design can be developed.

Compilations of case histories of embankment dam failures and accidents show that frequently, conduits were considered a factor in the failures or accidents. Case histories such as those shown in appendix B are examples of embankment dams where conduits were associated with failures and accidents. The case histories in appendix B include a variety of situations where defects in the conduit and poor design or construction decisions contributed to the failures. Several modes of failure are discussed in this document related to conduits, which include both the backward erosion piping and internal erosion modes of failure. Appendix B includes very few case histories that involve backward erosion piping associated with conduits. This is because soils that are highly susceptible to backward erosion piping have seldom been used to backfill around conduits. Most case histories of failures and accidents involving conduits are related to one of the internal erosion modes of failure. Both this *Introduction* and chapter 7 provide more discussion on modes of failure.

In 1998, a survey of State dam safety programs was conducted for the Interagency Committee on Dam Safety (ICODS) Seminar No. 6 on piping associated with conduits through embankment dams (Evans, 1999, p. 1). Fourteen states provided responses to the survey. The respondents indicated that 1,115 embankment dams with conduits would likely need repair within the next 10 years. Of these 1,115 embankment dams, 53 percent had conduits constructed with corrugated metal pipe (CMP), 23 percent were constructed with steel pipe, and 20 percent were constructed with concrete pipe.

Conduits within embankment dams are often designed using standards not specifically intended for penetrations through dams. For example, certain pressure pipe standards (e.g., those from the American Water Works Association) may not be applicable (without design and construction modifications) for use in pressurized conduits through embankment dams. The purpose and performance characteristics of conduits through embankment dams differ from those required for water supply pipelines. The use of certain types of manufactured pipe for conduits through embankment dams is a concern, since these materials were developed and standardized for applications other than embankment dams. The unique performance requirements for conduits in embankment dams include:

- *Service life*.—Most embankment dams are designed assuming a minimum 100-year service life with minimal maintenance. Manufactured pipe needs to be durable in the expected wet, dry, and freeze/thaw environments found within a conduit.

- *Accessibility*.—As the height of the embankment dam increases, the practicality of accessing the conduit for repairs decreases. Manufacturing and installation quality control needs to be high to ensure dependable installations.

- *Strength*.—The structural loading on manufactured pipes can be very high due to positive projecting, rather than trench loading conditions, and very high embankments. The pipe needs to be structurally designed for all possible loading conditions for applications within embankment dams.

- *Risk*.—The development of small defects within the conduit can lead to serious failure modes threatening the entire embankment dam. Designs needs to be robust and conservative.

- *Movement*.—Conduits within high embankments dams, built on compressible foundations, may experience significant displacement as the dam settles. The conduit joints need to be capable of absorbing such movements while remaining watertight. The conduit placement needs to anticipate subsequent settlement in order to remain positively sloping for gravity drainage.

Inexperienced designers may inadvertently apply inappropriate design standards or misuse design standards to save on time and provide cost savings. Examples of the misapplication of design standards include:

- *Inappropriate design references*.—State highway department standard plans for culverts and culvert structures are sometimes simply referred to in construction specifications and drawings to save the designer from actually designing the conduit. Culvert designs are not intended for use within embankment dams.

- *Inappropriate application of standards*.—The NRCS has developed several standardized conduit and joint detail drawings for use in embankment dams. Such drawings have been used to successfully build thousands of small embankment dams. Such drawings have also been misused. In one known case, the standard detail drawings were used to unsuccessfully install a pressurized conduit on a high hazard embankment dam on a soft foundation. As with all standardized designs and drawings, the design and construction assumptions made in preparing the drawings need to be satisfied for the specific application and site.

- *Inappropriate use of materials*.—Reinforced concrete pressure pipe has been used for pressurized conduits within embankment dams. Reinforced concrete pressure pipe utilizes gasketed joints, which could be subject to leakage, if improperly constructed. In a typical 100-foot high embankment dam there could be over 80 gasketed joints, all with the potential for leakage.

Conduits often penetrate other types of embankment structures or are used for utility purposes. These types of penetrations are not addressed in detail in this document. Some of the guidance presented in this document may apply to these types of penetrations and should be carefully evaluated by designers for implementation. Users of this document will need to evaluate the applicability of the proper guidance to their project. Conduit penetrations not specifically addressed within this document include:

- *Conduits within levees.*—Guidance on conduits through levees is available from other sources, such as the USACE's *Design and Construction of Levees* (2000).

- *Utility conduits.*—Utility conduits are utilized for various functions, such as water, wastewater, sewer, electrical, telecommunications, and gas lines. As urbanization pushes farther out into previously undeveloped or agricultural areas, more utility conduit crossings of embankment dams are being required. While many of the new utility conduits installations are made through low hazard dams, the continued urbanization may make previously low hazard structures become significant or high hazard dams.

  Typically, requests for utility crossings are made to local and State agencies. These agencies provide the necessary review and right-of-way permitting. If at all possible, these conduit crossings should be located outside the limits of the embankment dam, so as not to provide a discontinuity within the dam or a transverse seepage path through the dam.

  Inexperienced designers associated with utility conduits may utilize designs that trench through the embankment dam for new installations or to repair or replace existing conduits and not use proper excavation, backfill, and compaction practices around the conduit. This can lead to failure of the embankment dam. Any utility conduit installation should be designed by a professional engineer experienced in the design and construction of embankment dams. If these conduits must be located within the embankment dam, they should be positioned in the upper crest of the dam, well above the design flood elevation. Typically, this is not a problem, as the utility owner requires permanent access to the conduit. If the utility conduit is for a water line, special precautions should be employed, so that a rupture of the conduit will not continue unchecked and cause erosion of the embankment dam. Such precautions should consider applicable guidance contained within this document, automatic shutoff mechanisms, frequent testing and inspection of the system, and visual monitoring.

  Another concern with utility conduit crossings are unauthorized installations. Embankment dam failures have occurred as the result of unauthorized utility conduit installations where no notice was given to the responsible agency for proper review and right-of-way permitting. All embankment dams should be marked with "no trespassing" signs. Unfortunately, these signs are often ignored during unauthorized utility conduit installations.

  One alternative to burying the utility conduit crossing within an embankment dam is to construct the crossing over the crest of the dam. This alternative has been successfully used by USACE for conduit crossings over levees and is best suited for small diameter pipes. To accomplish this alternative, additional earthfill is added on the crest of the dam, so a ramp is constructed over the utility conduit to allow for the crossing of vehicular traffic. Typically, a minimum of 2 feet of cover is provided over the conduit and a 6-percent grade is utilized on the ramps. Additional earthfill is ramped around the utility conduit on both the upstream and downstream faces of the embankment dam as needed to provide protective cover. The grade on these ramps is usually about 10 percent. This alternative eliminates any concerns associated with excavation into the embankment dam.

- *Conduits within tailings and slurry impoundment dams.*—This document is intended to apply to traditional embankment type dams. The design and construction of conduits through tailings and slurry impoundments often utilize different guidelines than those presented in this document.

  Tailings and slurry dams are an integral and vital component of mining operations. Tailings dams permanently retain mining, chemical, and industrial waste products (e.g., ground-up rock that remains after the mineral value has been removed from the ore). Figure 12 shows an example of a tailings dam. A slurry dam permanently retains waste created by the processing and washing of coal. These structures retain the waste products and allow them to settle out, enabling reclamation (recycling) of the slurry water, and permanent retention and eventual restoration of the site.

  The coarser fraction of the waste material is commonly used to construct the dam, with the finer waste being pumped as slurry behind the dam. Typically, tailings and slurry dams are constructed over the life of the mine, with the dam being raised as needed to provide additional disposal capacity. The dams may be raised by downstream, upstream, or centerline construction. In many cases, the dams reach several hundred feet in height.

  As with any dam, an important aspect of these impoundments is handling water, in this case both storm runoff and water pumped in with the slurry. Some of the dams are totally diked structures while others have contributing watersheds. Often the impoundment water is reclaimed for use in processing or in other mining activities. The seepage from these impoundments can cause chemical deterioration due to its acidity or alkalinity. In some cases, the nature of the leachate requires that the impoundment be provided with an impervious liner. A "decanting system" typically removes free water from behind the dam. Designers use a variety of methods and materials to decant water from slurry and tailings dams. Decanting systems often consist of an extendable intake structure (e.g., tower or sloping chute) and a conduit to convey discharge away from the tailings dam. Figure 13 shows an example of an intake structure for a decanting system. The intake structure is normally constructed progressively as the deposition level rises to avoid the costs of a high, unsupported structure

Figure 12.—An example of a tailings dam.

before the impoundment is constructed. Reinforced cast-in-place concrete, precast concrete, steel, and plastic conduits have all been used. Some designers prefer to avoid having a conduit pass through the dam and use either floating pump installations or siphons to decant the water. Use of these options is especially favored in areas where the impoundment can be located high in the watershed to minimize runoff inflow, and in areas, such as portions of the western United States, where rainfall is low. Designers also cite the advantage in this approach of eliminating potential problems with decant conduit risers, such as structural stability and debris clogging.

Some tailings dams are required to be provided with impervious liners due to the acidity of the leachate. In these cases, if a decant conduit is used, a watertight connection must be achieved between the liner and the conduit. The presence of the liner affects these installations by limiting the potential for seepage along the conduit.

Some tailings dam failures and problems have been attributed to problems with compaction of the backfill around the decant conduit. A notable occurrence was a failure at a phosphate tailings dam in Florida in 1994. While this case involved CMP, it highlighted the difficulty in obtaining adequate compaction in the haunch area under the pipe. Postfailure investigation of two other decants that had been installed at the same facility indicated gaps or loose areas in the haunch area backfill. Interestingly, although plastic and steel pipes have been used extensively in slurry impoundments, no failures are known to have occurred, and only a few problems have been attributed to inadequate haunch area compaction in these applications.

In the past, decant pipes were constructed of CMP. However, in many cases the acidity of the refuse caused corrosion problems. Protective coatings were

Figure 13.—An example of a decant intake structure.

employed to combat this problem, but there also were problems with the watertightness of the joints. This problem became particularly apparent when dam safety regulators began to require pressure testing of the pipes.

As a result of problems with corrosion and joint watertightness, and to deal with increasing fill heights over the pipes, designers turned to two other types of decant pipes: thick-walled welded steel pipe, and high density polyethylene pipe (HDPE). Pipes were often designed to withstand the fill height loading from several stages of construction, and the pipe would be replaced by installing another pipe at a higher elevation, with the original pipe being filled with grout.

Designers considered corrosion-protected welded steel and HDPE pipes to be beneficial for the type of foundation conditions and construction practices found at these dams. The locations of these dams are limited to areas near the processing plants, meaning that designers need to deal with varied, and often less than ideal, foundation conditions. Furthermore, as these pipes may be extended up- or downstream, their length can become relatively long, sometimes exceeding 1,000 feet. Over such lengths, a flexible pipe could tolerate some differential movement due to varying foundation conditions. Additionally, many of these dams have underground mining in their vicinity and the possibility of mining-induced ground movement needs to be considered.

The pipes used for the decanting systems have typically been installed without being encased in concrete. In most cases, the pipes have been installed with hand compaction of the backfill in the haunch area. Hand held compaction equipment has often been used. Flowable fill has been used in a few cases. A more recent practiced has been to place the pipe high in the dam cross section, so that it is above the normal phreatic level; then the pipe is grouted and replaced by another pipe when the next stage is constructed. A concern for a pipe placed high in the dam is that a large storm could result in a raised phreatic level which may subject the pipe to a situation analogous to "first filling" of the dam. That is, a problem with seepage along the pipe may only become evident at a critical time with respect to the amount of water stored in the impoundment.

In an attempt to address potential problems with seepage along a conduit, older installations made use of antiseep collars. In more recent years, filter diaphragms have been installed. In spite of installing the pipes with hand compaction of the haunch area, a practice that has led to problems in other applications, no significant problems have been attributed over the last 25 years to piping or excessive seepage through the backfill of a decant conduit for slurry impoundments.

The International Commission on Large Dams (ICOLD) has prepared a number of technical publications (Bulletins Nos. 44, 74, 97, 98, 101, 103, 104, 106, and 121 [1989a, 1989b, 1994b, 1995a, 1995b, 1996a, 1996b, 1996c, and 2001]) related to the design, construction, and operation of these types of dams (many of these have been developed in partnership with the United Nations Environment Programme [UNEP]).

Tailings and slurry dams have inherent differences compared to embankment dams used for the storage and control of water. The reasons that tailings and slurry dams do not fit within the normal context of "embankment" dams include (see ICOLD publications for further information):

1. They are designed to be abandoned and not operated.

2. Construction is usually simultaneous with its operation.

3. Generally constructed with mill tailings, mine waste, and earth- or rockfill.

4. The primary use is the disposal of waste and slurry from the processing operations. They usually impound water only for sedimentation, reclamation, and mill operation. Water retention is considered to be incidental to their intended operation of waste disposal.

5. The waste is typically discharged along the upstream slope of the dam, forming a delta of settled fines, with the water pushed back away from the dam.

6. The hydraulic gradient existing within the dam is typically less than that existing within in a traditional embankment dam.

7. These dams often are required to impound and release water with a low pH, which can cause corrosion and deterioration of the conduit.

8. Typically, the free water that is impounded is a small percentage of the total stored volume. The majority of the stored volume is hydraulically deposited fine waste.

9. These dams typically cannot be breached at the end of their useful service life and the reservoir area returned to its original condition. These dams must retain their waste products for hundreds of years.

10. These dams are often raised many times to stay ahead of the rising impoundment water.

11. These dams are normally subjected to only a nominal amount of drawdown of free water.

12. The settled fines typically provide a low permeability zone, which acts to restrict seepage.

13. Due to the much larger mass of these dams, decant conduits are generally much longer than conduits in traditional water storage dams. Some dams have conduits over 1,000 feet in length.

As a result of these factors, the performance experience indicates that the combination of hydraulic gradient and backfill material characteristics may have been sufficient to prevent internal erosion and backward erosion piping problems. Also, it may be possible that particles of fine waste carried with the seepage act to choke off potential seepage paths. Experience has shown, for example, that moving the discharge point of the slurry to a point upstream of a localized seepage area is often effective in eliminating the seepage.

Since these types of dams are raised concurrent with disposal, and construction occurs over the life of the mine, which could be a few years to over 30 years, these dams provide a unique opportunity to monitor the structural performance of decant conduits. In applications where the height of fill proposed over the conduit creates concerns about pipe deflection, deflection is typically monitored at various intervals of fill height. Based on these measurements, parameters affecting deflection, such as the modulus of soil reaction (E') can be back-analyzed, and future pipe deflection, and the point at which remedial actions may be required, can be better modeled and estimated.

The "best practices" provided in this document should be applied to the decant conduits installed in tailings and slurry dams. However, these best practices creates a dilemma in the case of tailings and slurry dams. As previously discussed, there are benefits to having a conduit that can tolerate some deformation in these dams. Furthermore, these impoundments do not typically have an "impervious core," and the added cost of reinforced cast-in-place conduits concrete is not as suitable for the shorter life of impoundment conduits, as compared to conduits through traditional embankment type dams. While the absence of significant problems does not rule out future problems, the existing record does provide some indication that alternatives to concrete encasement may be reasonable in tailings and slurry impoundment applications. The following recommendations are provided for installing conduits in these types of dams:

1. Although extensive problems have not been encountered with decant pipes through these dams, good conduit design and installation practices need to be followed. A primary recommendation is that designers recognize the large body of evidence that indicates that adequate compaction cannot be achieved in the haunch area by conventional hand held compaction methods. Using these methods, full contact between the pipe and the backfill cannot be ensured. For guidance on compaction, see section 5.3.

2. Decant conduits should be provided with an adequately designed filter. The filter should extend far enough out from the conduit to intercept areas where cracks may occur due to hydraulic fracturing or differential movement of backfill/embankment materials. For guidance on filters, see chapter 6.

3. The filter should not be considered as an adequate defense, by itself, against problems with seepage along the conduit. The permeability of the backfill material and its level of compaction need to be sufficient to restrict seepage and reduce the hydraulic gradient along the conduit. The filter is intended to collect the limited seepage that occurs through well compacted and suitable backfill. The filter could be overwhelmed and rendered ineffective by excessive seepage.

4. If the pipe is not to be encased in concrete, with sloping sides that allow compaction by heavier equipment, then an alternate construction method, that provides for adequate compaction and full contact in the haunch area, needs to be specified.

5. Use of an alternate construction method should only be considered where it can be shown that the combination of hydraulic gradient and backfill

material characteristics indicate adequate protection against internal erosion and backward erosion piping.

6. Whatever conduit installation method is to be used, the specifications should include a detailed step-by-step procedure for installing the conduit and for achieving full contact with the bedding and backfill. The type of equipment to be used to achieve the specified backfill densities should be specified. Quality control during construction should be the responsibility of a registered professional engineer who is familiar with the project specifications and the potential problems. The specifications should indicate how it will be determined that full contact between the conduit and the backfill has been achieved and the required backfill moisture/density specifications have been met. The engineer should be required to inspect and accept the conduit bedding and backfill before the backfill is placed over the conduit.

Even though these dams differ significantly from embankment type dams, they can experience failure. Regulatory agencies, dam owners, and designers may find application of the guidance provided in this document can improve the overall integrity of their structure. They should fully consider the basis for these best practices and decide on the applicable guidance to use. Where the design and construction of a conduit through these types of dams deviates from these best practices, the designer should ensure that potential problems are otherwise accounted for in the design.

Chapter 1

General

Conduits have been placed through embankments for centuries. However, placing a conduit within an embankment dam increases the potential for seepage and internal erosion or backward erosion piping. Water may seep through the earthfill surrounding the conduit, through cracks in the embankment caused by the conduit, or into or out of defects (e.g., cracks, deterioration, or separated joints) in the conduit. If the conduit is flowing under pressure, and defects exist in the conduit, the water escaping the conduit can erode surrounding soils.

Replacement of a conduit through an embankment dam is difficult, time consuming, and expensive. Designers should adopt a conservative approach for the design of conduits. The purpose of this chapter is to provide guidance for both constructing new conduits and renovating or replacing existing conduits in embankment dams. When evaluating existing conduits, designers should attempt to determine how closely the design of the existing conduit complies with criteria for new conduits. If the existing conduit lacks state-of-the-practice defensive design measures, it may be considered inadequate by modern standards. These design measures should provide both primary and secondary defensive measures to reduce the probability of failures. Inadequate conduit designs, poor construction, and improper maintenance can adversely affect the safety of embankment dams.

1.1 Historical perspective

Most designers of embankment dams have attempted to include defensive design measures to address potential seepage along conduits extending through earthfill or earth- and rockfill embankments. Even so, many observed failures and accidents of embankment dams have occurred, involving conduits or the earthfill near the conduits. For large embankment dams, about one-half of all failures are due to internal erosion or backward erosion piping. In about one-half of these failures, internal erosion or backward erosion piping was known to have initiated around or near a conduit (Foster, Fell, and Spannagle, 2000, p. 1032). This means that about 25 percent of all embankment dam failures are a result of internal erosion or backward erosion piping associated with conduits.

Until about the mid-1980s, the most common approaches for controlling seepage were antiseep collars (also known as cutoff collars) and careful compaction (special compaction using small hand held compaction equipment) of backfill around conduits. Antiseep collars are impermeable diaphragms, usually of sheet metal or concrete, constructed at intervals within the zone of saturation along the conduit. They increase the length of the seepage path along the conduit, which theoretically lowers the hydraulic gradient and reduces the potential for backward erosion piping.

Antiseep collars were designed primarily to address intergranular seepage (flow through the pore spaces of the intact soil). Antiseep collars did not fully address the often more serious mechanism of failure (internal erosion), that occurs when water flows through cracks and erodes the compacted earthfill near the conduit outside the zone of influence of the antiseep collars in the compacted earthfill near the conduits. In the 1980s, major embankment dam design agencies including the U.S. Army Corps of Engineers (USACE), and the Bureau of Reclamation (Reclamation) discontinued using antiseep collars on conduits for new dams. Reasons why antiseep collars were abandoned include:

- Antiseep collars impeded compaction of soils around the conduit.
- Antiseep collars contributed to differential settlement and created potential hydraulic fracture zones in the fill.
- Designers realized that problems associated with conduits were more likely to be caused from internal erosion mechanisms than from intergranular seepage.
- Designers achieved increased confidence in the capability and reliability of filters to prevent internal erosion failures.
- Antiseep collars can form a foundation discontinuity that could result in differential settlement and cracking of the conduit.

The Natural Resources Conservation Service (formerly Soil Conservation Service, SCS) also discontinued using antiseep collars on new embankment dams, but continues to allow them on small, low hazard dams and only under certain restrictive conditions.

Figures 14 through 16 show examples of the construction difficulties involved with compaction around antiseep collars. Appendix A gives a detailed history of the design rationale used for antiseep collars and reasons for their being discontinued. Figures 17 and 18 show examples of the ineffectiveness of antiseep collars in preventing embankment dam failure resulting from internal erosion near conduits.

Figure 14.—Antiseep collars impeded the compaction of soils around the conduit. Hand tampers were used next to the antiseep collars.

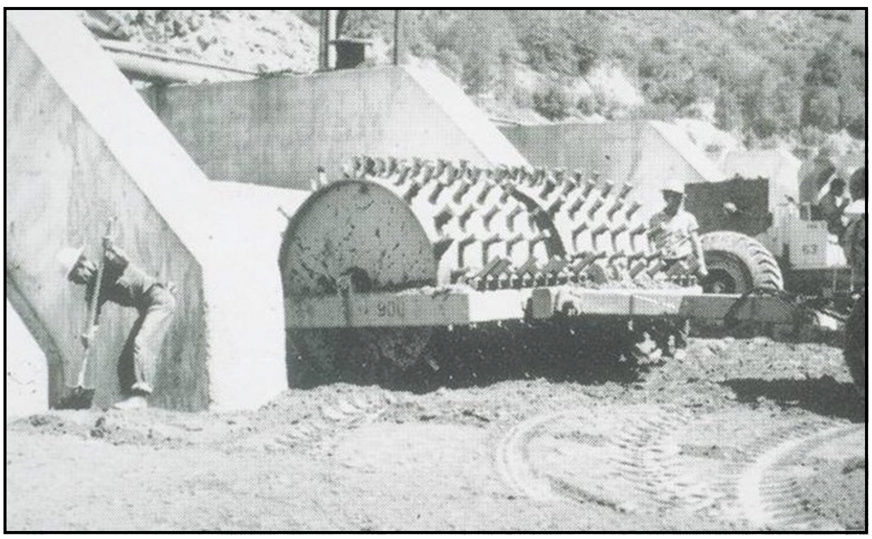


Figure 15.—Compaction around antiseep collars was difficult using large equipment.

Most embankment dam designers, dam regulators, and dam-building agencies now recommend a zone of designed filter material surrounding the penetrating conduit. Some designs use a filter diaphragm located about midway between the centerline of the embankment dam and downstream toe. Other designs use a filter collar around the downstream portion of the conduit. Often, a chimney filter serves as a diaphragm to protect the conduit, as well as satisfying other functions of

Figure 16.—Good compaction around antiseep collars was difficult to achieve.

embankment dam design. See chapter 6 for guidance on the design and construction of filters. Since filters have become a standard design element in embankment dam designs, very few failures have occurred that can be attributed to internal erosion or backward erosion piping near conduits.

1.2 Locating the conduit

A number of factors influence the layout of a conduit, such as the type and cross section of the embankment dam, topography, geology, and hydraulics. Conduits through embankment dams are often referred to as "cut-and-cover" conduits. Conduits through embankment dams should be avoided, when safe and cost-effective alternatives are available. An alternative to a conduit through the embankment dam is a tunnel located in the abutment, wherever geology, topography, and economics are favorable. The advantages of a tunnel include:

- *Eliminates potential failure modes*.—The tunnel is not physically associated with the embankment dam. Using a tunnel completely eliminates the potential failure modes normally associated with a penetration through an embankment dam.

- *Facilitates construction*.—A tunnel can often facilitate stream diversion around the damsite during construction.

- *Simplifies embankment placement*.—A tunnel can allow unobstructed embankment placement, since it no longer hinders construction of the earthfill.

Figure 17.—Failure of an embankment dam following first filling. The failure was attributed to internal erosion because the time required for seepage to develop through the compacted embankment and cause failure was very short. Also, the soils are not the type ordinarily considered susceptible to backward erosion piping. Antiseep collars were not effective in preventing the failure.

Figure 18.—Antiseep collars were not adequate to prevent the internal erosion failure of this embankment dam. The internal erosion that occurred on first filling of the reservoir occurred in dispersive clay soils that are not susceptible to backward erosion piping.

- *Eliminates compaction requirements.*—A tunnel eliminates the need for special compaction requirements around the conduit.

- *Allows for independent construction of tunnel.*—Tunnel construction can be performed independently of the embankment dam construction. Typically, the construction of the conduit through an embankment dam is a critical path feature for construction of the dam.

- *Eliminates the need for special filters.*—A tunnel eliminates the need for special filter placement and drainage requirements, which can typically slow the progress of embankment dam construction.

However, there are disadvantages associated with a tunnel, such as:

- *Increased cost.*—A tunnel is often more expensive than a conduit through an embankment dam. This is especially true for smaller diameter conduits. However, for larger diameter conduits or where pressurized systems are required, the relative cost differences can be reduced. The reduction in cost difference is due to a lesser need for redundant safety features, such as steel pipe liners, special filter and drainage requirements, and more efficient embankment dam construction.

- *Soft ground concerns.*—A tunnel may be problematic in soft ground conditions. This could result in higher design and construction costs. Also, the portal conditions must be able to accommodate the entrance and terminal structures.

- *Potential for overruns.*—A tunnel typically involves more risk for cost and schedule overruns than a conduit through an embankment dam.

- *Requires additional engineering experience.*—Fewer engineering firms maintain a qualified staff for planning, design, and construction services for tunnels than for conduits through embankment dams.

- *Construction data lacking.*—Since tunnels are not very often constructed, up-to-date construction cost data are not always readily available for comparison of costs to conduits through embankment dams.

Tunnels are seldom used for small embankment dams and may be a more costly option for some larger dams. In those embankment dams, a conduit penetrating the dam may be preferred. Conduits have typically been located at about the embankment/foundation interface. They are often located so as to align the conduit discharges with the original watercourse, bypassing streamflow during construction, and potentially emptying the reservoir by gravity. This means that for many sites, the conduit is located on alluvial soils that can be deep and compressible. This also means the conduit is often located near the maximum section of the embankment dam, which contributes to greater structural loading on the conduit. The designer should consider the following guidance in locating the conduit (Reclamation, 1987c, p. 3):

- *Avoid differential settlement.*—Whenever possible, the conduit should be located where the profile is entirely on bedrock, or entirely on soil. Differential settlements can occur where the overburden soil thickness is extremely variable or foundation properties differ. The bedrock profile underlying the conduit location should not have abrupt changes in a short horizontal distance.

- *Locate the conduit in a trench.*—Locate the conduit in a trench section in firm rock when the rock is at or near the ground surface (figure 19). For this option, the construction specifications should include provisions for rock excavation to be performed to eliminate or minimize open fractures or other damage to the rock beyond the limits of the excavation. Concrete should extend to an upper limit of the top of the conduit or to the original rock surface, if lower than the top of the conduit.

- *Locate the conduit on a bench.*—Locate the conduit on a bench excavated along the base of an abutment when geological conditions and topography are favorable. Placing concrete on the abutment side or placing the conduit against the excavated rock reduces or eliminates requirements for earthfill compaction against one side of the conduit (figure 20).

- *Consider the potential for nonuniform settlement.*—Foundation conditions along the length of the conduit are often nonuniform, and concentrated settlement is common in some areas. As the height of the embankment dam is raised during the construction of the dam, periodic inspection of the interior of the conduit should be performed. The frequency of such inspections should be determined based on anticipated foundation conditions as well as any uncertainties. Some conduits have experienced distress during and after construction as a result of unidentified foundation conditions. If distress is observed in the form of cracking or separation of joints, prompt remedial action is required. Reclamation has monitored and recorded these concentrated settlements at a number of their embankment dams after construction was completed. The results of monitoring show that nonuniform settlement along the conduit is common after completion of construction.

- *Flatten slopes where conduits span a cutoff trench.*—Where a conduit spans a cutoff trench, the side slopes of the cutoff trench may require flattening to reduce differential settlement between the compacted backfill in the cutoff trench and the foundation soils adjacent to it.

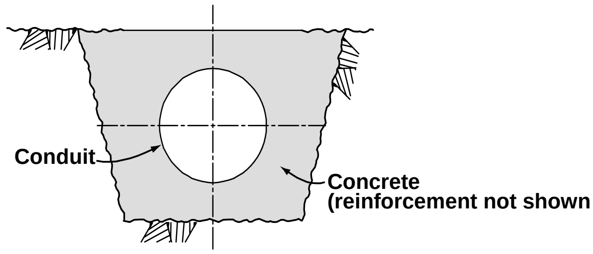


Figure 19.—Conduit constructed in a trench in firm rock.

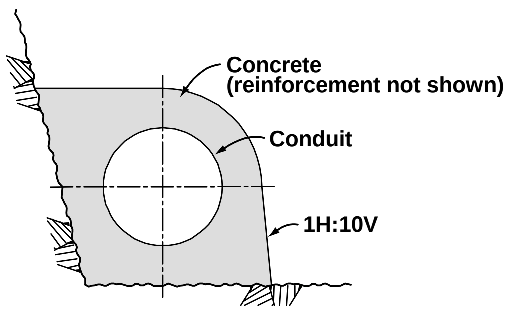


Figure 20.—Conduit cast against excavated rock slope.

- *Limit number of conduit penetrations*.—Designs should use only one conduit, when feasible, to minimize problems associated with penetrations of the embankment dam. Installing several conduits, particularly near one another, compounds the construction difficulties and increases the likelihood of problems associated with conduits through the embankment dam. However, the designer should be aware that with only one conduit, if problems develop that limit the ability to control the release of water, this may result in a dam safety concern. Therefore, the design should be robust using proven methods.

- *Avoid locating conduit joints at discontinuities*.—Locate joints for the conduit where underlying discontinuities do not occur. If the conduit alignment intersects a slurry trench cutoff or vertical drainage zone, the conduit should be designed where these discontinuities are not at a joint, but near the midway point between joints. For guidance on conduit joints, see section 4.3.

- *Consider seismic deformation*.—Seismic activity can result in significant deformation of the embankment dam. Deformations, such as settlement and spreading can open conduit joints and cause cracking and displacement of the conduit.

- *Avoid the use of bends*.—Locate the conduit so that bends in the alignment or profile are not required for the portion under the embankment dam. This will facilitate future inspection and renovation (i.e., sliplining). This will also provide improved compaction near the conduit and eliminate any stress concentrations resulting from the bend.

Some techniques that have been found to be applicable for designing conduits on compressible foundations include:

- *Excavate and replace compressible foundation soils*.—To reduce differential settlement or to reduce total settlement, excavate compressible foundation soils and replace with less compressible compacted soil.

- *Properly locate controls*.—Position the control gates and valves upstream of im pervious zone in the embankment dam.

- *Avoid pressurizing the conduit*.—Avoiding pressurized conduits through impervious embankment dams, unless the pressure conduit is placed within a larger conduit. To prevent pressurizing of the conduit, a free standing welded steel pipe supported by cradles can be placed within a larger reinforced concrete conduit. Access is provided along the side of the steel pipe. The steel pipe is considered to be ductile and will deform and still maintain a watertight conduit. When possible, field weld the steel pipe joints after the initial foundation settlement of the conduit has occurred.

- *Bridge over weak areas*.—Longitudinal reinforcement extending across the joints of the reinforced concrete encasement surrounding the welded steel pipe liner can provide a rigid beam effect and bridge over weak foundation areas to minimize locally concentrated deflections.

- *Utilize longitudinal reinforcement across joints*.—Large horizontal movements often occur at randomly selected conduit joints, rather than uniformly along the conduit length. This type of concentrated movement can open gasketed conduit joints that are not designed for large horizontal movements. The use of longitudinal reinforcement across the joints and continuous welded steel pipe liners are effective in reducing concentrated openings within conduits.

- *Provide camber*.—A conduit that is not located on bedrock must be designed so that the amount of predicted foundation settlement does not damage the conduit or its function. A conduit constructed on a compressible foundation should be cambered to accommodate the predicted foundation settlement, to achieve a proper final grade.

In lieu of constructing tunnels or conduit penetrations through embankment dams, siphons can often provide alternative reservoir drawdown capability for low hazard dams. Siphons are particularly useful for recreation reservoirs that do not make regular releases. However, proper design precautions must be utilized to ensure long term performance. For guidance on the design of siphons, see section 11.4.1.

1.3 Foundation investigations

Thorough foundation investigation and interpretation of the data obtained are required to determine whether a safe and economical conduit can be built at a selected site. The designer should always participate with the planning of the subsurface exploration program. Guidance for planning, conducting, and interpretation is available in Reclamation's *Design of Small Dams* (1987a) and *Engineering Geology Field Manual* (1998b), and the USACE's *Geotechnical Investigations* (2001a). The designer should be aware that final alignment of the conduit may require adjustment after the complete foundation has been exposed.

Chapter 2

Conduit Materials

Various materials have been used in the design and construction of conduits through embankment dams. The reasons for utilizing these different materials have included cost, availability, operations, maintenance, and constructability. The most common materials used in the construction of new and renovated conduits have been:

- *Concrete.*—Reinforced cast-in place and precast
- *Plastic.*—Thermoplastic and thermoset
- *Metal.*—Steel, ductile iron, cast iron, and CMP

The strength and performance characteristics of each conduit material depend on its chemistry and the relationship of its components. For example, concrete is produced using cement, sand, aggregates, and reinforcement, whereas metal is a homogenous, isotropic material.

Certain design and construction advantages and disadvantages are associated with each material. Each material requires specific design and construction considerations. Some of these materials, are not recommended for use in the design and construction of conduits through significant and high hazard embankment dams. For example, CMP is seldom used in any embankment dams other than low hazard dams and needs to be carefully evaluated for the specific dam site. For guidance on the use of specific materials in renovation, replacement, and repair of conduits, see chapters 12, 13, and 14.

2.1 Concrete

Concrete materials used in conduit construction have included:

- Reinforced cast-in-place
- Precast concrete pipe

These materials are discussed in the following sections.

2.1.1 Reinforced cast-in-place concrete

Reinforced cast-in-place concrete is placed and allowed to cure in the location where it is required to be in the completed conduit. Reinforced cast-in-place concrete is made by mixing cement, fine and coarse aggregates, sand, and water. Admixtures are frequently added to the concrete immediately before or during its mixing to increase the workability, strength, or density, or to lower its freezing point. A framework of reinforcing steel is constructed, and forms to contain the wet concrete mix are built around the reinforcement. The wet concrete mix is placed inside the forms and around the reinforcing steel. Typically, consolidation of the concrete mix is obtained by vibration. The final solidified mass becomes reinforced cast-in-place concrete. Reinforced cast-in-place concrete conduits are built at the construction site. Figure 21 shows typical reinforcement used with cast-in-place concrete.

Reinforced cast-in-place concrete conduits (figure 22) have a long history of use by the major dam design agencies. Reinforced cast-in-place concrete conduits are very adaptable in their application and can be designed to fit specific project requirements and site conditions. A variety of design shapes are possible. For guidance on selecting the proper shape see section 4.1. Properly designed and constructed reinforced cast-in-place concrete should have a service life of 100 years or longer.

The advantages of using reinforced cast-in-place concrete for conduits include:

- The longitudinal reinforcement typically extends across the conduit joints. This prevents the joint from separating and developing a leak.

- A variety of conduit shapes are available to provide better distribution of loadings to the foundation.

- Conduit shapes can be designed to provide for good compaction of earthfill against the conduit.

- Allows for redundant seepage barrier protection, since waterstops and reinforcement typically extends across conduit joints. Welded steel liners are often used to provide additional seepage barrier protection.

The disadvantages of using reinforced cast-in-place concrete conduits include:

- Construction costs are often higher than for other conduit materials, particularly for small diameters.

Figure 21.—Reinforcement being unloaded for use in cast-in-place concrete.

Figure 22.—Concrete placement for a reinforced cast-in-place conduit.

- Quality of concrete depends on quality control and construction inspection in the field.

- Aggressive water or soil chemistry can limit service life, unless proper precautions are taken in design.

2.1.2 Precast concrete

Precast concrete refers to concrete pipe that is cast somewhere other than its final location. Precast concrete pipe sections are transported to the location where the conduit is constructed (figure 23). Three types of precast concrete pipe have typically been used in the construction of conduits through embankment dams: reinforced concrete pipe (RCP), reinforced concrete cylinder pipe (RCCP), and prestressed concrete cylinder pipe (PCCP).

Precast concrete pipes are typically circular in cross section. Rectangular precast conduits (also known as precast concrete boxes) are seldom used in embankment dams, because joints cannot be constructed that are reliably watertight.

The advantages of using precast concrete for conduits include:

- Manufactured to tight tolerance in a controlled environment.
- Quality is unaffected by adverse field casting conditions.
- Can be installed quickly, thus minimizing the amount of time required for stream diversion.
- Articulation of joints and the ability to accommodate varying settlement along the entire length of the conduit without high structural stresses.

The disadvantages of using precast concrete for conduits include:

- Longitudinal reinforcement does not extend across the conduit joints. Joints can open as a result of embankment dam settlement or elongation, unless a continuously reinforced concrete cradle is provided along the full length of the conduit.
- Due to shipping and handling limitations, short pipe lengths are required to reduce weight. This will result in many pipe joints for the entire length of the conduit and increase the number of locations for potential leakage.
- Gasketed joints are the only defense against leakage.
- Compaction of earthfill is difficult under the haunches of the pipe, unless a concrete cradle is provided.
- Aggressive water or chemistry can limit service life, unless proper precautions are taken in design.

Figure 23.—Precast concrete pipe being unloaded from delivery truck.

Some design agencies, such as Reclamation, do not permit use of pressurized or nonpressurized precast concrete conduits through embankment dams due to concerns with watertightness, the lack of longitudinal reinforcement extending across conduit joints, and the difficulty of adequately compacting earthfill against the conduit below its springline.

Other design agencies, such as NRCS, use precast concrete pressure pipe (American Water Works Association [AWWA] C300 [2004a], 301 [1999b], and 302 [2004b]) extensively for all embankment dams other than low hazard dams. The typical NRCS application is a pressure rated pipe in a nonpressurized conduit situation where the entrance structure is an ungated riser or tower, and the terminal structure is an ungated plunge pool or stilling basin. *Earth Dams and Reservoirs* (1990) contains NRCS design guidance for conduits in embankment dams.

2.2 Plastic

Plastic pipe is often used in the renovation of conduits (e.g., sliplining or lining of existing conduits). Plastic pipe that is used in the construction of new, significant and high hazard embankment dams should always be encased in reinforced cast-in-place concrete to assure quality compaction against the conduit. Use of plastic pipe in new, low hazard embankment dams is generally limited to small diameters (less than 12 inches). Plastic pipe used in low hazard embankment dams is often not encased in reinforced cast-in-place concrete for economic and construction-related reasons. However, use of a filter diaphragm or collar is a valuable defensive design measure, even for low hazard classification sites with favorable conditions. Some designs may not employ a filter diaphragm around the conduit, but eliminating this valuable feature should be carefully considered and justified, based on extremely favorable soil conditions, good conduit construction materials and methods, reliable construction practices, and favorable foundation conditions. Plastic pipe is generally considered to have a shorter service life (approximately 50 to 100 years) than concrete, but may be preferred in situations where aggressive water or soil chemistry could attack concrete.

Plastic pipe consists of resins composed of polymerized molecules mixed with lubricants, stabilizers, fillers, and pigments. Plastic pipe used in the construction or renovation of conduits has included thermoplastic and thermoset plastic. These materials are discussed in the following sections.

2.2.1 Thermoplastic

Thermoplastics are solid materials that change shape when heated. Thermoplastics commonly include polyethylene (PE) and polyvinyl chloride (PVC). Thermoplastic pipe is produced by the extrusion process. The extrusion process continuously forces molten polymer material through an angular die by a turning screw. The die shapes the molten material into a cylinder. After a number of additional processes, the final product is cut into the specified pipe lengths.

The advantages of using thermoplastic pipe as a new conduit or for the sliplining of an existing conduit include:

- Lightweight material that facilitates installation.
- Resists corrosion and is not affected by naturally occurring soil and water conditions. May be preferable in certain conduit applications where aggressive water or soil chemistry would limit the life of concrete or metal pipe.
- The smooth interior surface reduces friction loss. Also, due to the very smooth surface of thermoplastic pipe, adherence of minerals (e.g., calcium carbonate) is minimized.
- The ability to heat fuse PE pipe joints provides a watertight joint.
- Resists biological attack.

The disadvantages of using thermoplastic pipe as a new conduit or for the sliplining of an existing conduit include:

- High coefficient of thermal expansion relative to concrete can cause movement of the slipliner, requiring the use of end restraints.
- Can easily be damaged or displaced by construction and compaction equipment unless it is encased in concrete.

- Compaction of earthfill is difficult under the haunches of the pipes unless encased in concrete to provide good compaction of earthfill against the conduit.

- Heat fusion of pipe joints requires special equipment and an experienced operator.

- Requires a concrete encasement for significant and high hazard embankment dams to provide a favorable shape for compaction of earthfill against the conduit.

Solid wall, high density polyethylene (HDPE) is the most commonly used thermoplastic material for sliplining of existing conduits. Figure 24 shows an example of HDPE pipe. HDPE pipe is an inert material and as such is not subject to corrosion or deterioration, has a long service life, and requires little maintenance. This is especially important in small conduits that are not easily renovated and cannot be easily inspected. HDPE has been used in sliplining of existing conduits, since the early 1980s. HDPE is typically available in sizes up to 63 inches in diameter. The manufacturer can fabricate HDPE pipe fittings, such as bends, flanges, reducers, and transitions. Specialized fittings can also be custom fabricated. HDPE pipe is typically black. However, HDPE pipe is also available with gray and white pigmentation to reduce glare and improve conduit inspection using closed circuit television (CCTV) equipment. For guidance on the use of HDPE pipe in conduit sliplining applications, see section 12.1.1.

PVC pipe (figure 25) is not as commonly used as HDPE pipe as a conduit or slipliner due to concerns with lack of watertightness and other inherent disadvantages. The major disadvantage with PVC pipe is the bell and spigot joint connections. This type of joint connection has the potential for leakage or can separate as the embankment dam settles. The bell and spigot joint integrity must be tested for leaks to ensure that the gasket has not rolled off during installation. Use of PVC bell and spigot joints should only be considered for nonpressurized, low hazard dam applications. PVC is typically available in sizes up to 48 inches in diameter.

Figure 24.—HDPE pipe to be used for sliplining of an existing conduit.

Figure 25.—PVC pipe has infrequently been used in conduit applications within low hazard embankment dams. The bell and spigot joint connection used for this type of pipe limits its use for most conduits.

2.2.2 Thermoset plastic

Thermoset plastics are rigid after manufacturing or curing and cannot be reformed. The most commonly used thermoset plastic for lining nonpressurized conduits is cured-in-place pipe (CIPP). CIPP is also referred to as an "elastic sock." CIPP consists of a polyester needle-felt or glass fiber/felt reinforcement preimpregnated with polyester resin (USACE, 2001d, p. 11). The preimpregnation process is usually done at the factory for quality control purposes. On the inner surface of the CIPP liner is generally a coating or membrane of polyester, polyethylene, surlyn, or polyurethane, depending on the type of application. The membrane provides a low friction and hydraulically efficient inner surface to the CIPP liner. Figure 26 shows CIPP being used to line an existing conduit. CIPP has been successfully used in renovating deteriorated pipelines, drain pipes, and conduits through levees for over 25 years. CIPP has been used for conduit renovation through embankment dams since about the mid-1990s.

The advantages of using CIPP lining for conduits include:

- Thermoset plastic pipe is corrosion resistant and is not affected by naturally occurring soil and water conditions. Thermoset plastic pipe may be preferable in certain conduit applications where aggressive water or soil chemistry would limit the life of concrete or metal pipe.

Figure 26.—CIPP liner exiting from an existing conduit via the hydrostatic inversion method.

- The smooth interior surface reduces friction loss. Also, due to the very smooth surface of thermoset plastic pipe, adherence of minerals (e.g., calcium carbonate) is minimized.

- Thermoset plastic pipe resists biological attack.

- Typically, the need for grouting of the annulus between the CIPP liner and existing conduit is eliminated, since it is tight fitting.

The disadvantages of using CIPP lining for conduits include:

- High material and installation costs.

- Not suited for conduits with significant bends or changes in diameter.

- Inability to accommodate internal and external loadings when the original conduit is severely damaged.

CIPP liners are generally applicable for lining of existing conduits ranging in diameter from 4 to 132 inches. Maximum lengths of CIPP liners generally range from 1,000 to 3,000 feet. CIPP liners are best suited for existing conduits that are not severely damaged or deformed. For guidance on the use CIPP in conduit renovation applications, see section 12.2.

2.3 Metal

Metal pipes used in the construction of conduits have included:

- Steel
- Ductile-iron
- Cast-iron
- CMP

These materials are discussed in the following sections.

2.3.1 Steel

Steel is a strong alloy of iron and carbon that contains a lower carbon content than cast iron (lower than 2 percent). The amount of carbon determines the steel's hardenability. Steel pipe is manufactured in a shop. The manufacture starts with steel plate conforming to a specified ASTM International (ASTM) standard and of the proper thickness. These plates are butt welded together. The plates can be rectangular pieces that are rolled to the curvature of the pipe wall, and welded together at circumferential and longitudinal joints. Another plate configuration is spiral. For this configuration, a long length of steel plate is rolled in a spiral pattern, and welded edge to edge. Steel plate is also welded together for specials, such as bends, wye branches, reducers, manholes, and transitions.

Steel pipe is typically hydrostatically tested to 1½ times the design pressure in the shop. Steel pipe is also often hydrostatically tested after installation. Hydrostatic testing of fittings can be performed in the field after installation. Straight pieces of steel pipe are normally fabricated in standard 40-foot lengths, which are shipped from the shop to the job site. The pieces of steel pipe are installed and field welded together for a rigid, waterproof joint. Joint welds are checked in the field using liquid-penetrant, ultrasonic, or radiographic methods. A flanged joint can be used to provide another rigid connection, which can be disassembled and reassemble, if needed. Flanges are typically used with gates and valves to provide a rigid connection. Flanges are also used to connect steel pipe with thermoplastic pipes, such as HDPE pipe.

Steel pipe with diameters 24 inches and smaller (at some shops, 36-inches and smaller) is manufactured to standard wall thickness and diameter. Pipe greater than 24 inches in diameter can be custom manufactured to any desired diameter. However, standard diameters are listed in AWWA M11 (2004c) for steel pipe with diameters greater than 24 inches. Minimum plate thickness for larger diameters is ¼ inch, available plate thicknesses increase by multiples of $^{1}/_{16}$ inch.

Steel pipe is protected with a variety of linings and coatings. Often the interior lining is different from the exterior coating, because of the different exposure conditions of the interior and exterior surfaces. Typically, the interior surface can be lined with the same paint system regardless of location. The exterior surface coating may vary, depending on location, encasement, or submergence. The exterior surface is usually bare steel, where encased in concrete. With the proper coating, good surface preparation, proper maintenance of the coating, and cathodic protection steel pipe can last over 100 years, and not need recoating for at least 25 years. The applicable coatings and linings selected to mitigate corrosion should consider the velocities within the pipe. Cement mortar should only be used on the interior surfaces of steel pipe with low velocities.

Steel pipe has been used in some sliplining applications (figure 27), but has more often been used as a liner in reinforced cast-in-place conduits. Steel pipe has been used in sliplining of existing conduits since the 1980s. Steel pipe used as conduit liner has been used since about the early 1920s. The advantages of using steel pipe for conduits include:

- Manufactured to a tight tolerance in a controlled environment.
- Long service life, if proper linings and coatings are used. Cathodic protection can be used in addition to coatings to address expected holidays in the coating for effective corrosion protection.

Figure 27.—Steel pipe slipliner being prepared for insertion into an existing conduit.

- Welded joints provide watertightness and steel pipe is often used as a lining within conduits constructed on compressible foundations.

- High compressive and tensile strength.

- Flexible and deformable under stress.

- High modulus of elasticity to resist buckling loads caused by external water loads or vacuum.

- Various types of joints possible, including butt welding, flanged, pipe coupling, and grooved-end coupling. Many of these joints permit flexibility of the pipe in case of expansion, settlement, etc. Figure 28 shows an example of a flanged joint.

- Bends, wye branches, reducers, manholes, transitions, and other specials can be fabricated.

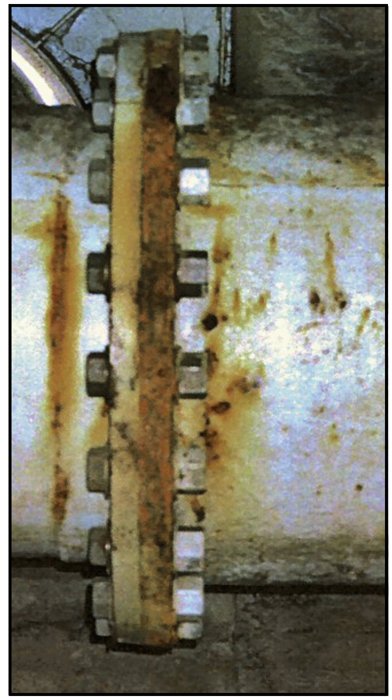


Figure 28.—Watertight joints can be provided for steel pipe by use of flange connections.

- Easy to connect additional steel pipe in the future by tapping and welding. Blind flange ends can be installed for easy future additions.

- Flanges provide a rigid connection to gates and valves.

- Has the ability to be easily used as a redundant system within reinforced cast-in-place concrete (i.e., steel pipe located within a larger access conduit).

The disadvantages of using steel pipe for conduits include:

- High material costs.

- The proper selection of linings and coatings and any associated maintenance are required to prevent corrosion.

- Requires a concrete encasement for significant and high hazard embankment dams to provide a favorable shape for compaction of earthfill against the conduit.

- Requires special linings at reservoirs where aggressive water may exist (i.e., acidic mine drainage).

2.3.2 Ductile-iron.

Ductile-iron pipe is manufactured by centrifugal casting. A controlled amount of molten iron is introduced into the rotating mold, which generates a centrifugal force that holds the iron in place against the mold until it solidifies. The pipe is then removed and furnace annealed. Ductile-iron has greater range of deformation, and is less brittle than cast-iron pipe. Ductile-iron pipe also has greater tensile and compressive strength than cast-iron pipe. Linings and coatings are required for ductile-iron pipe; commonly asphalt paint and cement-mortar lining are used. The first manufacture of ductile-iron pipe was in 1955. Ductile iron has been commonly used in utility water and sewer systems. Ductile iron is used infrequently in conduit applications. Ductile iron pipe is available in diameters up to 64 inches and in lengths up to 24 feet.

The advantages of using ductile-iron pipe for conduits include:

- Manufactured to tight tolerance in a controlled environment.
- Long service life, if proper linings and coatings are used.
- High tensile and compressive strength.
- Flanged joints (figure 29) provide improved watertightness over bell and spigot connections.

Figure 29.—Flanged joints of ductile iron pipe improve the watertightness of the joint.

The disadvantages of using ductile-iron pipe include:

- Ductile iron pipe is heavy and makes handling difficult.

- Requires a concrete encasement for significant and high hazard embankment dams to provide a favorable shape for compaction of earthfill against the conduit.

- Requires cathodic protection in corrosive soils.

2.3.3 Cast-iron.

Cast-iron is metallic iron containing more than 2 percent dissolved carbon within its matrix and less than 4.5 percent (as opposed to steel, which contains less than 2 percent). Cast-iron (figure 30) pipe cannot be wrought, so it must be manufactured by casting in a foundry. The molten iron is poured into a vertical annular mold. The mold is removed after the iron cools and solidifies. Commercial availability of cast-iron pipe typically is up to 15 inches diameter. Cast-iron pipe has been around for centuries, and used for storm water and sewer systems. The available length for single pieces is up to 40 feet. Many cast-iron pipes have been in service over 100 years.

Cast-iron pipe has been used in the past for conduits through embankment dams, but is currently not considered acceptable for new embankment dam construction by any of the federal dam-building agencies.

The disadvantages of using cast-iron pipe include:

- Cast-iron pipe is heavy and makes handling difficult.

- Joints are bell-and-spigot. Cast-iron pipe cannot be welded, so that flanged joints are not possible.

- Not normally commercially available in the realm of diameters of most conduits. Custom fabrication would be very expensive.

- Lacks a favorable shape for compaction of earthfill against the conduit.

- Cast-iron pipe is brittle and can crack, if not properly handled.

2.3.4 CMP

CMP is fabricated from factory-produced sheet steel with corrugations added to provide stiffness and strength. The sheets are typically coated with polymers, zinc

Figure 30.—Cast iron pipe is not considered acceptable for use in conduit applications.

(galvanized), aluminum, or aluminized zinc alloy. Additional coatings, such as bituminous, have been applied for added protection against corrosion and abrasion. CMP was first used for conduits in the late 1890s. Generally, round pipe ranges from 6 inches to 26 feet in diameter. Other shapes and sizes of CMP are available, but have had limited applications in conduits through embankment dams. CMP is typically joined with coupling bands that extend over several corrugations on each end of two adjoining pipes. The coupling bands are designed to be mechanically tightened against the pipe corrugation via rods, lugs, angles, and bolts. A gasket material is used between the band and the pipe (figure 31). CMP has a service life of about 25 to 50 years. However, depending on reaction to certain soils and water conditions, there are cases where CMP has deteriorated in less than 7 years.

Many embankment dam failures have been associated with the use of CMP conduits. CMP has had a history of joint separations due to differential settlement, joint separations due to lateral spreading of the embankment dam, and deterioration. Major federal dam-building agencies, including NRCS, and USACE, limit their use of CMP to low hazard embankment dams. Reclamation does not permit CMP to be used for conduits through their embankment dams. Although CMP has the advantage of being lightweight and easily installable without the need of heavy construction equipment, there are many serious disadvantages.

The disadvantages of using CMP for conduits include:

- Deterioration has resulted in many embankment dam failures.

- Joint separations from differential settlement and embankment dam spreading can result in nonwatertight joints.

Figure 31.—A CMP conduit being installed.

- Joints are often incorrectly assembled in the field, resulting in nonwatertight joints.
- Not applicable for pressurized conduits due to lack of watertight joints.
- CMP is considered a flexible pipe, and flexible pipe design requires the earthfill surrounding the pipe to provide structural stability and support to the pipe. If the surrounding backfill does not provide adequate support, flexible pipes are subject to distortion and deflection.
- Circular shape and corrugations on the exterior surface makes compaction of the earthfill against the conduit difficult to achieve.

Chapter 3

Hydraulic Design of Conduits

The discharge of water through a conduit requires a good understanding of the purpose for which the structure is being designed. For instance, the invert profile of the conduit should be sloped to provide drainage in the downstream direction. Where feasible, the conduit should discharge at an elevation higher than the highest tailwater or at an elevation where there is no influence from tailwater. Free flow conduits should not flow greater than 75 percent full (i.e., 75 percent of the diameter or height of the conduit) at the downstream end, to minimize the risk of surging flow developing in the conduit as the result of inadequate air for open channel flow conditions.

This chapter discusses some of the pertinent aspects of hydraulic design of conduits. For detailed guidance on hydraulic design, the reader is directed to references such as Reclamation's *Design of Small Dams* (1987a), and USACE's *Structural Design and Evaluation of Outlet Works* (2003b) and *Hydraulic Design of Reservoir Outlet Works* (1980).

3.1 Outlet works

The main purpose of an outlet works through an embankment dam is to control the release of water from a reservoir. An outlet works typically consists of a combination of structures. The outlet works is often comprised of the some or all of the following components (this list is not all inclusive, and the type of components may vary, based on project requirements):

- *Approach channel.*—The channel upstream from the intake structure. This channel is generally unlined, excavated in rock or soil, and with or without riprap, soil cement, or other types of erosion protection.

- *Entrance structure (typically referred to as an intake structure).*—A structure located at the upstream end of the outlet works. Entrance structures often include gates or valves, bulkheads, trashracks, and/or fish screens.

- *Conduit.*—A closed channel used to convey water through the embankment dam.

- *Control features.*—Typically gates or valves located in the intake structure, conduit, gate chamber, or a downstream structure.

- *Terminal structure.*—A structure located at the downstream end of the outlet works. Terminal structures often include gates or valves and may include some type of structure to dissipate the energy of rapidly flowing water and to protect the riverbed from erosion.

- *Discharge channel.*—The channel downstream from a terminal structure. This channel conveys releases back to the "natural" stream or river. This channel may be excavated in rock or soil with or without riprap, soil cement, or other types of erosion protection.

For guidance on the design and construction of entrance and terminal structures, see section 3.4. Design and construction guidance on approach and discharge channels, control features, and gate chambers are outside the scope of this document. Additional guidance relating to various components of an outlet works is available in references, such as Reclamation's *Design of Small Dams* (1987a), and USACE's *Structural Design and Evaluation of Outlet Works* (2003b) and *Hydraulic Design of Reservoir Outlet Works* (1980).

Discharge requirements through the outlet works may fluctuate throughout the year, depending upon downstream water needs or reservoir flood control requirements. Outlet works typically serve a number of different purposes (Reclamation, 2001b):

- *Emergency evacuation.*—The outlet works should be sized to meet established reservoir evacuation guidelines that apply for the State in which the embankment dam was constructed or for the agency/organization responsible for the dam. For example, Reclamation (1990b, p. 13) specifies depths and volumes of the reservoir to be evacuated during specified timeframes based on the levels of "risk" (potential for an incident to occur at the dam) and "hazard" (level of downstream consequences as the result of misoperation and/or uncontrolled release of part or all of the reservoir). Deviation from established evacuation guidelines may be justified for existing reservoirs if (1) the risk associated with first filling of the reservoir has passed, or (2) the risk reduction for increasing evacuation capacity does not justify the cost of modifications.

- *Reservoir filling rates.*—The first filling of a reservoir is a critical time. Some embankment dams have failed due to hydraulic fracture caused by the pressure of water as it penetrates the embankment dam too rapidly. The rate of reservoir filling is generally regulated to monitor the response of the embankment dam to increasing hydrostatic loading. The outlet works may be required to pass some portion of the reservoir inflow to keep filling rates within the desired range. Typical filling rates are in the range of 0.5 to 2 feet per day.

More restrictive filling rates (e.g., first filling) may be required when the reservoir exceeds historically high levels. The designer must assume that first filling can be sudden and unexpected, and the outlet works must have sufficient capacity to accommodate this type of event. The Picketberg and Wister Dam case histories in appendix B are examples of embankment dam failures that have occurred upon first filling of the reservoir.

- *Diversion.*—The outlet works may be utilized for the diversion of stream or river flows during construction of the embankment dam. The sizing of the outlet works conduit is based upon the size of the flood that might be reasonably expected to occur during construction. Historically, diversion flood capacities are in the range of 5, 10, or 20 years, with consideration given to larger flood levels, if the consequences of failure during construction are large.

- *Operational.*—The outlet works is typically used to pass downstream release requirements, such as irrigation releases, environmental enhancement for wetlands, fisheries, or water quality, and municipal and industrial releases.

- *Flood control.*—The outlet works may be sized to restrict the amount of flow that can pass through the system, thereby storing excessive flood flows in the reservoir and limiting flood flows downstream of the embankment dam.

3.1.1 Arrangement of control features

Depending on the requirements of the project, the outlet works may be controlled or uncontrolled. Controlled outlet works are used at multipurpose reservoirs that provide storage for conservation, irrigation, etc. and for single-purpose flood control projects in which control of the discharge is required. Uncontrolled outlet works are used at some flood control reservoirs, where predetermined discharges (varying with the head) are required to meet the flood control requirements. The type and size of the controls depend on the purposes that the outlet works will serve.

The location of the control features within the outlet works affects the risk associated with internal erosion and backward erosion piping incidents. Downstream control features can allow pressurized conditions to occur in the upstream portion of the conduit. Pressurized conditions create a greater potential for water escaping under pressure, potentially eroding the surrounding earthfill or foundation soils. Careful consideration is required in selecting the location of control features.

Control of the outlet works discharge is accomplished by gates and valves. The gates and valves are typically motor operated, hydraulically operated, or manually operated. Operators for gates and valves are typically required to have backup systems to open them under emergency conditions. Regulating gates and valves are used to control and provide regulation of the outlet works flow. Regulating gates and valves are designed to provide a wide range of operation from closed to fully open. The closure times for gates and valves should be closely evaluated to keep water hammer pressures within reasonable limits. Guard gates are designed to provide closure only when the regulating gates become inoperable or when unwatering of the conduit is required to inspect or repair the section of conduit downstream of the guard gates or to inspect or repair the regulating gates. In some applications, an emergency gate may be used in conjunction with or in lieu of a guard gate. An emergency gate is typically provided only as a standby or reserve gate and is used when the normal means of water control is not available for use. Generally, slots are provided for stoplogs or bulkheads to be installed at the conduit entrance to allow for unwatering and inspection of the conduit. In some cases, if stoplogs or bulkheads can be quickly installed during an emergency, guard gates may not be required. However, stoplogs and bulkheads are not intended for emergency closure under unbalanced conditions or when the outlet works is operating. Specially designed stoplogs and bulkheads would be required.

The control features should allow for complete inspection by man-entry or CCTV. Certain types of gates or valves (e.g., butterfly valves) can act as an obstruction and may preclude the use of robotic camera-crawler equipment, since it may not be able to pass under or around the gate or valve. Alternate access using manholes may be required to provide access around the obstruction.

An important consideration in any closed conduit design for an outlet works is the proper use of air venting. Air vents can permit air to enter the conduit to prevent collapse or to prevent the formation of low pressures within flowing water, which could lead to cavitation and its possible attendant damage. Air vents can also be used to bleed air from a conduit prior to operation. Figure 32 shows an example of an air vent leading from the conduit that daylights onto the surface of the embankment dam. For guidance on the location, airflow rates, and structural considerations of air vents, see Reclamation's *Air-Water Flow in Hydraulic Structures* (1980).

The location for the control of the outlet works can be placed at the upstream end of the conduit, at the downstream end, or at some intermediate point. For illustrative purposes, four arrangements for locating the control features within the outlet works have been adapted from Reclamation's *Design of Small Dams* (1987a, p. 446):

- Arrangement 1—Intermediate control with downstream access (figure 33)
- Arrangement 2—Intermediate control without downstream access (figure 34)
- Arrangement 3—Upstream control (figure 35)

Figure 32.—An air vent is required in closed conduits downstream from the controlling gate or valve to prevent collapse or the formation of low air pressures.

- Arrangement 4—Downstream control (figure 36)

These arrangements are discussed in the following sections. Figures 33 through 36 illustrate the arrangement of control features only. The internal zoning of the embankment dam's filters and drains are not shown in these figures.

3.1.1.1 Arrangement 1—Intermediate control with downstream access

In this type of arrangement (figure 33), a control gate or valve (i.e., guard or guard and regulating) is located at an intermediate point (typically at or upstream of the embankment dam centerline) between the intake and the terminal structures, with additional regulatory gate(s) or valve(s) located downstream in a control house. The specific aspects of this arrangement are:

- *Flow conditions.*—Pressure flow would exist upstream of the intermediate control. Pressure flow could also exist downstream of the intermediate control, if the regulating gate in the control house is partially or fully closed.

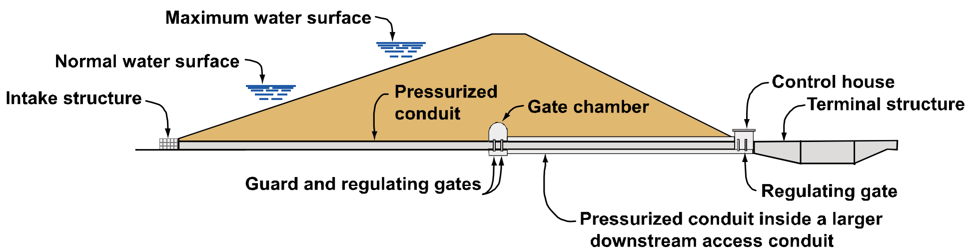


Figure 33.—Arrangement 1—Intermediate control with downstream access.—The control feature is located at an intermediate point within the conduit.

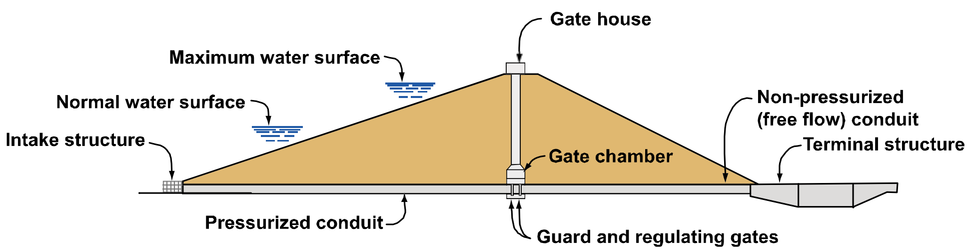


Figure 34.—Arrangement 2—Intermediate control without downstream access.—The control feature is located at an intermediate point within the conduit.

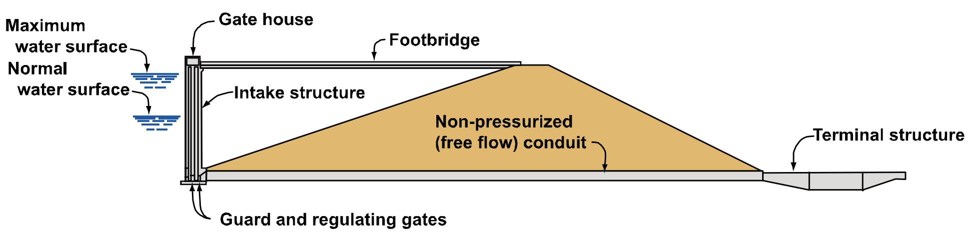


Figure 35.—Arrangement 3—Upstream control.—The control feature is located at the upstream end of the conduit.

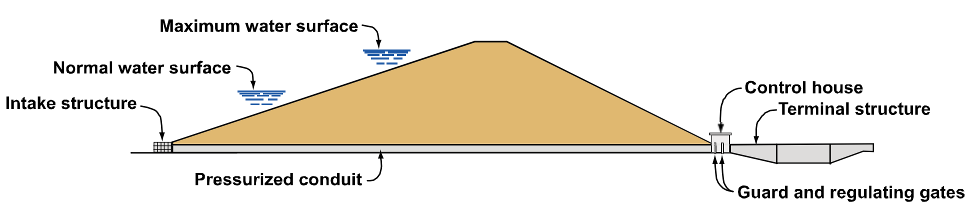


Figure 36.—Arrangement 4—Downstream control.—The control feature is located at the downstream end of the conduit.

Figure 37.—A steel pipe is located within a larger downstream access conduit.

- *Access*.—The outlet works can be operated through an interior conduit (typically a steel pipe), which is located within the larger downstream access conduit while the downstream portion of the conduit is inspected. Figure 37 shows an example of this type of arrangement. Access for inspection and maintenance of the larger downstream access conduit can be more frequent with this arrangement. Access for inspection and maintenance of the conduit upstream of the intermediate point can be limited (i.e., bulkheads must be installed).

- *Emergency closure*.—Emergency closure is possible at the intermediate control point.

- *Risk*.—This type of arrangement is typically used for high embankment dams with significant to high downstream consequences. This arrangement is considered to have less risk than arrangements 2, 3, and 4, since the ability exists to provide closure at an intermediate location. Since the external and internal hydrostatic pressures are usually balanced upstream of the intermediate control, the development of a defect in the conduit in this area will be less of potential problem. The conduit located within the larger downstream access conduit provides another degree of protection, since the ability to inspect allows for problem detection.

3.1.1.2 Arrangement 2—Intermediate control without downstream access

In this type of arrangement (figure 34), gates or valves are located at an intermediate point (typically at or upstream of the embankment dam centerline) between the intake and the terminal structures. The specific aspects of this arrangement are:

- *Flow conditions.*—Pressure flow exists upstream of the intermediate control, and open channel flow exists downstream of the intermediate control.

- *Design considerations.*—The internal pressure upstream from the intermediate control is approximately equal to the full reservoir head. The internal and external hydrostatic pressures will be closely balanced, and the potential for leakage into or out of the conduit will be minimized. As external hydrostatic pressure around the conduit diminishes with increasing distance from the reservoir, there may be excess internal pressure, and the conduit must be kept watertight to avoid leakage through joints or cracks, which could allow water to be forced out of the conduit and into the surrounding soils. Normal practice limits the length of the pressure portion of a conduit to that part of the conduit upstream from the crest of the embankment dam or to approximately the upstream third of the dam. The upstream conduit should be designed to resist the full external hydrostatic pressure when it is dewatered for inspection or maintenance. The use of a steel liner for the upstream conduit should be considered, whenever there is concern regarding the watertightness of a pressure conduit.

- *Access.*—Access for inspection and maintenance of the downstream conduit can be limited, since the gates or valves located at the intermediate point must be closed. Once closed, the downstream conduit can be accessed. However, the upstream conduit will remain inundated. Access to the upstream conduit requires bulkheading of the conduit entrance. Access to the gates or valves (normally located within a structure called a gate chamber) is typically provided through an access shaft from the crest of the embankment dam.

- *Emergency closure.*—Emergency closure is possible at the intermediate control point. Typically, this type of arrangement provides tandem gates or valves located at the intermediate control point. The upstream gate or valve serves as a guard, and the downstream gate or valve provides regulation.

- *Risk.*—This type of arrangement is typically used for high embankment dams with significant to high downstream consequences. This arrangement is considered to have more risk than arrangement 1, but less risk than arrangements and 3 and 4, since the ability exists to provide closure at an intermediate location. Since the external and internal hydrostatic pressures are usually closely balanced upstream of the intermediate control, the development of a defect in the conduit in this area will be less of potential problem.

3.1.1.3 Arrangement 3—Upstream control

In this type of arrangement (figure 35), the gates or valves are located at or immediately downstream of the intake structure. The specific aspects of this arrangement are:

- *Flow conditions*.—Open channel (free-flow) flow exists throughout the conduit downstream from the gates or valves.

- *Design considerations*.—Designed for external loadings and outside water pressures on the conduit. Near full reservoir head will be exerted on the exterior of the conduit until adequate thickness of impervious embankment is provided over the conduit. Due to large external hydrostatic pressure, the conduit must be kept watertight to avoid leakage through joints or cracks, which could allow embankment materials to be carried into the conduit.

- *Access*.—Access for inspection and maintenance is greater than arrangements 1, 2, or 4 (i.e., closing the gates or valves allows inspection of almost the entire conduit. An upstream bulkhead must be installed to inspect the upstream side of the gates or valves and the remaining portion of conduit). In most cases, this type of arrangement requires an intake tower and access bridge for gate or valve operation or bulkhead installation, which add significant design and construction costs, especially in areas with potentially high seismic activity. Figure 38 shows an example of a footbridge. Sometimes submerged intake structures containing gates or valves have been used instead of intake towers.

- *Emergency closure*.—Emergency closure is provided at the intake structure upstream of the regulating gate or valve.

- *Risk*.—This type of arrangement is considered to have more risk than arrangements 1 and 2, but less than arrangement 4. If the conduit develops a defect downstream from the intake structure, a high pressure differential will exist due to the external hydrostatic pressure from the full reservoir head and no internal pressure within the conduit. A conduit defect in the area downstream from the intake structure could result in water flowing into the conduit. In this arrangement, no emergency closure exists downstream from the intake structure. Another factor for the higher risk assignment is the potential for the free-flow conduit not being properly sized and operation resulting in a pressurized condition. If the conduit is properly sized and operated, this arrangement does not have the concern with high pressure flow being forced out of the conduit then into the surrounding fill.

Figure 38.—A footbridge or access bridge is often required to operate gates or valves located at the intake tower.

The use of upstream control without an intake tower is common on low hazard embankment dams. In this application, an inclined slide gate is located on the upstream face of the embankment dam. The gate stem is often buried to avoid damage from ice floating in the reservoir or by vandalism. Trashracks are located on the intake structure to prevent plugging of the conduit with debris. An alternative to the gate stem would be the use of a hydraulic gate operator. For this application, instead of the gate stem extending from the top of the embankment dam to the gate, a hydraulic cylinder is mounted in the intake structure. Hydraulic lines buried within the upstream face of the embankment dam connect to a manual pump and hydraulic reservoir at the crest.

3.1.1.4 Arrangement 4—Downstream control

In this type of arrangement (figure 36), gates or valves are located at or just upstream of the terminal structure (on the downstream side of the embankment dam). The specific aspects of this arrangement are:

- *Flow conditions*.—Pressure flow exists throughout the entire length of conduit from the intake structure to the gates or valves at the terminal structure.

- *Design considerations*.—The external hydrostatic pressure around a conduit normally diminishes with increasing distance from the reservoir. At the downstream portion of the pressure conduit, there may be excess internal hydrostatic pressure. The potential exists for leakage out of the conduit through joints or cracks. A steel pipe liner is normally used with pressure conduits.

- *Access*.—Access for inspection and maintenance can be seldom (i.e., either the reservoir must be drained, or divers must perform inspections after gates or valves are closed, or the conduit must be unwatered after an upstream bulkhead is installed). At some sites, a submerged upstream gate or valve is provided for closure to facilitate access for inspection. Arrangement 4 is discouraged unless the embankment dam is low hazard with minimal downstream consequences.

- *Emergency closure*.—Emergency closure is typically not possible upstream of the control point, unless a submerged intake structure with a mechanically or hydraulically operated gate or valve is provided.

- *Risk*.—This type of arrangement is considered to have more risk than arrangements 1, 2, and 3. If the conduit develops a defect upstream from the downstream control structure, a high pressure differential will exist due to the internal hydrostatic pressure from the full reservoir head and the lack of external hydrostatic pressure. A conduit defect in the area upstream from the control structure could result in water flowing out of the conduit. In this arrangement, no emergency closure exists upstream from the control structure. Even if an upstream emergency closure gate is provided in a submerged intake structure, a leak from a defect in the conduit may not be identified in time to prevent an embankment dam failure.

3.2 Spillway

Spillways utilizing conduits are generally shaft or drop inlet type. These types of spillways typically consist of an entrance (crest) structure with or without control devices, a conduit, and a terminal structure. Figure 39 shows an example of a drop inlet type of spillway called a morning glory. The drop spillway is often referred to as a "principal spillway." Figure 40 shows an example of a riser structure for a principal spillway. A spillway provides flood control regulation for floods, either in combination with an outlet works, or as the only flood control facility. Typically, the spillway is used to release surplus water or floodwater that cannot be contained in the allotted reservoir storage space. The discharge capacity of a spillway conduit is determined by the results of flood routings and is influenced by the flood surcharge volume available above the spillway crest. Where little flood surcharge volume is available, the spillway must be large enough to pass the peak of the flood. If the reservoir has a large storage capacity above the normal water surface, a portion of the flood volume can be retained temporarily, and the spillway discharge capacity may be considerably reduced.

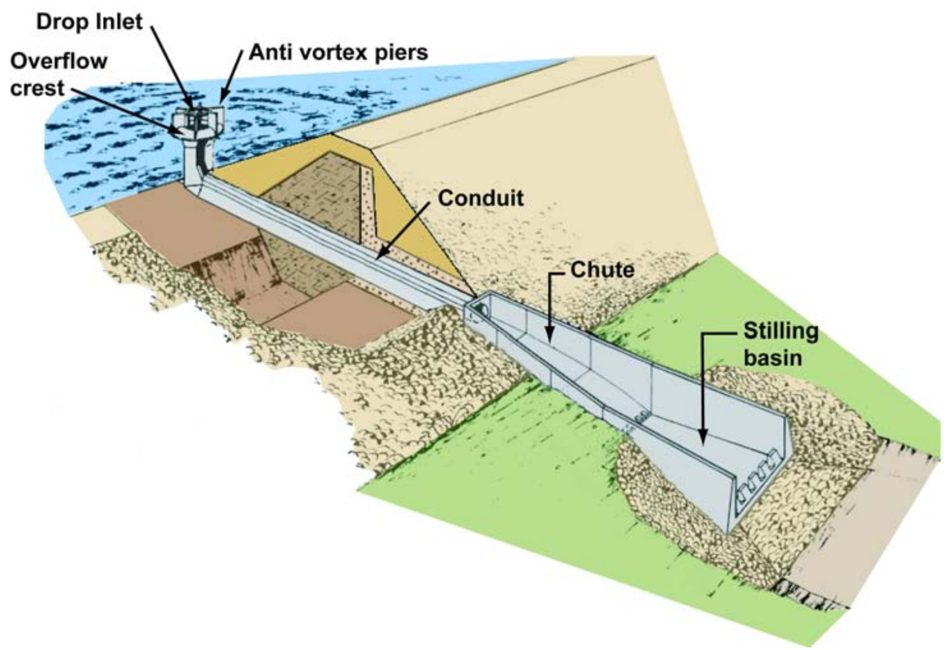


Figure 39.—A drop inlet spillway conduit through an embankment dam.

Ideally, the spillway should be designed to operate with crest control throughout the entire expected range of discharge. However, the range of expected discharge is based on the current hydrologic data. Spillway flood flow rates may change due to updated probable maximum precipitation quantities; changes in the basin runoff characteristics could vary significantly with time; and the project operation may be revised at a future date, which may result in an increase above the original spillway design flow. Any of these factors, separately or in combination, could be sufficient to cause a spillway designed for crest control to shift to conduit control in the upper range of expected discharge. The spillway conduit is considered a closed channel and generally takes the form of a vertical or inclined shaft connected to a horizontal or sloped conduit. In most cases, a spillway conduit is designed to flow partly full throughout the entire length. Another condition that could cause the control shift at essentially any discharge is partial plugging of the conduit. Plugging could occur either by external debris (i.e., logs or ice) or an internal problem resulting from cavitation damage. To ensure free flow in the conduit, the ratio of flow depth to conduit diameter or height should be limited to about 75 percent or less. Some sites have utilized log booms or specially designed trashracks that reduce surface currents to prevent floating debris from entering the conduit. Additionally, air vents may be required to ensure adequate air supply is provided to prevent unstable flow in the conduit. In areas where high velocity flow may occur, aeration of the flow may be required to prevent cavitation damage.

Figure 40.—The riser structure for a principal spillway.

Spillways utilizing conduits are not applicable to all sites, and an open channel overflow spillway or tunnel may be preferable. The limitations of a spillway utilizing a conduit include:

- The required flood discharge capacity may necessitate a large diameter conduit or multiple conduit barrels. The size and shape of the conduit can have undesirable consequences, since it represents a discontinuity through the embankment dam.

- Future increases in the size of the design flood are difficult to accommodate with a spillway conduit.

- Discharge capacity limitations of conduit may require the use of an auxiliary or emergency spillway to provide required flood control capability.

3.3 Power conduits

Power conduits (also known as "penstocks") are used to transport water from an intake structure located in the reservoir to a downstream facility for the generation of power. Figure 41 shows an example of penstocks extending through an embankment dam. The power conduit typically operates in a pressurized condition. The power conduit is usually constructed of steel pipe encased by reinforced

Figure 41.—Penstocks extending through an embankment dam.

cast-in-place concrete. Power conduits are often combined with the outlet works conduit by the use of a wye branch to the powerplant. Power conduits are normally designed and constructed with the same criteria used for outlet works conduits through embankment dams.

3.4 Entrance and terminal structures

Entrance and terminal structures are placed at the upstream and downstream ends of conduits, respectively. Entrance structures are often referred to as intake structures for outlet works and inlet structures for spillways. Properly designed entrance and terminal structures are important to the safe operation of the conduit. Figures 42 and 43 show typical intake and terminal structures for an outlet works conduit. Figures 44 and 45 show examples of outlet works where no entrance and terminal structures have been provided. In both of the situations shown in figures 44 and 45, serious dam safety deficiencies exist, since the upstream entrance could become plugged or the downstream toe of the embankment dam could erode. Reclamation's *Design of Small Dams* (1987a, p. 451) provides a good source of information concerning purpose and design considerations for entrance and terminal structures. The following has been adapted from that reference:

> Intake structures.—In addition to forming the entrance to the conduit, an intake structure may accommodate control devices, support necessary auxiliary appurtenances (such as trashracks, fishscreens, and bypass devices), and include temporary diversion openings and provisions for installation of bulkhead or stoplog closure devices.

Figure 42.—Typical outlet works intake structure.

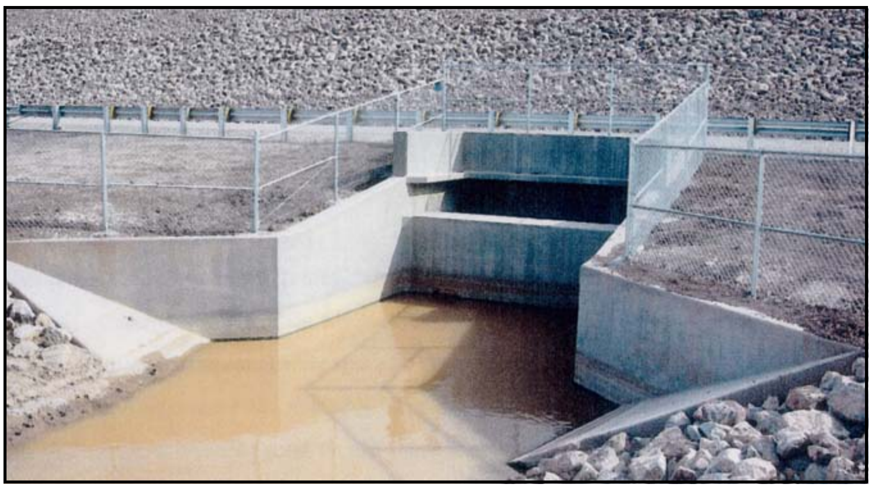


Figure 43.—Typical outlet works terminal structure.

The type of intake structure selected should be based on several factors: the functions it must serve, the range in reservoir head under which it must operate, the discharge it must handle, the frequency of reservoir drawdown, the trash and debris conditions in the reservoir (which will determine the need for or the frequency of cleaning of the trashracks), reservoir ice conditions or wave action that could affect the stability, and other similar considerations. Depending on its function, an intake structure may be either submerged or extended in the form of a tower above the maximum reservoir water surface. A tower must be provided if the controls are placed at the intake, or if an operating platform is needed for trash removal, maintaining and cleaning fishscreens, or installing stoplogs. Where the structure serves only as an entrance to the conduit and

Figure 44.—No intake structure exists for this outlet works. This conduit is prone to plugging with trash and debris.

Figure 45.—No terminal structure or erosion protection exists for this outlet works. The embankment around the exit portal has experienced significant erosion.

> where trash cleaning is ordinarily not required, a submerged structure may be appropriate.
>
> The conduit entrance may be placed vertically, inclined, or horizontally, depending on intake requirements. Where a sill level higher than the conduit level is desired, the entrance can be a drop intake similar to the entrance of a drop inlet spillway. A vertical entrance is usually provided for intakes at the conduit level. In certain instances, an inclined intake structure may be placed along the upstream slope of the dam or along the reservoir bank upstream of the dam.

The designer should exercise caution in the design of the trashracks for the intake structure. As releases are made through the intake structure, debris can accumulate on the trashracks. Continued accumulation of debris will gradually begin to clog the trashracks to a point where the internal and external hydrostatic pressures on the intake structure and conduit are no longer balanced. Unless these structures have been properly designed to resist this type of loading, the pressure differential may cause a collapse of the structures. Another concern the designer should be aware of is the accumulation of sediment and debris in the reservoir. Severe storms can wash tree stumps and other large debris (e.g., logs) into the reservoir. Also, if a forest fire occurs in the watershed, this can cause mud, ash, and debris to enter the reservoir. For trashracked intake structures, this debris can accumulate on the trashracks and clog them. For intake structures without trashracks, this can result in plugging of the conduit. The lack of regular testing of gates and valves and reservoir flushing can contribute to this situation. For flood control conduits, the clogging or plugging can result in loss of discharge capacity, which could lead to overtopping of the embankment dam. Clogging and plugging can also affect the operation of downstream turbines (ICOLD, 1994a).

Reclamation's *Design of Small Dams* (1987a, p. 452) provides a good source of information concerning terminal structures. The following has been adapted from that reference:

> Terminal structures.—The discharge from a conduit, whether it be pressure or free flow, will emerge at a high velocity, usually in a nearly horizontal direction. If erosion-resistant bedrock exists at shallow depths, the flow may be discharged directly into the river. Otherwise, it should be directed away from the toe of the embankment dam by a deflector. Where erosion could be a potential problem, a plunge basin may be excavated and lined with riprap or concrete.
>
> When more energy dissipation is required, the hydraulic jump basin is most often used for energy dissipation of discharges. However, flow that emerges in the form of a free jet, as is the case for valve-controlled outlets of pressure

conduits, must be directed onto the transition floor approaching the basin so it will become uniformly distributed before entering the basin. Otherwise, proper energy dissipation may not be obtained.

For further guidance on the design and construction of entrance and terminal structures, see Reclamation's *Design of Small Dams* (1987a), and USACE's *Strength Design for Reinforced Concrete Hydraulic Structures* (1992), *Hydraulic Design of Reservoir Outlet Works* (1980), and *Structural Design and Evaluation of Outlet Works* (2003b).

Chapter 4

Structural Design of Conduits

Conduits through embankment dams differ from nonwater-retaining structures. Conduits have many unique structural design requirements, which the designer must consider in any design. These requirements include:

- Cracking must be minimized to avoid the effects of internal erosion and backward erosion piping. Minimizing cracking will also reduce the vulnerability of reinforcement corrosion. Also, conduits located on weak or compressible foundations must remain watertight during horizontal and vertical movements caused by settlement and spreading of the embankment dam.

- High velocity flow can result in cavitation or erosion.

- Flow within conduits can fluctuate over the year, depending on project requirements.

- Due to the release requirements of the downstream users, conduits may be difficult to shut down for frequent maintenance or repair.

- Tight tolerances are required to maintain properly functioning gates and valves.

The following sections discuss some of the important aspects to consider in the structural design of the conduit. For additional guidance on the structural design of conduits, see Reclamation's *Design of Small Dams* (1987a), and USACE's *Structural Design and Evaluation of Outlet Works* (2003b) and *Culverts, Conduits, and Pipes* (1998a).

4.1 Conduit shape

The primary considerations in selecting the proper shape of the conduit are:

- To promote good compaction of earthfill against the conduit

- To eliminate or minimize the impacts of low density areas caused by difficulties in compaction of the earthfill
- To eliminate or minimize the potential for stress arching in the embankment dam leading to low density zones and hydraulic fracture of the dam
- To eliminate or minimize the potential for differential settlement leading to low density zones and maintain a positive embankment pressure on the conduit
- To allow access for periodic inspection by either man-entry or CCTV
- To allow for future repairs, renovation, or changes in operating requirements
- To allow for the most economical structural design while still addressing all of the items above considerations

Depending on the arrangement of the control features, type of conduit, purpose of the conduit, etc., specific shapes are used. Figures 46 through 50 show examples of shapes typically used for single barrel reinforced cast-in-place concrete conduits. Figure 51 represents a shape commonly used with precast concrete conduits. A box shape has been used in both reinforced cast-in-place and precast concrete applications (Note: Most box shaped conduits have used vertical sidewalls. However, figure 52 shows a preferred shape with sloping sidewalls). A filter is

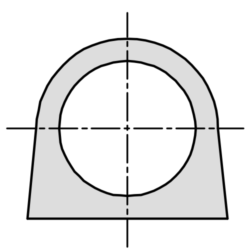


Figure 46.—Conduit shape A

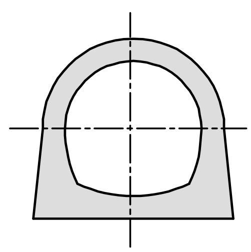


Figure 47.—Conduit shape B

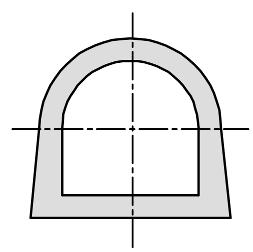


Figure 48.—Conduit shape C

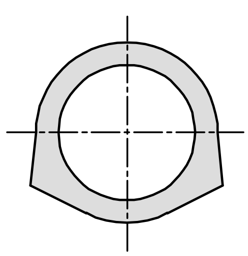


Figure 49.—Conduit shape D

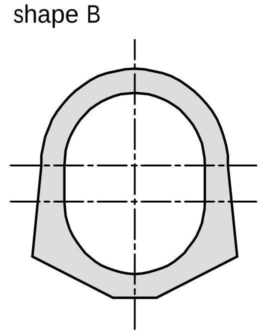


Figure 50.—Conduit shape E

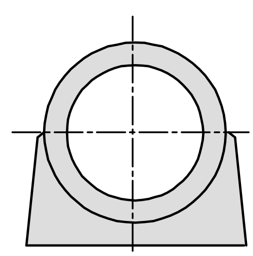


Figure 51.—Conduit shape F

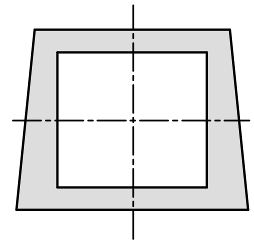


Figure 52.—Conduit shape G

needed with any shape of conduit selected. A filter provides a defense against internal erosion and backward erosion piping resulting from differential settlement of the conduit, hydraulic fracture of the embankment, or deterioration of the conduit. See chapter 6 for guidance on the design and construction of filters.

In some special situations, double or triple barrel conduits may be required. Operational and/or flow capacity requirements usually determine the number of barrels required. Structural design of multibarrel conduits should adhere to the guidance in this section. Figure 53 shows an example of a double barrel conduit under construction.

The following sections discuss conduit shapes that have been used by the major embankment dam design organizations.

4.1.1 Conduit shapes A, B, and C

Conduit shapes A, B, and C (figures 46-48) tend to be less adaptable to changes in loading and stresses than fully circular sections, but provide an exterior surface that is superior for compacting earthfill materials against. Depending on the loading, stress concentrations may be large enough in or near the base that shear stirrups may be required or concrete thicknesses must be increased.

Figures 54, 55, and 56 show the interiors of conduit shapes A, B, and C, respectively. Conduit shape A is typically used for pressure flow. Conduit shapes B and C are typically used for nonpressurized flow conditions. Conduit shapes B and C are also used as the larger downstream access conduit in arrangement 1, as discussed in

Figure 53.—A multibarrel outlet works conduit under construction.

Figure 54.—An example of the interior of conduit shape A.

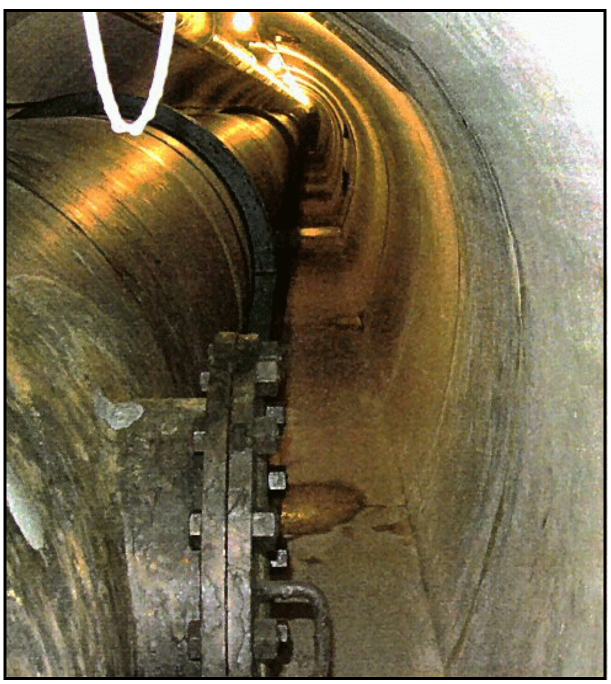


Figure 55.—An example of the interior of conduit shape B.

Figure 56.—An example of the interior of conduit shape C.

section 3.1.1.1. The steel pipe located within these types of conduits is supported on concrete saddles as shown in figure 57. Interior and exterior shapes that are curved can better accommodate high earthfill and water loads. Conduit shapes with flat interior bottoms are generally only used where limited loadings are applied to the conduit. The sides of conduit shapes A, B, and C should be sloped to 1H:10V or more through the im pervious zone of the embankment dam to allow equipment to compact the earthfill directly against the conduit. Contractors may use straight cords to avoid using curved forming techniques for the exteriors of shapes A, B, and C. If straight cords are used, the designer must ensure that stress concentrations do not change and adequate concrete thickness is provided.

4.1.2 Conduit shape D

Conduit shape D (figure 49) has structural attributes similar to those of the circular section (i.e., tends to be more adaptable to changes in loading and stresses that may be caused by unequal fill or foundation settlement). The interior of shape D will be similar to the shapes in figure 54. The sides of the conduit should be sloped to 1H:10V or more through the impervious zone of the embankment dam to allow equipment to compact the earthfill directly against the conduit. Contractors may use straight cords to avoid using curved forming techniques for the exterior of this shape. If straight cords are used, the designer must ensure that stress concentrations do not change and adequate concrete thickness is provided.

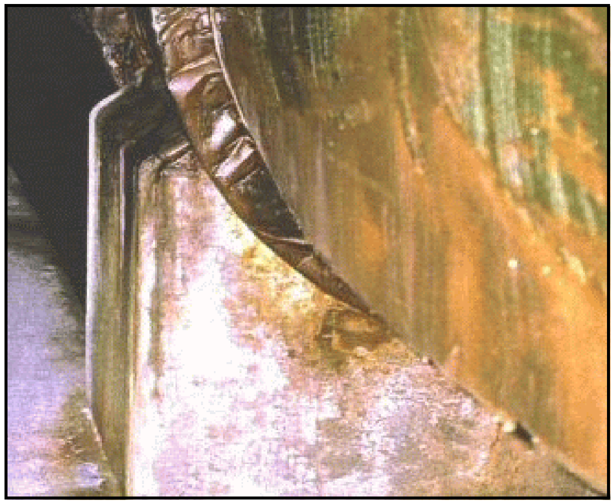


Figure 57.—Concrete saddles are used to support steel pipe located within a larger access conduit.

4.1.3 Conduit shape E

Conduit shape E (figure 50) is formed by separating two semicircular sections by short side sections. Figure 58 shows an example of the interior of conduit shape E. Conduit shape E generally achieves maximum economy of materials by mobilizing more of the relieving fill pressure. The sides of the conduit should be sloped to 1H:10V or more through the im pervious zone to allow equipment to compact the earthfill directly against the conduit. Contractors may use straight cords to avoid using curved forming techniques for the exterior of this shape. If straight cords are used, the designer must ensure that stress concentrations do not change and adequate concrete thickness is provided.

4.1.4 Conduit shape F

An externally shaped circular conduit is more adaptable to changes in loading and stresses that may be caused by unequal fill or foundation settlement (i.e., better distribution of loads). As a flow surface, a circular internal cross section is used primarily for pressure flow conditions, since it is a very hydraulically efficient shape. The interior of a circular conduit would be similar to figure 54. The use of an externally shaped circular conduit through an embankment dam should be carefully evaluated due to concerns with the difficulty or inability to uniformly compact the earthfill around the conduit. Precast concrete pipe is the most often used externally shaped circular conduit. The earthfill beneath the haunches of the conduit cannot be adequately compacted with pneumatic tired equipment and requires compaction with hand held tampers. Efforts to obtain proper compaction using hand tampers could cause movement or displacement of smaller conduits. Improper compaction of the earthfill around the conduit and movement of the conduit can result in differential

Figure 58.—An example of the interior of conduit shape E.

settlement and hydraulic fracture. The use of externally shaped circular conduits differs between the major dam building agencies. Reclamation does not use externally shaped circular conduits in their designs due to concerns about the inadequate compaction of earthfill against the conduit. NRCS allows use of externally shaped circular conduits, if they are constructed on cradles or bedding. The use of externally shaped circular conduits (e.g., precast concrete pipe) requires thorough inspection and strict adherence to proper construction techniques to achieve quality assurance of earthfill compaction. Figure 51 shows an externally shaped circular conduit on a cradle. The sides of the cradle should be sloped to 1H:10V or more through the im pervious zone of the embankment dam to allow equipment to compact the earthfill directly against the conduit. For guidance on the use of cradles and bedding, see section 4.1.6.

4.1.5 Conduit shape G

A box (or rectangular) shape has historically been used by some agencies, such as the NRCS. Other agencies, such as Reclamation, no longer use this shape due to the objectionable consequences. Box shapes using both reinforced cast-in-place and precast concrete have been used in the past with varying degrees of success. The shape shown in figure 52 illustrates a box shape with the preferred 1H:10V or more side slopes for improved compaction of earthfill against the conduit. Historically, the box shape has used vertical sides. The designer must fully consider the advantages and disadvantages of this shape before making a final selection for use. Figure 59 shows an example of the interior of a box shape.

The advantages of a box shape include:

- The flow capacity of a box shaped conduit is typically greater than other shapes. Using one conduit large enough to convey the design flow is preferable to having multiple smaller conduits penetrating the embankment.

- Other shapes may be more costly because of the more complicated forming required. However, contractors often submit a request to use a series of straight chords in lieu of constructing the circular outside cross section to avoid using curved forming techniques. This request is usually acceptable, as long as the thickness requirements for the particular shape are not compromised.

- Forming the transition section between inlet risers and box shaped conduits is easier than the forming required for the transition section and circular conduits.

The disadvantages of a box shape include:

- Stress concentrations occur within the concrete at the corners of the box. Reinforcement designs must thoroughly address this issue.

- Arching of the embankment fill is more likely for the box shaped conduit. This may result both in stress concentrations and low lateral stresses favorable to hydraulic fracture in zones of the earthfill surrounding the conduit. The filter diaphragm or chimney filter design for this shape conduit may need to be more robust than for conduits with more favorable configurations for these reasons.

- The sharp outside corners at the top of the box shaped conduit can cause undesirable stress concentrations in the fill. Small tension zones occur in the fill adjacent to the upper portions of the conduit. The tensile stresses that develop may cause formation of tension cracks in the fill. These cracks, combined with the possibility that fill will pull away from the side walls (unless they are sloped), may induce internal erosion near and along the conduit. Casagrande and Covarrubias (1970, p. 17) discuss the problems with uneven stress levels in earthfill next to vertical walls transverse to embankments in more detail.

- While reinforced cast-in-place concrete conduits with a box shape have been used with suitable precautions, using box-shaped precast concrete conduits is discouraged in embankment dam design. Constructing precast concrete conduits with joints that are adequately watertight is difficult. Even if joint fillers are used in the joints, the probable movement of articulated joints of this

Figure 59.—A box shape is not commonly used for conduits. The designer needs to carefully consider the advantages and disadvantages of this shape.

shape is likely to result in poor watertightness. Reinforced cast-in-place box conduits are designed with reinforcement extending across the joint and waterstops to improve watertightness.

- The economy in the box shape is typically due in part to the straight sides, making forming less expensive, and the flat bottom, making foundation excavation easier. However, best practice for compaction requires the sides of the conduit to be sloped to 1H:10V or more through the im pervious zone of the embankment dam to allow equipment to compact the earthfill directly against the conduit. The preferred shape for a box conduit is shown in figure 52.

4.1.6 Cradles and bedding

Externally shaped circular conduits should be constructed on concrete cradles to avoid problems with compacting beneath the haunches of the conduit. Cradles are typically used in conjunction with precast concrete pipe. The cradle should be formed concrete that provides vertical, longitudinal, and lateral structural support to the conduit. The cradle should extend for the full length of the conduit and should encase the lower half of the conduit extending up to the springline. Conduit shape F illustrates a circular conduit with a concrete cradle extending up to the springline.

The design of the conduit support through an embankment dam will depend upon the hazard class potential associated with the dam, compressibility of the foundation, and the use of the conduit. As noted previously, precast concrete pipe should not be used in pressurized applications within significant and high hazard embankment dams, because failure of a single pipe joint or joint gasket could allow pressurized water to come in direct contact with the embankment. Conduits should be constructed on rock or firm foundations whenever possible. When a conduit is founded on a compressible foundation, the designer must exercise care in design of the conduit because of the large settlements that can occur. These settlements can open joints and cause pipe joints to fail.

Different approaches have been used to design conduits on compressible foundations. NRCS uses a joint design for the cradle that allows the articulation and spreading of the conduit and its support system. To allow for joint articulation, the joint is placed at the location of the pipe joint and cradle reinforcement is not allowed to pass through the joint. In addition, the spaces between joints are filled with a compressible material, such as high-density sponge rubber or bituminous fiberboard to allow for articulation of the cradle joints. The NRCS design guidelines for the configuration of pipe cradles are provided in *The Structural Design of Underground Conduits* (1958). Reclamation does not allow precast concrete pipe to be used for conduits within embankment dams and therefore would not use cradles.

The concrete cradle should bond to the conduit. The sides of the concrete cradle should always be sloped at 1H:10V or flatter through the im pervious zone to allow equipment to compact earthfill directly against the cradle. There should be no sharp or protruding corners associated with the cradle that could cause undesirable stress concentrations in the fill. Blocks and wedges are required to support the conduit on grade until the concrete cradle has been placed and cured.

Designers of conduits through low hazard embankment dams often use concrete bedding beneath fully circular conduits. Bedding generally comes up to about 25 percent of the conduit height to provide support and facilitate compaction under the haunches. Bedding often has joints located at the circular pipe joints so as to not interfere with pipe movement. Figure 60 shows a precast circular conduit using bedding for support. For guidance on the use of bedding in conjunction with fully circular conduits, see NRCS's *The Structural Design of Underground Conduits* (1958) and USACE's *Culverts, Conduits, and Pipes* (1998a). The selection of whether to use cradles or bedding is typically a function of the height of the embankment dam. Cradles are often used for higher dams, where more lateral support is required. Regardless of whether a cradle or bedding is used, the use of a filter diaphragm or collar is a valuable defensive design measure that should be employed, even for low hazard classification sites with favorable conditions.

The designers of low hazard embankment dams have sometimes considered the use of flexible conduits (i.e., HDPE). Cradles and bedding should not be used with

Figure 60.—Precast concrete using bedding as support.

flexible conduits, since they require deflection to develop strength in the conduit walls, and cradles and bedding would prevent this deflection. If a flexible conduit is constrained by a cradle, even one that extends up to the springline, the conduit could be overstressed beyond its design strength. Consideration for fully encasing the flexible conduit in concrete should be evaluated, and the sides of the concrete should be sloped at 1H:10V or flatter. Also, the use of a filter diaphragm or collar should be considered.

4.2 Structural design and construction considerations

A conduit must withstand internal fluid and vacuum pressures, external hydrostatic loadings and buckling pressures, embankment loads, surface surcharge loads, construction loads, operational and maintenance loadings, and combinations of these loads. Designers should also consider the effects of horizontal and vertical movements that may occur from settlement and spreading of the embankment and foundation. These movements may result in loads on the conduit in excess of loads predicted from normal static computations. Excessive movements, both vertical and lateral, can occur when conduits have foundations that are either weak or compressible, or both. Poorly compacted embankment dams can spread from shear deformations, which also can lead to lateral spreading of the conduit. Some shales may be relatively incompressible but have anisotropic shear strength conditions that allow excessive lateral movement without much compression.

Loading conditions typically analyzed include:

- *Usual*.—This loading condition includes normal operating conditions with the reservoir at or near the normal water surface, involving combinations of vertical soil load (due to the weight of the fill column above the conduit); horizontal soil load; external and internal hydrostatic pressure loads; and the vertical foundation reaction (generally assumed to be equal to the vertical soil load plus weight of the conduit).

- *Unusual*.—This loading condition includes loads associated with a high reservoir water surface and high discharges due to flood conditions. However, since floods are typically short lived, the conduit may not come under increased external hydrostatic pressure. The difference in loading conditions between usual and unusual may be limited to increased internal hydrostatic pressure.

- *Extreme*.—This loading condition is associated with usual loading conditions plus earthquake loading. Depending on the "criticality" of the conduit (i.e., consequences due to failure or severe damage of the conduit under seismic loading), a range of earthquakes should be considered, including seismic loading to the maximum credible earthquake (MCE). Conduits are "low profile" structures and tend to have a high fundamental frequency. Unless the conduit is founded on deep layers of soil where peak ground accelerations could be magnified, peak ground accelerations are typically assumed to act on the conduit. If the fundamental frequency of the conduit is greater than 33 hertz, a pseudostatic approach generally gives reasonable results; otherwise a more detailed seismic analysis may be required (i.e., response spectrum method or time-history method). Additional factors that may affect loading conditions are the type of foundation, method of bedding, flexibility of the conduit, and soil characteristics (internal angle of friction, unit weight, homogeneity, consolidation properties, cohesiveness, and moisture content).

- *Construction*.—This loading condition pertains to loads resulting from construction activities. These activities may include construction vehicles or equipment moving or working adjacent to or on the conduit and are considered short term loadings.

In soils, a variety of factors should be considered when designing a conduit, including angle of internal friction, density, homogeneity and water content of the soil. Various combinations of conditions and loadings will need to be evaluated by the designer to ensure a long service life for the conduit.

The Marston theory of embankment pressures has typically been adopted for calculating loads on a conduit that is partially or fully projecting above the original ground surface. Using the Marston theory, vertical load on the conduit is considered to be a combination of the weight of the fill directly above the conduit and the frictional forces, acting either upward or downward, from the adjacent fill. This loading is also known as the "projection" condition. When the adjacent fill settles more than the overlying fill, downward frictional forces are induced, which can increase the resultant load on the conduit by as much as 50 percent of the weight of the fill above the conduit (figure 61). Conversely, a greater settlement immediately above the conduit results in an arching condition, which reduces the load on the conduit by as much as 50 percent of the weight of the fill above the conduit (figure 62). This loading is also known as the "trench" condition. This condition may occur for conduits placed in a trench. Some publications indicate that higher increases in load may be applicable (by as much as 200% over the prism load based on the Marston theory); see NRCS's *The Structural Design of Underground Conduits* (1958, p. 1-7). The practice of constructing conduits within trenches with vertical side walls in embankment dams is not recommended. Loss of positive contact of the fill next to the conduit is possible due to the effects of arching. For guidance on the selection of proper excavation for side slopes, see section 5.1.

The designer should use caution in designing conduits through embankment dams when the overburden is greater than about 100 feet. This is especially true for conduits that are not founded upon firm rock foundations and not constructed of reinforced cast-in-place concrete. For large embankment dams, the fill height for which a conduit can be economically designed is limited. Conduits designed for embankment dams with fill heights greater than 100 feet should only be attempted by very experienced designers. Greater fill heights result in extremely high stresses, excessive conduit wall thicknesses, and/or reinforcement requirements. For high fill applications, designers may want to consider a tunnel rather than a conduit.

The Marston theory is typically considered as a very conservative approach to quantifying loads upon a conduit for a fully projecting condition (Reclamation, 2001a, p. 8). More detailed tools are available that allow for two- and three-dimensional and time-dependent analysis. This type of analysis involves the use of soil interaction models. Soil interaction models can accommodate large displacements, strains, and nonlinear material behavior. Programs, such as Fast Lagrangian Analysis of Continua (FLAC) and Plaxis are ideally suited for modeling the stages of construction for the conduit, namely excavation, construction of the conduit, and then construction of the embankment over the conduit. Modeling and analyzing the stages of construction enables the program to accurately calculate stresses within the conduit after the embankment has been placed. These stresses are then used in the design of the conduit. Sensitivity studies should always be run to account for possible variations in material properties, foundation settlements, and construction conditions. The designer may find it prudent to compare conduit loadings developed using soil interaction models with the results obtained using classic loadings from references, such as *Design of Small Dams* (Reclamation, 1987a) and *Culverts, Conduits, and Pipes* (USACE, 1998a).

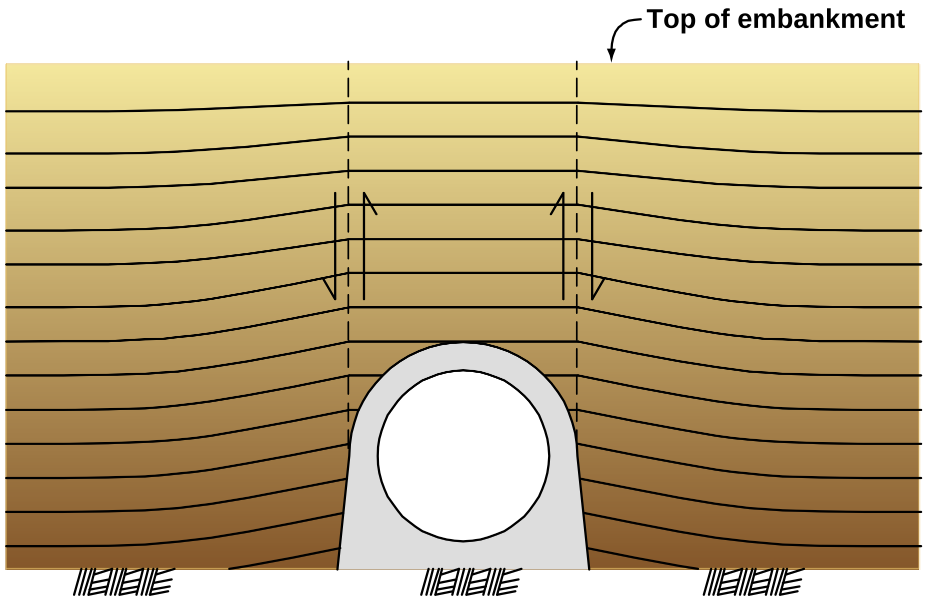


Figure 61.—Conduit constructed prior to earthfill placement. Friction factors increase embankment load on the conduit as adjacent earthfill settles more than earthfill overlying the conduit.

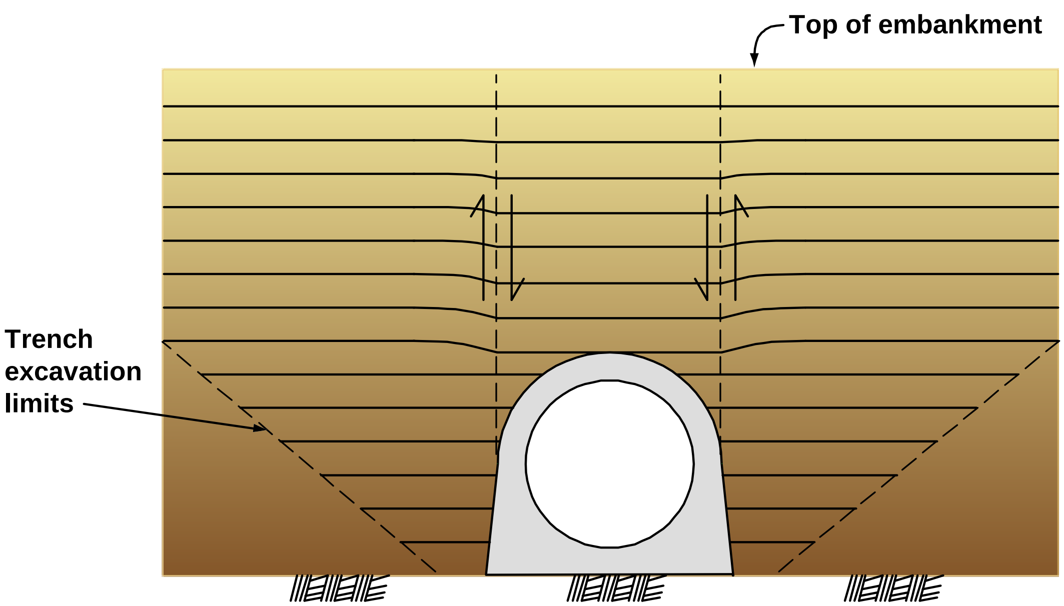


Figure 62.—Conduit constructed within a trench excavated into the embankment. Friction factors decrease embankment load on the conduit as earthfill over conduit settles downward relative to adjacent embankment.

4.2.1 Concrete

Concrete conduits are generally considered to be rigid structures. Plastic and metal pipes used in the construction of conduits are considered to be flexible. A flexible pipe is generally considered to be able to deflect without structural distress to the pipe or to any coating or lining. A flexible pipe derives its external load capacity from its flexibility. Under load, the pipe tends to deflect, developing soil support at the sides of the pipe. Flexible pipe used in conduit construction within significant and high hazard embankment dams should be encased in cast-in-place reinforced concrete to provide shapes that allow for good compaction of embankment materials against the conduit. Flexible pipe used in conduit construction within low hazard embankment dams is often not encased in cast-in-place reinforced concrete.

As discussed in section 2.1, concrete used for conduits is typically either reinforced cast-in-place or precast concrete. Specific guidance pertaining to these materials is discussed in the following sections.

4.2.1.1 Reinforced cast-in-place concrete

Reinforced cast-in-place concrete for conduits is typically designed by either the working stress design (WSD) or strength design (SD) methods or the alternate design method, which is similar to WSD, but includes some SD features. The WSD method proportions reinforced cast-in-place concrete members for prescribed service loads at stresses well below the ultimate and assumes linear distribution of flexural stresses and strains. The SD method requires service loads to be increased by specified load factors and computed nominal strengths to be reduced by specified phi factors. Various editions of the American Concrete Institute building code (ACI 318) describe these design methods.

Reinforced cast-in-place concrete conduits are normally subject to different loadings, more severe exposure conditions, and more restrictive serviceability requirements than buildings. The major embankment design agencies and ACI have more restrictive versions of WSD and SD methods that are appropriate for reinforced cast-in-place concrete conduit design. A brief summary of the reinforced cast-in-place concrete design philosophies used for conduits by these agencies includes:

- *Reclamation*.—Some existing conduits in Reclamation's inventory of embankment dams are as much as 100 years old. Reclamation designed these conduits with the WSD method. Reclamation has also used the WSD method for modifying any existing structures that were originally designed using WSD. The WSD method is preferred for any feature considered to be an integral part of a hydraulic structure, such as a spillway or outlet works, where crack control limitations are important considerations.

For high-flow-velocity, high-flow-volume structures, cracking can cause significant hydraulically induced structural problems, such as cavitation, uplift, and binding of gates or valves. This is considered especially important where a conduit passes through or under an embankment dam and seepage could be detrimental to the safety of the dam or where shutting down a structure for maintenance or repair can be difficult or very costly. Embankment dams and appurtenant structures are usually designed for larger load factors than those used for buildings. Allowable stresses of 1,800 lb/in^2 (compressive strength for concrete) and 24,000 lb/in^2 (minimum specified tensile stress for Grade 60 reinforcement) are used in WSD.

With the issuance of ACI 318-02 (2002), the alternate design method was removed from the code. However, ACI 318-02 Commentary Section R1.1 states, "the Alternate Design Method of the 1999 code may be used in place of applicable sections of the 2002 code." In order to fully address the requirements for hydraulic structures for its dams, Reclamation is currently developing guidance for the design of reinforced cast-in-place concrete structures with unique design requirements. This guidance will include recommended codes, design aids, and references for use in design. This guidance will allow use of the strength design method for structures where crack control and/or deflection limitations have been adequately addressed. Designers may want to consider this guidance when it becomes available, for use in future design work.

- *USACE*.—The USACE uses the SD method in accordance with ACI 318, as modified in USACE's *Strength Design for Reinforced Concrete Hydraulic Structures* (1992, pp. 1-2 and 1-3). Load factors that bear a close resemblance to ACI 318 are modified by a hydraulic factor to account for the serviceability needs (crack control) of hydraulic structures. This modification factor is intended to ensure that the resulting design was as conservative as if the working stress design were used.

- *NRCS*.—The NRCS uses either the WSD or SD methods for site cast "service hydraulic structures," which includes conduits through embankment dams. The current NRCS WSD criteria follow ACI 318-77 (1977) with several exceptions, including (1) allowable concrete compressive stress limited to 0.40 f'c and (2) allowable steel tensile stress limited to 20,000 lb/in^2.

  The current NRCS SD criteria also follow ACI 318-77 with several exceptions, including (1) single load factor of 1.8 applied to all loads and (2) steel design yield strength limited to 40,000 lb/in^2 for all grades. Current NRCS WSD and SD criteria are styled to produce basically the same concrete design results. Both are intended to provide lower stress levels than the ACI Code for buildings to ensure long term durability in aggressive wet/dry, freeze/thaw environments.

The NRCS is developing updated concrete design guidance under contract to a consultant. This guidance will adopt current ACI codes and explain their application to different NRCS concrete structures. The guidance will also include concrete joint design details and numerous example problems. The expected publication date is 2006.

The designer should consider adoption of ACI 350 Code Requirements for Environmental Engineering Concrete Structures. ACI 350-01 (2001) states: "The code portion of this document covers the structural design, materials selection, and construction of environmental engineering concrete structures. Such structures are used for conveying, storing, or treating liquid, wastewater, or other materials, such as solid waste. They include ancillary structures for dams, spillways, and channels." These structures are subject to uniquely different loadings, more severe exposure conditions and more restrictive serviceability requirements than normal building structures. ACI 350-01 further states:

> The liquid-tightness of a structure will be reasonably assured if:
>
> a) The concrete mixture is well proportioned, well consolidated without segregation, and properly cured.
>
> b) Crack widths and depths are minimized.
>
> c) Joints are properly spaced, sized, designed, waterstopped, and constructed.
>
> d) Adequate reinforcing steel is provided, properly detailed, fabricated, and placed.
>
> e) Impervious protective coatings or barriers are used where required.

Reinforced cast-in-place concrete conduits are usually transversely designed as rigid structures, whereby higher vertical loads relative to horizontal loads are supported by the transverse bending and shear strength of the conduit. Various loading conditions that maximize potential vertical loads while minimizing potential horizontal loads and vice versa are normally investigated to conservatively determine the required transverse bending strength of the conduit. Uplift pressures should be assumed to act uniformly across the entire width of the conduit. Internal hydrostatic pressures must also be considered in the design. References, such as Reclamation's *Design of Small Dams* (1987a) and USACE's *Conduits, Culverts, and Pipes* (1998a) provide further details on the structural design of concrete conduits.

Reinforced cast-in-place concrete conduits may also need to be longitudinally designed for tension stresses due to the friction of the spreading embankment along

Figure 63.—Longitudinal reinforcement across a conduit joint experienced tensile failure caused by lateral spreading of the embankment dam.

the conduit and for bending stresses due to nonuniform foundation conditions along the length of the conduit section. Figure 63 shows an example of longitudinal reinforcement that experienced tensile failure caused by lateral spreading of the embankment dam.

Reclamation's experience has shown that cracking in reinforced cast-in-place conduits due to shrinkage and temperature can be minimized by placing conduits in 12- to 16-foot sections (figure 64). The interfaces between conduit sections are typically control joints. Control joints are used to provide for control of initial shrinkage stresses. Waterstops should be used across all control joints, and a bond breaker, such as curing compound, should be applied to control the joint surfaces to direct cracking toward the joints. The longitudinal reinforcement is continuous across the control joint to limit movement between adjoining ends of conduit sections. The conduit sections should be constructed in an alternating pattern, such that any concrete volume shrinkage occurs prior to adjoining conduit sections being placed. The preferred placement method for transverse sections of concrete in small

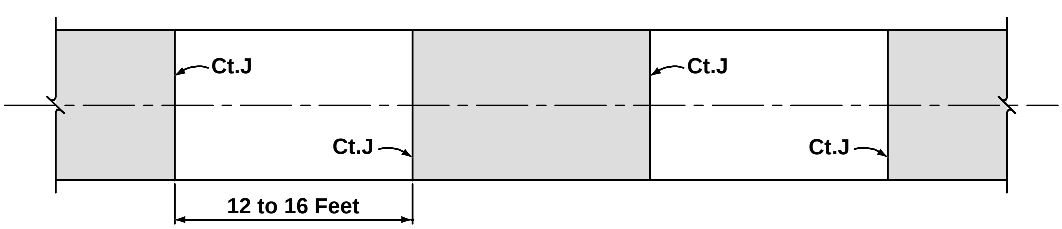


Figure 64.—Concrete placement for a reinforced cast-in-place conduit.

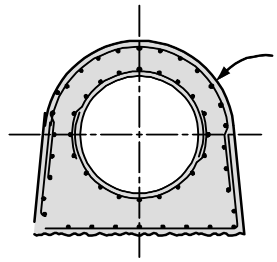


Figure 65.—Concrete placement for a reinforced cast-in-place conduit.

conduits (less than 3 feet in diameter) is by continuous placement, to ensure monolithic integrity (figure 65). In larger conduits, horizontal construction joints typically located at springline have been used to facilitate concrete placement. Horizontal construction joints are also useful in preventing flotation of steel liners during concrete placement. For guidance on the use of control and construction joints and waterstops, see section 4.3. For guidance on construction practices for the placement of concrete, see Reclamation's *Design of Small Dams* (1987a, p. 659), and the USACE's *Standard Practice for Concrete for Civil Works Structures* (2001b).

4.2.1.2 Precast concrete

Precast concrete pipes (RCP, RCCP, PCCP) are designed as rigid structural elements in the same fashion as reinforced cast-in-place concrete conduits. Internal and external loads are computed, and various load combinations are considered as acting on a unit length of pipe. Thrusts and moments at various points around the perimeter of the pipe are calculated. Required reinforced concrete proportions, including concrete thickness, reinforcing steel amount, steel cylinder thickness, and prestress tension, are determined as required for component concrete, reinforcing steel, and prestressing wire strengths, respectively.

Reinforced concrete design procedures and extensive examples specifically for RCP and RCCP are contained in AWWA M9, (1995). Prestressed concrete design procedures for PCCP are standardized in AWWA C304, (1999a). The designer should use these procedures carefully since they are basically targeted toward pipelines where internal pressures are high, but external loads are low relative to most embankment dams. Also, for prestressed pipes, the procedures assume that pipelines are usually full of water over their service life. Some embankment dams, particularly common NRCS flood control dams, are seldom full of water, and lesser relative humidity may allow concrete shrinkage and loss of prestress. The reader is directed to the *Introduction* for examples of how design standards have been misused.

The reinforced concrete design of some types and sizes of precast concrete pipes has been standardized by manufacturers. ASTM C 361 contains tables of reinforced concrete design proportions for various sizes and classes of pipe. Sizes range from 12 to 108 inches in inside diameter. Classes range from A-25 to D-125 where A, B, C, D represent fill heights over the pipe of 5, 10, 15, 20 feet respectively, and 25, 50, 75, 100, 125 represent internal water pressure in pounds per square inch. The designer should use these standardized designs cautiously since the standard assumes a simple soil prism load instead of a positive projecting condition typical of most embankment dams. Unique designs can be accommodated in the Standard Specification for higher external loads.

An alternative to a theoretical reinforced concrete design procedure is the indirect design procedure based on product testing. Most concrete pipe plants have the equipment to do a three-edge bearing load test on full size pipe specimens. The NRCS requirements for precast concrete pipe tested in accordance with ASTM C 497 shall demonstrate the following bearing loads:

- For RCP or RCCP manufactured according to ASTM C 361, AWWA C300 (2004a), or AWWA C302 (2004b), the load required to produce a 0.01-inch crack, 1 foot in length

- For PCCP manufactured according to AWWA C301 (1999b), the load required to produce a 0.001-inch crack, 1 foot in length, or the load 10 percent greater than the specified three-edge bearing strength, whichever occurs first

The NRCS has commonly used PCCP in most of their high and significant hazard and larger low hazard embankment dams over the past 50 years. NRCS worked with the American Concrete Pressure Pipe Association to develop design curves as a basis for proof of strength of AWWA C301 (1999b) PCCP. The curves, based on test data, show conservative relationships between the resultant concrete core stress and the three-edge bearing strength for various pipe sizes. Resultant concrete core stress can be calculated from the concrete thickness, cylinder thickness, prestress wire amount, and wire wrapping stress. NRCS's *Certification of Prestressed Concrete Cylinder Pipe*, (1982) contains these design curves and procedure.

The NRCS uses two construction specifications for concrete pipe, Construction Specification 41 (2001a) and Construction Specification 42 (2001b). Construction Specification 41 describes the materials and acceptable construction procedures for reinforced concrete pressure pipe conduits. This specification is commonly used for contracts involving principal spillway conduits on embankment projects designed by NRCS. Construction Specification 41 refers to Material Specification 541 (2001c) which describes the minimum material requirements for reinforced concrete pressure pipe.

Construction Specification 42 is for other types of conduits, including nonreinforced conduits, such as culverts and drainage pipe. Specification 42 refers to several different material specifications, depending on the specific application being constructed. Material Specification 541 is referenced for reinforced concrete pressure pipe, Material Specification 542 (2001d) is for concrete culvert pipe, and Material Specification 543 (2001e) is for nonreinforced concrete pipe. A few of the early embankment dams constructed by NRCS used nonreinforced conduits, but the majority of the embankment dams constructed by NRCS have used reinforced conduits.

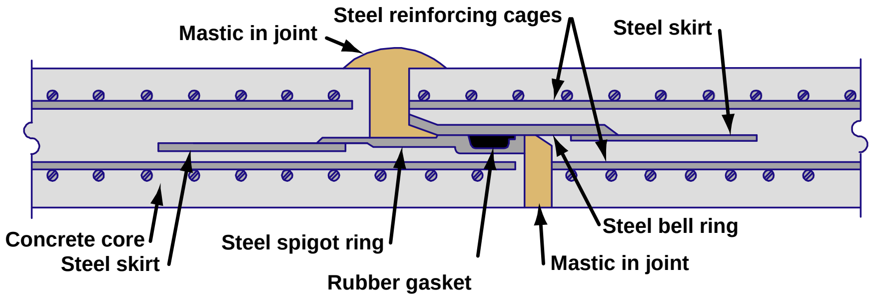
The NRCS process for designing PCCP in embankment dams uses NRCS TR5, *The Structural Design of Underground Conduits*, (1958) to determine required three-edge bearing strength for a conduit considering embankment, foundation, and cradle or bedding conditions. Such three-edge bearing strength is specified on the construction plans. Pipe supplied to the construction site is either tested as described previously, or documentation is submitted indicating pipe component parameters, which can be checked against the NRCS Specification Note No. 5 design curves to ensure adequate three-edge bearing strength.

As with reinforced cast-in-place concrete, individual precast pipe sections may also need to be longitudinally designed for tensile stresses due to spreading of the embankment dam along the conduit. For PCCP, the strength of the steel cylinder resists these tensile stresses. Sample calculations can be found in NRCS's *Use of AWWA C302 Pipe for Principal Spillway Conduit* (1970).

Figures 66 through 69 show the different arrangements of reinforcing steel and/or prestressing wire used in the various types of precast concrete pipe.

4.2.2 Plastic

Currently, the primary source of design information for plastic pipe is from manufacturers. However, most of this information is targeted to sewer and water distribution pipes and does not address the unique factors involved in using plastic pipe within embankment dams. FEMA is sponsoring the development of a supplemental "best practices" guidance document pertaining solely to plastic pipe used in embankment dams. This document will contain detailed procedures and guidelines for design, inspection, maintenance, and repair of plastic pipe. The guidance document will be based on experience provided from experts in the fields of civil and geotechnical engineering and construction. The expected publication date is 2006. This document will be made widely available for use by the dam safety community. Interim guidance can be found in NRCS's *Structural Design of Flexible Conduits* (2005). This reference provides design guidance for flexible pipe materials, including metal and plastic.



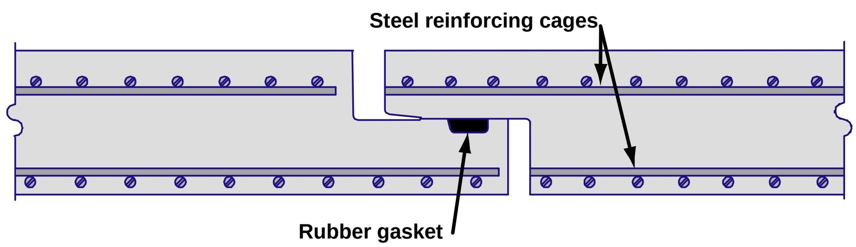


Figure 66.—Reinforced concrete pipe (RCP) details. The top figure illustrates pipe with steel joint rings, and the bottom figure illustrates a concrete joint.

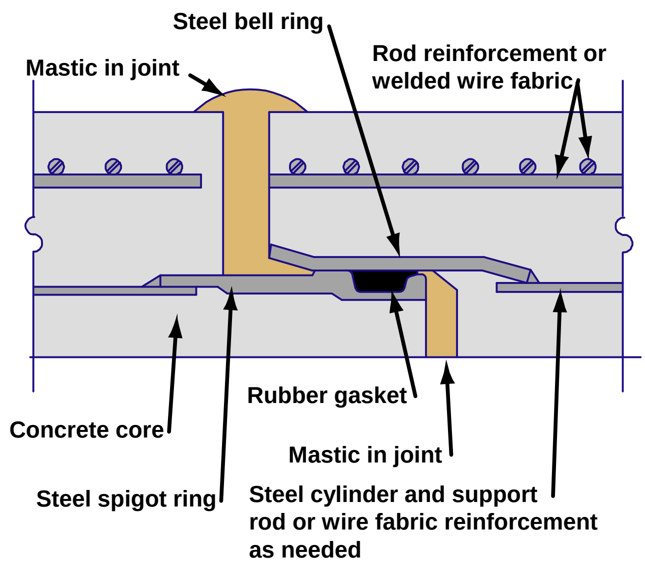


Figure 67.—Reinforced concrete cylinder pipe (RCCP) details.

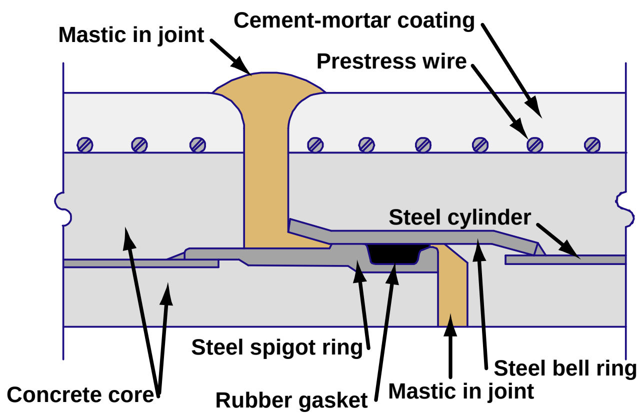


Figure 68.—Prestressed concrete cylinder pipe (PCCP) details (lined cylinder).

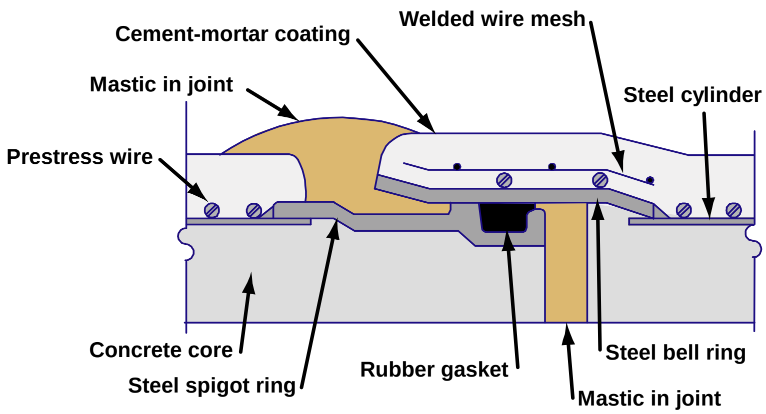


Figure 69.—Prestressed concrete cylinder pipe (PCCP) details (embedded cylinder).

For guidance on design and construction parameters pertaining to:

- *Thermoplastic.*—See section 12.1.1.
- *Thermoset plastic.*—See section 12.2.

Plastic pipe used within low hazard embankment dams is often not encased in reinforced cast-in-place concrete. Use of plastic pipe in new, low hazard embankment dams is generally limited to small diameters (less than 12 inches). However, use of a filter diaphragm or collar is a valuable defensive design measure, even for low hazard classification embankment dams with favorable site conditions. Some designs may not employ a filter diaphragm around the conduit, but eliminating this valuable and relatively inexpensive feature should be carefully considered and justified based on extremely favorable soil conditions, good conduit construction materials and methods, reliable construction practices, and favorable foundation conditions.

4.2.3 Metal

As discussed in the previous section, NRCS's *Structural Design of Flexible Conduits*, (2005) provides design guidance for flexible pipe materials, including metal and plastic. In general, the following guidance should be considered for conduits using metal pipe:

- *Steel pipe*.—Steel pipe should be designed in accordance with industry accepted methods, such as found in AWWA M11 (2004), Amstutz (1970), and Jacobsen (1974). For guidance on the design of steel pipe used within conduits, see section 12.1.2.

- *CMP*.—CMP should only be used for nonpressurized applications in low hazard embankment dams. CMP should be designed in accordance with industry-accepted methods as found in the American Association of State Highway and Transportation Officials' (AASHTO), *Standard Specifications for Highway Bridges* (2002) or the American Iron and Steel Institute's (AISI), *Handbook of Steel Drainage and Highway Construction Products* (1994). USACE's *Culverts, Conduits, and Pipes* (1998a) provides guidance for CMP used in rural levee systems and drainage culverts. Certain aspects in that reference may apply to CMP used in low hazard embankment dams.

Metal pipe used within low hazard embankment dams is often not encased in reinforced cast-in-place concrete. However, as discussed previously with plastic pipe, the use of a filter diaphragm or collar is a valuable defensive design measure, even for low hazard classification embankment dams with favorable site conditions.

4.3 Watertightness

The major dam-building agencies require conduits within an embankment dams to have watertight joints. The degree of water tightness depends on the anticipated hydrostatic head either inside or outside of the conduit. For example, pressurized reinforced cast-in-place concrete conduits are waterstopped and have longitudinal reinforcement extending across the joint. In some cases, a welded steel liner may be used for additional protection. The following sections discuss guidance pertaining to the watertightness of concrete conduits. Plastic and metal pipe are frequently used in the renovation of existing conduits; for guidance on watertightness using these materials, see chapter 12.

If the joints between the ends of conduit sections separate or develop other defects, the conduit may develop leaks. This leakage can lead to the development of internal erosion or backward erosion piping failure mechanisms. The designer should carefully consider the important parameters related to watertightness, such as:

- Conduit joints
- Barriers within joints

Guidance pertaining to these parameters is discussed in the following sections.

4.3.1 Conduit joint

Conduit designers should be aware that foundation conditions are usually not homogenous along the alignment of the conduit. Variable foundation conditions can result in abrupt changes in the foundation settlement of a conduit beneath an embankment dam, causing large relative movements and failure of the conduit. A properly designed joint will limit vertical and transverse displacement of conduit sections relative to each other as the embankment dam settles. A properly designed joint will also accommodate rotation and longitudinal movement while retaining watertightness in the conduit. Figure 70 shows a joint within an outlet works conduit that has settled differentially and is allowing seepage to enter through the joint. Designers must estimate the maximum joint elongation that may occur from the compressibility of the foundation as accurately as possible. The predicted joint elongation depends on the shear strength of the foundation, the estimated settlement of the foundation, the configuration of the embankment dam, and the lengths of conduit joints used in the design. If the predicted elongation is greater than the designed joints can accommodate, changes to the design are necessary. Design changes may involve using shorter lengths of conduit, removing compressible foundation soils and replacing them with compacted backfill, and flattening the slopes of the embankment dam.

Embankment dam settlement may not always be uniform, as predicted by analyses, and is often be erratic and can result in abrupt joint displacements in certain situations. Abrupt joint displacements may be the result of localized joint movement, and settlement can be more extensive than predicted by theoretical analysis. Figure 71 illustrates how actual settlement can differ from theoretical settlement. Abrupt joint displacements may be more likely for conduits that are constructed using precast concrete pipe than for conduits constructed with reinforced cast-in-place concrete. The reason for this difference is that reinforced cast-in-place concrete conduits are constructed with longitudinal reinforcement extending through the joint (control joint), which allows the conduits to bridge over a weak foundation and spread the load more uniformly over more of the conduit foundation.

Figure 70.—Water seeping through a joint in an outlet works conduit. The joint has experienced differential settlement. This joint had no longitudinal reinforcement extending across the joint. The mortar joint filling has cracked and deteriorated.

Based on observations of conduits through 20 selected NRCS dams constructed during the 1960s on compressible foundations (Casagrande Volume, 1973, p. 235):

- 70 percent of the joint opening occurred during construction of the embankment dam.

- Additional measurement of joint openings several years after construction showed negligible increases.

Conduits constructed on compressible foundations are more likely to experience joint spreading problems. Special attention should be given to evaluating the compressibility and shear strength of these soil types. Performing field vane shear tests and similar evaluations are appropriate to evaluate the undrained strength of these types of foundations. Available references and procedures for predicting the amount of conduit spreading based on foundation consolidation and shear strength parameters are not well known and can be inappropriately used by inexperienced designers. The NRCS's *Computation of Joint Extensibility Requirements* (1969) uses the predicted vertical strain beneath the conduit, the shear strength of foundation soils, and the geometry of the embankment and foundation in an attempt to predict the horizontal strain at the conduit. Additional information on conduit extension can be found in the Casagrande Volume (1973, pp. 209-237). For an example of a project where spreading of conduit joints occurred, see the case history in appendix B for Little Chippewa Creek Dam.

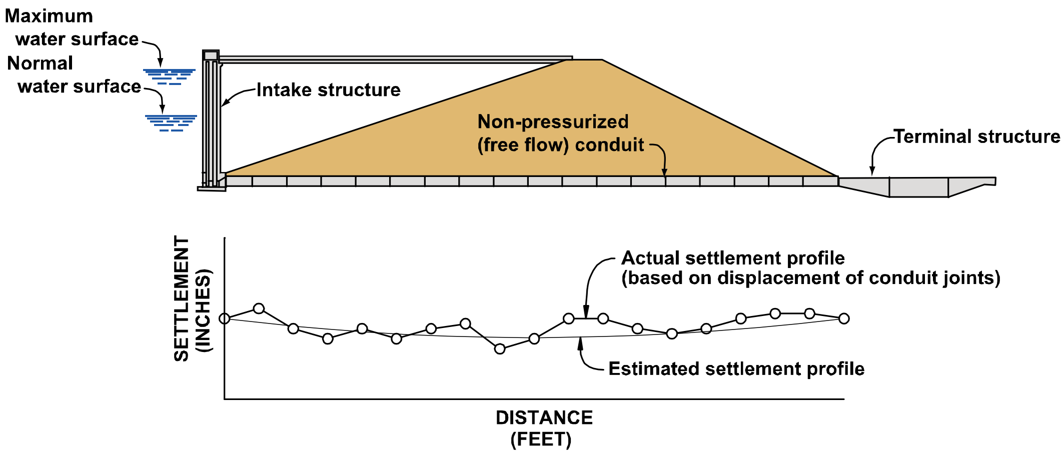


Figure 71.—Actual embankment dam settlement can differ from predicted settlement.

Excessive lateral movement of the embankment/foundation system can also occur when thin weak layers in the foundation are loaded beyond their shear strength. These movements may result in slope instability problems but can also damage the conduit if it is located over the offending layer. Slope flattening and berms are commonly used design measures to prevent such movements, and this will result in a longer conduit than otherwise would be needed. Foundations under conduits should have relatively uniform compressibility characteristics to prevent differential settlement and movement of conduit joints. The Hernandez Dam case history in appendix B illustrates problems that can occur when conduits are located partly over compressible fill and partly over nonyielding bedrock.

Special precautions should be taken for joints where the conduit connects to a structure, such as an intake structure (figure 72). This location may be in an area susceptible to differential settlement due to the differing weights of the two structures and the foundation beneath them. An engineered fill to limit settlement may be needed under the intake structure, when the structure and conduit cannot be located on bedrock or a firm foundation. If the intake structure is constructed on a pile foundation, special precautions are also required for the first few joints of the conduit because high stresses can develop as a result of bending stresses caused by differential settlement. Extending the conduit and locating the intake structure beyond the limits affected by the embankment dam can reduce these stresses.

Special precautions are also required, if the conduit operates under pressure or high velocities, has discharges that create a surging effect, or if the conduit is constructed on a compressible foundation. This may include the use of welded steel pipe to serve as a liner within the conduit. The steel pipe liner provides ductility and a watertight seal.

Figure 72.—Special design considerations are required in locations where differential settlement between two structures can occur.

Chapter 2 discussed materials used for the construction of conduits, such as reinforced cast-in-place concrete, precast concrete, plastic, and metal. The following sections provide guidance pertaining to joints used with these types of materials.

4.3.1.1 Reinforced cast-in-place concrete

Reinforced cast-in-place concrete conduits undergo physical changes in length, width, height, shape, and volume when subjected to environmental and mechanical conditions surrounding them (USACE, 1995e, p. 2-1). These changes may be the result of drying shrinkage, creep, settlement, and other effects. As these changes occur, internal stresses may form within the concrete, resulting in cracking. Most reinforced cast-in-place concrete conduits require joints to control or limit cracking and are typically placed in conduits about 12 to 16 feet apart. The location of joints can also be utilized to facilitate construction.

Four types of joints are commonly used in construction of reinforced cast-in-place concrete (Reclamation, 1987a, p. 799):

- *Contraction joints*.—Contraction joints are used in concrete to provide for the volumetric shrinkage of a monolithic unit or movement between monolithic units. These joints provide for a complete separation of the monolithic unit into smaller structural elements. No bond is expected between the concrete surfaces of the smaller structural elements. Sealing (curing) compound applied to the joint surfaces can be used to prevent bonding. Except as otherwise provided by dowels, reinforcement is never continuous across a contraction joint. A minimum of 7 days should elapse between adjacent placements at contraction joints. Waterstops should be placed across contraction joints. However, excessive movement at the joint may damage the waterstop. Therefore, contraction joints are not typically used in the construction of conduits, since they may not ensure a watertight joint. Contraction joints are often used in the construction of entrance and terminal structures.

- *Control joints*.—Control joints are typically used in reinforced cast-in-place concrete conduit to provide for control of initial shrinkage stresses and cracks of monolithic units. Control joints are constructed as described for contraction joints, except that reinforcement is always continuous across the joint. The reinforcement prevents the longitudinal forces from opening the joints. Waterstops should be placed across control joints to provide a watertight seal. The surface of concrete first placed at joints should be coated with sealing compound, so no bond develops between the ends of adjoining conduit sections. Figure 73 illustrates a typical control joint used in conduit construction. These joints are very effective in minimizing differential movement between conduit sections. A minimum of 7 days should elapse between adjacent placements at vertical control joints and 3 days between adjacent placements at horizontal joints.

- *Expansion joints*.—Expansion joints are used in concrete to prevent damage due to the compressional forces from movement caused by the expansion of abutting concrete. Expansion joints separate adjoining structural elements. Dowels and keyways can be used across these joints to resist movement. Preformed joint filler is placed in all expansion joints. The joint filler should cover the entire surface of the concrete at each joint and be laid against the concrete and held rigidly in place while concrete is placed on the other side of each joint. All joints in the filler should be tightly fitting butt joints. Expansion joints are not typically used in the construction of conduits, since they do not provide for a watertight joint. However, expansion joints are often used in the construction of entrance and terminal structures.

- *Construction joints*.—Construction joints are typically used in reinforced cast-in-place concrete conduit to facilitate construction; to reduce initial shrinkage stresses and cracks; to allow time for installation of embedded metalwork; or to allow for subsequent placing of other concrete. For conduit construction, bond is required at these joints, and reinforcement is continuous across the joint. The surface of the joint should be cleaned to expose aggregate before placement of the next concrete lift. A minimum of 7 days should elapse between adjacent placements at vertical joints and a minimum of 3 days between adjacent placements at horizontal construction joints. As much as

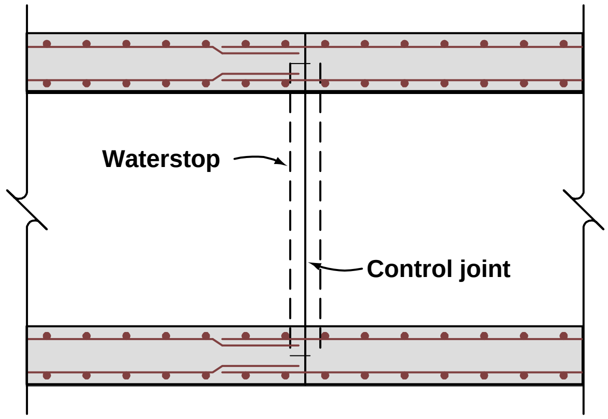


Figure 73.—Typical control joint used in reinforced cast-in-place concrete conduit construction. Longitudinal reinforcement is continuous through the joint.

possible, vertical construction joints should be avoided because of concerns with getting a good bond on a vertical joint surface.

The joints previously described are planned joints that allow for good procedures to treat the joint. Sometimes during construction, interruptions occur during the placement of concrete and require an unplanned joint. This type of joint is referred to as a "cold joint." Great care should be exercised to avoid cold joints because this type of joint introduces the possibility for higher porosity, weakened shear and tensile strength, and decreased durability. Typical cold joint treatments include high pressure washing or wet sandblasting to remove mortar coatings, or other contaminants, followed by a high volume, low pressure washing and vacuuming to remove all excess water and debris. If the surface of the cold joint is not properly cleaned, this will result in a lack of bond between the existing surface and new concrete. After cleaning, the surface should be maintained in a damp condition before placing the new concrete.

Time delays are specified between placement of adjacent sections at joints to allow the concrete to dissipate heat resulting from the peak hydration temperature. The longer the concrete is given to cool, the smaller the stresses at control and construction joints and the openings at contraction joints due to the volumetric change in the adjacent sections. The length of the time delay can be affected by the concrete mix, the thickness of the sections, the placement temperature of the concrete, and the ambient temperature surrounding the concrete as it cures.

Temperature-measuring instruments can be used as an aid in minimizing the curing time between adjacent concrete placements while also minimizing the joint separation between the same placements. These instruments can also be used in estimating the in-place strength of concrete, based on its temperature history. Without these temperature instruments, specification paragraphs must be written with conservative requirements for how long a contractor must wait before closure sections can be placed. With temperature instruments, measurements can be taken so that it can be determined when the concrete temperature has dropped sufficiently to allow the closure placement to be made. This can result in making placements several days earlier. If there are a number of placements required, then the total time savings could be a couple of weeks. For certain projects, this could be a critical matter.

The resistance thermometer, thermocouple, and thermistor are types of instruments that will measure temperature in concrete (USACE, 1987, p. 7-1). Temperature changes are the primary causes of volume change and stress. The temperature rise within the concrete causes an outward expansion during the early life of the concrete. The temperature of the internal mass is higher than that of the exposed surfaces. Thus, as the outer surface cools and tends to shrink, compressive stresses develop internally, and tensile stresses externally. In order to determine the effect of temperature on the stress and volume change, temperatures can be measured at a number of points within the structure. Thermocouples are suitable for measuring temperature under certain conditions and at several locations. However, resistance thermometers are preferred over thermocouples because they have been found to be more dependable, more precise, and less complicated in their operation. Sufficient details should be shown on the contract drawings and adequate specifications provided to obtain required installation. The instruments must be properly placed and secured during installation. Care should be exercised during concrete placement because lead wires can be easily damaged. Identification tags should be attached to the cable to accurately identify the instrument (figure 74).

Newer models of temperature-measuring instruments allow for wireless meters. A handheld computer permits the data to be compiled and analyzed to provide a real-time concrete strength value.

The advantages of using temperature instruments include:

- They allow for improved timing of construction activities, resulting in shorter durations between placements.

- Temperatures in specific critical locations within the conduit can be measured.

- More accurate representation of concrete strength is possible.

- Strength measurements can be obtained any time.

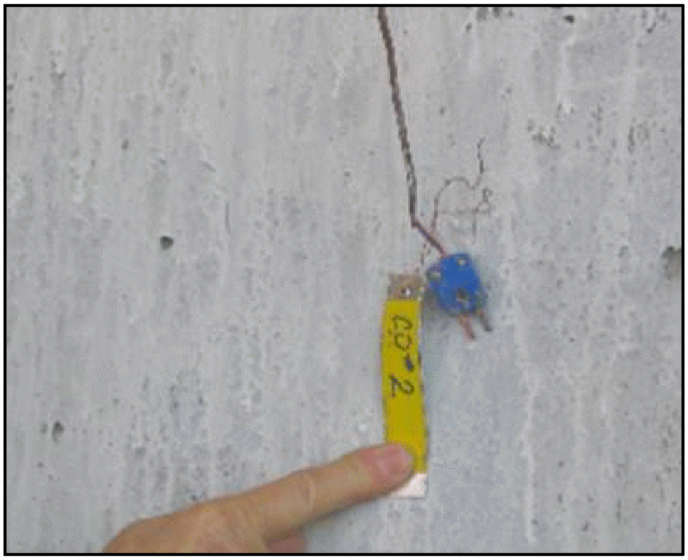


Figure 74.—An identifying tag used with a thermocoupler.

For additional guidance on temperature-measuring instruments, see USACE's *Instrumentation for Concrete Structures* (1987).

4.3.1.2 Precast concrete

All precast concrete pipes incorporate joints that enable the individual sections to be manufactured elsewhere and assembled onsite to form a continuous, watertight conduit. All precast joints are tapered bell and spigot type, which can compress a rubber O-ring gasket as adjacent pipe sections are drawn together. PCCP and RCCP are always fabricated with steel bells and spigots, which are welded directly to the steel cylinder. See figures 67, 68, and 69. RCP may be fabricated with either steel or plain concrete joint surfaces. See figure 66. Steel bell and spigot joints are always specified for larger dams where a high confidence of watertightness is required. Concrete bell and spigot joints are acceptable for only low hazard dams. There is no difference in the steel ring joints between AWWA C300, C301, and C302 pipe—RCCP, PCCP, and RCP respectively. All three types of pipe are allowed in NRCS embankment dams. Preference is based solely on economics, which consider the external and internal strength required as well as the weight of the pipe to transport and install.

The critical consideration for precast pipe joints is the degree of longitudinal rotation and longitudinal elongation that the joint can accommodate without overstressing the ends of the pipe or losing watertightness. For AWWA C302 (RCP), C300 (RCCP), and C301 (PCCP), the movement capacity of the pipe is specified by joint length and joint limiting angle. Joint length is defined as the maximum distance through which the spigot can move, relative to the bell or sleeve, from the fully engaged to the fully extended condition of the joint when the adjoining pipe sections are in parallel, concentric alignment while maintaining full confinement of the gasket. The joint limiting angle of the joint is defined as the maximum deflection angle between adjoining pipe sections that the joint will permit before the outer surface of the spigot comes into direct contact with inside of the mating bell or sleeve. Part of the elongation and rotational capacity of the joint to accommodate expected settlement and movement along the conduit is lost due to installation tolerances and designed conduit camber. Standard and deep joints are usually available from most manufacturers. Where joint capacity is inadequate, shorter lengths of pipe can be used to decrease the movement of each individual joint. Figures 66, 67, 68, and 69 illustrate the joint details of AWWA C300, C301, and C302, as RCCP, PCCP, and RCP respectively.

4.3.1.3 Plastic

Methods used to join plastic pipe are discussed in section 12.1.1.

4.3.1.4 Metal

Methods used to join metal pipe are discussed in section 12.1.2.

4.3.2 Barrier within joints

Chapter 2 discussed materials used for the construction of conduits, such as reinforced cast-in-place concrete, precast concrete, plastic, and metal. The following sections provide guidance pertaining to the barrier within joints used with these types of materials.

4.3.2.1 Reinforced cast-in-place concrete

Waterstops are used to prevent the movement of water through joints of reinforced cast-in-place concrete conduits. Figure 75 illustrates how a waterstop is typically placed across the joint of a reinforced cast-in-place conduit. Waterstops are available in a variety of materials and shapes. The most common waterstops used in conduit construction are typically made of preformed flexible materials, the basic resin of which is virgin PVC. The waterstop is fabricated, such that the cross section is dense, homogeneous, and free from porosity and other imperfections. The waterstop is specially shaped, so it will interlock with the concrete. Figure 76 shows a waterstop with ribbed sides and a centerbulb profile. This type of waterstop (also referred to as dumbbell shaped) is very versatile, and the centerbulb can accommodate lateral, transverse, and shear movements. Larger centerbulb diameters are available to accommodate larger anticipated movements.

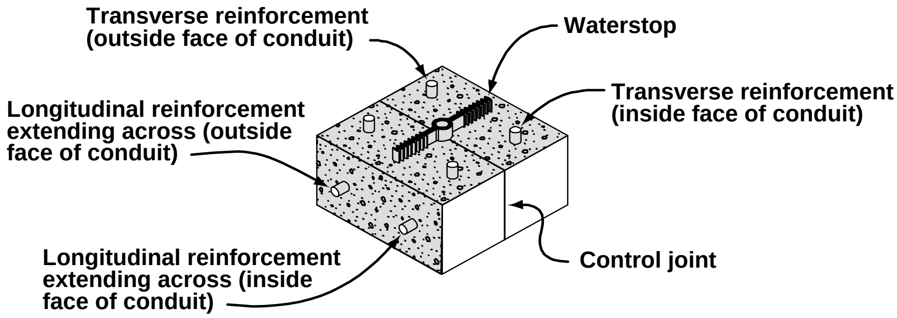


Figure 75.—Waterstop is placed across the joints of conduits to stop water from coming through the joint.

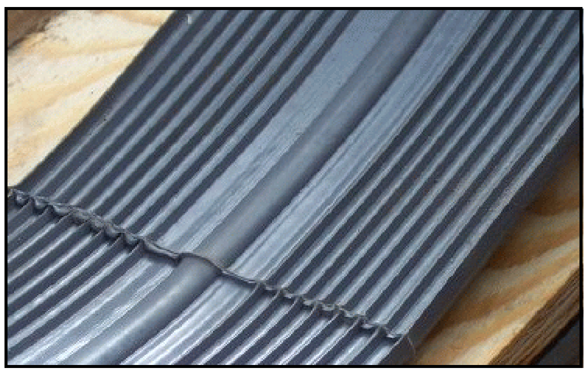


Figure 76.—Typical waterstop used in conduit construction to prevent the movement of water through joints. The ends of this waterstop have been spliced together.

The most commonly used sizes of waterstops are either 6- or 9-inch width. The anticipated hydrostatic head within the conduit should be considered in selecting the thickness and width of the waterstop to be used. Usually, assuming a waterstop width about three times the size of the maximum size aggregate in the concrete is sufficient. In some applications, the designer may want to consider the use of double waterstops.

Although waterstops are an important seepage control feature, proper installation during construction is often overlooked. Common installation errors include:

- *Poorly secured waterstop*.—Poorly secured waterstop can result in uneven embedment or an undulating alignment. A poorly secured waterstop can move during concrete placement and may become ineffective in preventing the movement of water through the conduit joint. Figure 77 shows an example of a poorly secured waterstop that experienced movement during concrete placement. The most secure way to support waterstop during concrete placement is with the use of properly built forms (commonly referred to as split forms). Figure 78 shows an example of a waterstop inserted through a specially cut slot in the form. Additional support is provided to the waterstop on the nonplacement side, to ensure no movement occurs. Proper waterstop installation requires that one-half of the waterstop be embedded on each side of the joint. Nailing, stapling, or insertion of wire through the waterstop should not be allowed, since this may affect the integrity of the material and shorten the seepage path around the waterstop. Often contractors use hog ring fasteners crimped along the edge of the waterstop and wired to reinforcing steel to secure the waterstop. If hog rings are used, extreme care should be exercised not to pierce the waterstop when crimping, since this could result in shortening of the seepage path around the waterstop. Also, reinforcing bars should not be allowed to penetrate the waterstop.

- *Poorly spliced waterstop.*—Since the PVC waterstop is thermoplastic, it can be spliced at the construction site as needed. The ends of waterstop to be spliced should be properly aligned, true, and straight. A miter-box guide and portable saw is typically used to cut waterstop. The proper splicing of waterstop requires the use of an electric thermostatically controlled waterstop splicing iron. Sometimes contractors will use a welding torch in an attempt to splice the waterstop together (Zomok, 2004, p. 3). A torch will cause PVC to burn, resulting in a poorly spliced waterstop joint. The joints should not be lapped. Approved manufacturer recommendations should be followed for splicing.

- *Poorly consolidated concrete.*—During concrete placement, the concrete surrounding the waterstop should be adequately vibrated, such that the waterstop is completely embedded in concrete. Inspectors should pay close attention during construction for proper waterstop installation. Any improperly installed or spliced waterstop should be removed and replaced. The contractor's proposed method of waterstop installation should be carefully reviewed prior to beginning any work. Approved manufacturers' installation recommendations should be carefully followed.

The use of waterstops across control joints is advised even for steel lined conduits. Guidance on the design of waterstops is available in the USACE's *Waterstops and Other Preformed Joint Materials for Civil Works Structures* (1995e). The Arkabutla case history in appendix B illustrates the importance of using waterstops in the construction of conduits.

4.3.2.2 Precast concrete

For all precast concrete pipe, a rubber gasket (figures 66-69) provides the primary barrier against movement of water or soil into or out of the conduit. The spigot end

Figure 77.—A poorly secured waterstop moved during concrete placement.

Figure 78.—Waterstop held firmly in place by use of a specially cut slot in the form.

of the joint is fabricated to contain a rectangular recess that holds a continuous solid rubber ring of circular cross section. The rubber gasket is compressed when the spigot is pushed into the bell end of the joint. Figure 79 shows an example of a rubber gasket being installed on a precast pipe. The gasket and steel rings, if used, are manufactured to high tolerances to ensure a reliable high pressure seal. Lubricating the gasket and inside face of the bell with vegetable soap can ease assembly. In embankment dams, a mastic sealing compound (figures 66-69) and metal or geotextile bands are typically placed around the outside length of the joint to prevent any movement of soil backfill into the joint space that might interfere with future joint movement.

Figure 79.—A rubber gasket is installed at the spigot end of the precast concrete pipe.

Quality installation is critical to ensure precast pipe joints are assembled watertight and will remain watertight after the pipe settles and moves. Several careful steps are required (NRCS, 2001b):

- Pipe section shall be set to specified line and grade and temporarily supported on precast blocks or wedges until the sections are joined and the cradle is cast. For guidance on use of cradles, see section 4.1.6.

- The connecting surface of the bell and spigot shall be thoroughly cleaned and dried.

- The gasket and the bell surface shall be coated with a light coat of soft vegetable soap compound.

- The spigot shall be seated to within 0.5 inches of its final position and the position of the gasket checked with a feeler gauge around the entire circumference of the pipe. Detection of any improperly seated gasket will require disassembly.

- A sealing compound (mastic) shall be applied to completely fill the exterior annular space between the placed pipe sections. Figure 80 shows mastic being applied to the annular space of a precast pipe.

- The sealing compound shall be covered with a metal or geotextile band where stones larger than ¼ inch may occur in the backfill material.

Figure 80.—Mastic being applied to exterior annular joint space of precast concrete pipe.

- Water or air pressure testing the completed conduit to approximately 10 feet of hydrostatic head is highly recommended before casting the cradle.

An alternative to gasketed joints in PCCP (lined-cylinder, AWWA C301) is welded joints (also known as internal welded tied joints). Welded joints have been used in water pipelines to form a watertight barrier for about 30 years. A continuous interior side fillet weld is required to provide the watertight seal. Typically, the rubber gasket is not installed in the spigot groove, since the gasket would burn during the welding operations. Welded joints should only be used for conduit diameters of 36 inches or larger to accommodate access by man entry. The designer will need to carefully evaluate if this alternative provides adequate watertightness for the given conduit application.

Chapter 5

Foundation and Embankment Dam

Previous chapters have discussed the importance of placing the conduit in the most favorable location within the embankment dam to reduce problems with foundation and embankment settlement. In this chapter, the reactions of soil and rock foundation horizons and how they can affect the design of the conduit are discussed. If conduits are located on foundations that are not uniform, differential settlement can lead to cracking and joint problems in the conduit. If foundations consist of low strength or highly compressible materials, unacceptable deformations and lateral movements can damage the conduit.

Other discussions in this chapter address how the settlement of embankments near conduits can create hydraulic fracture mechanisms. Design approaches effective in preventing this problem are included. The importance of careful design of excavations made to install conduits is extensively discussed. Special attention is recommended for any excavations made transverse to the centerline of the embankment where the excavation backfill may be different in compressibility than the adjacent foundation materials. Recommendations for backfilling soils near conduits are provided. Problematic soils, such as broadly graded soils and dispersive clays are defined, and potential problems associated with them are also discussed.

5.1 Excavation and foundation preparation

Ideally, sound rock provides the best foundation conditions for conduits. However, ideal conditions are rare, and many embankment dam sites have marginal foundation conditions.

If the underlying foundation is highly compressible, subject to collapse upon saturation, or has other objectionable properties, the conduit could be damaged from excessive settlement. In some cases, unsuitable foundation soils must be removed and replaced to prevent damage to the conduit. Backfill used in excavations for conduits must be compacted uniformly under the conduit. Excavations should be wide enough to accommodate motorized compaction equipment. The side slopes of the excavation must be flat enough to avoid differential settlement of the embankment dam near the conduit.

5.1.1 Rock foundation

The foundation line, grade, and density should be uniform. Controlled blasting or other excavation procedures should be followed to avoid damaging the foundation. Smooth blasting techniques, such as line drilling are typically considered. Rocks and/or irregularities at the foundation contact that might create a stress concentration should be removed. Cleaning and backfilling should treat existing defects, such as soft or pervious soil fillings in the rock, fault gouge, fractures, erosion channels, or solution cavities that cannot be removed. These defects require removal to an adequate depth (usually three times the width) and replacement with lean concrete slush grout, dental concrete, or specially compacted earthfill. Slush grout should only be used to fill narrow cracks in the foundation and not large areas. Slush grout typically consists of cement and water or, in some cases, cement, sand, and water. Slush grout can harden and crack under load and for this reason, is used only in small areas. Dental or shaping concrete should be used to fill larger irregularities or discontinuities in the foundation.

If the excavated foundation surface is subject to slaking when exposed to the atmosphere, the foundation surface should be protected with suitable earthfill, a concrete pad (mud slab), or an acceptable sealing compound until conduit construction commences. Protecting the foundation can reduce the potential for differential settlement. Shale, chalk, mudstone, and siltstone formations are most prone to slaking problems. If concrete pads are used to protect a foundation, they are usually placed within 24 hours of exposing the foundation to provide protection from weather and construction activities. If the entire foundation cannot be exposed, the concrete pad may have to be placed incrementally. The surface of the concrete pad should be treated as a construction joint, and proper attention given to cleanup to ensure good bond to the conduit.

5.1.2 Soil foundation

Conduits located on soil foundations require analysis to predict the amount of foundation settlement and spreading that may affect the conduit. The conduit may need to be constructed with a camber to compensate for the predicted settlement, and any joints in the conduit must be designed to accommodate the predicted spreading. The designer should be aware that foundation compressibility under embankment dams often is not uniform, and abrupt displacements can occur. Abrupt vertical and horizontal movements can result in overstressing and cracking of conduits and opening of joints.

In soil foundations, excavation may be required to provide a good interface between the conduit and foundation and to remove objectionable materials. Foundation materials that have poor strength and permeability properties will also require removal. These materials may include organic material, such as roots and stumps, sod, topsoil, wood trash, or other foreign material. Other objectionable materials that may require removal include very low shear strength, highly compressible and collapsible soils.

If excavation for the conduit is required in earth materials, the trench should be wide enough to allow equipment to perform backfill compaction parallel to the conduit. The side slopes of any excavation may need to be flattened to avoid differential settlement. Any excavation for a conduit must consider the potential differential settlement that could occur, caused by different properties of the compacted backfill in the excavation and the foundation soils. This problem is most important where foundation soils are soft and compressible or collapsible. Flattening the side slopes of excavations may be required to prevent hydraulic fracture of the overlying embankment. Section 5.2 discusses hydraulic fracture of embankments in more detail.

Conduits may be required to be located on a compacted soil base to provide a uniform foundation. Rather than attempting to compact the soil foundation to exactly the required grade, consideration should be given to overbuilding the embankment in the area of the conduit by 1 to 2 feet and excavating down to the structural grade of the conduit. Depending on the nature of the embankment, it may be desirable to construct a concrete pad directly over the prepared foundation to protect the foundation integrity and minimize degradation when exposed to air, moisture, or construction activity. The concrete pad should be placed within the lateral limits of the conduit. If a wider concrete pad is required on both sides to facilitate construction, it should be constructed with a vertical joint with a bond breaker. The bond breaker will allow for easy removal of the concrete pad extending beyond the conduit edges.

When soft foundation soils are encountered, some designers may propose use of piles to support a conduit. Use of piles is not recommended, because the conduit may become undermined, allowing uncontrolled seepage to occur under it. This has occurred in at least two pile-supported spillway conduits in Maryland, where voids up to 5 feet deep were found beneath one structure. In the other structure, complete failure of the spillway conduit occurred less than 2 years after construction was complete. For details concerning the latter spillway conduit, see Bohemia Mill Dam case history in appendix B.

5.2 Cracking and hydraulic fracture of embankment dams

Most embankment dams crack, but only a few develop problems from cracking. Transverse cracks that develop in an upstream and downstream direction are of the most concern. Once a crack forms and water enters the crack, three possibilities can result (ASDSO, 2003):

1. Water penetrates soil adjacent to the crack and the soil begins swelling. If the crack is small and not continuous through the embankment dam, it can swell shut and not develop into a problem.

2. Water runs through the crack, but the crack is small, so the velocities are low, and the soil is resistant to erosion. A wet spot may appear, but no internal erosion takes place.

3. The soil in the embankment dam, such as dispersive clay, is erosive. Internal erosion begins, and a concentrated leak develops. More and more internal erosion occurs, and the embankment dam fails from the breach that is formed.

Cracks in embankment dams caused by hydraulic fracture may provide a pathway for internal erosion. If soils in the pathway of the crack are highly erodible, the crack will enlarge quickly, leading to a breaching type of failure. Hydraulic fracture is common near conduits, because the conduits create differential strains in surrounding embankment soils.

Hydraulic fracture of embankment dams can occur when the piezometric head of water within the dam is greater than the lateral effective stress on the earthfill. Sherard (1986, pp. 905-927) discusses hydraulic fracturing in detail. Figure 81 illustrates how on first filling, a wetting front moves through the embankment dam. Figure 82 shows an example of hydraulic fracture in an embankment dam. For an example of a project that experienced hydraulic fracture near a conduit, see the case history for Piketburg Dam in appendix B.

Problems often occur on first filling of the reservoir. About 42 percent of all embankment dam failures due to internal erosion or backward erosion piping occur on first filling (Foster, Fell, and Spannagle, 2000, p. 1025). As discussed in section 9.1, filling the embankment dam's reservoir for the first time requires caution. Slow filling of the reservoir is important to allow the wetting front to slowly penetrate into the embankment dam. This will allow the soils to swell and deform, which helps prevent hydraulic fracture. Typical filling rates are in the range of 0.5 to 2 feet per day. The designer should consider the rate of reservoir rise when determining the hydraulic capacity of the conduit. For guidance on the hydraulic design of conduits, see chapter 3.

Excavations for conduits increase the potential for differential settlement, and special care is recommended for any excavations used near and under conduits. The excavation should be wide enough to accommodate motorized compaction

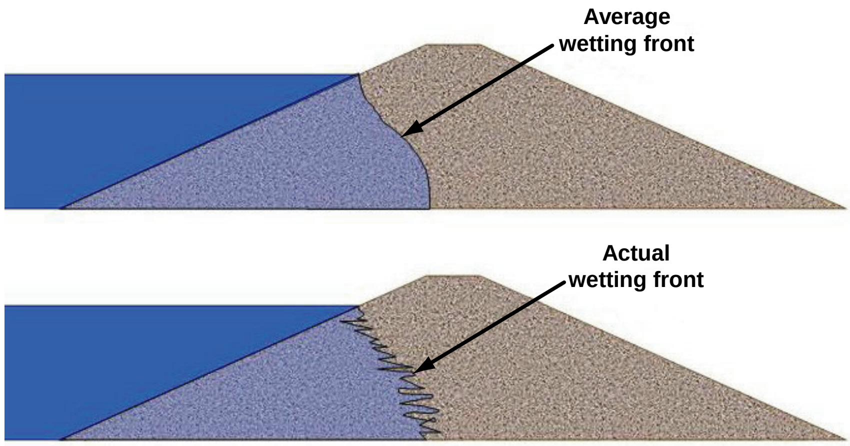


Figure 81.—On first filling, a high hydraulic gradient exists in the embankment dam as a wetting front moves through the dam. The wetting front will not be smooth. Projections will exist due to the different permeability of the embankment dam. The water pressure pushing against the soil can easily be greater than the lateral stress, and hydraulic fracture can result. Figure courtesy of ASDSO.

Figure 82.—The failure of this embankment dam located in South Carolina was attributed to hydraulic fracture. The eroded seam located to the right of the conduit may have been the hydraulic fracture that formed and allowed internal erosion and failure of the embankment dam. The embankment was composed of dispersive clays.

equipment, and the side slopes should be flat to reduce differential strain. The USACE (2004a, p. 8-2) recommends:

> Special attention must be given to the junction of embankments with concrete structures, such as outlet works . . . to avoid piping along the zone . . . . A 10 vertical on 1 horizontal batter on the concrete contact surfaces will ensure that the fill will be compressed against the wall as consolidation takes place. . . . It may be desirable to place material at higher water contents to ensure a more plastic material which can adjust without cracking, but then the effects of increased porewater pressures must be considered.

Factors that can contribute to hydraulic fracture in embankment dams include:

- Differential settlement that exceeds about 1 foot per 100 feet (measured longitudinally along the embankment dam). Settlement that exceeds this limit of acceptable strain can lead to concern for hydraulic fracture.

- Trenches that are transverse to the embankment dam create differential settlement, especially if the trench backfill has different compressibility than foundation horizons. Conduits often require excavations to provide a uniform foundation for the structure. Shaping the side slopes of an excavation that is transverse to the embankment centerline is essential. USACE (2004a, p. 6-6) recommends:

  > Excavations for outlet conduits in soil foundations should be wide enough to allow for backfill compaction parallel to the conduit using heavy rolling compaction equipment. Equipment used to compact along the conduit should be free of framing that prevents its load transferring wheels or drum from working against the structure. Excavated slopes in soil for conduits should be no steeper than 1 vertical to 2 horizontal to facilitate adequate compaction and bonding of backfill with the sides of the excavation.

  The above recommendation suggests that side slopes of excavations should be 2H:1V. This recommendation is appropriate for favorable soil properties. Flatter side slopes should be used for less favorable conditions. Excavation slopes of 3H:1V to 4H:1V are commonly recommended for unfavorable situations. Flatter than normal side slopes for excavations are advisable when the following situations exist:

  1. When an excavation that is transverse to the centerline of an embankment dam is backfilled, the compacted soils in the excavation may have considerably different stress/strain properties than the foundation horizons that have been excavated. These differences can result in

conditions favorable to hydraulic fracture in the overlying embankment. Two examples are:

a. *Soft foundation alluvial horizons.*—Soils compacted into the excavation will be significantly less compressible than the soft soils in the foundation. This will result in a sharp difference in settlement in the excavation backfill than in the soft foundation soils adjacent to it. A variation of this situation is one where the excavation is made in low density, collapsible foundation soils that are sometimes encountered in western United States. These soil types would probably have much higher strain potential than the excavation backfill, creating conditions favorable to hydraulic fracture in the overlying embankment.

b. *Extremely dense foundation materials.*—If an excavation is made in weathered shale, glacial till, overconsolidated clays, or other materials with very low compressibility, the soil used to fill the excavation may be significantly more compressible than the adjacent foundation materials. The result can be differential settlement that can create conditions favorable to hydraulic fracture in the overlying embankment.

2. If soils used to construct the embankment are extremely susceptible to internal erosion, excavations transverse to the embankment dam that create conditions favorable to hydraulic fracture should have special attention. Flattening the side slopes of excavations is strongly recommended. Commonly, for problematic conditions, slopes transverse to the centerline of the embankment dam are made to be 4H:1V or flatter. Examples of soils that are highly susceptible to internal erosion are low plasticity, clayey silts, and dispersive clays.

- Closure sections in embankment dams may also contribute to differential settlement. Closure sections should be avoided, if possible.

The Wister Dam case history (see appendix B) illustrates the dangers inherent with closure sections in embankment dams. Many conduit rehabilitation projects involve making a transverse excavation to the embankment dam and removing the conduit to replace it. The arching effect of soils in the closure section can result in hydraulic fracturing. Conduits should not be installed in closure sections unless no other alternatives are available. The USACE (2004a, p. 9-3) discusses factors related to closure sections as follows:

> Because closure sections of earth dams are usually short in length and are rapidly brought to grade, two problems are inherent in their construction. First, the

> development of high excess porewater pressures in the foundation and/or embankment is accentuated, and second, transverse cracks may develop at the juncture of the closure section with the adjacent already constructed embankment as a result of differential settlement. . . . Cracking because of differential settlement may be minimized by making the end slopes of previously completed embankment sections no steeper than 1 vertical on 4 horizontal. The soil on the end slopes of previously completed embankment sections should be cut back to well-compacted material that has not been affected by wetting, drying, or frost action. It may be desirable to place core material at higher water contents than elsewhere to ensure a more plastic material which can adjust without cracking, but the closure section design must then consider the effects of increased porewater pressures within the fill.

5.3 Selection and compaction of backfill

Proper selection and compaction of backfill material against the conduit will minimize the potential for differential settlement.

5.3.1 Selection of backfill material to be placed against conduit

If the conduit is being placed in a zoned earthfill embankment dam, backfill for the conduit should usually have properties that are compatible with the adjacent embankment zones. Core zone backfill should only be used around the conduit through the core section, with shell backfill soils used outside the core. Where the conduit passes through the core of an embankment dam often material with higher plasticity is used near conduits. Plastic materials can be placed at a water content wet of optimum (between 1 percent and 3 percent wet of optimum) to increase plastic behavior. An exception is where rock shell zones include large angular rocks that could impose point loads on the conduit. For that condition, encircling the conduit with a cushioning soil zone of smaller sand and gravel should prevent this problem.

Ideally, the earth material adjacent to conduits in the im pervious zone of fill should be reasonably well graded, have a maximum particle size no greater than 1½ inches, including earth clods, a minimum of 50 percent by weight passing a No. 200 sieve, and a plasticity index between 10 and 30 percent. The water content of the material as previously discussed should be between 1 percent and 3 percent wet of optimum. Dispersive clay and treatments are discussed in section 5.3.3.

Flowable fill (also known as controlled low strength materials) is not recommended for backfilling around conduits in significant and high hazard embankment dams, due to the following reasons:

- Flowable fill does not bond to either the conduit material or the adjacent foundation in which it is in contact. Measures are required to intercept flow along the interface between the flowabable fill and foundation or conduit.

- The flowable fill will develop interior cracks (shrinkage) that should be intercepted with filter diaphragms to ensure no movement of soil particles.

Flowable fill may be applicable for low hazard embankment dam applications, if used in conjunction with a filter diaphragm or collar. Use of a filter diaphragm or collar is a valuable defensive design measure, even for low hazard classification sites with favorable conditions. The use of a lean concrete in lieu of flowable fill may allow for elimination of the filter diaphragm or collar, but eliminating this valuable feature should be carefully considered and justified based on extremely favorable soil conditions, good conduit construction materials and methods, reliable construction practices, and favorable foundation conditions. Conditions where flowable fill may be applicable for low hazard embankment dams include:

- Backfilling trenches dug in relatively nonyielding materials, such as bedrock or glacial till in which a conduit is installed. Flowable fill provides a uniform material surrounding the conduit, which has strain properties similar to those of the adjacent foundation. This would allow somewhat steeper side slopes for the excavation.

- Design of the flowable fill provides similar deformation characteristics in the cured fill as in the adjacent foundation materials.

5.3.2 Compaction of backfill material against conduit

Recommendations for compaction of soils and rock zones against the conduit are as follows:

- *Minimum strength.*—Prior to placing embankment adjacent to the conduit, the concrete must have attained minimum strength. Minimum strength should be based on anticipated/estimated loading conditions (i.e., construction surcharge, embankment load, etc.). As a rule of thumb, placing embankment should not begin until curing of the concrete is completed (typically 7 to 14 days after concrete placement) and the concrete has achieved its design strength.

- *Average moisture content.*—The average moisture content during compaction should be in the range of 1 percent dry to 3 percent wet of optimum content, where optimum water content is defined by a Standard Proctor energy (ASTM D 698) compaction test. The compacted unit weight of the backfill around the conduit should be equivalent to that required for the surrounding soil.

- *Angular particles.*—Earthfill placed within 2 feet of the conduit should not contain large angular particles that could damage the conduit from compactive effort used in compacting soils near the conduit. In rockfill zones, a cushion zone of smaller granular particles should be used to prevent damage to the conduit from the point loading of rocks in the earthfill.

- *Permeabilities.*—Earthfill immediately adjacent to conduits should be compacted, so that no layers of material with permeabilities higher than in the adjacent earthfill extend in an upstream and downstream direction along the conduit.

- *Ramping of earth material.*—The earthfill should be ramped against the conduit on a slope of 6H:1V (figure 83) to help force the earthfill against the conduit and to avoid contacting the conduit with the frame of the pneumatic roller used for compaction. Pneumatic rollers should be operated in a direction parallel to the conduit. The pneumatic roller may form rutting and a smooth surface on the earthfill layer that will need scarification before new layers are placed.

- *Lateral movement.*—Earthfill should be maintained at approximately the same elevation on both sides of the conduit during backfilling. This will help to prevent lateral movement of the conduit caused by unequal compaction energy applied to the sides of the conduit.

- *Disking.*—The area adjacent to a conduit is normally a highly trafficked area, due to activities involved with the installation of the conduit. Disking, as well as being sure to eliminate drying cracks and moistening of surfaces before adding subsequent lifts are required to prevent smooth surfaces between lifts.

- *Compaction.*—Compacting soil next to large conduits requires different approaches than for compacting soil next to smaller conduits. A single recommended approach is not possible for a wide range of conditions. Generally, compacting soil surrounding larger conduits may employ pneumatic-tired rollers or similar equipment, which is operated parallel to the conduit. On smaller conduits, operating large equipment near the conduit can damage the conduit, and hand compaction may be required. Compacting soil within 2 feet of a conduit with heavy equipment, such as tamping rollers or vibratory steel-wheeled rollers is usually inadvisable.

  Hand controlled mechanical compactors (also known as tampers or wackers) have been used frequently in the past. Handheld compactors should not weigh less than 100 pounds. A much thinner lift thickness than the rest of the embankment dam is required when using handheld compactors. Hand compaction is often slow, labor intensive, and tends to lag surrounding embankment dam placement. Hand compaction requires more effort to obtain proper moisture and density, may require special gradation of soil particles, and requires intense inspection and at times is a source of irritation to both contractor and owner. This results in a tendency to concentrate more on progress than good construction techniques. Operators of handheld

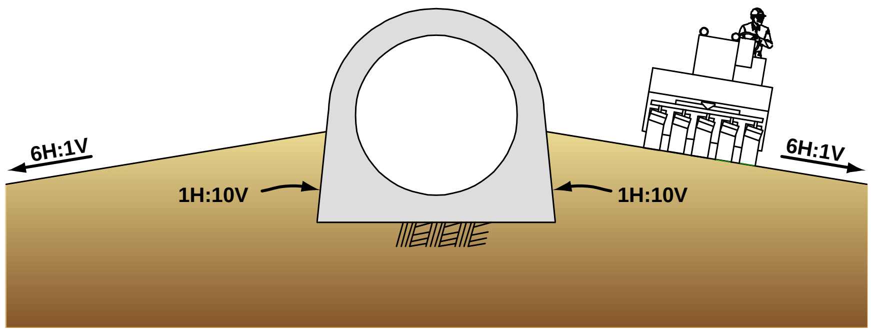


Figure 83.—Recommended earthfill ramp and conduit side slopes.

compactors often try to shorten compaction time by utilizing thicker lifts and not as much compaction. For these reasons, handheld compactors should be avoided, if possible.

Close inspection is needed to ensure proper results, and extensive testing is required to ensure a quality earthfill. Even with the best of efforts, substandard earthfill material next to the conduit may occur, which increases the probability of poorly bonded lift surfaces, low density zones and poor bond between the earthfill and structure and result in the development of seepage paths (Reclamation, 1987c, pp. A-1 and A-2). In some special cases, certain areas of large conduits may not be accessible to heavy compaction equipment and the use of handheld types of compactors may be unavoidable. Handheld compactors may be required where space available for large compaction equipment is limited. Designers should consider this potential and avoid the need for hand compaction to the extent possible.

Construction specifications should also ensure that hand-compacted earthfill is not compacted at too dry a water content. A wetter earthfill material will be more deformable and will result in better densities and bond. The earthfill next to the conduit should be compacted at water content between 1 percent and 3 percent wet of Standard Proctor optimum.

- *Control testing*.—The frequency of control testing should be as often as necessary to ensure that the earthfill adjacent to the conduit is properly compacted. At least one dry unit weight and moisture content control test should be taken during each shift. Use of a penetrometer to locate low dry unit weight zones, as a supplement to regular control testing, can greatly increase the effectiveness of the inspector.

- *Dry unit weight control*.—Dry unit weight control of the earthfill within 1 or 2 inches of the conduit surface is difficult using conventional test procedures. The inspector should make use of a penetrometer, a knife blade, or whatever device is necessary to make sure that the earthfill is compacted tightly against the structure and no voids are present. The soil's moisture content should be checked, so that it matches the originally intended water content.

- *Closure section*.—Conduits are often located in closure sections within embankment dams. Section 5.2 discusses several important design considerations for closure sections. Another important factor for the closure section is the potential for exposed surfaces to become desiccated before the closure section is filled. Before adding compacted soil to an area that has been exposed, the soil should be carefully inspected for evidence of desiccation cracking. Soil with desiccation cracks must be removed, moistened, and recompacted before allowing subsequent earthfill operations to resume. Poorly bonded lifts will result from placing compacted soil on a surface that has been allowed to dry. Hydraulic fracture can create a pathway for internal erosion in this zone of the earthfill. Often a few feet in depth of the existing embankment surface is moistened and reworked.

5.3.3 Dispersive clay backfill

Embankment dams usually contain a zone of lower permeability soil to reduce the seepage through the embankment. In small embankment dams, the entire dam may be constructed of the same soil, termed a homogeneous construction. If this zone of low permeability, clayey soil develops cracks, particularly transverse cracks, from hydraulic fracture, desiccation, or other causes, the integrity of the embankment dam may be compromised. Water flowing through a crack in any soil will erode and enlarge the crack, unless the crack is able to swell closed before erosion occurs. If the crack continues to erode, this can lead to a breaching of the embankment dam. Figures 84 and 85 show failures known to be associated with highly dispersive clay embankments. Figure 84 shows a small embankment dam that failed when water flowed along a transverse crack in the dam. The transverse crack was caused by hydraulic fracture of the earthfill associated with differential settlement near the conduit. Failures of embankment dams constructed of dispersive clays without appropriate defensive design measures have been common (figure 85).

These lower permeability zones are intended primarily to reduce seepage in embankment dams. They may be successful in this regard, but if they develop cracks, they can still perform unsatisfactorily. The erosion resistance of these zones depends on several factors, including the gradation, degree of compaction and compacted water content, plasticity and electrochemical composition. The most erosion-resistant zones are high in plasticity with an electrochemical composition that results in strong interparticle attraction, compacted to a high percent saturation to reduce their permeability. The least erosion-resistant soils are termed "dispersive clays."

Figure 84.—An embankment dam in Mississippi constructed of dispersive clay soil, failed on first filling with low reservoir level. Failure was likely caused by cracks from differential settlement, hydraulic fracture, or poor compaction about the outlet works conduit. This embankment dam had no filter.

Figure 85.—This embankment dam constructed with dispersive clays failed on first filling of the dam.

Soils with low plasticity indices are also more likely to experience rapid internal erosion than soils with higher plasticity, as water flows through the soils or along an interface between the soil and an object, such as a conduit or bedrock in contact with the soil. The failure of Teton Dam in 1976 in Idaho illustrates the speed with which failure can occur in internal erosion involving low plasticity embankment soils.

Some embankment dams contain zones of broadly graded soil with sufficient fines content to be considered low in permeability, but the soils are found to be subject to suffosion from internal instability. The low resistance to internal erosion of broadly graded soils is well documented in references by Garner and Sobkowicz (2002), Sherard (1979), and LaFleur, Mlynarek, and Rollin (ASCE, 1989). The internal instability of these soils results from the ability of finer soil particles to be mobilized between larger particles in these broadly graded soils. This soil movement, termed suffosion, results in sinkholes, if erosion progresses long enough. Designing filters using current criteria has been shown to be effective in blocking internal erosion in these soil types.

Dispersive clays differ from "normal" clays because of their electrochemical properties. The paper by Sherard, Decker, and Ryker (1972b, p. 589) has good discussions on the reasons dispersive clays' chemistry influences their behavior so strongly. From that paper,

> The main property of the clay governing the susceptibility to dispersion piping is the quantity of dissolved sodium cations in the pore water relative to the quantities of other main basic cations (calcium and magnesium). The sodium acts to increase the thickness of the diffused double water layer surrounding individual clay particles and hence to decrease the attractive force between the particles, making it easier for individual particles to be detached from the mass.

Flow through cracks in dispersive clays can quickly erode the cracks and lead to rapid enlargement of the cracks. Failures caused by internal erosion in dispersive clay dams are common. Several case histories presented in appendix B discuss failures associated with internal erosion in dispersive clay soils. The intent of chimney filters and filter diaphragms around conduits is to intercept flow occurring in cracks in the embankment dam, to seal the flow of water by developing a filter cake at the face of the filter. See section 6.4 for additional discussion concerning filters.

Identifying dispersive clays requires special tests that are not always routinely performed. The tests most commonly used for identifying dispersive clays are the crumb test (ASTM D 6572), the double hydrometer test (ASTM D 4221), and the pinhole test (ASTM D 4647).

The pinhole test is a direct measure of the erodibility of a soil. A 1-mm hole is formed in a specimen of soil, and water is forced through the hole at increasing heads. A dispersive clay soil will erode rapidly under a low 2-inch head within 10 minutes of flow initiation. A non dispersive clay usually undergoes little erosion, even under sustained flow through the hole under a head of up to 40 inches. The test is performed using standard procedures established in ASTM D 4647.

Two tests have been developed to study the erosion characteristics of soil in addition to the pinhole test, which is used exclusively to evaluate dispersive clays: the slot erosion test and the hole erosion test (Wan and Fell, 2004). Australian researchers at the University of New South Wales developed these tests, which are not widely used in the United States. The erosion characteristics are described in these supplemental tests by the erosion rate index, which indicates the rate of erosion due to fluid traction, and the critical shear stress, which represents the minimum shear stress when erosion starts.

Results of the two laboratory erosion tests are strongly correlated. Values of the erosion rate index span from 0 to 6, indicating that two soils can differ in their rates of erosion by up to 6 times. Coarse grained soils, in general, are less erosion resistant than fine grained soils. The erosion rate indices of coarse grained soils show good correlation with the fines and clay contents, and the degree of saturation of the soils. The erosion rate indices of fine grained soils show moderately good correlation with the degree of saturation. The absence of soils with the clay minerals called smectites and vermiculites, and apparently the presence of cementing materials, such as iron oxides, improve the erosion resistance of a fine grained soil.

The hole erosion test is proposed as a simple index test for quantifying the rate of internal erosion in a soil, and for finding the approximate critical shear stress corresponding to initiation of internal erosion. Knowledge of these erosion characteristics of the core soil of an embankment dam aids assessment of the likelihood of dam failure due to internal erosion.

See the NRCS's *Dispersive Clays* (1991) for more detailed discussions of methods for identifying dispersive clays. Another reference (Sherard, Dunnigan, and Decker, 1976, pp. 287-301) also discusses identification of dispersive clays.

A filter around conduits is important where dispersive clays are used for embankment dam construction. The dimensions of a filter should be increased because dispersive clays are so dangerous to the integrity of an earthfill. An embankment chimney drain/filter that extends completely across an earthfill, from one abutment to the other, extending upwards to the normal pool height or higher, is often used. See chapter 6 for design and construction of filters.

Treating dispersive clays with chemical amendments may be effective in reducing their erodibility, but this alternative may be costly, except with specific embankment zones. Treating dispersive clays used as backfill in cutoff trenches and on the outer slopes of embankments are examples. Several chemicals have been used to modify dispersive clays. The most common additive is hydrated lime. The NRCS reference on dispersive clays (NRCS, 1991) discusses chemical amendment in more detail. However, the designer is cautioned that treatment with lime will increase the material's brittleness.

Chimney filters that coincide with a filter diaphragm or collar around conduits are the preferred method for preventing failures associated with dispersive clays. Embankment dams constructed of dispersive clays should use a substantial filter diaphragm around the conduit, if a chimney filter is not included in the design of the dam.

5.4 Frost susceptibility and ice lenses

Some soils, especially silts, are frost susceptible. When conduits are in contact with frost-susceptible soils, large ice lenses may form in the soil if the conduit is exposed to freezing temperatures. When these ice lenses melt, voids are left in the earthfill that are subject to internal erosion, if they are connected to the reservoir and continuous in a direction transverse to the embankment dam. This is a suspected failure mode in the Anita Dam and Loveton Farms Dam failures (see appendix B) and Kelso Dam in southern Ontario, Canada (Milligan, 2003, pp. 786-787). Guidance concerning frost susceptibility and ice lenses includes:

- *Process of ice lens formation.*—When wet soil freezes, most of the water in the soil pores becomes ice, and it expands about 10 percent (Sowers, 1979). However, in frost-susceptible soils, it has been found that not all of the water freezes. Capillary soil water may remain in liquid form at temperatures of 28 °F, and some liquid water may even still exist at temperatures as low as -4 °F (Penner, 1962, p. 1). When the water in the larger pores freezes, the moisture content in the soil is reduced, and unfrozen capillary water in the surrounding soil tends to migrate toward the frozen zone because of surface tension. The frozen water becomes an ice lens, which will draw water from the surrounding soil as long as unfrozen water is available and the temperature at the lens remains cold. The result is that the ice lens grows larger, and the total expansion of the soil is much larger than that which would occur with the expansion of the original amount of water present in the soil.

  The growth of these ice lenses creates very large pressures in the soil, creating frost heave within the embankment dam. These forces can damage rigid conduits and can cause excessive deformation of flexible conduits resulting in pathways for internal erosion. When the ice lens defrosts, it can create a void or low density zone that can initiate the internal erosion process.

- *Frost-susceptible soils.*—Silts and silty soils are the most frost susceptible, because the voids can be of capillary size, and the permeability of the soil is sufficient for migration of pore water. Fine grained clays are not conducive to formation of large ice lenses, because they are too impermeable to allow substantial migration of the soil water. Sands and gravels with less than 3 percent fines content (material finer than 0.075 mm, No. 200 sieve) are not generally frost susceptible (Reclamation, 1998a, p. 54).

  Thousands of embankment dams constructed of frost-susceptible materials have experienced no apparent problems. For both case histories in appendix B where ice lenses are suspected as contributing factors in dam failure, the conduits were relatively large metal pipes. Such large pipes may allow sufficient cold air flow through them to cause the adjacent embankment soils to freeze, while the heat in flowing water in smaller conduits may minimize the problem. Also, the metal pipes are relatively thin, so cold temperatures are transmitted to the surrounding soil relatively quickly. Cold temperatures would take longer to affect the soils surrounding conduits with thicker concrete walls.

- *Design considerations.*—Backfilling adjacent to conduits with clayey materials would minimize the potential for formation of ice lenses in the embankment dam. Also, construction of a properly designed filter diaphragm or collar near the downstream end of the conduit would control seepage along the outside of the conduit and minimize the potential for failure by internal erosion.

Chapter 6

Filter Zones

Zones of designed filter material have become the accepted method of preventing failures caused by uncontrolled flow of water through the embankment materials and foundation soils surrounding a conduit through an embankment dam. This chapter discusses the theory behind the concept for using filter zones to prevent erosion of earthen embankments near conduits caused by the uncontrolled flow of water through soils surrounding conduits that penetrate the embankment.

The type and configuration of the filter zone depend on site conditions and soils used in the embankment dam. Three basic designations for filter zones associated with conduits are discussed: filter diaphragms, filter collars, and chimney filters. Examples of typical designs used by the major design agencies are included.

6.1 Theory of filter seal development

The concept behind the function of a filter in sealing a concentrated flow was developed largely from laboratory experiments under the guidance of Sherard as reported in several references (Sherard, Dunnigan, and Talbot, 1984). Figure 86 (top) is reproduced from this reference, and it illustrates filter cake development in the laboratory experiments. Figure 86 (bottom) shows the action of the filter in sealing a concentrated leak in an embankment dam.

6.2 Federal agency policy on filters for conduits

The following policy concerning filter zones has been summarized from three of the major federal agencies that have been traditionally involved with embankment dam construction:

- *Bureau of Reclamation*.—From Reclamation's, *Embankment Dams—Embankment Design* (1992, p. 21):

> Structures through embankments should be avoided unless economics or site geology dictates their use. If they are used, the primary means of controlling

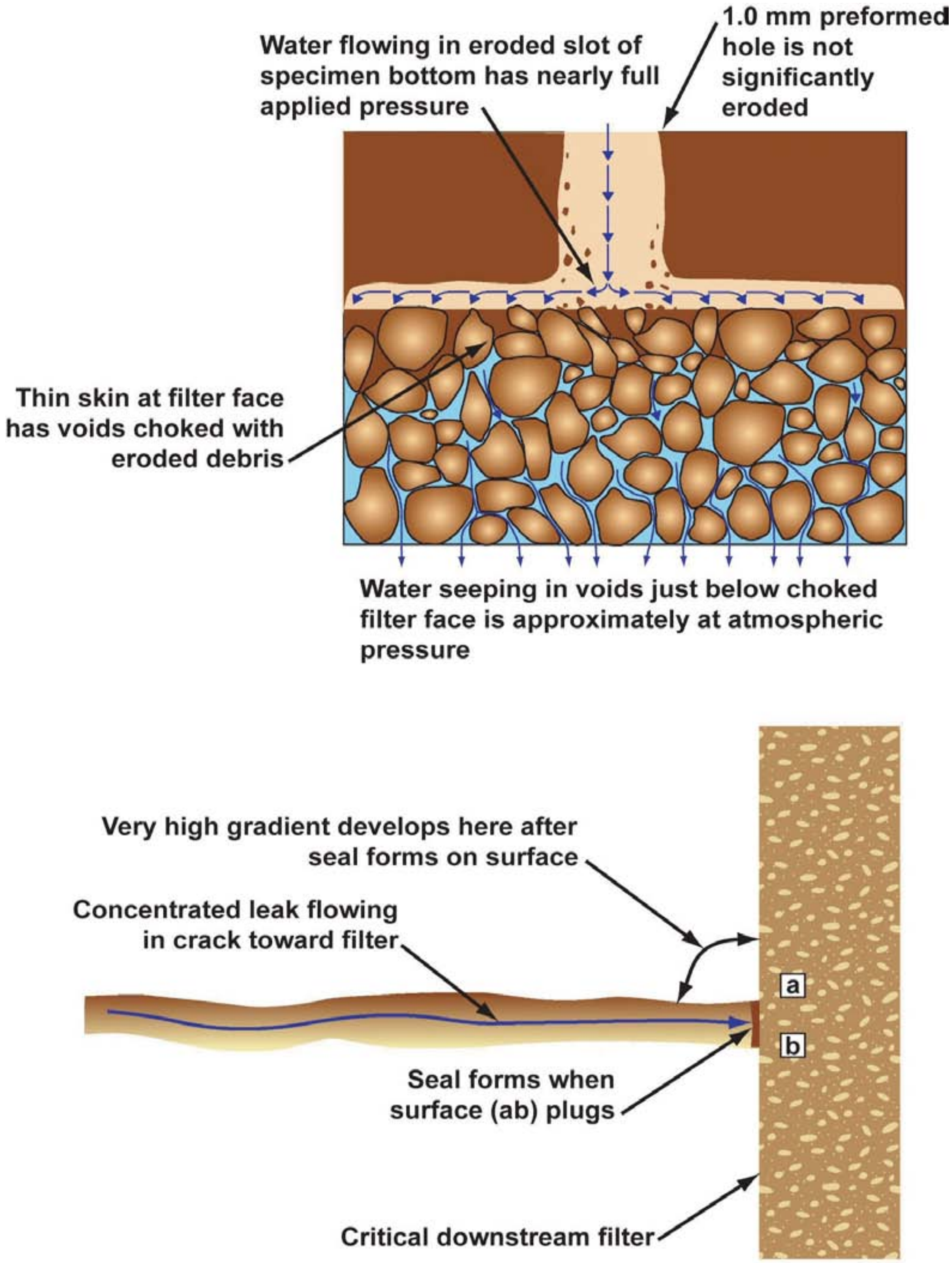


Figure 86.—Illustration of the mechanism of development of a filter seal resulting from the accumulation of eroding soil particles at the face of a designed filter zone, showing filter cake development (top), and sealing of concentrated leak (bottom). The filter seal results in a thin zone of soil with a slurry-like consistency and a permeability similar to that of the soils that eroded to form the seal (after Sherard, Dunnigan, and Talbot, 1984).

> seepage or leakage along the surface of the structure or through adjacent im pervious zones is the use of a properly designed filter and drainage zones around the conduit downstream of the impervious core along with quality constructed fill adjacent to the structure.
>
> Current policy is that cutoff collars should not be used as a seepage control measure and any other protruding features on a conduit should be avoided.

- *Natural Resources Conservation Service*.—From NRCS's *Earth Dams and Reservoirs* (1990, p. 6-7):

> Use a filter and drainage diaphragm around any structure that extends through the embankment to the downstream slope. . . . It is good practice to tie these diaphragms into the other drainage systems in the embankment or foundation. Foundation trench drains and/or embankment chimney drains that meet the minimum size and location limits are sufficient and no separate diaphragm is needed.

- *U.S. Army Corps of Engineers*.—From USACE's *General Design and Construction Considerations for Earth and Rock-Fill Dams*— (2004a, p. 6-6):

> When conduits are laid in excavated trenches in soil foundations, concrete seepage collars should not be provided solely for the purpose of increasing seepage resistance since their presence often results in poorly compacted backfill around the conduit. Collars should only be included as necessary for coupling of pipe sections or to accommodate differential movement on yielding foundations.
>
> . . . Drainage layers should be provided around the conduit in the downstream zone of embankments without pervious shells. . . . In embankments having a random or an impervious downstream shell, horizontal drainage layers should be placed along the sides and over the top of conduits downstream of the impervious core.

Filter zones are provided in embankment dam designs to meet various requirements and conditions. Filters serve the following purposes (ASDSO, 2003):

- For water seeping through the natural voids of the soil (embankment dam or foundation), a drainage system is designed to intercept this seepage and carry it to a safe outlet.
- A filter consists of a graded sand and/or gravel material designed to prevent the migration of soil particles from the base soil being drained.
- The filter supports the soil discharge face, and no movement of soil occurs with water flow.

- Filters are placed next to the soil. If needed, one or more coarser zones are placed behind the filter to serve as the drain. Collection pipes may be used to carry the water to a safe outlet.

- If the filter has sufficient permeability, it can serve as both the drain and the filter.

- Filters also intercept cracks, openings, or other anomalies where water flow has the potential to develop a concentrated leak.

- In cracks or openings, filters intercept soil moving in suspension with the water; a filter cake is formed that seals the crack and prevents further erosion.

FEMA is sponsoring the development of a "best practices" guidance document for filters used in embankment dams. This document will contain detailed procedures and guidelines for design and construction of filters. The design manual will be based on experience provided from experts in the fields of geotechnical engineering and construction. The expected publication date is 2007.

Filters used in conjunction with conduits through embankment dams generally fall into three broad categories: chimney filters, filter diaphragms, and filter collars. These filters are discussed in the following sections.

6.3 Chimney filters

A chimney filter that extends upward to the highest probable pool level and extends across the length of the embankment from abutment to abutment is a common element for most high and significant hazard embankment dams. Chimney drains are also valuable for sites with a high permanent water storage level, because they intercept and lower the phreatic line and maintain a stronger downstream zone of unsaturated soil. Figure 87 shows a double gradation zone chimney filter being installed in the construction of a modern embankment dam.

Chimney filter zones are a valuable protection against internal erosion in transverse cracks that could occur in the embankment dam. Chimney filters are also commonly used when embankment dams contain zones of widely varying gradation. The chimney filter provides a transition and filtering capability between these zones. Multiple filter and drain zones may be required in embankment dams that include zones of soils with a wider range in gradation. Examples are embankment dams with zones of impervious, finer grained soils with coarse shell zones of rock or gravel fill.

Figure 87.—Two stage chimney filter within an embankment dam located in Texas.

Embankment dams with chimney filters do not normally require a separate feature to control internal erosion or backward erosion piping along the conduit. The chimney filter will usually encompass the conduit and serve the function of a filter diaphragm as well as the other primary functions of a chimney filter, such as providing transition filter capability and controlling the phreatic line. Figure 88 (top) shows a zoned embankment dam with a chimney filter constructed along the interface between the central core of the dam and the downstream shell. The chimney filter encompasses the conduit at the intersection and serves as a filter diaphragm as well as a chimney filter.

Figure 88 (bottom) illustrates a design where an embankment chimney filter also serves as a filter diaphragm around the conduit. This example is for a homogeneous embankment with a vertical chimney filter. Chimney filters are typically installed in new construction or within embankment dams undergoing extensive renovation.

Figure 89 provides another example of a typical design used for a high hazard embankment dam. This design includes a chimney filter, a drain at the downstream edge of the cutoff trench, and a foundation blanket drain. The chimney filter and cutoff drain satisfy the functions of a filter diaphragm around the conduit. The blanket drain serves as an outlet for the collected seepage flow. The design shown is typical for embankment dams that have distinctly different materials in the core zone and the exterior shells of the dam. The foundation blanket drain may also function to collect seepage in bedrock or permeable foundation horizons, and function to convey collected seepage to the downstream toe. Not all high hazard embankment

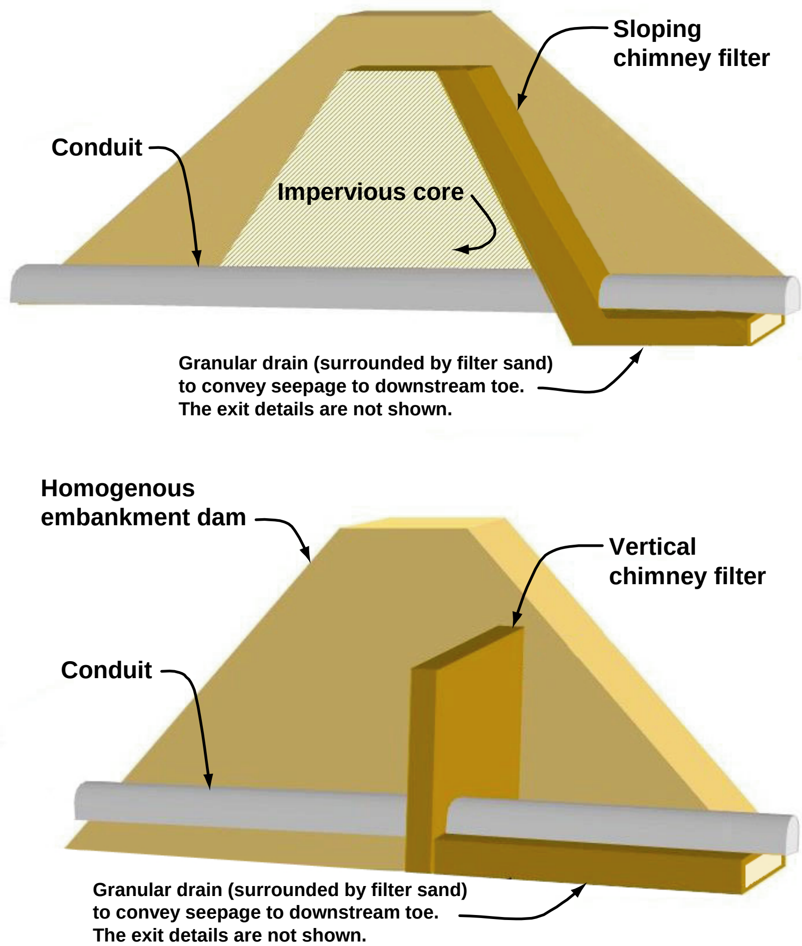


Figure 88.—Typical configurations for chimney filters used in the design of embankments with conduit penetration. A chimney filter may be constructed either on a slope (top) or with a vertical configuration (bottom) within the embankment dam.

dams require the same configuration of foundation drainage features shown in figure 89, but the design shown is a typical one.

Some low hazard embankment dams may not include a chimney filter for the entire length of the dam, if the following factors are present:

- The embankment dam is constructed of soil(s) with a good resistance to both internal erosion and backward erosion piping. Particularly important is that dispersive clays are not used in the construction.

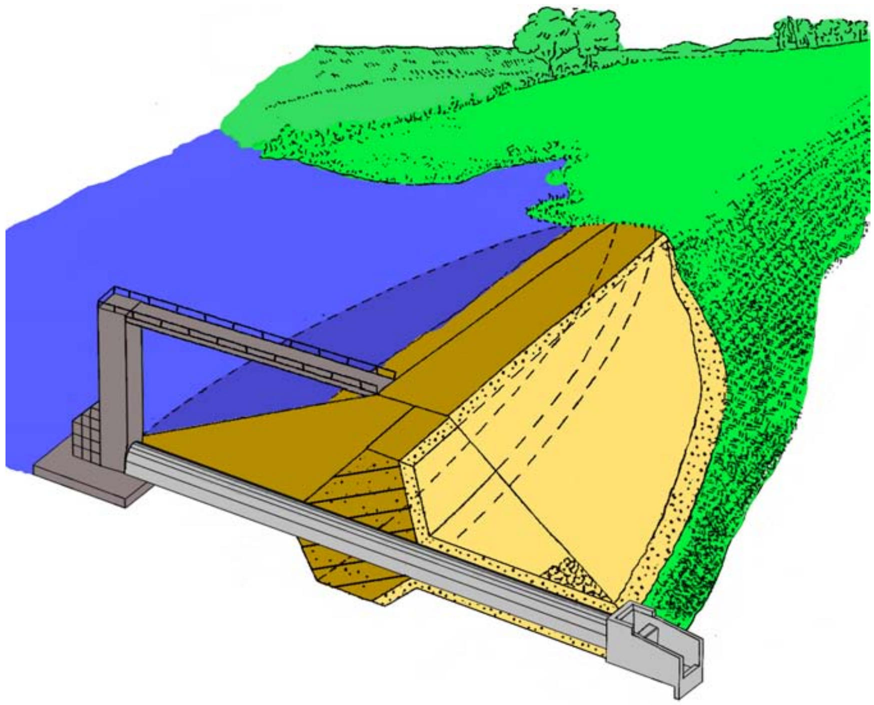


Figure 89.—Typical design used for embankment dams with distinctly different materials in the core zone and exterior shells of the dam.

- Zones of widely varying grain size are not used adjacent to one another in the cross section. Examples include core zones of fine grained soil with exterior zones of coarser gravel or rockfill.

- Abrupt changes in the cross section transverse to the centerline are not present. Examples are bedrock ledges, closure sections, and excavated trenches.

- Good oversight of construction is provided, including quality assurance and control that are consistent and effective.

- Special care is taken in following recommendations for compaction of fill surrounding the conduit, as discussed in section 5.3.2.

If a chimney drain is not included in the design of a low hazard embankment dam, the design measures for preventing internal erosion or backward erosion piping along the conduit may consist of three basic choices:

1. A filter diaphragm is a valuable defensive design measure even for low hazard classification embankment dams with favorable site conditions. The filter diaphragm will intercept flow along discontinuities along the pipe or through

cracks in the earthfill immediately surrounding the conduit that are caused by differential settlement associated with the conduit. Cracks in the earthfill surrounding the conduit may also be caused by desiccation during interruptions of fill placement, hydraulic fracture, and other mechanisms discussed in this document. Section 6.4 discuss filter diaphragm considerations in detail.

2. A filter collar is sometimes used rather than a filter diaphragm. This configuration protects against flow directly along a conduit, but does not address potential flow in cracks that may occur in the soils surrounding the conduit. Section 6.5 discusses in more detail the guidelines for using a filter collar rather than a filter diaphragm.

3. Some design agencies, including the NRCS, still allow the use of antiseep collars as a defensive design measure to address flow along conduits for low hazard embankment dams with favorable soil and site conditions. Some regulations by State and other entities may also require the use of antiseep collars. As discussed in detail in appendix A, the theory behind the development of antiseep collars is flawed, and their continued use may be considered a relic of conventional design. Because antiseep collars impede the uniform compaction of backfill along the conduit, and their theoretical basis is not sound, their use has been largely abandoned. The only likely benefit of antiseep collars would be an interruption of flow that might occur from poor construction practices on circular conduits where the pipe is dislodged by construction efforts or backfill under the haunches of the pipe is loose. However, antiseep collars are not recommended as the "best practice" approach to the design of conduits.

6.4 Filter diaphragms

A filter diaphragm is an important component of design for both new construction and the renovation of older embankment dams. Some low hazard embankment dams constructed of soils that are inherently resistant to internal erosion or backward erosion piping may not include a chimney filter zone for the entire length of the dam. However, these embankment dams should still contain a filter diaphragm around any conduits within the dam.

As discussed in section 6.3, chimney filters are used in many embankment dam designs, both new and renovation. The chimney filter zone can usually satisfy the function of a filter diaphragm. A separate component is not required for those embankment dams, if the chimney filter surrounds the conduit in a similar fashion as the filter diaphragm would.

A filter diaphragm (figures 90 and 91) is a designed zone of filter material constructed around a conduit in which a chimney filter is not being used (usually in

Figure 90.—Construction of a filter diaphragm within an embankment dam.

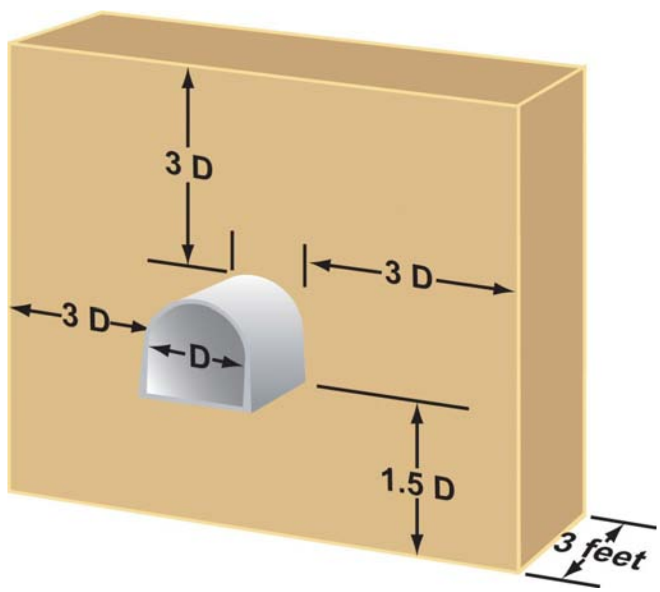


Figure 91.—Typical configuration for filter diaphragm used in the design of an embankment dam.

the renovation of an existing embankment dam). This zone can act both as a drain to carry off water and as a filter to intercept soil particles being transported by the water. The filter diaphragm will intercept both intergranular flow through the embankment dam and flow through cracks in the earthfill or along the interface between the conduit and the earthfill. Any fines being eroded from the embankment will be filtered by the diaphragm of sand that surrounds the conduit. The fines carried by the flowing water will accumulate on the surface of the diaphragm and develop a filter cake. The filter cake that develops on the upstream face of the filter diaphragm reduces the flow and prevents further erosion of any cracks caused by this flow. The filter diaphragm must extend far enough from the conduit that it can intercept all potential water flow paths associated with the conduit. A filter diaphragm is typically installed during new construction or with conduit renovations.

If the only postulated flow path is immediately along the contact between the earthfill and the conduit, the filter diaphragm may not need to extend far from the conduit. As an example, some agencies only use a filter diaphragm 18 inches thick, which is similar to a filter collar. In other cases, the embankment dam may be subject to hydraulic fracture in zones that are far above and on either side of the conduit. In the absence of a chimney filter, the filter diaphragm may need to be much wider and taller than the dimensions of the conduit to intercept those cracks.

A filter diaphragm that extends farther from the conduit is often recommended for designs where significant differential settlement is associated with the conduit and/or the trench used to install the conduit. This type of filter diaphragm is often used for embankment dams constructed without a chimney filter, when soils with a very low resistance to internal erosion, such as dispersive clays, are used to construct the dam. This type of diaphragm is a zone of designed gradation filter sand that completely encircles the conduit. The shape of the filter diaphragm is usually either rectangular or trapezoidal, and the diaphragm is typically 3 feet thick. Figure 91 shows a typical configuration for a filter diaphragm. For further guidance on recommended dimensions for a filter diaphragm, see NRCS's *Earth Dams and Reservoirs* (1990) and *Dimensioning of Filter Drainage Diaphrams for Conduits According to TR60* (1989).

The NRCS filter diaphragm typically has the following characteristics. Agency policy and the judgment of the individual designer may dictate different dimensions:

- *Configuration.*—The filter diaphragm is a rectangular or trapezoidally shaped zone of filter sand that is about 3 feet wide in a direction perpendicular to the conduit; see figure 91.

- *Location.*—Locating the filter diaphragm along a conduit depends on several site conditions. If the embankment is zoned, the filter is often located at the juncture of the impervious core zone and downstream shell zone. In a homogeneous embankment dam, the location of the filter diaphragm is usually based on the following requirements:

    1. Downstream of the cutoff trench.

    2. Downstream of the centerline of the embankment dam when no cutoff trench is used.

3. Upstream of a point where the top of the filter diaphragm has at least a thickness of soil overlying it that is a minimum of one-half of the difference in elevation between the top of the diaphragm and the maximum potential reservoir water level.

The rationale for the third requirement is that if an open crack occurred within the embankment dam and the full reservoir water pressure was acting on the crack, that pressure would be transmitted along the crack with little head loss to the point where the crack intercepted the filter diaphragm. At that point, the crack would presumably be sealed from sloughed particles carried along the crack to the face of the filter diaphragm. Then, at that interface between the open crack with a seal and the filter diaphragm (see figure 86 [bottom]), full reservoir hydrostatic pressure could exist. The criterion is intended then to ensure that the weight of overlying soil in the embankment counters this hydrostatic pressure. The rule requiring the thickness of one-half the reservoir head is based on the simplification that the unit weight of moist earthfill is approximately twice the unit weight of water.

- *Vertical/horizontal limits*.—The filter diaphragm should extend below and to either side of the conduit far enough to intercept potential flow along excavation/embankment interfaces. Usually, the filter diaphragm extends into the foundation a dimension equal to at least 1.5 times the diameter of the conduit, unless bedrock is encountered at a shallower depth.

- *Lateral limits*.—The filter diaphragm usually extends laterally a distance at least equal to 3 times the diameter of the conduit or a minimum of 10 feet from the sides of the conduit. In some situations the filter diaphragm may need to be wider than these minimum suggested dimensions. For instance, if an excavation has been made for the conduit, the filter diaphragm should notch into the excavation slopes by at least 2 feet; see figure 92.

A designer should consider several factors in determining the dimensions to use for a filter diaphragm, as follows:

- *Whether a filter diaphragm is a "stand-alone" element of the embankment dam's design, or if it is a coincidental part of a chimney filter in the dam*.—Many embankment dams include a chimney filter that extends across the entire length of the dam. When the chimney filter is located where it can also encompass the conduit passing through the embankment dam, a separate filter diaphragm is not required. Figure 88 shows two configurations commonly used for chimney filters that would serve the function of a filter diaphragm, as well as the functions of a chimney filter.

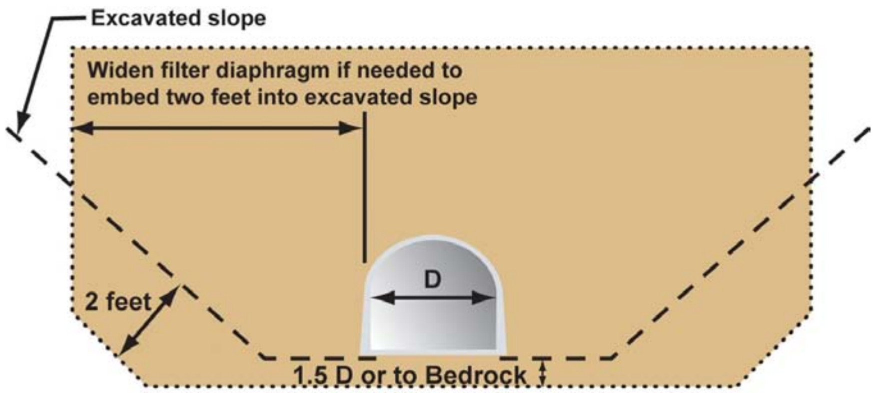


Figure 92.—Typical configuration for filter diaphragm used in design of an embankment dam. The filter diaphragm should extend into the foundation soils, where an excavation is made for the conduit.

- *The type of equipment used to construct the filter diaphragm*.—If a chimney filter is used in the design of the embankment dam and it serves the coincidental purpose of a filter diaphragm, then dimensions appropriate for a chimney filter should be used. Many times, chimney filters are sized for the width of equipment (about 10 to 12 feet) to accommodate production equipment in placing and compacting this zone in the embankment cross section. In small embankment dams where the cost of the chimney filter may be excessive, a narrower width may be considered.

- *The method of constructing the filter zone*.—If the chimney filter zone or filter diaphragm is constructed concurrently with the adjoining fill zones, using a width of 10 to 12 feet is suited to most construction equipment. A chimney filter designed with multiple filter zones complicates construction. Figure 87 shows such a design. If the filter zone is placed in a trench cut in compacted fill, a width of 3 feet is often specified, because that is typically the width of backhoe buckets used for this method of construction. A paper by Hammer (2003) includes good discussions of the advantages and disadvantages of various methods for constructing chimney filter zones. The discussions would apply equally well to sites where only a filter diaphragm was used instead of a full chimney filter section. The trenching method is illustrated in figures 6, 7, and 8 of that paper.

- *The predicted zones of embankment susceptible to hydraulic fracture resulting from the presence of the conduit in the fill*.—The filter diaphragm should extend vertically and laterally far enough to intercept all zones of the fill and foundation that are susceptible to hydraulic fracture attributable to the presence of the conduit in the dam. Hydraulic fracture is discussed in section 5.2. Hydraulic fracture zones not caused by the conduit, but by other factors, such as steeply dipping bedrock surfaces, are usually addressed with chimney filter zones in addition to a filter diaphragm.

- *The ability of the filter diaphragm to prevent propagation of a crack through the zone.*—The purpose of a filter diaphragm is to intercept cracks in the earthfill and collect and filter any flow eroding the walls of the crack. The filter zone must be thick enough to prevent a crack from propagating through the filter. Many designers consider a thickness of 3 feet as adequate to satisfy this requirement. Other factors important in crack propagation include the gradation of the filter used for the diaphragm, its degree of compaction, and the potential for cementation of the filter. For less favorable conditions, wider filter diaphragms may be advisable.

Intergranular seepage passing through the filter diaphragm may be collected and conveyed downstream to the toe of the embankment dam with various design approaches. The outlet drain to convey the collected flow may be a combination of granular filters and it may or may not include a perforated collector pipe. Figure 93 shows one type of outlet drain for a filter diaphragm. This figure shows an outlet drain consisting of a zone of gravel surrounded by a fine sand filter, without a collector pipe. Collector pipes may also be included in the designs for outlet drains for filter diaphragms, particularly to provide a safety factor for conveying larger than expected flow quantities. Many designers contend that outlet drains should be designed to have a capacity to convey all of the collected flow in the granular zones alone, without considering the additional capacity provided by a collector pipe—the reason being that the collector pipe could eventually be damaged or otherwise become inoperative, and the granular zone would still be functional.

The estimated flow quantity that filter diaphragms are required to convey depends primarily on the predicted quantity of intergranular seepage, not flow through cracks that are intercepted by the diaphragm. If properly designed, the filter diaphragm will form a seal on the face of any intercepted cracks, and subsequent flow through the face of the crack at the filter will be similar to intergranular seepage.

In addition to the dimensions of filter diaphragms, designers must also decide whether to use a sloping zone or a vertical configuration for the diaphragm. Each configuration has advantages and disadvantages:

- *Sloping configuration.*—Filter diaphragms and chimney filters may also be constructed with a sloping configuration, as illustrated in figure 88 (top). This configuration is more common on larger embankment dams and those with distinct zones in the dam. The filter zone is often placed at the juncture between the core and shell zones in the dam as shown in figure 88 (top). This configuration reduces the effect of differential settlements between the filter zone and the adjacent embankment zones. Because sloping zones are typically

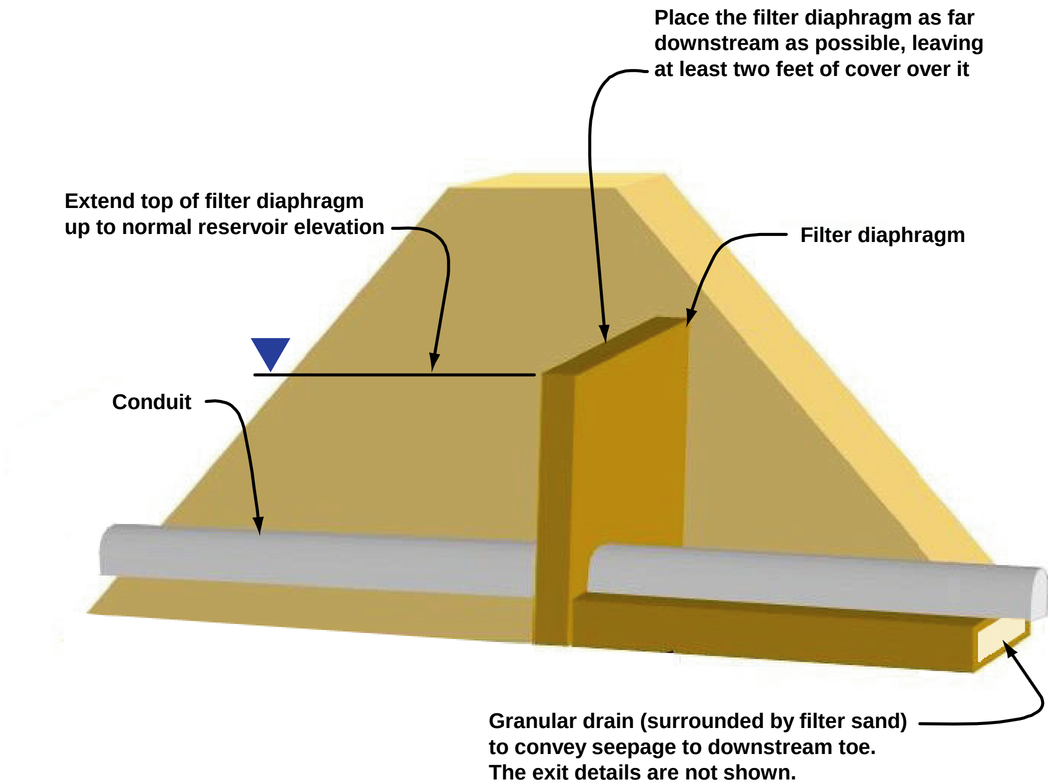


Figure 93.—Typical configuration for a filter diaphragm used in the design of an embankment dam. The figure shows the location of the filter diaphragm as far downstream as possible, leaving adequate cover over it.

wider than vertical zones, differential settlement occurs over a wider distance, also lessening the potential for cracking associated with the differential settlement. Collapse of the filter zone on wetting is still a concern, and proper compaction control is needed, as it is for a vertical zone. Sloping configuration zones have a lower potential to cause cracking of the surrounding embankment zones for reasons listed and are preferable when the design permits. Constructing a sloping filter diaphragm would be considerably more difficult for small homogeneous embankments, and this configuration is seldom used for those designs.

- *Vertical configuration*.—This configuration for a filter diaphragm is the one commonly used. Figure 88 (bottom) shows a vertical zone for the filter zone surrounding a conduit (In this case the zone is a combination filter diaphragm and chimney filter, but illustrates the shape for a design using a smaller filter diaphragm as well). Vertical filter diaphragms are commonly constructed using the trenching method, as shown in the paper by Hammer (2003). This shape of filter diaphragm is common in embankment dams that do not have distinct zones, and the engineering properties of the embankment soils on both sides of the diaphragm are similar.

Filter zones are typically composed of somewhat well graded relatively clean sand, such as ASTM C 33 fine concrete aggregate. The compressibility characteristics of this zone likely are different than those of the earthfill in which it is placed. This may create concern over differential settlement between the embankment soils and the filter diaphragm. A special concern is the potential for collapse of the filter zone upon wetting. To reduce this potential, filters are typically compacted to a moderate degree as described in section 6.8. Because any potential differential settlement is oriented parallel to the embankment, less concern occurs than if the differential movement were transverse to the embankment dam.

For an example of a project that used a filter diaphragm, see the case history in appendix B for Waterbury Dam.

6.5 Filter collars

A filter collar consists of a zone of filter material (usually sand) that completely surrounds a specified length of conduit. This type of filter is recommended, if the only flow that is considered likely is that along the contact between the conduit and the surrounding earthfill, and embankment soils are not dispersive clays.

A filter collar should be limited to sites with few problems. If conditions exist that could cause hydraulic fracture, or if soils in the embankment dam are very low in erosion resistance (such as dispersive clays), more substantial filter zones, as discussed in section 6.4, should be used rather than a filter collar. A filter collar is generally used in conduit renovation or new construction.

For renovations, the filter collar wraps the downstream one-third length of the conduit, and the filter is about 18 inches thick. The thickness depends upon design requirements. The USACE's *Design and Construction of Levees* (2000, p. 8-5) and *Culverts, Conduits, and Pipes* (1998a, p. 1-3) show a typical design. Figures 94 and 95 illustrate an example of this type of filter design for a conduit renovation.

The dimensions for the filter should vary with the size and complexity of the embankment dam. Larger filter collars and multiple zones may be needed for more complex, significant hazard to high hazard embankment dams or those with problematic soils. The gradation of the filter collar should be designed for filter compatibility with the surrounding soils in the embankment dam. At the downstream end of the filter collar, a zone of gravel may be placed at the end for

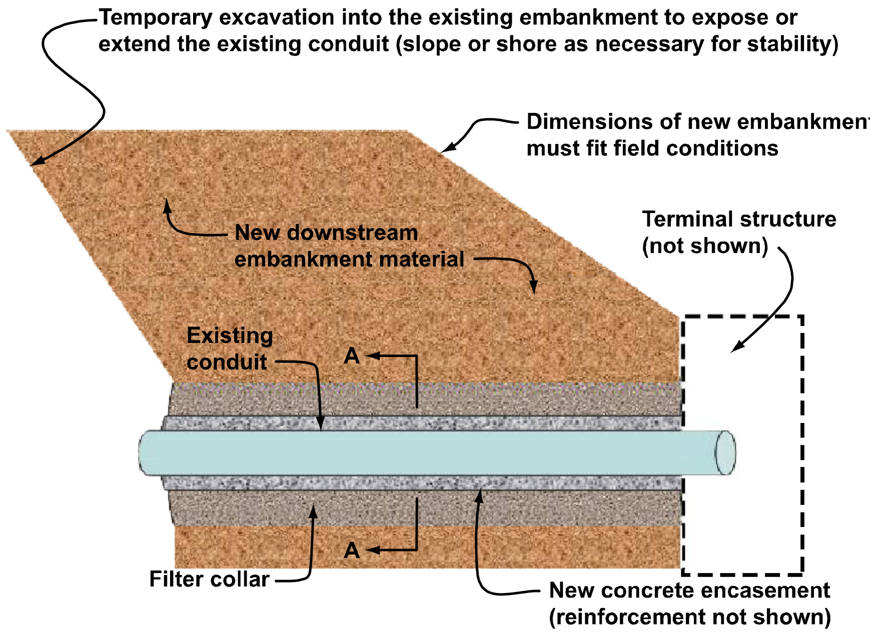
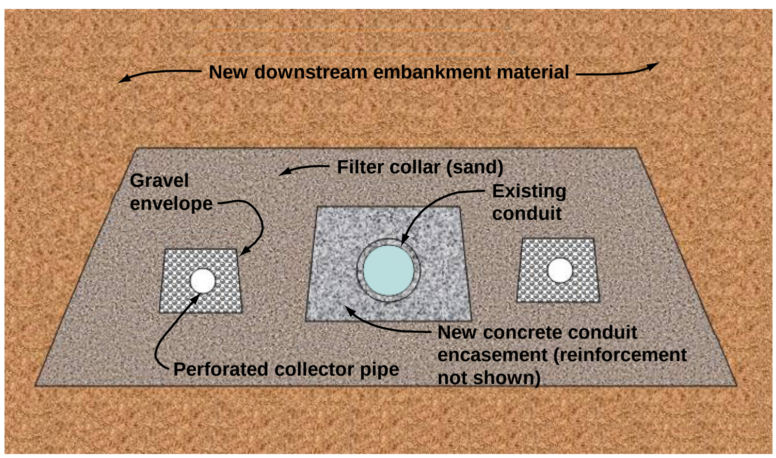


Figure 94.—Filter collar surrounding a conduit renovation.



Section A-A

Figure 95.—Cross section showing a filter collar surrounding a conduit renovation. The drain pipes located on both sides of the conduit collect and convey any seepage within the filter collar to a downstream exit location.

filtering the sand and providing a controlled outlet for the collected seepage. The gravel should meet filter criteria for the sand filter.

At the very downstream toe of the embankment dam, a short section of perforated drain pipe (similar to a toe drain) is often provided to collect and convey any seepage water out of the filter collar. This collector pipe will help keep the toe area from becoming boggy. The pipe will also provide opportunity to measure seepage flow and monitor for sediment transport. Provisions should be made in the design of the collector pipe to allow for inspection by CCTV equipment. See section 9.5.4.4 for a discussion of CCTV inspection.

Burying the collector pipe too far into the embankment should be avoided. The gravel envelope drain should be capable of providing the needed drainage. The designer should provide access provisions for the collector pipe to enable inspection and cleaning. Also, should the collector pipe become damaged, it should be located such that it can be removed and replaced.

6.6 Filter and drain gradation design

Designing the gradation of the sands used in the filter is important if they are to function properly. Standard filter design methods, such as the NRCS design procedure shown in *Gradation Design of Sand and Gravel Filters*, National Engineering Handbook, Part 633, Chapter 26 (1994), the USACE's *General Design and Construction Considerations for Earth and Rock-Fll Dams* (2004a), and Reclamation's *Embankment Dams—Protective Filters* (1999) are typical. Materials suitable for filters will rarely be available onsite and are usually purchased from concrete aggregate suppliers or processed from materials near the site. For many designs, ASTM C 33 fine concrete aggregate is found to meet criteria. However, designers should always determine a range of compatible filter gradations that will protect the soils used in the surrounding embankment to increase construction flexibility. Sands used to construct filter diaphragms, filter collars, or chimney filters should be filter compatible with the embankment zones being protected, and they must also be able to deform and fill any cracks that may be propagated to the filter. If the filter does not have a property referred to as "self-healing," the crack could propagate through the filter, and the filter would not satisfy its intended function. Vaughan and Soares (1982, p. 29) and the USACE (1993, p. 8-6) have described a simple test for evaluating the self-healing ability of a filter. Factors that influence the ability of a filter to be self-healing are the percentage of fines (percent finer than the No. 200 sieve) and the plasticity of the fines. Filter designs usually require a low percentage of nonplastic fines (usually less than 5 percent as measured after placement in the embankment dam and compaction of the filter) to ensure that filters have adequate permeability and self-healing characteristics. Fine, poorly graded filters have less desirable self-healing characteristics than broadly graded, coarse filter materials. However, segregation of broadly graded filters can be a serious problem.

The no erosion filter test described by Sherard and Dunnigan (1989, pp. 927-930) is useful in evaluating whether a specific filter gradation is filter compatible with a specific base embankment soil. While existing filter criteria generally ensure filter compatibility, site-specific testing using the no erosion filter test is advisable for some situations. For high hazard embankment dams, where designers want additional documentation on the filter compatibility between a specific filter source and the embankment soils, or where designers wish to explore whether relaxing strict filter criteria would be safe, conducting laboratory filter tests with actual site materials is advisable.

Other considerations for the filter and drainage zone materials include:

- Granular materials should be hard and durable, so they will not break down during transportation, placement, and compaction. Overcompaction can reduce permeability and increase fines.

- The specified gradation should filter the embankment dam's core material and be permeable enough to avoid excess water pressure buildup.

- The filter gradation should be designed to avoid segregation during placement.

- A two-layer filter may be required for zoned embankment dams where the core and shell zones are very different.

- Designs for drainage zones favor permeability considerations over filtering criteria. Ordinarily, filters that are separate from the other embankment dam drainage zones are not expected to convey much flow, because their purpose is to intercept and prevent flow through cracks.

- The most likely damage that can occur to sand and gravel filter/drainage zones or material during construction is contamination or segregation.

- Segregation will occur when the filter or drain material is dumped from an end-dump truck or other hauling unit, where the material falls more than about 2 feet. Close inspection is always necessary to make sure segregation is not occurring. The use of narrowly graded materials helps to prevent this problem.

- Wide gradations (gradations that include a wide range of particle sizes) can be internally unstable. This is a problem when the finer portion of the filter can pass through the coarse portion with water flow; washing the finer portion out and leaving a very coarse soil matrix that will not serve as a filter for the base soil. This can be checked by mathematically dividing the grain-size curve into two gradations. If the coarse gradation does not meet the filter requirements for the fine gradation, the filter is internally unstable (ASDSO, 2003).

- Avoid placement of filter materials in freezing temperatures. Frozen filter material cannot be properly compacted.

Recently, an additional method has been proposed that expands on the existing research on protective filters. The method (Foster and Fell, 2001, pp. 398-407) establishes a "continuing erosion boundary" that is based on the analysis of the results of laboratory tests and the characteristics of dams that have experienced internal erosion incidents. The method can be used to determine whether filters that are coarser than required by modern filter criteria will eventually seal, or experience continuing erosion leading to possible failure of the embankment dam in the event that internal erosion begins. The method is intended to help evaluate filters in existing embankment dams only and should not be used to design new filters for dams.

6.7 Construction of the filter

Construction of the filter should be performed carefully to ensure that high quality is obtained. The construction needs to ensure that the filter is placed completely around the conduit. Sloppy placement techniques can result in voids in the filter or inadequate bond with the conduit encasement.

Placement of the sand filter and adjacent materials in the embankment must be performed to avoid contamination of the filter. During construction, the sand filter zone should be maintained above adjacent materials to preclude contamination. Construction traffic crossings over filter zones should be minimized. The surface of the filter at the crossing can be covered with plastic and the plastic covered with earthfill to reduce contamination at the crossing.

Placement techniques should ensure that segregation of the filter does not occur. Segregation will cause portions of the filter to be overly coarse, which can allow embankment material to flow through, negating the purpose of the filter. Segregation can be avoided by careful selection of the handling equipment. The following has been adapted from Reclamation's *Embankment Dams—Protective Filters* (1999, p. 16).

> A common cause of segregation is the manner in which material is handled. Material placed in a pile off a conveyor, or loaded from a chute, or from a hopper segregates because the larger particles roll to the side of stockpiles or piles within the hauling unit. Material dumped from a truck, from loader, or other placing equipment will almost always segregate, with the severity of the segregation corresponding to the height of the drop. When material is dumped on the fill, segregation occurs.

> Segregation can be minimized in several ways. First, the designer should avoid using widely graded filters that are more prone to segregation. Rather than using a single, widely graded filter, a designer could specify a dual band of filters. A fine filter layer to protect the finer embankment materials would be outletted into a coarser layer used to outlet the collected flow. Secondly, construction techniques to control segregation should be specified and enforced. Use of rock ladder, spreader boxes, and tremies or "elephant trunks" for loading hauling units, and hand working the placed materials will help prevent segregation. If material is dumped, limiting the height of drop will help. Often the height of drop is limited to 4 feet maximum. Placing filter/drain material with belly dumps is a better method than others because the height of drop of materials is limited by the equipment. Limiting the width of the belly dump opening by chaining or other means can increase their ability to limit segregation. Using baffles in spreader boxes and other placing equipment can help reduce segregation. The personnel inspecting the filter/drain production, placement, and compaction should be trained in the techniques effective in preventing segregation. They should be aware of contract provisions in specifications that are intended to prevent segregation and be prepared to enforce those specifications.

Filter zones must be compacted properly to avoid problems. Vibratory compactors (usually smooth drum or plate), are more efficient in densifying filters than "kneading" compactors (such as sheepsfoot or padfoot), without causing much breakdown. Breakdown of the filter material's particles can cause the gradation of filters to change. The most harmful result of breakdown is the increase in the percentage of fines (usually defined as the material passing the No. 200 sieve size). Excessive fines in the material will drastically reduce the material's permeability and adversely affect other of the filter's attributes. Overcompaction should be avoided. Often the gradation of the filter is specified as "in-place, after compaction," thus ensuring that the intended gradation is obtained. Also, specifying filter material that is comprised of hard and durable particles is important in helping minimize breakdown.

Previous sections have discussed factors that can affect the integrity and quality of the filter diaphragm or collar around a conduit. A paper by Hammer (2003) contains valuable additional guidance on constructing drain zones within embankment dams. Recommendations in that paper include other factors in addition to those discussed in previous sections of this document. Recommendations are included for methods to avoid contamination of filter zones, advantages and disadvantages of various schemes for constructing vertical drainage zones, and others. The recommendations in that paper should also be considered when constructing filter collars and diaphragms associated with conduits, as well as when constructing other types of embankment drainage zones.

6.8 Specifications and density quality control and quality assurance for filters and drains

Contract specifications contain requirements for compacting sand for filters. Quality control personnel are responsible for ensuring that the specifications are met. Quality assurance personnel then ensure that the quality control methods and equipment are satisfactory.

Filters are compacted using two principal types of specifications. Compaction of sands is important for filter construction. Controlled compaction of the filter sands is important to prevent settlement of the sands on wetting and liquefaction during seismic activity. At the same time, filters should not be overly compacted because that can reduce their ability to "self-heal" or adjust to any movements in the underlying embankment and foundation. The two types of specifications used for controlling compaction of sands and gravels are:

- *Method placement specification*.—A method placement specification requires the filter sand to be compacted in lifts of a stated maximum thickness using specified equipment operated in a specified manner. The specification assumes that the designer has previous favorable experience with a specified method and feels sure that the filter sand will have adequate properties, if it is compacted using these procedures. This type of specification does not require a specific density or water content, but relies on the specified procedure to produce desirable filter.

  Quality control and quality assurance for this type of specification concentrate on observations and documentation of the processes used to place the filter, compared to the acceptable methods listed in the specifications.

  An example of a method placement specification is shown in the NRCS's *Drainfill*, National Engineering Handbook, Part 642, Specification No. 24 (2001f, p. 24-2). Class I compaction requires each layer of drainfill to be compacted "... by a minimum of two passes over the entire surface with a steel-drum vibrating roller weighing at least 5 tons and exerting a vertical vibrating force of not less than 20,000 pounds at a minimum frequency of 1,200 times per minute, or by an approved equivalent method."

  For filters, using smaller compaction equipment, such as walk-behind vibratory rollers and plate compactors, may be required if working space is limited.

  Test fills can be performed in advance of construction to check on the adequacy of method type placements.

Construction specifications typically also include other requirements. Examples from the NRCS's *Drainfill*, National Engineering Handbook, Part 642, Specification No. 24 (2001f, p. 24-1) include:

> Drainfill shall be placed uniformly in layers not to exceed 12 inches thick before compaction. When compaction is accomplished by manually controlled equipment, the layers shall not exceed 8 inches thick. The material shall be placed to avoid segregation of particle sizes and to ensure the continuity and integrity of all zones. No foreign material shall be allowed to become intermixed with or otherwise contaminate the drainfill.
>
> Traffic shall not be permitted to cross over drains at random. Equipment cross-overs shall be maintained, and the number and location of such crossovers shall be established and approved before the beginning of drainfill placement. Each crossover shall be cleaned of all contaminating material and shall be inspected and approved by the engineer before the placement of additional drainfill material.
>
> Any damage to the foundation surface or the trench sides or bottom occurring during placement of drainfill shall be repaired before drainfill placement is continued.
>
> The upper surface of drainfill constructed concurrently with adjacent zones of earthfill shall be maintained at a minimum elevation of 1 foot above the upper surface of adjacent earthfill.
>
> When placed, drainfill shall be in a wet or near saturated condition. Each layer of drainfill shall be saturated immediately prior to compaction.
>
> Drainfill shall be placed in such a manner as to prevent segregation of particle sizes.

Application of water in front of the vibratory roller drum during compaction is crucial. The inability to achieve the desired density is typically due to insufficient water application.

- *End result specification*.—An end result specification requires the filter sand to be compacted to a specified value of dry density. Usually, the required density is specified by reference to a standard test. The traditional method for specifying filter sand compaction uses relative density terminology and tests. ASTM D 4254 describes the test used to measure the minimum index density of a filter sand; ASTM D 4253 describes the test for maximum index density of a sand. NRCS's *Drainfill*, National Engineering Handbook, Part 642, Specification No. 24 (2001f, p. 24-2) shows an example of an end result specification. Class A compaction requires each layer of drainfill to be compacted ". . . to a relative density of not less than 70 percent as determined by ASTM D 4254."

Various degrees of compaction have been historically used. Relative density specifications have typically required placement to relative density values in the range of 50 to 85 percent. Values for specified relative density may be as low as 50 percent for low hazard projects, to as high as 85 percent for larger drain zones in high hazard projects. Relative density specification may not be practical for small projects, because the equipment needed to perform the tests for index density are not readily available for field use.

An alternative to using relative density tests is a special type of Standard Proctor (ASTM D 698) compaction test. Research (McCook, 1996) has shown that good correlations exist between relative density test values for filter sands and a dry density value obtained using dry sand in a ASTM D 698 mold. The one-point test uses the ASTM D 698 (Method A) energy (25 blows per lift of a 5.5-pound hammer dropped 12 inches, using 3 lifts to fill the mold). For many sands, 70-percent relative density corresponds to the dry density obtained in the special ASTM D 698 test on dry sand. A value of 50 percent relative density correlates with 95 percent of the dry density obtained in the special ASTM D 698 test. An example of a specification using this alternative is NRCS's *Drainfill*, National Engineering Handbook, Part 642, Specification No. 24 (2001f, p. 24-2) :

> The compacted density shall be greater than 95 percent of the maximum dry density as determined by the method in ASTM D 698. The ASTM test procedure D 698 shall be modified to consist of a 1-point test performed on a representative sample of oven-dried drainfill.

When an end result type of specification is used to control the placement of filters in drainage and filter zones, such as a filter diaphragm, quality control testing is required to verify that the placed sand meets the specified requirements. Accuracy of the quality control testing is essential to prevent misunderstanding of the in-place density. Quality control tests that underestimate in-place density will lead to additional compaction in order to achieve the specified density. This additional compaction can lead to additional breakdown of the filter material. Quality control tests of compacted sands are performed most frequently using one of the following two methods:

- *Sand cone method.*—ASTM D 1556.

- *Nuclear gauge method.*—ASTM D 2922. Note that a separate ASTM Standard Test Method applies to measuring the water content by the nuclear gauge, ASTM D 3017. The ASTM Standard Test Methods for performing nuclear gauge measurements of dry unit weight and water content are being revised. The proposed revision will combine the two test methods into a single ASTM Standard Test Method. At the time of the writing of this guidance document, an ASTM Standard Test Method number had not been assigned for the revised standard.

The sand cone method is a direct measurement of in-place density (weight of solids, wet of water, and volume), whereas the nuclear method is an indirect measurement technique (measurement of backscatter radiation from a prescribed source). By definition, indirect measurement techniques require some form of conversion or empirical relationship. Knowledge and use of these relationships should be well understood by the user to guard against drawing incorrect conclusions on the nature of the material being tested.

To obtain accurate results, these tests must be performed carefully with all the precautions listed in the ASTM Test Methods carefully followed. Nuclear tests that are performed in a trench condition require that corrections to the measurements be made according to the gauge manufacturer's recommendations. Because the nuclear gauge measures water content indirectly by counting hydrogen ions, water content measurements must also be corrected by comparing readings to the oven-dry method (ASTM D 2216).

Careful calibration of the sand used in the sand cone method is important to obtaining reliable results. Experienced personnel are essential to obtain reliable results for both tests. The reader is cautioned on two points. First, opinions differ on the acceptability of the test procedures described above. Second, this discussion is only relevant to uniformly graded sand material (filter material). This section does not apply to other soil types, such as broadly graded material containing cobbles.

6.9 Use of geotextiles

Due to the lack of long term performance information on the use of geotextiles in embankment dams, it is current practice that they are not used in locations that are both critical to dam safety and inaccessible for repair. The use of geotextiles can be considered in some cases that may be critical to dam safety, but the geotextiles must be accessible for replacement. The designer must assess the potential hazard posed by failure of the geotextile and the time available to respond and repair or replacement the geotextile (France, 2000, p. 2-5).

Some limitations to evaluate when considering the use of geotextiles (ASDSO, 2003):

- As with any filter, a geotextile will clog when water containing soil in suspension enters the filter face.

- For preventing development of a concentrated leak in a crack or opening in the embankment dam, it is desirable for the filter face to clog in the area of the crack. Other areas should remain open so normal seepage through pores of the soil can be intercepted and safely discharged through the drainage system.

- Properly designed sand filters support the soil discharge face and prevent the movement of fines that would clog the filter.

- Geotextiles by themselves do not support the soil discharge face as a granular filter does.

- Geotextile installation must be made in such a way that the geotextile has intimate contact with the soil discharge face, with the distance between contact points being similar to a granular filter; if not, soil movement will occur and clog the geotextile.

- A coarse granular fill or a geocomposite on the downstream side of the geotextile generally does not provide the needed uniform pressure on the geotextile to provide the needed support to the soil discharge face on the upstream side of the geotextile.

- Inside the embankment dam, geotextiles will have very large soil pressures on both sides of the geotextile that will hold it firmly in place with no chance to distribute stresses that are produced by differential movement within the soil mass along the plane of the geotextile.

- When a crack occurs in the embankment dam, it will likely tear the geotextile in the plane of the crack.

- Damage can occur during geotextile installation from equipment passing over it, from protrusions in the underlying material, or from moving sheets of the geotextile over a rough surface. The damage may not always be detectable.

- The structural integrity of the embankment dam depends on complete continuity of the filter drainage zone, and when constructed with a geotextile, it must be without holes, tears, or defects. This is difficult to achieve in a typical construction operation.

Chapter 7

Potential Failure Modes Associated with Conduits

Water flowing through conduits can escape through defects in the walls or between separated joints of the conduit. Soils can also be carried into a conduit through these defects. If a conduit is flowing under pressure and water is forced out of defects within the conduit, this can lead to a very serious problem that must be addressed by emergency action, since catastrophic embankment dam failure could result. If a conduit is not flowing under pressure, defects within the conduit may allow soils surrounding it to be carried into the conduit by seepage and hydraulic fracture. Water escaping through defects from within nonpressurized conduits will probably have a lower velocity and lower pressure and should be less damaging to the surrounding soils, than if the conduit were pressurized. This may allow for remedial measures to be undertaken in less of an emergency mode. Generally, defects in conduits are much more serious for conduits designed for pressure flow than for nonpressurized flow.

Attempting to place a filter on the outside of a conduit at a defect is likely to be ineffective, particularly for a pressurized conduit. The quantity of flow from the defect in a pressurized conduit will likely exceed the capacity of a filter designed to protect adjacent soils. In a nonpressurized conduit, the filter designed for a given size defect may be inadequate when the defect increases in size. Replacing or renovating conduits with defects are the only reliable long term solution to preventing damage to surrounding soils. See chapters 12 and 13 for guidance on replacement and renovation of conduits.

Water flowing through soils surrounding a conduit may also cause failure of the embankment dam. A conduit within an embankment dam is a discontinuity that may create stresses in surrounding soils that are conducive to hydraulic fracture. A conduit may impede uniform compaction of soils in its vicinity. The various ways embankment dams may fail (where conduits are the sole or primary contribution to the failure) are discussed in this chapter. Many other types of failure modes for embankment dams exist that are not associated with conduits and are outside the scope of this document.

The important factors that determine the timing and severity of problems related to soil movement associated with conduit defects include:

- *Type of material used in construction of the conduit.*—Some materials, such as corrugated metal, can corrode and develop defects much sooner than conduits constructed of more durable materials, such as concrete. Conduits overlain by high earthfills are more likely to be stressed beyond their strength, resulting in the development of cracks.

- *Dimensions of the crack or hole, in relation to the gradation of the surrounding backfill soil.*—Even small defects in conduits can result in movement of finely graded surrounding soils into the conduit.

- *Resistance of the surrounding backfill to internal erosion and backward erosion piping.*—Very fine sands and silts are extremely prone to particle movement from intergranular flow of water into defects in conduits. All soils will erode, if subjected to sufficient concentrated flow, such as might occur in cracks in the earthfill, but plastic clays and clayey coarse-grained soils that are not dispersive resist erosive forces better than silts and cohesionless coarse-grained soils.

- *Cracks in surrounding soil connected to water sources.*—If cracks in surrounding soil connect to water sources, erosion of the crack walls can increase dramatically and lead to catastrophic failure of the embankment dam. This can occur for erosion of materials into the conduit or along the conduit.

- *Existence of differential head.*—The potential for internal erosion or backward erosion piping is directly related to the differential head causing the flow of water, whether the flow is intergranular seepage or flow through cracks in the soil. High gradients increase the likelihood of internal erosion or backward erosion piping. Even if the head in the reservoir is not high, continued flow through cracks in the soil surrounding the conduit is likely to result in excessive erosion of the soil.

- *Type of flow.*—Conduits flowing under pressure are more likely to develop problems associated with conduit defects than nonpressurized conduits. The consequences of the problems that develop related to defects will be greater in pressurized conduits than those associated with defects in conduits that are not pressurized.

- *Backfill able to support a tunnel.*—Water escaping from defects in a conduit may erode surrounding soils. The ability of the soils to support a tunnel will determine the type of problem that develops. Backward erosion piping requires soil to be present that can support a tunnel feature. Otherwise, sinkholes or other types of features may be more likely to be the expression of the erosion.

There are four main potential failure modes involving conduits through embankment dams. These failure modes are discussed in the following sections.

7.1 Failure Mode No. 1: Backward erosion piping or internal erosion of soils into a nonpressurized conduit

For this failure mode, the conduit is surrounded over at least part of its length by soil with a low resistance to backward erosion piping. If the conduit develops a defect from deterioration, or a joint in the conduit becomes open from movement and seepage is occurring through the surrounding embankment, seepage forces may carry soil particles into the conduit. For this failure mode, the conduit is presumed to have an interior pressure lower than the seepage pressures in the surrounding soil. Figures 96, 97, and 98 show conduits with defects where water is entering the conduit. In figures 96 and 97, the defects are separated joints in a conduit. In figure 98, the defect is a poorly constructed joint in a CMP.

If the soil surrounding the conduit defect is resistant to backward erosion piping and the defect in the conduit is small, the time for serious erosion of surrounding soils to develop could be lengthy. Inspections of the conduit should disclose the presence of defects and allow for timely repair before serious problems develop. However, if the reservoir head is high and the defect in the conduit is large enough, the potential for either backward erosion piping or internal erosion is significant. Backward erosion

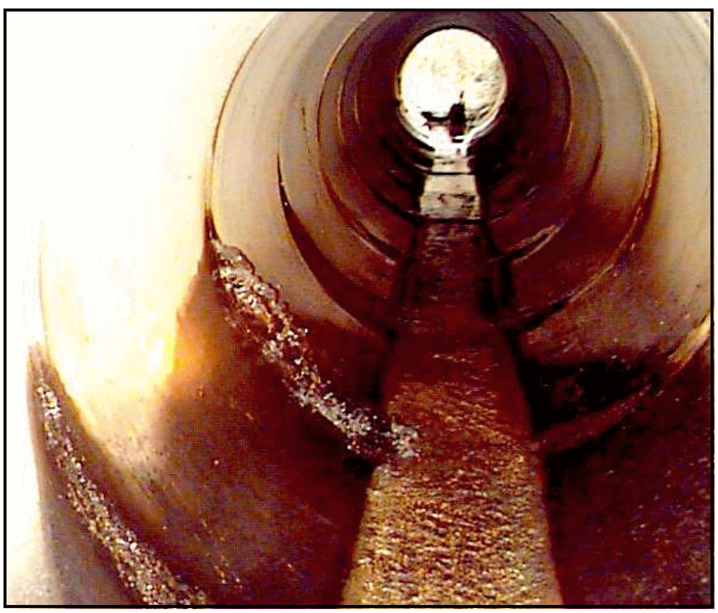


Figure 96.—Leaking joints in a 60-inch diameter RCP spillway. Several large voids were also observed in the adjacent earthfill on the upstream slope.

Figure 97.—Soil particles being carried into an outlet works conduit through a joint.

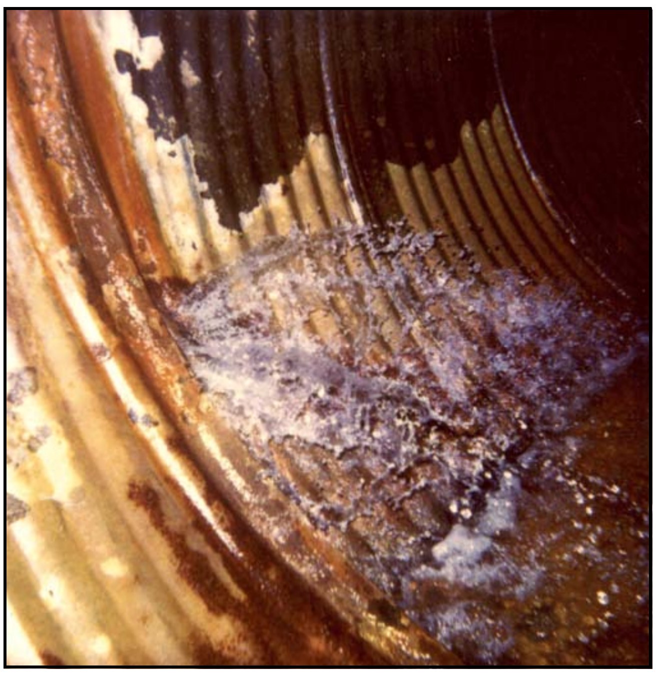


Figure 98.—Leakage from an unauthorized "field joint" constructed by the contractor about 5 feet downstream from the spillway riser structure. An inspection revealed that nearly all of the joints were exhibiting severe leakage and loss of embankment material.

piping could occur from seepage forces surrounding the defect in the conduit if the soils are susceptible. If the defect is large enough and the reservoir head is high enough, the loss of particles from the surrounding soil body caused by backward erosion piping could be severe.

Internal erosion could occur if a preferential flow path (like a hydraulic fracture) develops that is connected to the conduit defect. If internal erosion occurs in soils surrounding a conduit defect in this failure mode, the potential for eventual failure is high, because all soils when subjected to continued flow along a preferential flow path are erosive over time. Highly erodible soils, such as nonplastic silts, broadly graded silty coarse-grained soils, and dispersive clays, could develop erosion features more quickly. The most likely manifestation of erosion in this failure mode is a sinkhole that develops on the embankment surface.

As previously discussed, Failure Mode No. 1 may involve either a backward erosion piping or internal erosion mechanism of particle erosion. The *Introduction* of this document includes extensive discussions of factors that should be evaluated to determine which of these mechanisms is likely for a specific situation.

The sequence in which this failure mode could develop is illustrated in figure 99 and described in the following steps. Note that the following description specifically involves the development of backward erosion piping in a situation where the conduit is surrounded by soils susceptible to this failure mechanism. A similar set of steps could be described for a scenario involving internal erosion rather than backward erosion piping, when the surrounding soils are not susceptible to backward erosion piping. For the sake of brevity, this description of similar steps in an internal erosion scenario is not repeated.

1. As water is impounded in the reservoir, seepage develops through the embankment dam. The time for this to occur varies with the permeability of the embankment zones. A phreatic line develops, and seepage forces are active in the saturated soils around the conduit.

2. Seepage can enter any defects in the conduit, if the conduit has an interior pressure lower than the water in the soil pores. If the seepage discharging into the nonpressurized conduit has sufficient gradient and soils are susceptible to backward erosion piping, soil particles may be carried with the water.

3. Backward erosion piping of the soils in the embankment dam will cause a tunnel to develop for soils that can support a tunnel. If the soils cannot support a tunnel, a sinkhole may occur instead. A failure can occur if the defect in the conduit is large enough to allow most of the reservoir water to escape.

4. If soils between the reservoir and the defect in the conduit are not susceptible to backward erosion piping, and no preferential flow paths occur in the surrounding soils, the defect in the conduit may not result in immediate problems.

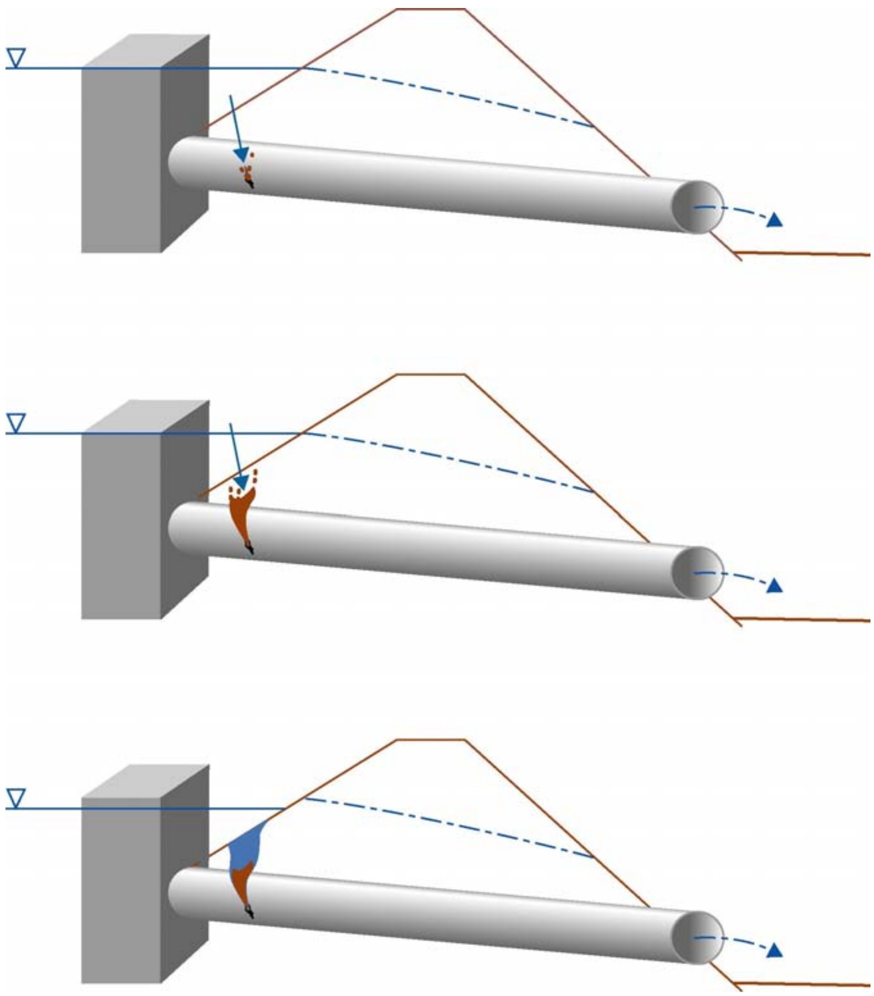


Figure 99.—**Failure Mode No. 1**.—Backward erosion piping or internal erosion of soils surrounding a defect in a nonpressurized conduit.

5. If a preferential flow path develops in the soils surrounding the defect in the conduit, such as a hydraulic fracture in the surrounding soils, then internal erosion will occur as water from the reservoir flows along the crack or other preferential flow path to the defect in the conduit.

6. The extent of the erosion that will occur depends on the velocity of the flow, the erosion resistance of the surrounding soil, the size of the preferential flow path, and the size of the defect in the conduit. The erosion that develops from internal erosion from a preferential flow path may have a similar appearance to that from backward erosion piping.

7. If the tunnel continues to develop from internal erosion and proceeds backwards until it reaches the reservoir, and the defect in the conduit is large enough, a breaching type of failure can occur. If the tunnel erosion does not progress completely until it reaches the reservoir, a complete breaching failure may not occur, but sinkholes may develop that must be repaired.

The *Introduction* of this document includes extensive discussions of factors that should be evaluated to determine whether internal erosion or backward erosion piping is the correct term to describe the mechanism of failure.

This type of failure mode was in progress at Tin Cup Dam (Luehring, Bezanson, and Grant, 1999). Numerous sinkholes developed in an embankment dam, when a masonry tunnel developed defects and the soils adjacent to the conduit were eroded into the conduit. Later, after the conduit was repaired, additional problems developed, as described under Failure Mode No. 2.

This type of failure mode can also occur where conduits have misaligned joints or irregularities in their walls. Joint offsets can cause high negative pressures to develop at overhangs during high velocity flow within the conduit. These offsets can create negative pressures at the offset from a Venturi effect. The negative pressures can pull or "suck" surrounding soils into the conduit through the opening, and voids can develop next to the conduit. Continued loss of surrounding soil could lead to development of a sinkhole, which, if it were to connect with the reservoir, could lead to serious consequences and eventually a disastrous failure of the embankment dam. Theoretically, this failure mechanism would develop as follows:

1. High velocity flow in a conduit with an joint offset or other irregularity in the walls causes a negative pressure to develop downstream of the offset or defect.

2. If a defect in the conduit wall or a joint that has separated occurs near the point of high negative pressure, the soil surrounding the conduit could be pulled into the conduit from the negative pressures, even though the conduit is flowing under pressure.

3. Continued removal of soil near the defect could result in a sinkhole, if it were allowed to continue and could even progress to connect to the reservoir or embankment surface.

This failure mechanism is less likely than the one where water under positive pressure is forced through the defect in the conduit and damages the surrounding soil (Failure Mode No. 2).

7.1.1 Design measures to prevent failure

Preventing this type of failure requires conduits to be properly designed and constructed with durable materials that are unlikely to develop defects. Chapter 2 discusses important design considerations for conduit materials. CMP's are particularly susceptible to this type of failure. Joints in articulated conduits must be designed to accommodate movement to tolerable limits to avoid separation of the joints.

Once soil around the conduit begins to move into a defect in a conduit, either from backward erosion piping or internal erosion, a serious problem exists. Quick action is usually advisable. Sinkholes can develop, and if the defect is large enough, perhaps an embankment dam breach could even develop. The only reliable long term solutions to preventing a failure or accident associated with this failure mode are to repair the defect in the conduit or renovate or replace the damaged section(s) of conduit. Short term remedial measures like grouting seldom are adequate to completely stop the seepage from moving the soil particles. Several options for addressing the defect in the conduit are available, including:

- Sliplining the conduit
- Removal and replacement of the conduit
- Repair of the conduit

Chapters 12, 13, and 14 have more extensive discussion on methods for renovation, removal and replacement, and repair of conduits.

Once a defect develops in a conduit, quick action is needed to prevent serious erosion of the surrounding soils. At Tin Cup Dam, an emergency repair involving sliplining a deteriorated 2- by 3-foot outlet works conduit (masonry pipe) with a 16-inch diameter HDPE pipe was implemented to address sinkholes that had formed above it (Luehring, Bezanson, and Grant 1999, p. 3). The annulus space between the HDPE slipliner and the masonry pipe was grouted, but the grout was later found to have floated the HDPE conduit, and sufficient grout was not injected to fill the annulus space completely. Later inspections showed that cavities were present next to joints in the masonry pipe that were not filled during the grout operations. Additional seepage problems became apparent soon after the repair. Consequently, extensive additional repairs were required the next year. This example illustrates how emergency repairs may avert an immediate threat, but may not be a suitable long term solution. This also illustrates that problems perceived to be associated with a conduit may have additional causes. In the final repair of the embankment dam, evidence was found of construction problems, including use of materials containing roots and other debris. Other poor construction practices and material incompatibility between portions of the embankment and coarse rock fill zones also contributed to problems at the site.

7.2 Failure Mode No. 2: Backward erosion piping or internal erosion of soils by flow from a pressurized conduit

When the conduit is flowing under pressure, the pressure in the conduit can exceed the pressure outside the conduit. If there are defects in the conduit, the high pressure flow can exit the conduit through the defects. The water flowing under pressure begins to exert hydraulic forces on the embankment soils. This could also occur, if a portion of the conduit has collapsed or articulated conduits separate at a joint. Water flowing in the conduit could then flow outward into the surrounding embankment through the defect in the conduit.

Conduits may collapse from deterioration, poor design and construction, and other causes, as discussed in chapter 8. If the conduit were to become blocked by debris, the internal pressure in the conduit could be much higher than the normal pressure at design flow. A conduit designed to flow without pressure may then become pressurized. Designers should consider this possibility. Separation of articulated conduits is discussed in section 4.3.1. The sequence of failure is described as follows and is illustrated in figure 100.

1. Water flowing out of the pressurized conduit begins seeping through the embankment dam and emerges at some exit face. The exit face may be the downstream toe, a downstream shell zone composed of very coarse gradation, or another seepage exit face. If the seepage face is unprotected by a properly designed filter, particles can be dislodged by the seepage water.

2. Seepage forces detach soil particles from the exit face, and backward erosion piping occurs if the soils are susceptible to this mechanism of failure and able to support a tunnel roof.

3. Backward erosion piping progresses backwards until a tunnel connecting the defect in the conduit and the exit face forms. If this backward erosion piping continues, it can lead to a failure of the embankment dam.

4. If the soils surrounding the conduit are resistant to backward erosion piping, the defect in the conduit is small, and the hydraulic force of water in the conduit is low, no immediate problems may occur. Soils not susceptible to backward erosion piping require a concentrated flow path for significant erosion to occur.

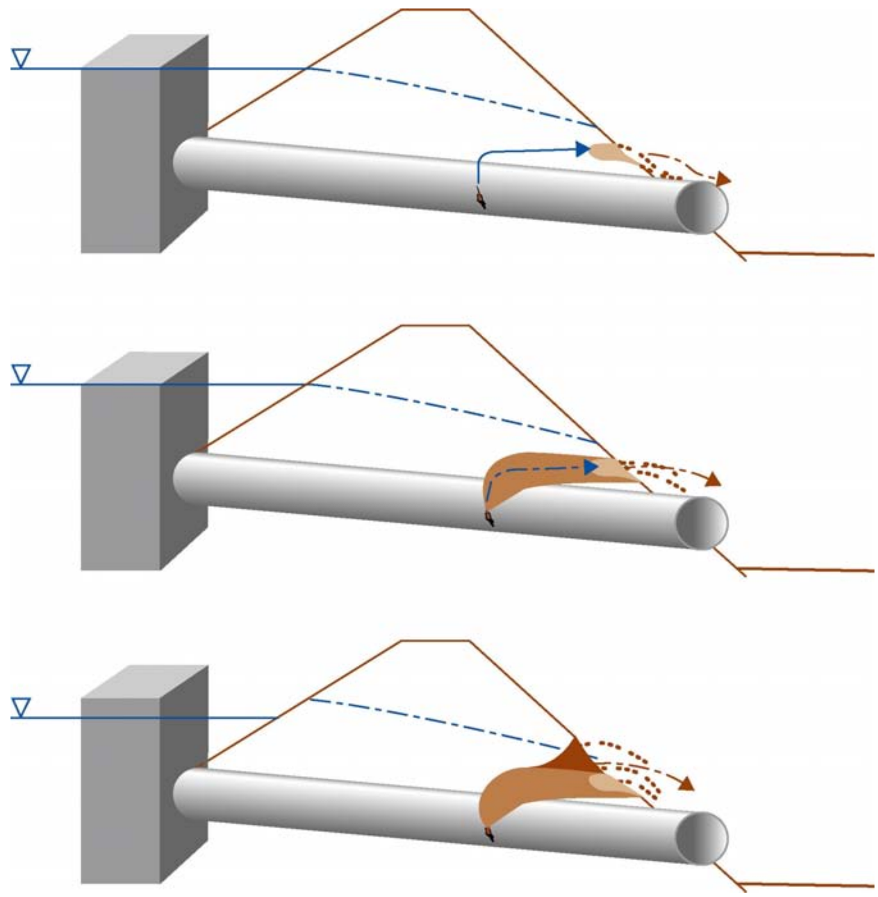


Figure 100.—**Failure Mode No. 2**.—Backward erosion piping or internal erosion of soils surrounding a pressurized conduit with a defect.

5. If soils surrounding the conduit develop a preferential flow path, such as a hydraulic fracture, and the internal conduit pressure is large enough, internal erosion may occur, rather than backward erosion piping. The hydraulic fracture created can erode and lead to development of a failure tunnel that is similar to that which develops in soils that are susceptible to backward erosion piping. If the erosion process continues, it can lead to a breaching failure of the embankment dam.

As previously discussed, Failure Mode No. 2 may involve either a backward erosion piping or internal erosion mechanism of particle erosion. The *Introduction* of this document includes extensive discussions of factors that should be evaluated to determine which of these mechanisms is likely for a specific situation.

An example of this failure mode is the breach of Lawn Lake Dam near Estes Park, Colorado, discussed in more detail in appendix B. A defective seam in the connection between a conduit and valve allowed water under pressure to erode the downstream soils by combination of internal erosion and backward erosion piping, and an embankment dam failure occurred. The failure occurred when lead caulking between the outlet conduit and gate valve deteriorated and allowed water under pressure to erode the embankment dam. The failure of Lake Tansi Dam (Heckel and Sowers, 1995) was also attributed to this type of failure mode.

In some cases, multiple failure modes may be involved at a single site. Tin Cup Dam developed sinkholes associated with Failure Mode No. 1, as described in the previous section, when the masonry outlet conduit collapsed. To address these problems, an HDPE pipe was sliplined in the masonry outlet tunnel and the annulus grouted. Additional sinkholes and distress symptoms related to Failure Mode No. 2 occurred when the downstream control gate that was installed as part of the first repair allowed pressurized flow in the conduit. More extensive repairs were required to address these second series of distress symptoms, including relocating the control gate back to the upstream side of the embankment dam and placing a downstream buttress fill (Luehring, Bezanson, and Grant, 1999, p. 7). Failure Mode No. 3, which is discussed in a following section, was probably also active at this site (see Failure Mode No. 3 for further discussion of Tin Cup Dam).

7.2.1 Design measures to prevent failure

Design measures that eliminate or reduce the possibility of a conduit deteriorating and developing a defect that would allow this failure mode include (1) using conduit materials that are resistant to deterioration, (2) ensuring watertight joints for pressure flow conduits, and (3) designing conduits to resist cracking from applied loads and foundation movements. Chapters 1 through 6 discuss many of these design measures in more detail.

Two general methods might be used to address this type of failure mode once it occurs. They are (1) barrier cutoffs, and (2) filter diaphragms and collars. A barrier cutoff consists of a grouted zone surrounding the conduit or sliplining of the conduit. The grout can be chemical or cementitious grout, depending on the size and shape of the suspected voids in the soil and the nature of the soils. For guidance on grouting around conduits, see section 14.1. Rarely would grouting be considered adequate without also installing an inverted filter over the area where seepage is occurring. An inverted filter is a series of layered filters placed on a soil surface that is discharging seepage. This filter is designed to filter any soil particles being discharged with the seepage and to provide capacity for releasing the collected water. The layers usually consist of a layer of finer sand placed on the ground surface where the seepage is discharging, which is covered by a layer of coarser gravel that is filter compatible with the fine sand. A third layer of small cobbles may overlay the gravel filter. In some cases, a fourth layer of rip rap size rock may be used to armor the filters beneath and protect them from damage. When multiple layers of filters are used to backfill a sinkhole, this system of filters may be placed in reverse order, with the coarser gradations placed in the bottom of the sinkhole, and progressively finer filters used to backfill the sinkhole. The intent of this system is to block additional movement of soils above the sinkhole into the feature. Ultimately, no remedial measure would be considered safe without repairing the conduit, because the hydraulic heads at the discharge point would be excessive for granular filter/drainage zones to control.

Filter diaphragms or collars that are limited in size are seldom sufficient to control this type of failure. Emergency action consisting of placing an inverted filter with rock cover over the discharge point of water or the face of the embankment dam may be appropriate. Rarely should this type of measure be considered a long term solution. If internal erosion rather than backward erosion piping is the cause of the problem, a filter blanket over the discharge area may become plugged, and flow will seek an alternative exit.

7.3 Failure Mode No. 3: Backward erosion piping or internal erosion of soils along the outside of a conduit caused by hydraulic forces from the reservoir

For this failure mode, water flows along the interface between the conduit through an embankment dam and the surrounding soil. This failure mode is usually associated with embankment seepage through the soils surrounding the conduit. The seepage along the interface between the conduit and surrounding soil may be concentrated enough to result in backward erosion piping, if the soils are susceptible. This failure mode is very similar to Failure Mode No. 2. The only difference in these two modes of failure is the source of the water. In Failure Mode No. 2, the source of the water causing internal erosion of the soils is a defect in a conduit. In Failure Mode No. 3, the source of the water is seepage from the reservoir that concentrates at the interface between the conduits and surrounding soil. The sequence of failure is described as follows and is illustrated in figure 101.

1. Seepage forces and concentrated flow develop along the contact between a conduit and surrounding soil.

2. Backward erosion piping can occur if the seepage exits downstream through an unfiltered face or into an overly coarse zone of the embankment dam and the soils surrounding the conduit are susceptible to backward erosion piping. Continued flow can result in the formation of a tunnel connected to the reservoir that will potentially result in a breach of the embankment dam.

3. If soils surrounding the conduit are resistant to backward erosion piping, but cracks or preferential flow paths occur from poor compaction techniques or later develop from hydraulic fracture, continued flow through the preferential

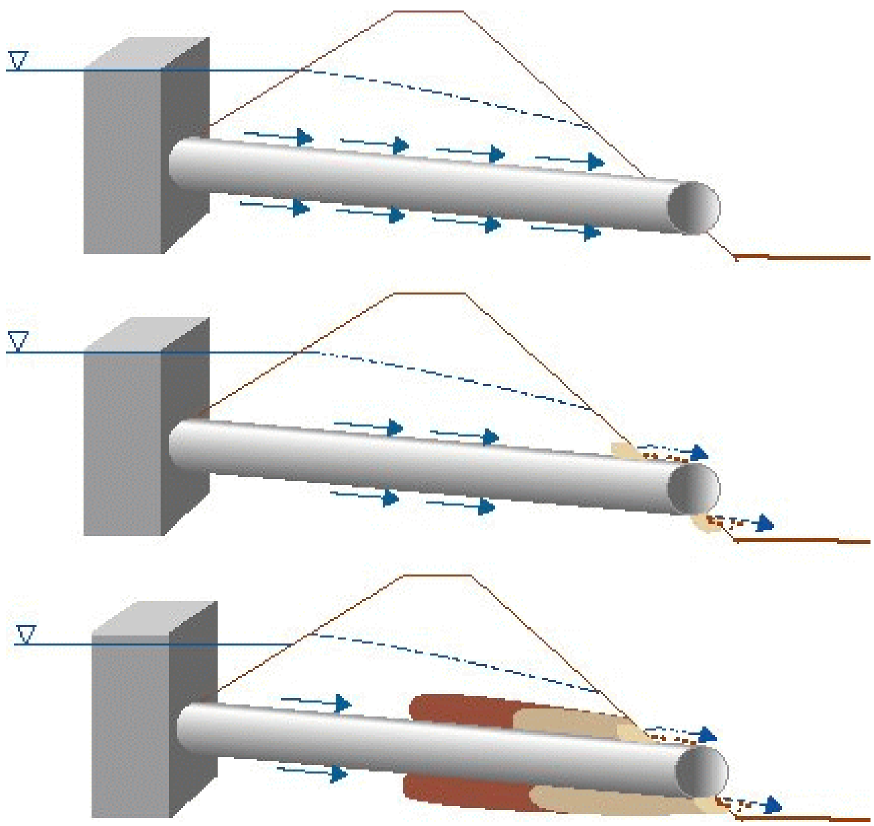


Figure 101.—**Failure Mode No. 3**.—Backward erosion piping or internal erosion of soils along a conduit at the interface between the conduit and surrounding soils.

flow paths will result in internal erosion. An erosion feature similar to that caused by backward erosion piping can then develop.

4. If the erosion process continues, it can lead to a breaching failure of the embankment dam.

As previously discussed, Failure Mode No. 3 may involve either a backward erosion piping or internal erosion mechanism of particle erosion. The *Introduction* of this document includes extensive discussions of factors that should be evaluated to determine which of these mechanisms is likely for a specific situation. Figures 8 and 9 illustrate failure mechanisms resulting from internal erosion along a conduit.

The Tin Cup Dam case history described by Luehring, Bezanson, and Grant (1999) is an example where this failure mode probably contributed to the development of extensive sinkholes and other distress symptoms at an embankment dam. As discussed previously in this chapter, it seems likely that multiple failure modes occurred at Tin Cup Dam, and water flowing along the outside of the conduit from the reservoir, Failure Mode No. 3, was one of them.

Compacting soil adjacent to a conduit is difficult, and compaction efforts can dislodge the conduit and create pathways for future concentrated flow. A cradle is needed, so that soil does not have to be compacted under the haunches of the conduit. This is important to prevent an easy pathway for internal erosion. For guidance on the design and construction of conduits and filters, see chapters 1 through 6.

Often, failures of embankment dams related to water flowing through or under the embankment have been near the conduit location. A natural tendency has been to assume that the pathway for water flow that caused the failure was directly along the conduit, identified as Failure Mode No. 3. This mode of failure appears most likely when soil is not compacted properly under the haunches of circular conduit, and a continuous zone of poorly compacted soil is subject to the hydraulic head of the reservoir. Examples of this type of failure mode are the Loveton Farms and Medford Quarry Dams Wash Water Lake case histories in appendix B.

One of the reasons that antiseep collars were used in embankment dam design was to prevent this mechanism of failure. The fact that many failures occurred even though antiseep collars were installed correctly, and that the collars could be seen to be intact after the failure caused investigators to consider that at least a portion of the flow path may have been away from the interface between the conduit and surrounding soil in some dam failures. Appendix A discusses in detail why antiseep collars have been discontinued as a primary defensive design element on most new embankment dams.

Figure 102 shows a conduit with its antiseep collars intact after an internal erosion failure of the embankment dam. In this case, it appears unlikely that the flow path for the failure was a continuous uninterrupted flow along the conduit, but at least part of the flow path was in the earthfill surrounding the conduit. In most cases, it has not been possible to determine the exact flow path that water followed in internal erosion failures, because the evidence was destroyed by the failure.
If hydraulic fracture and other causes of cracks in compacted backfill were ignored as the potential cause for failures, one might incorrectly assume that all failures that occur in the vicinity of conduits are attributable to flow along the conduit. In Failure Mode No. 4 (discussed in the following section), hydraulic fractures occur in the soil mass beyond the immediate vicinity of the conduit, usually associated with differential settlement in the fill caused either by the conduit or excavations made to install the conduit, or uneven bedrock profiles near the conduit.

Failures of compacted dispersive clay embankment dams, such as those experienced by the NRCS and documented in Sherard (1972), probably involve Failure Mode

Figure 102.—Conduit with intact antiseepage collars. The collars would have interrupted flow along conduit. Internal erosion or backward piping erosion likely occurred through hydraulic fractures in surrounding earthfill, resulting in failure of this embankment dam.

No. 4 more often than Failure Mode No. 3, where water is assumed to flow directly along the conduit. The reason for this conclusion is the known high quality of the compaction effort used to place low permeability clays around the conduits of these structures, plus several eyewitness accounts where the flow path was known to be as much as 15 feet above and to the sides of the conduit. The known cause of those failures was hydraulic fracture of the embankment dam, not always immediately in the vicinity of the conduit.

7.3.1 Design measures to prevent failure

A filter diaphragm or collar surrounding the conduit is the currently accepted method used to prevent this type of failure mode. Filter diaphragms and collars are discussed in detail in chapter 6. In summary, Failure Mode No. 3 refers to the condition where the predicted flow path for backward erosion piping or internal erosion is directly along the interface between the conduit and surrounding earthfill. Failure Mode No. 4, discussed in the following section, covers situations where the pathway for the erosion of the earthfill is a significant distance away from the interface of the conduit and embankment. Filter diaphragms or collars may need to be significantly larger to protect against Failure Mode No. 4 than are needed to protect against Failure Mode No. 3.

7.4 Failure Mode No. 4: Internal erosion of hydraulic fracture cracks in the earthfill above, below, or adjacent to the conduit

Conduits are one of the primary causes of differential settlement of an embankment dam that can result in hydraulic fracture of the embankment in the vicinity of the conduit. When an earthfill experiences hydraulic fracture, a pathway is created along which water from the reservoir can flow easily and erode the soil in contact with the crack.

Failure Mode No. 4 is one where hydraulic fracture of the embankment dam in the vicinity of a conduit is attributable to the differential settlement caused by the conduit, and flow through the crack erodes the embankment to the point where a breaching type failure occurs. Hydraulic fracture of earthfill is discussed extensively in section 5.2. This failure mode differs from Failure Mode No. 3, since the seepage path forms at a location away from the soil-conduit interface.

These kinds of failures are most common when a reservoir fills suddenly shortly after completion of the embankment dam, and the earthfill is highly erodible. The sides of cracks may erode very quickly when water from the reservoir flows through the crack. The eroded failure path can enlarge to a size that can empty a reservoir rapidly.

If a crack is not intercepted with a filter zone, an embankment dam failure can result when the crack enlarges from erosion. Even high plasticity clays that are not dispersive can erode over time. The sequence of failure for Failure Mode No. 4 is described as follows and is illustrated in figure 103.

1. After a crack forms in the soils surrounding the conduit, if the embankment soils are highly erodible, the crack rapidly enlarges from erosion of the sidewalls of the crack. The water discharging at the downstream face of the embankment dam is muddy, and a vortex may form at the entry point on the upstream slope.

2. The erosion tunnel enlarges to the point that the reservoir is emptied and the breaching process is completed.

3. A tunnel-shaped hole will exist after the failure, if the eroded tunnel is narrow enough to support the roof of the tunnel. If the tunnel collapses from erosion and widening caused by a lack of support for the roof, the failure will have the appearance of an open breach in the embankment dam.

4. As previously discussed, Failure Mode No. 4 almost always involves the mechanism of internal erosion, and very rarely can backward erosion piping be correctly attributed as the cause of such failure.

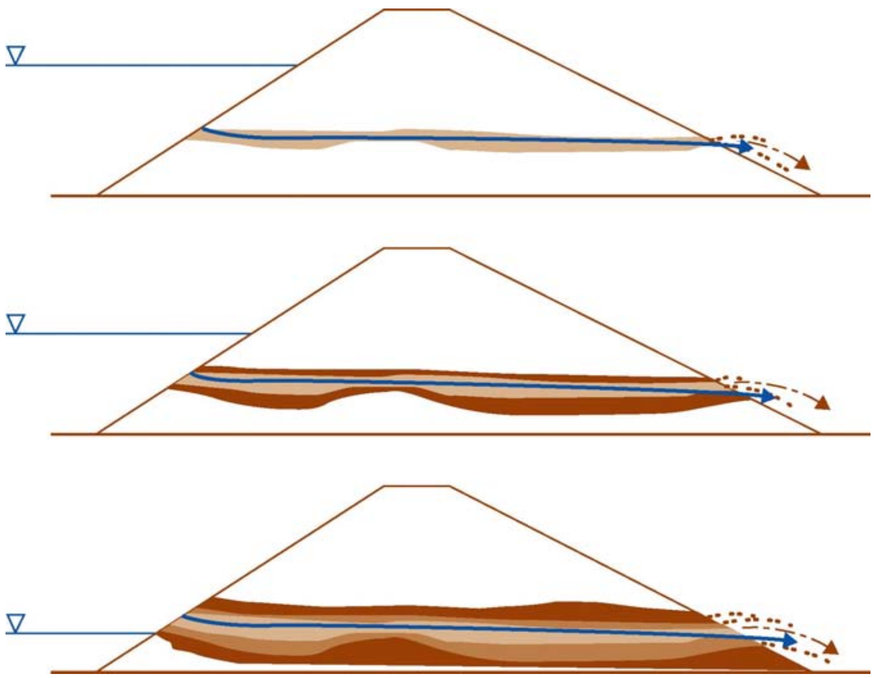


Figure 103.—**Failure Mode No. 4**.—Internal erosion of the earthfill above and on either side of a conduit caused by concentrated flow in a hydraulic fracture or other preferential flow path in the compacted earthfill. Hydraulic fracture cracks in the embankment dam may result from differential settlement caused by the presence of a conduit within the earthfill.

5. This type of failure occurs most frequently on first filling of the reservoir so that intergranular seepage rarely has had time to develop. One of the requirements for backward erosion piping to be defined in the context of this document is that it results from intergranular seepage, which justifies this conclusion.

6. Another reason that backward erosion piping is seldom the cause for failures in the earthfill above a conduit is that most embankment dams are not constructed of soils susceptible to backward erosion piping without proper design features to prevent the backward erosion piping.

As previously discussed, Failure Mode No. 4 almost always involves the mechanism of internal erosion, and very rarely can backward erosion piping be correctly attributed as the cause of such failure. This type of failure occurs most frequently on first filling of the reservoir, so that intergranular seepage rarely has had time to develop. One of the requirements for backward erosion piping to be defined in the context of this document is that it results from intergranular seepage, which justifies this conclusion. Another reason that backward erosion piping is seldom the cause for failures in the earthfill above a conduit is that most embankment dams are not constructed of soils susceptible to backward erosion piping without proper design features to prevent the backward erosion piping. The *Introduction* of this document contains extensive discussions of factors that are important in distinguishing between internal erosion and backward erosion piping mechanisms of particle erosion. Figure 7 illustrates the sequence in development of Failure Mode 4.

Figure 104 shows a small embankment dam that failed by internal erosion. The conduit created differential settlement in soils above the conduit that resulted in hydraulic fracture. The embankment soils were highly dispersive clays.

If the erosion tunnel widens enough, the tunnel can collapse, and a tunnel-shaped failure surface is not observed after the failure. The failure is simply a breach in the embankment dam.

The near failure of the USACE's Wister Dam is a good example of this scenario of internal erosion. The failure of the embankment dam during first filling was narrowly averted by quickly lowering the pool and employing other intervention measures. The embankment dam was constructed of highly dispersive clays without a chimney filter. The problems occurred in a closure section of the embankment dam. See the Wister Dam case history in appendix B. Sherard (1986, p. 911) provides further details on this interesting case history.

Another example of internal erosion resulting from the existence of hydraulic fracture cracks within an embankment dam is the Upper Red Rock Site 20 Dam. See appendix B for a detailed discussion of this case history.

Before the NRCS gained an understanding of the behavior of dispersive clay soils, over 15 embankment dams constructed of dispersive clays failed. Most of the failures occurred near the conduits through the embankment dam. The conduits contributed to differential settlement, which led to hydraulic fracturing (Sherard, 1972; Sherard, Decker, and Ryker, 1972a). Another example of this type of failure is the Anita Dam case history in appendix B. Investigators attributed one possible cause for the formation of a flow path for water to be freezing and thawing of soils adjacent to the conduit. Hydraulic fracture could also have contributed to the failure.

7.4.1 Design measures to prevent failure

Several design measures are available in preventing this type of failure mode from developing.

The first design measure involves reducing the potential for cracking and internal erosion of the fill. The mechanism responsible for this type of failure mode is

Figure 104.—Failure of an embankment dam due to internal erosion of hydraulic fracture cracks upon first filling of the reservoir.

hydraulic fracture. Hydraulic fracture is discussed in section 5.2. Dispersive clays are the most susceptible to this failure mode, and special attention should be given to testing for the presence of dispersive clays in all embankment dams. See section 5.3.3 for additional discussion of dispersive clays.

The second measure involves constructing a properly designed zone of filter material around the conduit to intercept cracks that develop from hydraulic fracture. Filter diaphragms, filter collars, or embankment chimney filter zones are common design elements. Most high and significant hazard dams will have as part of their design a full chimney filter. Low hazard embankment dams constructed of nondispersive soils may only include a filter diaphragm or filter collar. Filter zones are discussed in more detail in chapter 6.

A third measure to address the potential for internal erosion failures in embankment dams is the use of additives incorporated into the fill to reduce erosivity. Lime treatment has been used to reduce the erosivity of dispersive clays, but its cost is seldom justified, except in critical parts of the fill, such as the contact between the central core and bedrock. The case history on the Piketberg Dam in South Africa is discussed in appendix B, and it showed that the addition of gypsum to treat the dispersive clays in the core of the embankment dam may not have been completely effective. For guidance on soil amendments, see section 5.3.3. Usually, relying on a filter zone is considered more positive than using soil amendments.

Chapter 8

Potential Defects Associated with Conduits

Defects associated with conduits can lead to the development of potential failure modes. If corrective action is not taken to repair the damage resulting from the defect, this can lead to a failure of the embankment dam. For a further discussion of potential failure modes associated with conduits, see chapter 7. For guidance on the renovation, removal and replacement, and repair of conduits, see chapters 12, 13, and 14.

Various materials have been used in the construction of conduits, such as concrete, plastic, and metal. Each conduit material reacts differently in embankment dam applications. A search of the USACE's Waterways and Experiment Station damage and repair data base indicated that the most common defect requiring repair in concrete conduits was leakage through cracks and joints (USACE, 1988, p. 96).

This chapter will discuss some of the most common types of defects associated with conduits. Periodic inspection of the conduit by man-entry or CCTV inspection is the only reliable method to detect the extent of damage. For guidance on inspection, see chapter 9.

8.1 Deterioration

Often, if deterioration is left unchecked, it will continue and progressively worsen. If repairs are promptly made, the conduit may be able to continue to function in a serviceable fashion. However, if deterioration is allowed to progress, there may come a time when a significant portion of the conduit must be entirely replaced. Action for timely repair may be more cost effective than postponing repairs and eventually having to replace major portions of the conduit.

8.1.1 Abrasion

Abrasion in conduits is an erosional process and is a function of velocity and turbulence in the flow, the hardness of the abrasive material, and the quality of the surface experiencing abrasion. Abrasion is caused by water flowing through a conduit at high velocities and containing silts, sands, gravels, or stones (figure 105).

Figure 105.—Abrasion/erosion damage to concrete from flowing water containing sand and silt.

This flow causes a scouring or grinding effect on the exposed surface. Most conduits do not carry significant amounts of abrasive materials in the flow. However, conduits used for diversion during construction or for reservoir sediment release are especially vulnerable. Increases in the velocity of the flow can increase the abrasive power.

In concrete conduits, abrasive damage has been experienced in concrete with low strength and poor quality aggregates. Abrasive flow usually erodes the cement mortar mix matrix, leaving an exposed, polished, and coarse aggregate surface. As the abrasion process continues, the concrete may be eroded down to the reinforcement. The extent of damage depends on the flow duration and velocity, concrete quality, and compressive strength. In concrete conduits, abrasion is generally not a factor when velocities are less than 15 ft/s. In metal conduits, abrasive flow can erode protective linings and coatings and expose the surface to corrosion.

Once damage from abrasion has begun, it will accelerate with each operation of the conduit, unless the source of the abrasive materials is removed. Cavitation may also be triggered by the abrasion damage (by creating a flow surface irregularity) and greatly increase the rate of destruction.

Polyethylene plastic pipe has been found to be very abrasion resistant. However, high velocity flow containing abrasive materials can still be problematic for any type of pipe.

8.1.2 Aging

The aging process can also cause deterioration in conduits. In concrete, properties change over time and eventually affect the integrity of the structure (Pinto, 1994, p. 1111). Both the quality of concrete (e.g., porosity), and physical and chemical factors influence the rate of concrete deterioration. Processes that can weaken concrete include:

- *Freezing and thawing*.—Repeated cycles of freezing and thawing can affect the durability of concrete. Concrete readily absorbs water and is vulnerable to damage, if the water within its system of pores can freeze and generate disruptive pressures. If the pores existing in the concrete are inadequate in size and number to accommodate the greater volume occupied by the ice, the concrete will fracture. The rate of progression of the freezing and thawing deterioration will depend upon the number of cycles, the degree of saturation during freezing, the porosity of the concrete, and the exposure conditions. Concrete experiencing damage by freezing and thawing is characterized by a disintegrated appearance. Deterioration due to freezing and thawing is especially severe in the northern and mountain zones of the United States. Deterioration from freezing and thawing progresses from the exterior surface to the concrete inward. As the concrete on the surface fails and is removed by spalling, the depth of freezing progresses inward (Reclamation, 2003, p. 7). Freezing and thawing typically is not a significant concern for conduits, since most of the conduit is submerged or has limited exposure. However, freezing and thawing can become a problem for entrance and terminal structures. Figure 106 shows a concrete intake structure that has been exposed to repeated cycles of freezing and thawing. In new construction, the entrainment of small bubbles of air into fresh concrete has been found to provide relief for pressures developed by free water as it freezes and expands. Repairs to existing structures require replacement concrete or epoxy-bonded concrete (Reclamation, 1997, p. 26).

Figure 106.—Concrete deterioration from freezing and thawing.

- *Alkali-aggregate reaction.*—Alkali-aggregate reaction (AAR) occurs when certain types of sand and aggregate (e.g., opal, chert, flint, or volcanic material with a high silica content) are exposed to sodium and potassium hydroxide alkalies in portland cement. In a moist environment, a gel is formed around the reactive aggregate, creating tension cracks around the aggregate and extensive expansion and fracturing of the concrete. This expansion, cracking, and loss of concrete strength can lead to pathways for seepage or localized collapse of the conduit. Concrete containing alkali-reactive aggregate may show immediate expansion and deterioration, or it may remain undisturbed for many years. Concrete experiencing AAR is characterized by pattern cracking on the surface. Figure 107 shows a concrete wall that has experienced AAR. In new construction, aggregate sources containing negligible potentially alkali-reactive materials, low alkali cements, and pozzolan replacement of a portion of cement, should be used. When abundant potentially alkali-reactive materials are available, low alkali portland cements and fly ash pozzolan have been found to eliminate or greatly reduce the deterioration of reactive aggregates. There is no proven method for eliminating AAR in existing structures (Reclamation, 1997, p. 6).

- *Sulfate attack.*—Sodium, magnesium, and calcium sulfates existing in soils and groundwaters react chemically with the hydrated lime and hydrated aluminate in the cement paste in concrete. The volume of the reaction byproducts is greater than the volume of the cement paste from which they are formed, resulting in disruption of the concrete from expansion. Concrete experiencing sulfate attack is characterized by a disintegrated appearance. In new construction, a sulfate resistant portland cement or a combination of suitable cement and pozzolan should be specified, when it is recognized that concrete will be exposed to soil and groundwater with sulfates. The application of a thin polymer concrete overlay or sealing compounds may be beneficial for existing structures experiencing sulfate attack. Otherwise, removal and replacement of concrete with a sulfate resistant cement should be considered (Reclamation, 1997, p. 23).

Polyethylene plastic pipe, if exposed to ultraviolet (UV) radiation and oxygen, can experience degradation affecting the physical and mechanical properties of the pipe. Ultraviolet light is present in sunlight. Typical applications using polyethylene pipe involve sliplining of existing conduits. In this type of application, exposure to UV light is limited. Any exposed surfaces would require long term UV protection. This protection is provided by compounding 2 to 3 percent carbon black into the material, which prevents UV penetration.

8.1.3 Cavitation

Cavitation is an erosional process and often causes deterioration in concrete, plastic, and metal conduits with high heads, where high velocity vortices are formed. The

Figure 107.—Concrete deterioration from alkali-aggregate reaction.

risk of cavitation can be evaluated by computing the cavitation index for flow, which is a function of velocity and pressure. Normally, for flow velocities less than 40 ft/s, cavitation will be minimal. Discontinuities or irregularities on flow surfaces and/or misalignments in conduits carrying high velocity flow can induce cavitation. These discontinuities, irregularities, or misalignments cause the flowing water to separate from the conduit surface, resulting in negative pressure zones and bubbles of water vapor. When these bubbles travel downstream and collapse next to the conduit surface, the high pressure impact removes small particles of the conduit surface (pitting). As the pitting continues, a progressively deepening cavity develops, which causes additional irregularities that leads to even larger cavities farther downstream (also known as a Christmas tree pattern). Cavitation is common just downstream of mechanical control equipment, such as gates or valves (figure 108) where pressure flow changes to free flow. Damage from cavitation and abrasion can appear to be similar. Cavitation damage appears as a plucking out of the surface material with no fine scale evidence of flow direction. Abrasion damage is normally flow directional.

The use of aeration devices (e.g., ramps and/or slots) installed along flow surfaces in modern structures has been found to be an effective method for preventing cavitation damage. All new structures should include aeration devices, and existing structures that have experienced cavitation damage can be retrofitted to include these. However, the most effective solution is to eliminate the source of the cavitation, rather than attempting to minimize the resulting damage. For further guidance on cavitation, see Reclamation's *Cavitation in Chutes and Spillways* (1990a).

Figure 108.—Cavitation damage to the cast-iron lining of a conduit immediately downstream of a slide gate, caused by high velocity flow.

8.1.4 Corrosion of metals

Corrosion of metals is a complex phenomenon involving many inherent structural and environmental factors. Corrosion is commonly a result of contact between dissimilar metals, or when metals are in contact with water, moist earthfill, or the atmosphere. Corrosion affects all types of metal and alloy pipe and reinforcing bars in concrete. Corrosion is the destructive attack on conduit materials by electrochemical reaction to the environment. Corrosion can also be described as the process whereby metals return to their natural state. Certain metals, such as platinum, gold, silver, and copper exist in nature in a stable metallic state. However, other metals require refinement by heating. Unless these refined metals are protected from the environment, they will eventually revert from their temporary refined metallic state back to a more natural state. The soil and water surrounding the conduit, and water flowing through the conduit can affect the rate of corrosion. The soil and water can contain different types of acids, alkalis, dissolved salts, organics, industrial wastes, mine drainage, etc. The rate of corrosion will vary, depending on chemical and physical properties and exposure to the environment. Factors that influence corrosion include (American Iron and Steel Institute, 1994):

- *Soil resistivity*.—Corrosion involves the flow of current from one location to another. The ability of soils surrounding conduits to conduct electrical particles can affect their tendency to corrode a conduit. Resistivity is a measure of the resistance to current flow of a material, usually expressed in units of ohm-cm. Conduits surrounded by clay soils with typical resistivity values of 750 to

2,000 ohm/cm will be more likely to corrode than conduits surrounded by sands that typically have resistance values of 30,000 to 50,000 ohm/cm.

- *Acidity (pH)*.—Most soils fall into a pH range of 6 to 8, which is neutral. Water and soils with lower pH values are acidic and can result in a more corrosive environment.

- *Moisture content*.—Soils that drain rapidly are less corrosive than soils that tend to hold water longer. Soils with high clay content are typically more corrosive than sandy soils.

- *Soluble salts*.—Salts that become ionized can decrease the resistivity of a soil.

- *Oxygen content*.—Increasing levels of dissolved oxygen can accelerate corrosion.

The process of corrosion can proceed either uniformly or in pitting of the surface. Uniform corrosion is where corrosion occurs evenly over the surface, resulting in a low rate of corrosion. Pitting corrosion is not uniform and is focused only on a small surface area, resulting in a high rate of corrosion, until a perforation eventually develops. Pitting can begin on surface imperfections, scratches, or surface deposits. Between pH 5 and 9, pitting is likely to occur, if no protective film is present.

In the past, CMP has been a commonly used material for conduits through embankment dams. Thousands of embankment dams in the United States and all over the world have CMP conduits installed in them. Corrosion is a common problem with CMP conduits (figure 109). Many State highway departments have made extensive studies on the use and durability of CMP for culverts under highway embankments. However, available information on the use of CMP for conduits through embankment dams is limited. A study of 50 existing CMP conduits in watershed dams located in the Midwestern United States was done in 1989 (Koelliker and Lin, 1990). The study determined that the estimated average life of the sampled CMP conduits was 43 years, but the lifespan ranged from 24 to 72 years. This study also found that leakage and associated corrosion at pipe joints was most often the primary limiting aspect of life expectancy. Many spillway conduit systems constructed with CMP experience corrosion at the joint connection between the conduit section and the riser (the vertical pipe or inlet that connects to the outlet pipe). Spillway risers are subject to deformation and movement (tilting) caused by ice loadings or erosion, which can open the joint connection with the outlet pipe. The riser itself is also susceptible to corrosion.

The most susceptible portion of a CMP to corrosion is the invert, since it is exposed to the flow of water for the longest length of time. CMPs that have inverts with sags could trap water and further increase the potential for corrosion. Corrosion of

Figure 109.—Corrosion has completely destroyed this CMP spillway conduit. Backfill materials that surrounded the conduit have been eroded by flow within the conduit.

CMPs generally consists of two types: soil side or water side. Most metal loss associated with corrosion occurs on the interior or water side of the pipe. Soil side corrosion is not usually a significant factor in conduit life. In the presence of oxygen and water, metal corrodes through an oxidative process that involves the formation and release of metallic ions. The water acts as an electrolyte to carry these ions, which form the basis for the corrosion of the CMP. The reaction of the metal with the dissolved oxygen in the water causes the deterioration most visible on the water side of the conduit (Federal Highway Administration [FHWA], 1991, p. 4). CMPs are subject to electrolytic corrosion due to galvanic action between the metal and the surrounding soil, groundwater, and water flowing through the conduit (Kula, Zamensky, and King, 2000, p. 2). The galvanic action results in corrosion of the CMP and a gradual decrease in wall thickness and structural integrity. Over time, corrosion of the CMP will result in the reduction of wall thickness, formation of pipe perforations, and the eventual collapse of the CMP.

The service life of the CMP is affected by its metallic makeup, coatings, linings, pH and resistivity of the backfill and water, moisture content of the backfill, and abrasion from material particles in the flow. Pipe manufacturers have applied coatings and linings to CMP to mitigate corrosion and extend the service life. CMP coatings have included metallic coatings (zinc [galvanized] and aluminum), and nonmetallic coatings (bituminous [asphalt], cement, and polymers). CMP linings have included asphalt and concrete. The natural scaling tendencies inherent in some waters provide additional protection. Scaling is the deposit and adherence of insoluble products on the surface of the CMP, which isolate it from the water and protect it from corrosion. The factor that most affects corrosion and scale formation in the CMP are the chemicals dissolved in or transported by the natural water. All coatings and linings have some minor flaws (holidays). Corrosion tends to concentrate at these flaws, since water can seep between the coating or lining and the base metal moisture can become trapped, increasing the rate of corrosion. Thus, it may be possible for a coated CMP to become deteriorated in less time than an uncoated CMP in the same environment. Coatings applied to existing surfaces of conduits are generally not very effective due to difficulties involved in obtaining a good bond with the conduit surface.

For guidance on estimating the service life of CMP conduits, see the National Corrugated Steel Pipe Association's *CSP Durability Guide* (2000) and FHWA's, *Durability of Special Coatings for Corrugated Steel Pipe* (1991). Caution should be exercised in attempting to determine the service life of CMP used in conduits through embankment dams. Many of these CMPs may have used no corrosion protection, and many of the coatings and linings available today were not available when the embankment dams were originally constructed. Prior to 1950, galvanized steel was the only metallic coating available for CMP. If a CMP has experienced denting during installation, this could result in corrosion in areas where the protective coating has been damaged. Figures 110 through 113 show the results of corrosion affecting CMP outlet works conduits.

Corrosion of reinforcement in concrete conduits can also occur when it becomes exposed. Reinforcement can become exposed by cracking or spalling of the concrete (figure 114), inadequate cover, or porous concrete. When reinforcement is exposed to corrosive elements, the iron oxides formed expand (requiring more space within the concrete than the original reinforcement), resulting in tensile stresses within the surrounding concrete. These tensile stresses cause cracking and delamination of the concrete. Rust stains on the conduit surface may be an indicator of reinforcement corrosion.

Cathodic protection attempts to retard electrochemical corrosion through the application of reverse direct current to the protected metal and to another metal which acts as a sacrificial anode. This sacrificial anode, typically consisting of either zinc, magnesium, graphite, or aluminum alloys, must be periodically replaced. New concrete installations in hostile corrosive environments should place special attention on crack widths and concrete cover, as well as consider the use of protective coatings, before considering this often problematic and costly means of protecting against steel corrosion. Galvanic reaction between dissimilar metals can also result in corrosion. This can occur when a galvanic reaction forms between reinforcing steel and stainless steel outlet works components.

Another form of corrosion is bacterial corrosion (Patenaude, 1984) caused by anaerobic sulfate-reducing bacteria. Bacterial corrosion typically has been found to occur on galvanized steel pipe. This type of corrosion can exist in two environments

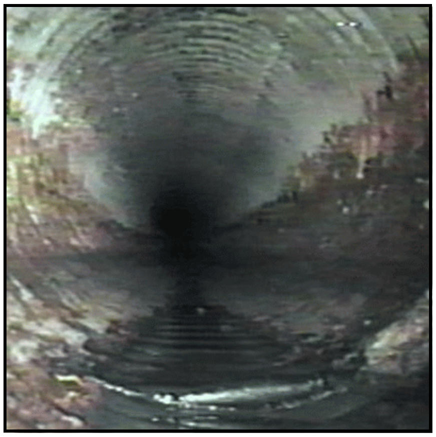


Figure 110.—CMP corrosion within an outlet works conduit.

Figure 111.—CMP corrosion on the invert of an outlet works conduit.

Figure 112.—CMP corrosion within an outlet works conduit caused by a leaking pipe joint.

Figure 113.—An outlet works conduit that has experienced corrosion and failure.

Figure 114.—Spalled concrete and exposed reinforcement in an outlet works conduit.

and have differing products of corrosion. Soil-related bacterial corrosion produces oxidation scale, which is active in organic, poorly drained soils of nearly neutral pH. This scale is usually black, but upon being exposed to aerated conditions in conduits, becomes rust colored. Water-related bacterial corrosion produces nodular oxidation, which exists on pipe surfaces associated with a water source of nutrients. Local perforations on the pipe invert characterize nodular oxidation. Nodular oxidation results from sulfate-reducing bacteria activity.

Polyethlene plastic pipe is not subject to galvanic action and will not corrode. Naturally occurring water and soil conditions will not affect the pipe.

8.2 Poor design and construction

Good design and construction practice can extend the service life of a conduit. However, poor design and construction practice can greatly shorten it. Much of the following discussion was adapted from USACE's *Evaluation and Repair of Concrete Structures* (1995b, pp. 3-1 to 3-14) for reinforced cast-in-place concrete. Some of the most common areas where poor design and construction practice can affect conduits are:

- *Poor design practice.*—Design errors may be divided into two general types: those resulting from inadequate structural design and those resulting from lack of attention to relatively minor design details. Common design errors include:

    1. *Inadequate structural design.*—Inadequate structural design exposes the concrete to greater stress than it is capable of carrying, or greater strain than its strain capacity. This may result in excessively high compressive stresses and appear as spalling. Similarly, high torsion or shear stresses may also result in spalling or cracking. Also, high tensile stresses will result in cracking. To prevent this from occurring, the designer must complete a thorough and careful review of all design calculations. Any renovation that makes use of existing conduit must be carefully reviewed.

    2. *Poor design details.*—While a conduit may be adequately designed to meet loadings and other overall requirements, poor detailing may result in localized concentrations of high stresses in otherwise satisfactory concrete. These high stresses may result in cracking that allows water to access the interior of the concrete. In general, poor detailing does not lead directly to concrete failure; rather, it contributes to the action of one of the other causes of concrete deterioration described in this chapter. A frequent cause of cracking in conduits is improperly spaced joints. Thermal cracking can also result in conduits where joint spacings are too long or are not provided in the conduit to accommodate for changes of length. In general, all of these problems can be prevented by a thorough and careful

review of plans and specifications for the project. In the case of existing conduits, problems resulting from poor detailing should be handled by correcting the detailing and not by simply responding to the symptoms.

- *Poor construction practice*.—Not following specified procedures and techniques may result in construction errors. While individually these errors may not lead directly to failure, when grouped together they could lead to the development of defects that could adversely affect a conduit's integrity. Construction errors can occur during new construction, renovation, and repairs. In concrete, cracking and spalling can be a symptom of poor construction practice. Common construction errors include:

    1. *Improperly located reinforcement*.—Reinforcement that is improperly located or is not adequately secured in the proper location may lead to two general types of problems. First, the reinforcement may not function structurally as intended, resulting in structural cracking or failure. The second type of problem stemming from improperly located or tied reinforcement is one of durability. This involves reinforcement that is improperly located near the surface of the concrete. As the concrete cover over the reinforcement is reduced by wear, it is much easier for corrosion to begin.

    2. *Improper alignment of formwork*.—Improper alignment of the formwork leads to discontinuities on the surface of the concrete. This occurrence is critical in areas that are subjected to high velocity flow of water, such as where cavitation-erosion may be induced.

    3. *Adding water to concrete*.—Water is usually added to concrete at the delivery truck to increase slump and decrease emplacement effort. This practice generally leads to concrete with lowered strength and reduced durability. As the water/cement ratio of the concrete increases, the strength and durability decreases.

    4. *Improper consolidation*.—Improper consolidation of concrete may result in a variety of defects, the most common being surface air voids (also known as bugholes), honeycombing, and cold joints. Surface air voids are formed when small pockets of air or water are trapped against the forms. A change in the mixture to make it less "sticky" or the use of small vibrators worked near the form has been used to help eliminate surface air voids. Honeycombing can be reduced by inserting the vibrator more frequently, inserting the vibrator as closely as possible to the form face without touching the form, and slower withdrawal of the vibrator. Obviously, any or all of these defects make it much easier for any damage-causing mechanism to initiate deterioration of the concrete. Frequently, a fear of "overconsolidation" is used to justify a lack of effort in consolidating

concrete. Overconsolidation is usually defined as a situation in which the consolidation effort causes all of the coarse aggregate to settle to the bottom while the paste rises to the surface. If this situation occurs, it is reasonable to conclude that there is a problem of a poorly proportioned concrete rather than too much consolidation.

5. *Movement of formwork*.—Movement of formwork during the period while the concrete is going from a fluid to a rigid material may induce cracking and separation within the concrete. Cracks open to the surface allow access of water to the interior of the concrete. An internal void may give rise to corrosion problems if the void becomes saturated.

6. *Settling of the subgrade*.—Poor foundation support can impart tensile stresses, resulting in cracking of the concrete conduit. This often occurs during the period after the concrete begins to become rigid, but before it gains enough strength to support its own weight; cracking may also occur.

7. *Settling of the concrete*.—During the period between placing and initial setting of the concrete, the heavier components of the concrete settle under the influence of gravity. This situation may be aggravated by the use of highly fluid concretes. If any restraint tends to prevent this settling, cracking or separations may result. These cracks or separations may also develop problems of corrosion, if saturated.

8. *Vibration of freshly placed concrete*.— Most construction sites are subjected to vibration from various sources, such as blasting and from the operation of construction equipment. Freshly placed concrete is vulnerable to weakening of its properties if subjected to forces that disrupt the concrete matrix during setting.

9. *Premature removal of shores or reshores*.—If shores or reshores are removed too soon, the concrete affected may become overstressed and cracked. In extreme cases, there may be major failures.

10. *Improper curing*.—Curing is probably the most abused aspect of the concrete construction process. Unless concrete is given adequate time to cure at a proper humidity and temperature, it will not develop the characteristics that are expected and that are necessary to provide durability. Symptoms of improperly cured concrete can include various types of cracking and surface disintegration. In extreme cases, where poor curing leads to failure to achieve anticipated concrete strengths, structural cracking may occur.

Figure 115 shows an example of poor construction practice (improper consolidation of concrete). For an example of how poor design and construction practice can lead to the failure of a concrete conduit, see the Olufson Dam case history in appendix B.

Poor design and construction practices particular to reinforced cast-in-place conduits were discussed in the previous paragraphs. However, poor design and construction practices affect all types of conduits. The following paragraphs briefly discuss effects of poor design and construction practices affecting other types of conduits, such as precast concrete, or CMP. The appearance of these defects can lead to preferential seepage paths and the development of potential failure modes for conduits. Some of these include:

- *Deformation.*—Deformation occurs when load or force changes the shape of the conduit. Deformation is typically caused by the application of excessive external load (e.g., improper selection of design loadings), loads from heavy construction equipment, or seismic activity. Figure 116 shows an example of where heavy construction equipment likely caused deformation of a CMP conduit. CMP is flexible and is designed to deform some as it transfers load into the surrounding backfill. The surrounding backfill provides stiffness and load carrying capacity. Improperly designed backfill or inadequately compacted backfill under the CMP haunches does not provide the needed lateral stiffness to the CMP. This can result in excessive deformations and structural failure or collapse of the CMP (Kula, Zamensky, and King, 2000, p. 3).

- *Differential settlement.*—Differential settlement occurs when the embankment materials next to the conduit are improperly or inadequately compacted or when the conduit is placed on a foundation of varying density. The conduit location and the resultant embankment loading can result in differential

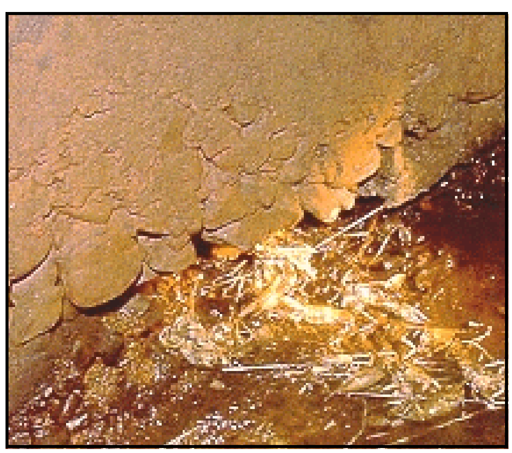


Figure 115.—A rock pocket at the bottom of a conduit side wall.

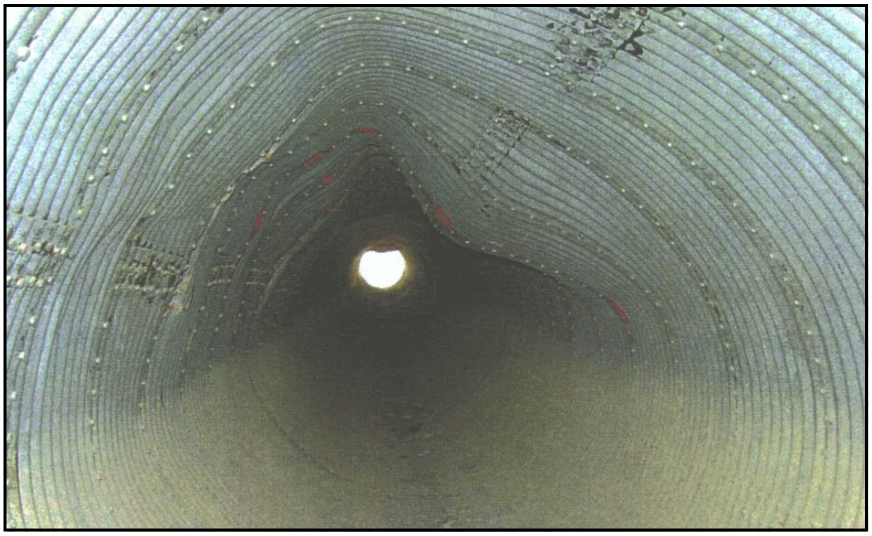


Figure 116.—Deformed CMP conduit. Deformation likely occurred during original construction, possibly from construction equipment traveling over the conduit with inadequate earthfill cover.

settlement problems. Differential settlement affects the structural integrity of the conduit by causing distress to the conduit in the form of misalignment (vertical or horizontal), shape distortion, joint offsets/separations, cracks, or spalls. Differential settlement occurs when one section of conduit settles more than the rest. This typically occurs at joints in the conduit (figure 117). The settling process can open these joints and provide a path for water either into or out of the conduit. Examples of differential settlement and the resulting damage are:

1. Spreading of the embankment dam, causing separations in the conduit joints. As compressible soils under the embankment dam consolidate, some spreading is inevitable. As soils spread laterally, sections of the conduit may separate, leaving joint openings through which water can then move.

2. Differential settlement due to foundation discontinuity, causing offsetting of joints.

3. Differential settlement of the embankment dam, causing loads greater than the conduit can accommodate, resulting in cracking and excessive deformation of the conduit.

- *Misalignment*.—Misalignment occurs when poor construction practice allows for alignment deviation or from improper or inadequate compaction of

Figure 117.—This conduit was severely damaged after the foundation settled more than 2 feet.

embankment materials next to the conduit. Misalignment can also be caused by compression of the foundation allowing rotation at the conduit sections.

- *Separation of joints*.—Separation of pipe joints occurs when the conduit experiences deformation, differential settlement, misalignment, or shear strains as a result of a weak foundation. Joint separation can result in a loss of conduit watertightness by allowing seepage to exit through the joint. The lack of joint gaskets being installed, or installation of the incorrect type of gasket, or the use of incorrect joint-connecting bands also affects watertightness. Seepage can lead to internal erosion or backward erosion piping of surrounding embankment materials and loss of support around the conduit.

Chapter 9

Inspection and Assessment of Conduit-Related Problems

Inspection of embankment dams, including their conduits and foundations, will detect many developing problems before they can affect the safety and reliable operation of the facility. Inspection should also assess the adequacy and quality of maintenance and operation procedures. Periodic inspection may reveal trends that indicate more serious problems are developing. The conduit is typically inspected as part of an overall inspection of the embankment dam and its appurtenant features. Typically, structural defects and deterioration develop progressively over time. A trained and experienced inspector can identify defects and potential problems before existing conditions in the embankment dam and conduit become serious. However, some situations can suddenly arise and cause serious damage in a short period of time. Examples of these situations are operations at full discharge capacity, seismic activity, or other special conditions. The need for special inspections should be evaluated after occurrence of any of these situations. The main focus of this chapter is on the inspection of conduits. However, reference is made to certain aspects of embankment dam inspection, since they have relevance to problems associated with conduits.

In 1986, 14 federal and State agencies developed a comprehensive training program (Training Aids for Dam Safety [TADS]) designed to train individuals involved with, or having responsibility for the safety of dams. The TADS program consists of modules that can be tailored to meet individual or organizational needs. The TADS program is widely used and recognized by the dam safety community. Further details on the TADS program are available from the Bureau of Reclamation. Additionally, training courses on dam safety inspection are available from various sources. Interested parties should consult the ASDSO website for a listing of available training opportunities. For information concerning inspection of penstocks see the American Society of Civil Engineers (ASCE) *Guidelines for Evaluating Aging Penstocks* (1995).

9.1 Types of inspections

Inspection intervals may vary, depending on the overall conditions determined from previous inspections and the existence of any dam safety concerns. Periodic inspections can vary in scope and purpose and by the organization or personnel (damtender, agency/district level, etc.) performing the inspection.

Dam safety organizations and embankment dam owners may employ a variety of inspections during the life of a conduit (figure 118). These inspections may include the following types (Reclamation, 1988, p. I-2):

- *Initial or formal.*—Initial or formal inspections include an in-depth review of all pertinent data available for the conduit to be inspected. Design and construction data are evaluated relative to the current state-of-the-art to identify potential dam safety problems or areas requiring particular attention. A thorough onsite inspection of all features is conducted, and an attempt is made to operate all mechanical equipment through their full operating range, if possible. Many State and federal agencies require formal inspections on a set frequency (e.g., every 6 years).

  The first time the reservoir behind an embankment dam is filled is critical to its integrity. The embankment dam will experience the hydraulic loading for the first time and will begin to adjust to this loading. During first filling, the wetting front begins to penetrate the embankment dam. History has shown that a much higher frequency of incidents occur at this time. Also, the conduit through the embankment dam will be tested for the first time.

  Good practice dictates that the embankment dam be monitored by frequent inspections during this crucial period. Round-the-clock surveillance is not uncommon for high hazard facilities. Special lighting provisions may be installed to permit adequate nighttime visibility.

  There may be several "hold" periods during initial fill to allow stresses in the embankment dam to partially stabilize and instrumentation to level off prior to the continuation of filling. The rate of reservoir rise may be limited to allow for the wetting front to slowly penetrate the embankment dam. A rate of reservoir rise in the range of 0.5 to 2 feet per day is a common. Limiting the rate of rise for small reservoirs that do not usually have large outlet systems may not be feasible. If the outlet conduit has a small capacity and large inflows follow a high precipitation event, no method for controlling the rate of rise exists.

  The first fill monitoring may be staggered to accommodate the amount of water available to fill the reservoir. For some embankment dams, many years may be required to reach their fully operational reservoir level. Often, an embankment

Figure 118.—Visual inspection for seepage on the downstream face of an embankment dam.

dam reaching a new record reservoir elevation during a flood is also considered to be in first fill status, necessitating heightened inspections. This is because portions of the embankment dam may have never received hydraulic loading until the flood stage was entered.

Following a major modification to an embankment dam, the dam may also be placed in a first fill monitoring situation, if the modifications were extensive. For example, if an existing conduit were completely removed and replaced, this would likely require first fill monitoring status. Complete removal and replacement of the conduit would require a section of the embankment to be excavated and replaced. For guidance on the removal and replacement of conduits, see chapter 13.

- *Periodic or intermediate.*—Periodic or intermediate inspections are conducted between formal inspections. An in depth review is made of all pertinent data available on the conduit to be inspected. However, the data review focuses on the current status of the conduit, and the data are not evaluated relative to current state-of-the-art criteria. A thorough onsite inspection of all features is conducted. All mechanical equipment may not be tested during any one inspection. Some equipment may be operated at another time or during the next inspection.

- *Routine.*—Routine inspections are typically conducted by field or operating personnel. The primary focus is on the current condition of the conduit.

Available data may not be reviewed and evaluated prior to the inspection, depending on the inspector's familiarity with the conduit. Inspections may be scheduled regularly or performed in conjunction with other routine tasks.

- *Special.*—A special inspection is conducted when a unique opportunity exists for inspection. For example, if low water conditions exist in a reservoir exposing a normally inundated structure, a special inspection may be arranged.

- *Emergency.*—An emergency inspection is performed when an immediate dam safety concern is present or in the event of an unusual or potentially adverse condition (i.e., immediately following an earthquake).

The actual terms and meanings used to define the types of inspection may vary between dam safety organizations and embankment dam owners.

The operating personnel responsible for daily operation and maintenance of the facility should also participate as inspection team members. Where applicable, water user organization representatives should also participate in the inspection. Additionally, the applicable State water resource agency may need to be advised for their possible participation in the inspection.

To the extent possible, inspections should be scheduled in different seasons. This will enable the structure or facility to be examined under differing reservoir levels, water delivery, and site conditions.

Before beginning inspection of a facility, the inspection team should discuss the order in which features are to be examined, to accommodate operations, as well as to ensure that time for the inspection team is appropriately allotted. In addition, the team should conduct a job hazard analysis (JHA) prior to the inspection, whereby procedures and equipment necessary to minimize or avoid potential safety and health hazards are discussed. Of primary importance is the need for detailed clearance (particularly if there are confined spaces), and lockout or "tag-out" procedures when accessing areas affected by equipment or gate/valve operations. For guidance on preparing a JHA, see section 9.4.

9.2 Factors influencing scheduling of inspections

Scheduling of periodic conduit inspections may be influenced by (Reclamation, 1988, p. III-7):

- *Sufficient notice* .—Embankment dam owners and operators may need sufficient time to make necessary arrangements, such as preinspections associated with lockout/tagout and confined space entry, or special equipment or approval for unwatering conduits, terminal structures, or pools. This process could require several weeks or months, depending on the facility.

- *Scheduling access*.—Access for the inspection should be scheduled when most or all of the major components of the conduit can be examined. Some features, such as intake structures and upstream conduits, are usually submerged and not accessible. Downstream conduits and terminal structures may or may not be able to be unwatered and made accessible for inspection. The embankment dam owner or operator may be requested to provide notification when reservoir conditions permit or when the reservoir can be drawn down to allow the inspection to be performed.

  If the feature to be inspected is normally inundated and inaccessible, certain factors (Reclamation, 1985, p. 4) should be considered in determining the extent and frequency for inspection, such as:

  1. Results of previous "hands on" inspection or evidence from the inspection of the normally accessible portions of the feature. Inspection of the normally accessible portion of a feature may provide information on the probable condition of the inaccessible portion. This information may include:

     a. *Condition of the feature*.—Cracking, joint separation, or significant deterioration.

     b. *Condition of the embankment dam and foundation*.—Excessive postconstruction settlement or alignment distortion of the downstream conduit; excessive embankment dam settlement or the existence of sinkholes on the upstream face along the alignment of the conduit.

     c. *Observed seepage*.—Seepage or wet areas observed at the downstream toe of the embankment dam.

     d. *Flow conditions*.—Changes in the discharge capacity of the conduit.

     e. *Damage and deterioration*.—Damage or deterioration of gates/valves and metalwork.

     f. *Water quality*.—Water quality known to be detrimental to concrete, conduit linings, or waterstops. Excessive amounts of sand or other material transported by the discharge.

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2. Operational history and performance of the feature, since its previous inspection.

3. Relative costs for providing access for inspection of the feature, including costs associated with lost water and power revenues.

4. Age of the feature.

5. Design and construction considerations, such as:

    a. *Changes in standards or guidelines*.—Design criteria, construction techniques, and/or quality of material at the time of construction fail to meet current standards or guidelines.

    b. *Foundation conditions*.—The conduit was constructed on foundation of varying compressibility, where there is a potential for differential settlement. This may result in cracking of the conduit or excessive opening of joints. Differential settlement is also possible between the conduit and gate chamber due to different pressures being exerted on them.

    c. *Foundation faults*.—The conduit crosses a foundation fault where there is the potential for movement or disruption of the conduit.

    d. *Unfavorable stresses*.—The conduit is located where conditions are conducive to arching, resulting in unfavorable stresses in the embankment dam and/or conduit. These stresses could be conducive to hydraulic fracture of the embankment dam or stress concentrations on the conduit.

    e. *Conduit within the core of the embankment dam*.—A significant portion of the conduit upstream from the gate chamber lies within the core of the embankment dam, so that any cracks in the conduit create the potential for water to be injected under pressure into the core. If erodible material is used to construct the impervious core, the potential for adverse consequences is increased.

    f. *Inadequate conduit joints*.—Inadequately sealed or encased conduit joints, which could lead to the escape of water under pressure, which creates the potential for water to be injected under pressure into the surrounding embankment.

    g. *Filters*.—Lack of adequate filters and drainage material around the conduit downstream from the im pervious zone of the embankment dam to safely convey seepage or leakage along the conduit to an exit point.

6. Critical function of the feature.

7. Any existing site conditions that may compromise the safety of the feature.

The appropriate frequency and extent to which the normally inundated features are examined will vary based on available information. The review personnel and decisionmakers will need to determine the appropriate frequency and extent based on the above factors. As an example, Reclamation has identified about 6 years as an appropriate frequency for a "hands-on or equivalent inspection frequency" for inaccessible features, such as conduit.

- *Operation*.—Certain problems may not normally appear when the feature is dry that appear when the feature is being operated. Also, when a feature is operating during a period of higher than normal releases, additional information may be gathered that may not have been available during normal operations.

The opportunity to optimize both access and operation during a single inspection typically is not possible. Inspection objectives may have to alternate from one inspection to the next. This may necessitate the need for scheduling "special" inspections during unusual conditions, in addition to regular inspections to provide a comprehensive understanding of the conduit safety. Special inspections may be required after floods, seismic activity, or other unusual or extreme events.

9.3 Periodic inspections by selected organizations

The frequency of periodic inspections varies among organization and embankment dam owners. Emergency situations may require much more frequent inspections, such as daily or hourly. Situations can arise suddenly that cause serious damage in a short period of time. Examples of these problems are operations at full discharge capacity, seismic activity, or other special conditions. The need for special inspections should be evaluated after occurrence of any of these situations.

A sampling of periodic inspections as required by selected organizations:

- *Reclamation*.—Reclamation employs the following process (Reclamation, 1998c, pp. 2-11) to monitor its significant and high hazard dams and attempt to detect any potential dam safety deficiencies:

  1. *Annually*.—Annual inspections are performed by inspectors who are generalist (as opposed to specialist) engineers very familiar with the

embankment dam and its operations, and can readily distinguish changes from year to year. All inspectors attend regular training in dam safety inspections.

2. *Periodic.*—On a 6-year cycle (alternating with the comprehensive facility review (CFR), each embankment dam is examined by a team originating in a Reclamation Regional Office, including the regional examination specialist. This examination is referred to as a periodic facility review (PFR) and includes a rather thorough review and reporting of all past dam safety and operation and maintenance (O&M) recommendations.

3. *Comprehensive.*—On a 6-year cycle (alternating with the PFR; the CFR and PFR are offset by 3 years), each embankment dam is examined/evaluated by a team of specialists from Reclamation's Technical Service Center that includes an examination specialist, mechanical engineer, and a senior dam engineer (either geotechnical or civil/structural specialist). This examination is referred to as a CFR and includes not only the PFR activities, but also technical evaluation of all design, construction, and analysis of the dam.

- *Federal Energy Regulatory Commission (FERC).*—Significant and high hazard embankment dams are inspected annually by FERC engineers and every 5 years for a Part 12D inspection by an independent consultant (FERC, 2005, pp. 14-43 to 14-45). FERC engineers inspect low hazard embankment dams at least every 3 years. An independent consultant also inspects some low hazard embankment dams every 5 years, if the dam is 30 or more feet high or the reservoir is 2,000 acre-feet or larger and the licensee or exemptee has not requested and received approval for an exemption from the Part 12D independent consultant inspection.

- *NRCS.*—The NRCS requires the sponsor/owner to be responsible for making inspections after they are turned over to the sponsors/owners (NRCS, 2003, pp. 1-2). Personnel trained in conducting the inspections perform special, annual, and formal (once every 5 years) inspections. If requested by the sponsor/owner, NRCS may participate in inspections; provide training to ensure that the sponsor/owner understands inspection techniques and the importance of completing corrective action; and provide technical assistance to address specific O&M needs. If an inspection reveals an imminent threat to life or property, the sponsor/owner shall immediately notify all emergency management authorities.

- *USACE.*—The USACE performs periodic, intermediate, and informal inspections on the basis of project size, importance, or potential hazard (USACE, 2004b, pp. 6-3 and 6-4):

1. *Initial periodic inspection*.—The first periodic inspection and evaluation of a new embankment dam is carried out immediately after topping out of the dam prior to impoundment of the pool.

2. *Second periodic inspection*.—The second periodic inspection for new embankment dams is performed no later than 1 year after impoundment is initiated.

3. *Subsequent periodic inspection*.—Subsequent periodic inspections are performed at 1-year intervals for the next 2 years. The next two inspections are performed at 2-year intervals and then extended to a maximum interval of 5 years. More frequent inspection intervals are scheduled, if conditions warrant.

4. *Intermediate inspection*.—For projects on a 5-year inspection cycle, an intermediate inspection of all or some of the features may be scheduled, if warranted. Selection is based on consequences of failure, age, degree of routine observation, a natural event (i.e., earthquake), performance record and history of remedial measures. Intermediate inspections are also made of any portion of a project exposed during unwatering that could not be accomplished during scheduled periodic inspection.

5. *Informal inspection*.—Appropriate employees at the project perform frequent informal inspections. The purpose of informal inspection is to identify and report abnormal conditions and evidence of distress.

9.4 Preparing for an inspection

The success of a conduit inspection depends upon good planning and preparation. Any inspection should consider:

- *Selection of the inspection team*.—The members of the inspection team will vary, depending on the needs and resources of the organization or embankment dam owner, type of the inspection, results of the data review, and any special requirements.

- *Review of project data*.—The amount of available data may vary greatly. The extent of project data review and evaluation depends on the type of inspection to be conducted.

- *Preparation of an inspection plan*.—A detailed inspection plan should be prepared to identify all features to be inspected, problem areas, and areas of potential problems. The inspection plan will also identify special logistics, access, or equipment requirements. An inspection checklist is typically prepared as part of an inspection plan. The checklist is used to identify specific inspection objectives and is also useful in developing the final inspection report.

Prior to any inspection, inspection personnel should review all pertinent and available design and as-built drawings, design criteria, geology, operational history, previous inspection and maintenance reports, and safety information. Typical documents that should be reviewed prior to an inspection are:

1. Technical record of design and construction
2. Design summary
3. Laboratory reports
4. Stress model reports
5. Geology reports
6. Site seismicity reports
7. Plans and specifications
8. As-built drawings
9. Final construction report
10. Construction progress reports
11. Travel reports
12. Correspondence files
13. Operation and maintenance records
14. Examination reports
15. Designers' operating criteria
16. Standing operating procedures
17. Reservoir operation records
18. Data books

After reviewing available documentation, a list of important and significant concerns should be prepared for use during the inspection.

A log should be established at the embankment dam that records the date, type of inspection performed, name of the inspectors, and the results. All inspections should be documented in the form of an inspection report with photographs, reservoir water levels, discharges from the conduit, and relevant instrumentation data, such as from nearby piezometers, and forwarded to the engineering staff or personnel responsible for technical review and evaluation. An ongoing visual inspection checklist should be developed to provide guidance and consistency in looking for signs of distress. If information is found that suggests the embankment dam, foundation, or conduit was not designed to current standards, specific items should be added to the inspection checklist to address specific deficiencies. All inspection reports should be maintained in a secure location for future reference. Good recordkeeping of inspection reports, technical reports, etc. will ensure that development of any adverse trends are identified and proper actions are taken to correct any problems.

For further guidance on inspection programs and checklists for inspection, see Reclamation's *Review of Operation and Maintenance Program Field Examination Guidelines* (1991).

A job hazard analysis should be prepared for embankment dam and conduit inspections, following approved safety guidelines. The basic elements of a JHA are outlined in Reclamation's *Operation and Maintenance Safety Standards* (1989b, pp. 65-66). Note: Other agencies and organizations may utilize their own set of standards for safety guidance. All personnel involved in the inspection should receive and review a copy of the JHA. As a minimum, a JHA should include:

1. Names of all participants and the agency, organization, or group they are representing.

2. Operations to be performed.

3. Special considerations, such as monitoring of atmospheric conditions prior to entry into confined spaces. Detection of adverse atmospheric conditions at any location requires that the confined space be mechanically ventilated or the examination be abandoned. Entry should only proceed upon confirmation of acceptable atmospheric conditions. All entrants into confined spaces are to have lockout/tagout and confined entry space

training and are required to wear an approved body harness to facilitate extraction of personnel should they become incapacitated.

4. Potential hazards associated with the confined spaces defined previously are engulfment by water; oxygen deficiency; walking/working surfaces; electrical hazards; lighting; molds, mildews, and spores capable of irritating the respiratory system; and other hazards (e.g., rodents, snakes, spiders and/or insects, or crayfish).

5. Mitigating measures.

6. Hazards and solutions.

7. Safety-related equipment, such as hard hats, safety boots, proper clothing, gloves, communication equipment, oxygen/gas detection meter, mechanical ventilation equipment, flashlights, first aid kit, rubber boots, safety lines and harnesses, extraction/hoist equipment, and eye protection.

8. Safety standards requirements.

9. Emergency services.

10. Signatures of the inspection team members indicating that they have reviewed the JHA and have been instructed in and understand the requirements and hazards associated with the entry into confined spaces for the purpose of conducting this examination.

Upon completion of the inspection, all participants should discuss the inspection to identify what could be improved in the JHA for the next time. Any findings or recommendations should be documented for inclusion in future JHAs. Any mishaps or near misses should be identified during the postinspection discussion.

A dive plan or dive hazard assessment should be prepared prior to any dive inspection. Most commercial diving companies have their own dive plans. Guidance on dive safety can be found in Occupational Safety and Health Administration (OSHA) Standards 29 CFR, Subpart T, *Commercial Diving Operations—General Industry* (2004), and the Association of Diving Contractors International's (ADCI), *Consensus Standards for Commercial Diving and Underwater Operations* (2004). Various government agencies have guidance on dive safety, such as Reclamation's Safety and Health Standards Section 29—*Marine and Diving Operations* (2002).

9.5 Performing the inspection.

Methods used for the inspection of the various features of a conduit mainly depend upon accessibility. Factors influencing accessibility include:

- *Inundation*.—Reservoir operations and water levels may make some features unavailable for normal inspection and require specialized inspection services (e.g., dive team, remotely operated vehicles).

- *Confined space*.—Certain features may require OSHA confined space permitting for man-entry, lockout/tagout procedures, and preparation of a JHA. An alternative to man-entry is the use of specialized inspection services (i.e., closed circuit television).

- *Size constraints*.—Limitations in size may prevent man-entry and require specialized inspection services (i.e., closed circuit television).

9.5.1 Inspection of entrance structures

In most cases, due to the entrance structure's location in the reservoir, it is either partially or fully inundated. If the entrance structure is partially inundated, inspection of the structure above the water level will be fairly straightforward. However, inspection of the portion of the structure below the water level, such as the intake or inlet, trashracks, fish screens, ice prevention systems, gates/valves, stoplogs, and bulkheads, will require specialized inspection services.

If the intake structure is a tower, it may have a wet well or some other access to the control mechanism. Closure of a guard gate or bulkhead may provide the ability for inspection of the interior of the tower. Problems common to entrance structures include deterioration, damage, and misalignment.

Descriptions of more specific problems related to trashracks, fish screens, ice prevention systems, gates/valves, stoplogs, bulkeads, and bridges are beyond the scope of this document. The TADS program as discussed earlier in this chapter should be referred to for more detailed information concerning the inspection of entrance structures.

9.5.2 Inspection of conduits

Generally, conduits with diameters 36 inches or larger can be inspected by man-entry, if proper OSHA precautions are taken. Conduits with diameters smaller than 36 inches are generally inaccessible for man-entry and require specialized inspection services.

9.5.2.1 Exterior inspection

Exterior inspection of the areas above and surrounding the conduit can provide many clues concerning the condition of the conduit. Items to look for include:

- Look for signs of infiltration of soil into the conduit. Depressions, sinkholes (figures 119 and 120), or cavities that exit onto the surface of the embankment dam along the centerline conduit alignment are usually an indication that internal erosion or backward erosion piping is occurring. These features often appear as holes that line up with one another. Such features should be marked with reference points and monitored to determine whether they are expanding with time. Sinkholes should be probed to determine the extent of the void, which may be dome shaped and enlarge with depth. The seepage and flow conditions on the downstream slope and through the conduit, should be examined for evidence of association with the sinkhole. Sinkholes are a cause for immediate concern and further investigation. Beware that some animals may take over these areas, and they may not be recognizable as sinkholes or cavities.

- Look for signs of seepage or indications that seepage is sometimes present. The best time to look for seepage may be when the conduit is operating in a pressurized condition or at full discharge capacity. Evaluate the following:

    1. If an area on the surface of the embankment dam is wet, the area should be marked or staked, and photographed, to see if it is expanding over time. If the seepage is flowing, measures should be taken, such as the installation of a weir, to collect and measure the quantity of flow. A seepage rate that is increasing faster than expected, relative to the reservoir level, may be an indication of internal erosion or backward erosion piping. Seepage in these areas may be characterized by increased vegetative growth or the presence of plants that thrive in wet areas. If instrumentation is available, measurements of seepage should be compared to previous measurements to reveal changes in flow rates. Piezometers should also be monitored.

    2. The quantity of seepage along the conduit or through the conduit's backfill may indicate that adequate compaction around the conduit was not achieved or internal erosion or backward erosion piping is occurring. The area where water outlets from a seepage diaphragm should be closely monitored. Seepage areas may be indicated by changes in vegetation or color. The limits of a newly wet area should be marked to determine whether the area is increasing in size. When possible, the seepage should be channeled away from the embankment dam and directed through a pipe, weir, or other device that will allow the quantity to be measured.

Figure 119.—Sinkhole in the crest of an embankment dam.

Figure 120.—Sinkhole around a spillway riser. Photo courtesy of Schnabel Engineering.

Measurement of flow by a stopwatch and bucket is a simple way to collect flow information. Installation of a weir and staff gauge is preferred for more uniform data collection under longer term conditions.

3. The quality of any seepage, especially whether it is carrying soil particles should be analyzed. Water seeping into, out of, or along a conduit can cause problems by carrying particles with the flow. If the quality and quantity of the water flowing into the conduit is different from the water flowing out of the conduit, then it is likely that open joints or cracks are allowing additional seepage flow to enter the conduit, or normal discharge to leak out. The appearance of the flow at the area where water outlets from a seepage diaphragm is of particular concern. Any water flowing in the vicinity of the conduit should be observed for evidence of fines being transported, such as cloudiness or discoloration. The internal erosion and backward erosion piping processes can occur intermittently, with fines being transported sporadically. Evidence of fines being carried in seepage is cause for concern, further investigation, and prompt action.

The monitoring of any condition involving seepage or discharge should also include the corresponding reservoir pool level. Any sudden change, or unusual trend over time, which does not correspond to changes in the reservoir level, could indicate a seepage problem. For example, an increase in the seepage rate while the pool level is constant could be an indication of internal erosion. Pool levels may be measured by a staff gauge, by calibrations placed on a fixed structure in the reservoir, or by water-level sensing devices.

- Look for signs of internal erosion or backward erosion piping where the conduit exits the downstream slope of the embankment dam near the terminal structure. Water flowing through cracks in the earthfill or along the conduit may erode soils and cause a cloudy effluent with turbulent flow. Deposits of sand may form at the exit point of seepage. Water escaping from intergranular seepage in granular soils may create sand boils, and the flow is less likely to be turbid. Other indicators of developing problems include deposits of sediment not associated with runoff, sinkholes, and signs of settlement, such as depressions on the surface of the embankment dam or its foundation.

- Any changes in the embankment dam or foundation in the vicinity of the conduit. Since the location of a conduit represents a unique condition in the embankment dam, and a potential seepage path through the dam, any changes in the vicinity of the conduit should be investigated. Such changes might include slope movement, changes in vegetation, areas of new or unexpected wetness or seepage, unusual piezometric readings, etc.

- Check the exposed areas of the conduit for cracking, weathering, and/or chemical deterioration.

- Look for any whirlpools in the reservoir in the vicinity of the conduit.

- During operation of the conduit, additional items of concern include:

  1. Any unusual noises, such as popping, banging, or vibrations should be investigated. Vibrations may occur, if the conduit is not properly supported. Vibrations could adversely affect the conduit and surrounding backfill.

  2. Color changes or fines observed in the discharge water coming out of the conduit.

  3. Pulsating or unstable flow.

  4. Unexplained reductions in discharge capacity.

9.5.2.2 Interior inspection.

Typical problems within the interior of conduits include deterioration, obstructions, joint offsets and separations, defective joints, cracking, and mechanical equipment misoperation (figures 121 and 122).

If the conduit is accessible, the inspector should use a measuring tape or pace off the locations of all damaged or questionable areas within the conduit. Damage or questionable areas should be documented using still, digital, or video camera equipment. If the conduit is inaccessible, CCTV inspection equipment should be utilized.

The interior inspection should look for:

- Water ponding on the invert of the conduit, which could be an indication of settlement-related problems in certain reaches of the conduit, especially if the conduit as-built drawings show a constant invert slope.

- The locations of cracks should be documented using a crack map or similar reporting method. Be aware of any previously reported cracks, and note any new cracks. The length and width of the crack should be measured. To get an indication of the continuity of cracks through a concrete structure, use a geologist's pick or similar hammer to tap the concrete and listen for changes of pitch that give clues to the condition of the concrete. At some selected sites where accessible conduits are constructed on compressible or nonuniform foundations, strain gauges, total stress cells, and crack meters have been used to monitor changing conditions. For guidance on performing a crack survey, see USACE's *Evaluation and Repair of Concrete Structures* (1995b, pp. 2-1 to 2-13). For

Figure 121.—Inspection of a CMP conduit looking for signs of deterioration.

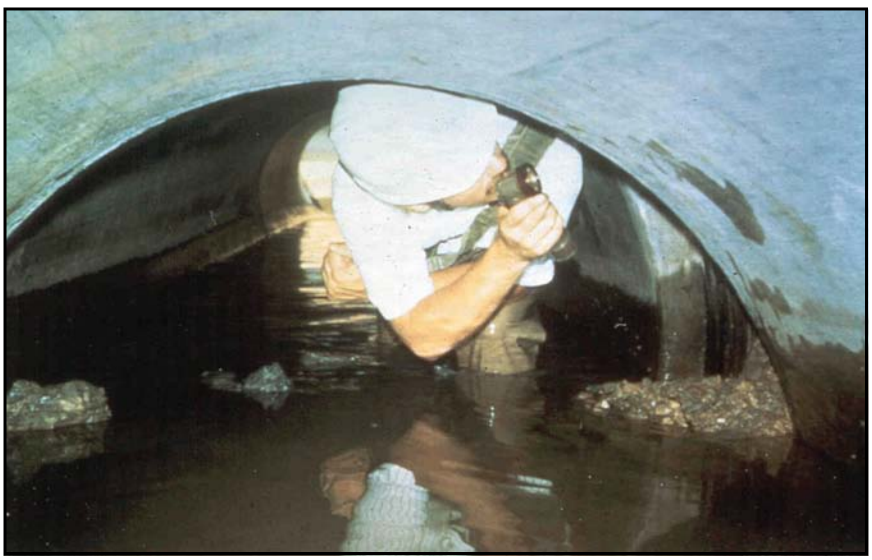

Figure 122.—Inspection being performed in difficult conditions. The joints of this 48-in concrete pipe separated when foundation movement occurred during construction of the embankment dam. For details, see the case history for Little Chippewa Creek Dam in app. B. Photo courtesy of Ohio Dam Safety Division.

an example of a how a crack survey was used within a conduit, see the Beltzville Dam case history in appendix B.

- Joint separations between conduit sections and at connections to entrance and terminal structures. In accessible conduits, joint meters have been used to monitor the opening and closing of joints in conduits. For additional guidance on crack and joint measuring devices, see USACE's *Instrumentation for Concrete Structures* (1987, pp. 5-1 to 5-24).

- Metallic corrosion of pipe or exposed reinforcement.

- Discoloration or staining of concrete surfaces.

- Damaged protective coatings.

- Deformation of the conduit circumference.

- Chemical deterioration of concrete.

- Leakage into or out of the conduit.

- Misalignment of conduit sections.

- Plugged drain holes.

- Voids behind the concrete near any observed cracks, joint separations, or misalignments. The ideal time to look for seepage through these areas is when the conduit has been recently unwatered and water may be draining into the conduit from the surrounding embankment.

- Spalled concrete from compression or reinforcement corrosion.

- Drummy or hollow-sounding concrete. The extent of deterioration may be difficult to determine. Sampling (coring) and testing of the material may be required. Samples taken from areas of deterioration are often compared with samples taken from good quality concrete. Testing may include determining the strength properties and use of petrographic examination.

- Erosion, abrasion, or damage in concrete downstream of gates/valves, offsets, and/or changes in slope.

- Cavitation damage.

- Binding of mechanical equipment.

- Blockages at the conduit entrance (i.e., trash or debris) or at the exit (i.e., vegetation, backed up water).

In attempting to inspect the interior of any conduit, there may be difficulties to overcome, such as:

- *Unwatering.*—A comprehensive inspection may be hindered, unless the conduit can be unwatered. Proper precautions should be considered prior to any unwatering situation. The possibility exists of external pressures being high enough to damage the unwatered conduit or vents being plugged, causing negative internal pressures to develop and collapse the conduit. This is a concern when pressurized conduits are unwatered. Unwatering a conduit may be impractical or impossible for one or more of the following reasons:

    1. Lack of a bulkhead or closure device.

    2. The need to limit reservoir drawdown. Lowering of the water surface may be restricted, which would prevent exposure of the conduit or entrance structure.

    3. Structural adequacy of the conduit to withstand external hydrostatic pressures in a unwatered condition.

- *Poor air quality.*—Poor air quality may exist within conduits. Poor air quality conditions may include lack of oxygen and the existence of hydrogen sulfide.

- *Inaccessibility.*—The conduit may be too small or too dangerous for man-entry inspection. The use of CCTV inspection equipment should be considered for inaccessible conduits. If this is not feasible, the inspection must then be based solely on the condition of the exposed and/or accessible portions of the conduit. Some details of the interior may be obtained by use of a bright light and the zoom feature of a camera.

For an examples of a man-entry inspections of a conduits, see the Dalewood Shores and Salmon Lake Dam case histories in appendix B.

9.5.3 Inspection of terminal structures

The terminal structure may be dry or partially inundated, depending on the time of year and the schedule of releases through the conduit. If the terminal structure is partially inundated, inspection of the structure above the water level will be fairly straightforward. However, inspection of the portion of the structure below the water level, such as the basin, chute blocks, baffle blocks, or end sill, will require specialized inspection services.

Problems common to terminal structures include deterioration, damage, obstructions, misalignment, backfill and foundation deficiencies.

Descriptions of more specific problems related to basin, chute blocks, baffle blocks, or end sills are beyond the scope of this document. The TADS program, as discussed earlier in this chapter, should be referred to for more detailed information concerning the inspection of terminal structures.

9.5.4 Specialized inspection

Specialized inspection includes the use of a dive team, climbing team, remotely operated vehicle (ROV), or closed circuit television.

9.5.4.1 Underwater inspections

Underwater inspection is typically accomplished by either scuba diving operations or surface-supplied air diving operations. Scuba diving equipment typically includes a breathing gas supply tank, which is carried by the diver. A scuba diver has more flexibility and maneuverability compared to surface-supplied diving operations. However, this method of inspection limits diver communication and should be limited to areas where the diver has an unobstructed path directly to the surface. Surface-supplied diving operations provide breathing gas to the diver via an umbilical and offer deeper dive capability, the potential for longer underwater stays, and communication between the diver and the surface, and should be utilized whenever the diver enters an overhead environment (diver does not have a direct vertical path to the surface).

Dive inspections are used for the examination of conduits, and entrance and terminal structures. However, the focus of this section will pertain only to dive inspections of conduits. The inspection of a conduit is often termed a "penetration dive."

Dive inspections are expensive, and the costs are greatly influenced by the depth of the dive, the elevation at which the dive is performed, and the temperature of the water. All specialized inspections involve a number of variables. As a general a rule of thumb, when comparing the costs involved with dive inspections to ROV inspections, dive inspections are about 3 to 5 times more expensive.

A dive inspection has the advantage of using a variety of instruments for testing the structural integrity of the conduit, such as a rebound hammer for providing data on concrete surface hardness, a magnetic reinforcing steel locator to locate and measure the amount of concrete cover or reinforcement, and an ultrasonic pulse velocity meter to determine the general condition of concrete based on sound measurements. Dive inspections also offer the potential for hands-on, tactile inspection of features in limited visibility or those covered with shallow layers of organics or sediments.

Some important considerations for any dive inspection are (Dulin and Crofton, 2004):

- *Certification*.—All divers and personnel associated with dive inspection should be certified commercial divers trained to meet the minimum requirements of ADCI's *Consensus Standards for Commercial Diving and Underwater Operations* (2004) through the training standard of an accredited Association of Commercial Diving Schools program. They should be compliant with all commercial diving training standards, have onsite documentation of first aid training, cardiopulmonary resuscitation (CPR), and meet other standards as applicable in compliance with OSHA and ADCI standards.

- *Dive team*.—The dive team should include the diving supervisor, a lead diver, and a backup diver for relief or emergencies. The diving team should have a Dive Master, whose primary talents are coordination of his crew and a solid understanding of what needs to be accomplished. Another member of the dive team should have a good understanding of mechanical equipment, what functions have to be maintained, and what has little importance to the equipment. Another member of the dive team should have solid experience with electronic equipment, such as ultrasonic thickness gauges, underwater still cameras, and communication equipment. All divers on the team should have the strength to accomplish the physically demanding tasks involved with the inspection.

- *Communication*.—Communication with a diver underwater is difficult. Everyone involved with the project needs to know the chain of command and what role each individual plays. The means of contact, both primary and secondary, should be fully understood by all parties who may be involved with any portion of the diving inspection.

- *Safety*.—A specific job hazard analysis should be performed to address all aspects of the diving operation. All parties who may be involved with any portion of the diving inspection should hold a kickoff meeting. Discussion should include the lockout tag-out (LOTO) procedure. A draft copy of the procedure should be provided to all attendees. The procedure should be finalized prior to commencement of any diving. No diving activity should start until the LOTO is finalized and accepted by all parties involved.

Diving in an environment where the diver does not have a direct route to the surface is a very specialized area of diving. No clear-cut criteria exist for defining conduits that can or cannot reasonably be inspected by divers. Many conduits that are large enough for a diver to enter may have factors that preclude them from being inspected. Certain factors must be weighed against one another and a judgment made as to the viability of a dive inspection. Factors that must be considered include:

- *Depth*.—As the depth of the conduit below the water surface increases, the difficulty of performing a dive increases. Divers have a limited amount of time on a given dive, and that time decreases with the increased pressures on deeper dives. Also, as the dive becomes deeper, more of the allowable dive time is spent descending to the conduit. Allowable dive times can be increased by means, such as using mixed gas, or diving in a pressurized "newt suit." This increased dive time at depth comes at an increased cost due to requirements for items like larger dive crews, more specialized equipment, and a limited numbers of companies that can actually do the work. As an example, a 25-foot deep dive at sea level using air would not have a no-decompression limit (NDL), an amount of allowable dive time before decompression is required, while an 80-foot deep dive under the same conditions would have a NDL of 40 minutes. Decompression diving can be utilized to increase the work time available to the diver, but would likely come at an increase in the costs associated with the dive.

- *Altitude*.—The altitude at which the conduit is located can greatly affect the viability of a dive inspection. This could really be considered a subfactor of the depth factor. Due to the lower atmospheric pressure at higher altitudes, the diver has an even more limited bottom time associated with a given depth of dive. For example, comparing the 80-foot deep dive previously discussed:

    1. At sea level, NDL of 40 minutes

    2. At 2500 feet, NDL of 30 minutes and would be treated as a 90-foot dive

    3. At 5000 feet, NDL of 25 minutes and would be treated as a 100-foot dive

    Using decompression diving is an option for addressing the impact of altitude on dive time, but once again this would likely come with an increased cost.

- *Water temperature*.—As the water temperature decreases, it can have the effect of decreasing the dive time available to a diver. This is not necessarily a quantifiable variable as it relates to dive time. Often the temperature effect can be mitigated to some degree by the level of thermal protection worn by the diver. Care should be exercised with decompression diving in extremely cold water, because a failure in the thermal protection measures (leak in suit, hot water heater shutdown, etc.) after the diver has passed the NDL will necessitate what could be a long, cold decompression stop with the risk of severe hypothermia.

- *Length*.—As with depth, the conduit length becomes a factor relating to the amount of time the diver has available at depth. If the conduit is extremely long, it can take much more time to inspect than the diver has available. The available dive time for a long conduit can be increased, but this can be costly. Safety also must be considered. Because the diver does not have a direct path to the surface, the farther the diver must penetrate into the confined space, the farther the diver is from a direct path to the surface.

- *Access*.—Often the entrances to conduits are equipped with trashracks on the inlet side. The ability to remove enough of the trashrack bars to allow easy entry and egress is important. Since divers in such an overhead environment will be utilizing some type of surface-supplied breathing gas, it is important that the access point be such that the hoses will be able to be fed into the conduit without hanging up. A second diver is required to be stationed underwater at the confined space entry point to tend the primary diver's umbilical.

- *Leakage and currents*.—The leakage of downstream gates or valves in a conduit is a safety factor that can affect whether a dive inspection can be safely performed. Currents can be unpredictable. Any inspection of this type should be performed, such that the diver enters the conduit against any current and then returns and exits with the current. In the case of an inverted siphon, this can be accomplished by entering from the downstream end, but in the case of an outlet works, a submerged conduit will more than likely need to be entered from the upstream end. Therefore, the condition of the gates or valves and how much leakage is exhibited is a big factor with respect to the viability of a dive inspection.

- *Conduit size*.—A conduit should really be large enough that the diver can turn around inside and exit head first. The size for this will obviously depend on the size of the individual diver and also the exact type of equipment required.

- *Visibility*.—The distance a diver can see is important to whether a dive inspection of a conduit is advisable. In poor visibility situations, the diver can use their sense of touch for inspection. Sometimes a diver can use a hand to probe areas that cannot be seen. In the event of zero visibility, there would likely be little reason to pursue a dive inspection, as the shear magnitude of the entire surface of a conduit would be extremely difficult to inspect by touch alone. Also, in a circular conduit, a diver does not have a real edge or other reference point to keep track of any findings. If a dive inspection (figure 123) is planned for a conduit, consideration should be given to making a large release prior to the inspection as a means of flushing sediments from the conduit and then allowing some amount of time for the water to settle out prior to diver entry. This time will depend on the type of sediments in the water, but could vary from a day to a week. If visibility is good, the diver may want to use a high resolution hand-held video camera to document conditions existing within the conduit. The video camera can be either self-contained or configured for topside viewing. A self-contained video camera is enclosed in a special waterproof case that allows for easy operation by the diver. For topside viewing, a cable is required from the camera to the monitor located on the top. Audio can be provided during the recording by the diver or topside personnel. Video cameras can also be mounted on the diver's helmet. However, no matter how good the video camera's resolution is, if visibility is poor, the camera will only be able to document a few square inches of surface at one time.

Figure 123.—Diver performing an underwater inspection.

Sometimes in pressurized conduits, it may be difficult for a diver to determine, if a defect is allowing water to leak through it. In these situations the diver may want to release colored dyes (e.g., food coloring) and observe if it gets sucked into the defect. Another option would be the use of a wand with a string or frayed rope attached to it. If water is leaking out of the conduit the string or frayed rope would be sucked into the defect (Stoessel, Dunkle, and Faulk, 2004, p. 2). Temporary repairs by the divers are possible by plugging these defects with Oakum or similar materials. However, a more permanent repair will need to be considered.

In certain situations, the combined use of divers and ROV or CCTV equipment may be required to complete the conduit inspection. The divers are used to gain access to the conduit and place the ROV or CCTV equipment in the proper location to begin the inspection.

For an example of an underwater conduit inspection, see the Salmon Lake Dam case history in appendix B.

9.5.4.2 Climb inspection

Although not often required for conduits, a climbing team may be utilized to perform inspection of the inaccessible portions of intake towers and the walls of terminal structures (figure 124).

9.5.4.3 Remotely operated vehicle

The ROV was first developed for industrial purposes to inspect oil and gas pipelines and offshore platforms. ROVs are now being utilized for underwater inspections of

Figure 124.—Climber performing an inspection on a terminal structure wall.

entrance and terminal structures and conduits. The focus of this section will pertain only to ROV inspections of conduits.

ROVs are normally linked to a surface power source, although untethered models are also available. However, untethered (autonomous) vehicles are typically larger and not used for inspection of conduits. ROVs that are linked to the surface have cables that carry electrical signals back and forth between the operator and the vehicle. Most ROVs are equipped with at least a video camera and lights. Additional equipment is commonly added to expand the vehicle's capabilities. These may include a still camera, a manipulator or cutting arm, water samplers, and instruments that measure water clarity, light penetration, and temperature.

An ROV consists of a video unit, a power source for propulsion, vehicle controllers (referred to as "joysticks"), and a display monitor. The ROV can provide real-time viewing. Most ROVs are either observation or working class vehicles. An observation class vehicle is small and compact and is used for visual inspection where nonintervention applications are required. Typically, observation class ROVs include a high resolution color video camera capable of zoom and manual or auto focus. Figure 125 shows an observation-class ROV entering the water. Precision color scanning sonar is an added option, but can be expensive. Some observation class ROVs may have a single function manipulator. Working class ROVs are typically capable of search, survey, inspection, and light intervention to depths of

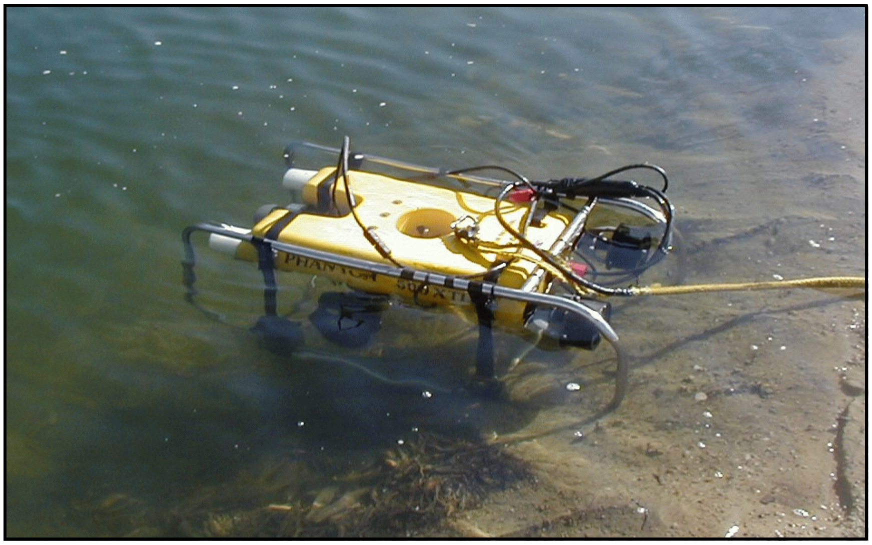

Figure 125.—An observation-class ROV entering the water to begin an inspection.

2,000 feet. Working class vehicles can typically support a payload capacity to allow for the attachment of sophisticated accessories. Most working class ROVs have multifunction manipulators.

An operator or "pilot" controls the vehicle from the surface. Using a joystick, a camera controller, and a video monitor, the operator moves the ROV to the desired location. The operator's eyes essentially "become" the camera lens. The vehicle's depth and heading can be recorded. A global positioning system (GPS) is generally not available on most ROVs and is an expensive and complicated added feature that cannot be used within the conduit. Joysticks are used to control the propulsion and manipulation of the ROV and any accessory equipment. ROVs typically have three thrusters, two horizontal and one vertical. The thrusters allow the vehicle to move forward and backward and to turn left and right. Some ROVs may have a fourth thruster mounted horizontally for lateral movement.

ROVs are capable of accommodating various attachments (i.e., a pincer claw) for grasping, cleaning, and performing other inspection tasks. However, the addition of attachments requires larger ROVs to accommodate the attachments. Specially designed ROVs can accommodate and operate non destructive testing equipment.

In the event that diving is prohibitive and dewatering of the conduit is not economically or technically practical, an ROV can be utilized. ROVs can compensate for the limitations inherent in underwater inspections performed by divers, since they can function at extreme depths and water temperatures, are not affected by altitude concerns, remain underwater for long durations, enter smaller diameter conduits, and repeatedly perform the same tasks without sacrifice in quality. Also, the costs involved for ROV inspection are considerably less than for dive inspection. Inspection by ROV may be preferable in certain situations prior to performing a dive inspection. This is especially important in regards to safety. An ROV that is damaged or destroyed can be replaced. However, this is not comparable to the loss encountered by a diver who is injured or killed.

Extreme caution is advised when performing an ROV inspection. The ROV operator should be qualified, experienced, and knowledgeable about the hazards involved. The potential exists for the ROV to become stuck in small diameter conduits due to offsets, sharp bends, or debris. The ROV can also become entangled in its umbilical cable (or the umbilical cable can become entangled with debris, such as tree branches). ROVs can be expensive depending upon the level of sophistication and costs involved with the retrieval of a stuck ROV can be very expensive and time consuming.

The ROV is typically inserted into the conduit from the upstream end. Depending on the entrance structure's configuration, assistance may be required from a diver to assist the ROV getting past trashracks. This approach can be used where the depth, length, and/or access limits a dive inspection's viability, but it is difficult to get the ROV into the conduit. The trashracks typically have a hatch cover that can be removed, or the ROV can also be lowered through a gate slot to access the conduit. If trashracks cannot be removed, a few of the bars may need to be cut and removed to allow insertion of the ROV. At some sites where the downstream conduit is located within a larger conduit, an ROV can be inserted from the downstream end of the conduit. For downstream end insertion, the ROV is placed within the unwatered section of conduit between the downstream guard and regulating gates/valves. The ROV cables are threaded through a special manhole in the pipe. Once the conduit section is rewatered and the guard gate opened, the ROV can proceed upstream and inspect the conduit. This method may be difficult, especially if umbilical cable needs to be continually fed through the opening, and should only be attempted by qualified and experience personnel.

Some of the limitations using an ROV for conduit inspection include (USACE, 1995b, p. 2-15):

- *Two-dimensional.*—The ROV inspection provides only a two-dimensional view and does not project the full extent of any defect. If the conduit diameter is large, the ROV inspection is much more likely to be limited to one small path along the conduit, whereas a diver can cover a much larger path or wider swath as the diver moves down the conduit.

- *Visibility.*—Murky water limits the effectiveness of an ROV inspection. With an ROV in a limited visibility situation, the only area inspected is the small area directly in front of the camera. A diver can use their sense of feel, in a limited visibility situation and focus in on any problem areas.

- *Orientation*.—In some situations, it may be difficult to determine the exact orientation or position of the ROV. This can impede accurate identification of the area being observed. Also, since ROVs often rely upon a compass, the steel in the conduit lining and/or concrete reinforcement can affect the navigation. If a CCTV camera-crawler is used in lieu of an ROV, the length of cable tether can be measured to determine the location within the conduit.

- *Maneuverability*.—In some "tight" areas the ROV may have more difficulty with maneuverability than divers would have in the same situation. Water currents can also affect maneuverability by causing the tether to become entangled.

The technology associated with ROVs is continually evolving. Continued advancements will allow the operator to overcome some of the existing ROV limitations by utilizing more sophisticated attachments and instruments to improve diagnostic capabilities.

9.5.4.4 Closed circuit television

The use of CCTV as an inspection method has undergone significant technological advancements. The introduction of robotic and automated systems, such as smart pigs, camera-crawlers, and other remotely controlled vehicles has allowed previously inaccessible conduits to be inspected. CCTV and man-entry are the most widely used methods of conduit inspection.

CCTV is a very useful method for examining small or inaccessible conduits (figures 126, 127, and 128). CCTV inspection provides significant improvements over other methods of inspection, such as man-entry inspection where an inspector crawls through the conduit (36 inches or larger) and documents the conditions, manual inspection where a sled with a camera is pushed through the conduit using long push rods, and mechanical inspection where a camera tethered to a wire rope is pulled through the conduit. CCTV inspection has the advantages of being able to examine conduits regardless of size limitations, has complete mobility, and provides real time video images.

CCTV inspection also can be used in conduits where confined space entry issues may require permitting prior to man-entry. OSHA regulations define a confined space as having limited access and egress, and not being designed for continuous human habitation. This would include not only small conduits, but also larger diameter conduits, where risks, costs, or system complexity may make remote inspection more advantageous.

Figure 126.—Seepage entering a CMP conduit through a defect.

Figure 127.—Corrosion within a 24-inch-diameter CMP outlet works conduit.

Generally, a CCTV inspection consists of a video camera attached to a self-propelled transport vehicle (crawler). Some crawlers utilize tracks, and others use wheels. The transport vehicle and camera are commonly referred to as a camera-crawler (figure 129). An operator remotely controls both the transport vehicle and camera. The camera can provide both longitudinal and circumferential views of the interior

Figure 128.—A CCTV inspection camera-crawler entering the downstream discharge portal of an outlet works conduit.

of the conduit surfaces. Video images are transmitted from the camera to a television monitor, from which the operator can view the conditions within the conduit. The video images are recorded onto videotape (VHS) or DVD for future evaluation and documentation. The operator can add voice narrative and text captions or notations as the inspection progresses.

Figure 129.—Camera-crawler used for CCTV inspection of conduits. Photo courtesy of Inuktun Services, Ltd.

CCTV inspection equipment was initially used for gas/oil and sewer pipelines. Over the last 10 to 15 years, CCTV inspection has expanded into many applications, such as conduits. In that time period, the robotic equipment used for CCTV inspection has changed significantly. The latest trend for equipment used in CCTV is for modular efficiency (interchangeable components), allowing greater versatility and a wider range of applications. The benefit of modular design is the reduction of added costs required for "application-specific" equipment and "custom designs."

Depending on the model, camera-crawlers used in conduits with very small diameters (about 4 to 14 inches) have cameras with some pan, tilt, and zoom capabilities, a wide range of tether pulling capacity (200 to 1,000 feet), and some steering capabilities. Camera-crawlers used in conduits with diameters of 15 inches or larger are steerable, have a greater cable tether-pulling capacity (500 to 1,500 feet), and have cameras that can provide a wider array of optical capabilities, including pan, tilt, and zoom. As the technology of CCTV inspection equipment advances, greater tether lengths and optical capabilities will become available. Actual tether limits obtainable in the field, vary greatly, depending upon a number of factors, such as conduit diameter, bends, invert slopes, and existing invert conditions, such as sediments, mineral encrustations, and bacterial growths.

In large diameter conduits, the video camera can be attached to a scissor mechanism mounted to the transport vehicle. The scissor mechanism, controlled by the operator, can raise or lower the video camera as needed for inspection. In addition, the video camera usually has a high powered zoom, which can be used to provide closeup views of areas that might be difficult for the transport vehicle to get near. These features allow examination of very large conduits with diameters as large as 40 or 50 feet.

If required, some models of camera-crawlers allow for the attachment of retrieval tools, such as alligator clamps, grippers, and magnets. These tools can be used to remove light debris or damage. The attachment of any type of retrieval tool will require additional clearance within the conduit to operate the retrieval tool. Some models of crawlers have robotic cutters attached to them. These cutters can be used to remove debris or protrusions in concrete, steel, or reinforcement. Most camera-crawler systems are portable and can be carried to conduit access locations (figure 130). The use of an all-terrain vehicle (ATV) may be beneficial for transport of equipment in difficult access areas.

Sometimes the conduit is too small and a transport vehicle cannot be used, or obstructions/invert conditions exist that prevent the transport vehicle from traversing the conduit. For these types of situations, a small color video camera (1.5 to 3 inches in diameter) with maximum pressure depth ratings up to 1,000 feet of water can be used. Figure 131 shows an example of this type of video camera. This video camera can be attached to metal or PVC poles (commonly referred to as push poles) and manually pushed up the conduit. Push poles are normally used for straight sections of conduit. The use of push poles for advancement is generally limited to about 400 feet of conduit length. If bends exist in the conduit, a flexible snake device (spring steel wire, coiled wire, or flexible polypropylene-jacketed fiberglass push rod) can be used instead of the push poles. A coaxial cable connects the video camera to a video cassette recorder and television monitor. Snake devices are generally limited to about 75 to 200 feet of conduit length.

Figure 130.—Most CCTV inspection equipment is portable and can be carried to conduit access locations.

Figure 131.—A small color video camera used for CCTV inspection.

The quality and adaptability of CCTV inspection equipment can vary greatly, depending on the requirements of the inspection. Any company or contractor selected to perform a CCTV inspection should have a wide range of available equipment for differing site conditions. No CCTV inspection equipment exists that is fully adaptable for all conditions, and a variety of crawler configurations and cameras may be required.

Camera-crawler inspection equipment is expensive to purchase, operate, and maintain. The environment being inspected is typically harsh and can pose many hazards and obstructions. Although rare, camera-crawler inspection equipment can become lodged in small diameter conduits if adverse offsets or obstructions exist. If camera-crawler inspection equipment becomes lodged within a conduit, it can partially block the conduit, reducing its discharge capacity. Also, due to the harsh environment, this type of inspection equipment can experience breakdown while operating within the conduit. The retrieval process for removing a lodged camera-crawler can be expensive and time consuming. If the camera-crawler inspection equipment becomes stuck in totally inaccessible portions of a conduit, complete abandonment and loss of the equipment is possible. For this reason, the operator of any inspection equipment must be very experienced and have a clear understanding of the capabilities and limitations of the equipment. The operator must be very cautious and should not push the equipment beyond retrievable limits. The ability to recognize inspection limitations is based largely on the operator's skill and prior experience. The operator must have a thorough understanding of potential dam safety defects, conduit materials, and obstructions within the conduit. Operators must understand that conduits within embankment dams are not like sewers, where only a limited amount of overburden typically exists and where excavation could facilitate camera-crawler retrieval. A conservative approach to inspection is best advised.

Experience with CCTV inspection has shown that past conduit design practices did not always allow for accommodation of equipment used for CCTV inspection. Also, certain configuration of entrance and terminal structures may not allow access for CCTV inspection due to existing trashracks, bends, baffles, etc. The design of any new conduit or the modification of an existing conduit should incorporate features to allow for complete inspection using CCTV inspection equipment. For an example of a conduit inspection using CCTV equipment, see the Pasture Canyon Dam case history in appendix B.

The success of performing a CCTV inspection depends upon the quality of the equipment and the experience of the operator. A CCTV inspection usually requires a two-person crew consisting of an operator and cable reel handler. Additional crew members may be required in difficult access locations. Guidance to consider in performing a CCTV inspection includes (Cooper, 2000, pp. 4-5):

- *Light.*—The amount of light is critical to the success of the inspection. Without the proper amount, areas of concern cannot be observed clearly enough. Lack of clarity hinders making definitive conclusions as to the integrity of the conduit. Also, the larger the diameter of the conduit, the more light that is needed. A trial-and-error procedure may be required to obtain sufficient light intensity. The ability to vary light intensity and control glare is an important feature to consider.

- *Camera.*—The video camera should be able to pan and tilt and also be capable of looking straight ahead. Zoom capabilities allow for close up viewing. Not all inspections involve horizontal conduits. Inspections of vertical drops are sometimes required. The video camera should be able to accommodate different conduit diameters, shapes, and orientations.

- *Footage meter.*—A footage meter should be superimposed on the videotape. This meter makes identifying specific locations within the conduit much easier. In lieu of a footage meter, the operator should verbally record on the videotape the location of the camera-crawler by measuring the length of cable tether.

- *Compass*.—A compass unit will provide azimuth and inclination readings superimposed on the videotape. This will assist in determining conduit alignment. However, a compass unit likely will not work in a steel conduit.

- *Narration*.—All inspection videotapes should include narration by the operator. The operator should describe in detail what is being seen. Narration should note any deposits, changes in the slope of the invert, condition of conduit joints, areas of deterioration, changes in shape, etc.

- *Drawings and photographs*.—Copies of all available design and/or as-built drawings of the embankment dam and conduit should be onsite during the CCTV inspection for immediate reference and confirmation of details and features observed during the inspection.

- *Measurements and data collection*.—The inspection and the technical evaluation will be greatly enhanced if the following data are collected at the time the CCTV inspection is performed: reservoir water level, any relevant data on nearby piezometer levels, history of past operations, and time/date.

- *Videotape library*.—The operator and other inspection personnel should review all previous inspection videotapes (if available) prior to doing the CCTV inspection. This will provide a baseline reference, so the rate of any continuing deterioration can be evaluated.

An important part of any CCTV inspection is the technical evaluation of the conditions observed during the inspection. A qualified professional engineer experienced in the design and construction of conduits should perform this evaluation. Interpretation of the results of the CCTV inspection should not be left to inexperienced personnel. The correct determination of conditions within the conduit is crucial in understanding potential failure modes involved. Many years may pass before the opportunity to perform another CCTV inspection is available. The engineer should prepare a report of findings (ROF), which documents all problem areas observed and recommends future actions. The ROF should also include pictures captured off the videotape or DVD showing areas of concern, a drawing or sketch showing the limits of the CCTV inspection, additional informational drawings if needed, and a detailed summary or log of observations that corresponds with time and linear footage on the videotape. Figure 132 shows a picture captured from videotape.

Other innovations in inspection systems are under development for sewers and for the oil and gas industry. These systems may eventually prove applicable to conduit inspection. These systems involve state-of-the-art laser scanners (digital imaging), and gyroscope technology. Laser scanner systems allow the operator to see the total conduit surface with color coding of conduit defects on a digital computer image.

Figure 132.—A joint has separated in the steel pipe of this outlet works.

Data processing and report preparation are completed using a manufacturer's proprietary software. Currently, laser scanners are not readily adaptable for conduit inspection, since they have some difficulties identifying infiltration, corrosion, and conduit ovality. Laser scanners also are limited to conduits in the range of 8 to 24 inches in diameter. Inspections utilizing laser scanners generally cost 50 to 75 percent more than for CCTV. However, the major benefit of laser scanners is the ability to produce a digital record, which reduces the subjective interpretation of results. Computerized evaluation will gain wider acceptance as a reliable inspection and evaluation tool as further technological advancements are made (Civil Engineering Research Foundation, 2001).

9.6 Cleaning of conduits

Small, inaccessible conduits are especially vulnerable to plugging issues. Cleaning is usually only an issue where man-entry is not possible. If a conduit requires cleaning, it should only be done after careful consideration of the potential effects on known or suspected deterioration within the conduit. The basic philosophy used in the cleaning of conduits should be to "do no harm." This means a very cautious approach is required for cleaning of conduits.

9.6.1 Reasons for cleaning

- *Inspection*.—Cleaning may be required to allow for operation of CCTV inspection equipment within the conduit.

- *Construction*.—Cleaning of the existing conduit may be required as part of the selected renovation method; see chapter 12 for renovation methods requiring cleaning of the existing conduit.

- *Maintenance*.—Cleaning may be required to improve the flow capacity within the conduit due to hard deposits, bacterial growths, sediments, or debris that may have collected in the conduit. Periodic operation of the conduit will flush out many of these types of collections. However, infrequent operation or nonoperation may allow for continued buildup of these collections.

    1. *Hard deposits*.—If a conduit has not been periodically operated, certain mechanisms may develop within the conduit. In conduits experiencing seepage into the conduit through a joint, solid deposits may develop where the seepage water evaporates. These deposits often contain calcium carbonate, which precipitates out of solution as the mineral calcite. Calcite will form deposits when the calcium ion and bicarbonate ion concentrations in the water increase to the point where they exceed the capacity to dissolve in water. Hard deposits of calcium carbonate precipitate may develop when the seepage water evaporates.

    2. *Bacterial growths*.—If a conduit has not been periodically operated, certain bacterial growths may develop within the conduit. Bacterial growths are common and can develop under a variety of conditions. Bacterial growth can occur anaerobically (without oxygen) and aerobically (with oxygen). Most of the time, bacterial growths are soft and easy to remove, but in some situations, these growths can become hard and mineralized. Aerobic bacterial growth can also create hazardous conditions by depleting the oxygen in the air of a confined space.

    3. *Sediments and debris*.—If a conduit does not discharge water completely out of the system or if the discharge channel is adversely sloped, water may partially or completely submerge the exit portal. If this occurs, sediments and debris can back up into the conduit, resulting in sediment deposits or debris accumulation.

9.6.2 Cleaning methods

The improper use or the selection of incorrect cleaning equipment may cause additional damage to a deteriorating conduit and further degrade its structural integrity. The type of conduit material (i.e., concrete, plastic, or metal) must be considered in selecting the appropriate cleaning method. Some conduit materials (such as CMP) are much more prone to defects. Cleaning of inaccessible conduits should only be considered after CCTV thoroughly inspects the conduit. If a deteriorating conduit is cleaned without the benefit of CCTV inspection, the conduit may become unknowingly damaged.

Indications of obstructions within the conduit may include reduced outlet flow capacity, etc. If obstructions are found during the CCTV inspection, the method of cleaning can be evaluated and a preferred method selected. Sometimes, CCTV inspection and cleaning are done on the same day. Some cleaning services have limited CCTV inspection equipment. Any cleaning should be attempted only in the presence of qualified and experienced staff representing the agency/owner of the embankment dam. Complete documentation (including photographs) of all activities at the site is highly recommended.

The success of any conduit cleaning depends upon accessibility, type of cleaning required, and the cleaning method used. A variety of cleaning methods are available:

- *Flushing*.—If debris and sediments are not significant, adequate cleaning may be obtained by merely flushing the conduit with water. Flushing can be accomplished by opening a gate or valve and allowing water to flow through the conduit or by inserting a flexible hose and pumping water into the conduit. In many cases, volume and low pressure is all that is needed to adequately clean the conduit.

- *Pressure washing*.—Pressure washing (figure 133) involves the use of a flexible hose attached to a metal nozzle that directs jets of water out in front of it to loosen debris and sediments in the conduit. The jet is created by a shaped restriction in the flow channel that forces water to accelerate and converts potential energy (pressure) into kinetic energy (velocity). The nozzle is propelled forward by reverse angle jets. The reverse angle jets also push debris and sediments backwards toward the end of the conduit, where the flexible hose exits. Pressure washing is best suited where biomasses or mineral encrustation are to be removed. The pressure selected for cleaning should fully consider the condition of the types of conduit material, age, and type of joints. The lowest possible pressure that effectively cleans the conduits should be used. The jets on the nozzle should be angled no more than about 30 degrees, so the jets are not aimed directly at the conduit wall. The nozzle should be kept rotating and moving and should not be allowed to remain in one spot during jetting.

- *Mechanical*.—Mechanical cleaning utilizes rotating brushes.

- *Cleaning pig*.—Cleaning pigs have wire brushes to scrape the walls of the conduit. A variety of brushes are available, depending on the type of cleaning required and the existence of any coatings on the interior surface of the conduit. Cleaning pigs are generally available in diameters up to 48 inches.

Figure 133.—Pressure washing cleaning head.

Again, it should be strongly emphasized that any cleaning should be given considerable thought before proceeding, to avoid causing any damage, or worsening existing defects within the conduit.

In some situations, minor cracks or joints experiencing seepage may eventually seal themselves by calcite deposition. This process occurs when calcite precipitates out of solution and forms a deposit. Deposition may occur as the seepage evaporates, leaving the calcite behind. Calcite deposits typically mineralize and harden over time. Figure 134 shows a conduit joint where calcite deposition has sealed a minor leak. If inspection shows locations within a conduit where this has occurred, cleaning with high pressure could remove enough of the calcite deposition to cause seepage to begin again. This possibility needs to be carefully considered prior to performing any cleaning operations within the conduit.

9.7 Forensic investigation

To better understand and to provide further knowledge concerning the failure mechanisms resulting from the internal erosion or backward erosion piping within an embankment dam, forensic investigation should be considered. Although traditionally a forensic investigation is conducted to establish the failure mechanism for legal cases, a detailed investigation can be very helpful in determining the causes of failures and to provide insight into design changes to reduce failures in the future.

For projects where a failed conduit is being removed and replaced, close coordination between designers, embankment dam owner, and the contractor will be required to preserve the soil adjacent to the conduit. The investigation team should

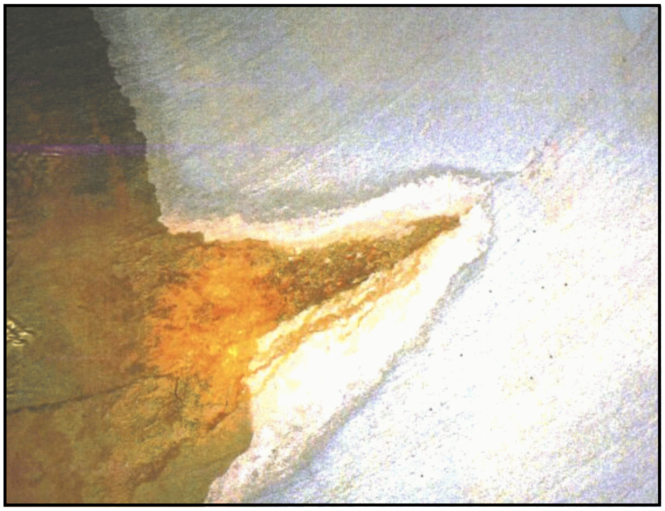

Figure 134.—Calcite deposition has sealed this leak at a conduit joint.

consist of experienced geotechnical and civil engineers, geologists, surveyors, and construction personnel. All anticipated items of interest (e.g., voids) should be clearly communicated to all parties involved prior to the commencement of embankment excavation. Test pits are usually excavated along the conduit, extending a specified depth below the bottom of the conduit. The contractor must take care to prevent damage to in-situ conditions before the investigation team can document them. Figure 135 shows an outlet works conduit excavation during a forensic investigation. Figure 136 shows how polyurethane grout flowed through the backfill surrounding an outlet works conduit during joint sealing operations. Close coordination between the forensic team and contractor were required in order to preserve this information for study.

Documentation of the conditions encountered is essential to be able to recreate the events leading to the failure. A surveyor with a transit, theodolite or total station, and one or more assistants with survey rod or reflector target should be available to precisely document the location (position and elevation) of items of interest. Numerous photographs should be taken, even of items that do not appear to have contributed to the failure in case they are needed later, since the soil structure surrounding the conduit will likely be destroyed by the investigation and the information will be forever lost, if not carefully documented.

A JHA should be prepared for all onsite forensic investigations. See section 9.4 for details on preparing a JHA. For details of a forensic investigation, see the Annapolis Mall Dam case history in appendix B.

Figure 135.—An outlet works conduit is being excavated during a forensic investigation. The top of an antiseep collar is exposed on the left side of the figure.

Figure 136.—Close coordination between the forensic team and contractor allowed for careful study of how polyurethane grout injected into the deteriorated joints of a conduit flows through surrounding backfill. In this case, the forensic investigation showed the injection of grout was relatively successful in sealing the joints of the conduit.

9.8 Instrumentation and monitoring

Instrumentation and monitoring are performed for three distinct reasons:

1. To aid in the evaluation of water pressure conditions surrounding a conduit and detect signs of a problem (i.e., first identification). Key detection elements include:

    a. Visual monitoring for unusual settlements or deformations above the conduit

    b. Visual monitoring for seepage emerging in or near the downstream end of the conduit

    c. Inspection of the interior of the conduit

    d. Structural measurement points in the conduit (where possible)

    e. Embankment measurement points in the vicinity of the conduit alignment

2. To gain a better understanding of an already detected problem for use in evaluation and design of a remediation

3. To monitor embankment and foundation water pressures during and following conduit remediation

Instrumentation in a conduit or embankment dam furnishes data to determine if the structure is functioning as intended and to provide a continuing surveillance of the structure to warn of developments that could endanger the safety of the embankment dam facility. Conduits are not normally instrumented unless there is a specific concern due to known adverse foundation conditions or other unusual circumstances. The means and methods available to monitor an emergency event or condition that could lead to a embankment dam failure include a wide spectrum of instruments and procedures from very simple to very complex. The need for instrumentation designed for monitoring potential and/or existing deficiencies at existing embankment dams must take into account the threat to human life and property downstream of the dam. Thus, the extent and nature of the instrumentation depends not only on the complexity of the conduit and embankment dam, and the extent of the deficiency being monitored and the size of the reservoir, but also on the potential for loss of life and property damage downstream of the dam (FEMA, 1987, p. 51; Reclamation, 1987b, pp. 1-3).

An instrumentation program should involve instruments and evaluation methods that are as simple and straightforward as the project and situation will allow. Instruments selected for use should be accurate, precise, and provide for repeatability of measurements. Beyond that, the designer and embankment dam owner should make a definite commitment to an ongoing monitoring program. If not, the installation of instruments will probably be wasted. Increased knowledge of any deficiency and emergency condition of the embankment dam acquired through an instrumentation and monitoring program is extremely useful in determining the cause of the deficiency, the necessary or probable remedy, and monitoring during and following corrective actions. Involvement of qualified personnel in the design, installation, monitoring, and evaluation of an instrumentation system is of prime importance to developing and achieving a successful and meaningful instrumentation and monitoring program.

A wide variety of devices and procedures are available for use in monitoring the behavior of and deficiencies along a conduit and at an embankment dam. Table 9.1 provides a listing of potential deficiencies and conditions and their causes that could be encountered along the alignment of a conduit. The table also provides a brief description of where the condition could be encountered and the instrumentation that could be used to monitor the condition. Additional discussion of each measurement is provided in the following sections. Most of these measurements are typically done for embankment dam concerns. However, there is some applicability to conduits. Further information or instrumentation and monitoring is available on ASCE's *Guidelines for Instrumentation and Measurements for Monitoring Dam Performance* (2000).

9.8.1 Structural deformation

Structural deformation of a conduit could lead to crack development or joints opening up along the alignment of the conduit. These deficiencies could result in the potential for internal erosion or backward erosion piping of embankment dam materials into or along the exterior of the outlet conduit. In the case of water seeping into the conduit through open joints or cracks, an unprotected exit point for the seepage exists, which could allow for the internal erosion or backward erosion piping of embankment dam materials into the conduit. For pressurized conduits, open joints or cracks in the conduit could allow for the saturation of the embankment dam materials around the conduit under a high seepage gradient condition, which could also lead to the internal erosion or backward erosion piping of embankment materials. Structural deformations may result from foundation settlement, lateral deformation of the embankment slopes above or below the conduit, or a collapse of the conduit due to a structural defect in the conduit or growth processes within concrete, usually resulting from alkali-aggregate reaction.

Table 9.1.—Instruments used for monitoring of conduits (ASCE, 2000)

Property measured	Cause	Measurement location	Typical instruments
Structural deformation	Vertical-settlement	Joints, alignment	Strain gauges, extensometer, joint meter, survey profiles
	Lateral-slope movement	Joints, alignment	Strain gauges, extensometer, joint meter, survey profiles
	Expansion-autogenous growth (alkali-aggregate reaction)	Any location of interest	Strain gauges, extensometer
Uplift pressures	Shallow structure and high groundwater	Within embankment dam	Piezometers, observation wells
		Within foundation	Piezometers, observation wells
Seepage quantity	Internal erosion or backward erosion piping	Any location of interest	Calibrated container, weir, flume, flow meter
Horiz. and vert. movements	Internal erosion or backward erosion piping	Any location of interest	Survey, staking, probing
Water quality	Internal erosion or backward erosion piping	Any location of interest	Turbidity meter, jar samples
Reservoir water level and flows	-	Reservoir or outlet channel	Elevation gauge

Structural deformation of the conduit can sometimes be first detected by defects noted on the surface of the embankment dam in the form of depressions, bulges, and cracks. For guidance on horizontal and vertical movement of embankment dams, see section 9.8.4.

9.8.2 Uplift pressures

Where the conduit is shallow and groundwater is high, uplift pressures on the conduit may be sufficient to the push the conduit or associated structures upward.

This movement could cause cracks to develop or joints to open up in the conduit similarly as discussed for structural deformations of conduits. Conduits in sandy and silty soils also could be susceptible to damage, if the soils are liquefiable.

If this condition is suspected along the alignment of a conduit, instrumentation, such as observation wells, could be placed near the conduit alignment. Installation of instruments to measure uplift pressures cannot be relied upon as the sole means of detection of these problems. Rather, instruments to measure pore pressure should only be placed as a means of providing information on the general water pressure conditions at the location of interest. If piezometers are installed after the conduit and embankment dam have been constructed, caution should be used in considering drilling close to a conduit, as low stress zones with the potential to hydraulically fracture often exist as a result of the structure. In a zoned embankment dam, locating the instrument in a zone other than the core should be considered.

Installation of instruments to measure pore pressure resulting from internal erosion or backward erosion piping cannot be relied upon as the sole means of detection of these problems. Rather, instruments to measure pore pressure should only be placed as a means of providing information on the general water pressure conditions within the embankment dam.

Designers should note that a trend is growing in the industry to eliminate the installation of instrumentation within the cores of embankment dams during construction. The performance of embankment materials is well understood, so there is little need to repeat past research. Also, it is very unlikely that the instrument will be placed in the correct place to detect a chance problem. Furthermore, it is recognized that the mere act of installing the instrument can adversely affect the quality of the embankment dam. Vertical risers associated with cables and tubing can disrupt the proper flow of compaction equipment. Instrumentation trenches can potentially introduce flaws that could lead to concentrated leakage.

Installing instruments in the cores of existing embankment dams to detect particular problems should still be considered. The instrument can be placed within the embankment dam by drilling techniques, but specific techniques that limit the potential for fracturing the embankment dam should be employed. Drilling into the embankment dam with techniques that use water or air to remove cuttings should be avoided, because blockages within the drill holes have been known to cause the buildup of high fluid pressures leading to fractures in the earthfill. For guidance on drilling within embankment dams, see section 14.3.1.

9.8.3 Seepage quantity

Seepage along a conduit or through an embankment dam is a valuable indicator of the condition and continuing level of performance of an embankment dam.

Particular attention should be given to seepage exiting ground conduits and the quantity of seepage flowing out of conduits. The quantity of seepage entering a seepage collection system is normally directly related to the level of the water in the reservoir. Any sudden change in the quantity of seepage collected without apparent cause, such as a corresponding change in the reservoir level or a heavy rainfall, could indicate a seepage problem. Similarly, when the seepage becomes cloudy or discolored, contains increased quantities of sediment or changes radically in chemical content, a serious internal erosion or backward erosion piping problem may be developing. Moisture or seepage at new or unplanned locations on the downstream slope or below the embankment dam also may indicate a seepage problem. Seepage should be monitored regularly to determine if it is increasing, decreasing, or remaining constant as the reservoir level fluctuates. A flow rate not changing relative to a reservoir water level can be an indication of a clogged drain, internal erosion or backward erosion piping, or internal cracking of the embankment dam.

Seepage may be measured with weirs of any shape, such as a V-notch, rectangular, or trapezoidal; flumes, such as the Parshall flume; water exiting a pipe measured with a stopwatch and bucket; and flowmeters. When a new seepage area that produces measurable flow is identified at an embankment dam, the seepage should be monitored and, in some cases, measured. A qualified engineer should promptly evaluate each new seepage area. In some situations, a change in the seepage regime precedes failures. The flow should first be confined and directed away from the embankment dam by excavating drainage channels or ditches. Then, the quantity of seepage can be measured by creating a large enough drop in the drainage channel to install a pipe, weir or flume or to facilitate the measurement of the flow by means of a stopwatch and bucket. The integrity of the seepage measurement devices should be maintained so that seepage does not bypass the device and the device is kept clear of obstructions.

Points where seepage measurement devices are added are often a good location to measure the amount of sediment that may be carried in the seepage. Sediment transport is often a sign of internal erosion or backward erosion piping failure modes. Providing an area adjacent to a weir where water flow is stilled can allow some of the sediment in the water to fall out and collect with time.

Seepage into conduits should also be monitored where it is determined to be important. Note that if the seepage into a conduit is transporting material, operations of the conduit may be transporting material out of the conduit. Frequently, the highest seepage gradient at a site is associated with seepage into a nonpressurized conduit. For this reason, inspection of the conduit is important. The internal erosion and backward erosion piping processes are frequently intermittent, and in many cases, the transport of materials in the seepage is sporadic. Inspection should look for signs of deposits, as well as clarity of the seepage.

9.8.4 Horizontal or vertical movements

Movements in the embankment dam or foundation have been known to damage conduits and create potential internal erosion backward erosion piping conditions. On soft, stiff, or weak foundations, it is important to realize that the conduit will be deformed over its length as it follows the deformations of the foundation. Conduits within large embankment dams have also experienced distress on rock foundations over areas where the foundation stiffness varies greatly due to the presence of shears, faults, or soft zones. Designs should account for these conditions, and consideration should be given to the possibility of distress in monitoring for horizontal and vertical movement. When monitoring a crack in the conduit, crack meters are also used to determine if the crack is formed due to temperature and shrinkage, or due to slope movement in the embankment dam.

Movements of embankment dams are generally caused by stresses induced by reservoir water pressure, unstable slopes (low strength), low foundation strength, settlement, thrust due to arching action, expansion resulting from temperature change, and heave resulting from hydrostatic uplift pressures. Monitoring displacements can be helpful in understanding the normal behavior of an embankment dam and in determining if a potentially hazardous condition is developing. The displacements, both horizontal and vertical, are more commonly measured on the surface of the embankment dam. Measuring displacements of points on the surface is usually accomplished by conventional surveying methods and the installation of permanent surveying points/monuments.

External vertical and horizontal movements are measured on the surface of embankment dams through the use of level and position surveys of reference points. Reference points may be monuments or designated permanent points on the embankment dam crest, slopes, or toe of the embankment dam or on an appurtenant structure.

For saturated areas on the downstream slope of an embankment dam, the perimeter of the hole or wet area should be surveyed to determine the extent of the area. As a minimum, the perimeter of the hole or wet area can be staked out with metal fence posts or wooden stakes (figure 137) and the length, width and location of the wet area recorded and photographed for future reference. For saturated areas on the embankment dam face, the degree of wetness should also be estimated and recorded, such as "boggy," or "surface moist but firm underfoot." Any flow of water from the wet area or into a sinkhole should be measured, if possible, and/or estimated and recorded. See section 11.3 for guidance on actions involving sinkholes.

Detecting surface evidence of slope instability is of primary importance. Such evidence includes slope bulging, sagging crests, foundation heave at or beyond the toe, and lateral spreading of foundations and embankments. During the operation of

Figure 137.—The perimeter of a wet area at the downstream toe of an embankment dam located with wooden stakes.

the embankment dam, measurements of lateral transitional movements from forces caused by pool loading, reservoir drawdown, gravity, and the effects of seepage pressures are required to help evaluate safe performance of the embankment dam.

The measured internal movements of embankment dams consist principally of vertical movements and relative horizontal movements caused mainly by the low shearing strength or the long term creep strain of the foundation or embankment materials. Internal movements generally result in external movement of the embankment dam's crest or side slopes. Internal displacement-monitoring plans can be very complex and expensive. Internal movement-monitoring devices consist of baseplates, inclinometers, tiltmeters, extensometers, and shear strips.

In the event of an emergency situation at a damsite, some relatively simple devices can be installed to monitor embankment dam movement, such as cracks and slides. If a small crack is observed on the embankment dam, it may be very important to know if the crack enlarges. An easy method of monitoring the crack is to drive steel rebar or wooden stakes on both sides of the crack to monitor additional separation and vertical displacement on one side of the crack relative to the other side. Also, the ends of the crack should be staked to determine if the crack is lengthening. This scheme can be used to monitor both longitudinal and transverse cracking.

Another special situation, which would require immediate attention, is the development of a slide on one of the embankment dam slopes. A simple yet reliable method to measure movement of the slide area would be an alignment method. A strong wire is stretched across the slide and tied to pins outside of the slide area. At intervals along the wire, pins are driven into the slide mass. If additional movement occurs, the amount is directly determined by measuring the distance between the pins and wire.

If a defect is suspected in a conduit, an inspection using man-entry or CCTV methods is required.

9.8.5 Water quality

Seepage comes into contact with various minerals within the soil and rock in and around the embankment dam and its foundation. This can cause two problems: the chemical dissolution of a natural rock, such as gypsum, or the internal erosion of soil. Dissolution of minerals can often be detected by comparing chemical analyses of reservoir water and seepage water. Such tests are site specific; for example, in a limestone area, one would look for calcium and carbonates, and in a gypsum area, calcium and sulfates. Other tests, such as pH, might provide useful information on chemical dissolution.

Internal erosion and backward erosion piping can be detected by comparing the turbidity of reservoir water with that of seepage water. An increase in turbidity may indicate internal erosion and backward erosion piping of the materials. A method of comparing observations is to collect a sample of the water in a large glass jar, which is marked with the date and location the sample was collected and retained for future comparison. Another jar should be used for the next water sampling. Glass jars should be filled periodically with the seepage flow and set aside to allow for any material to settle out. By comparing jars, one can determine if material is moving and if it is increasing. However, this method does have some limitations, since material transport is not usually continuous and can be episodic. For certain tests, such as iron bacteria, the sample must be kept refrigerated until tested.

The frequency of instrument readings or making observations at an embankment dam depends on several factors and could include the following items:

- Relative hazard to life and downstream property damage that the failure of the embankment dam represents
- The importance of the instrument in detecting a failure mode
- The nature and urgency of an emergency condition being investigated and monitored at the damsite

- Height or size of the embankment dam
- Volume of water impounded by the embankment dam
- Age of the embankment dam
- History of the performance of an instrument
- Frequency and amount of water level fluctuation in the reservoir
- Frequency of staff visits for other reasons, such as operations

In general, as each of the above factors increases, the frequency of the monitoring should also increase. For example, very frequent (even daily) readings should be taken during the first filling of a reservoir and more frequent readings should be taken during emergency events and high water levels in the reservoir under storm and seismic events. As a rule of thumb, simple visual observations should be made during each visit to the damsite. In the event of an emergency at the damsite under potential dam failure, and/or imminent dam failure, the frequency of the instrumentation monitoring and visual observation could vary from weekly to daily to hourly or less, depending on the nature and urgency of the situation. Lights are frequently employed during critical times to facilitate nighttime observations. In almost all cases, the consequences would be greatest if failure occurred at night.

Documentation and recording of the instrumentation readings and data and visual observations are very important in the monitoring and evaluation of an emergency situation at a damsite. The documentation should include tabulations of the instrumentation readings and data, written documentation of the visual observations and findings, and photographs of key elements or features of the investigation at the site during the occurrence of an emergency. The documentation should include the instrumentation description, location and readings, the date and time of the readings and observations, the reservoir water surface and tail water levels, the releases being made from the embankment dam, weather conditions, evaluation of the present condition of the embankment dam and comparison of previous information, and the recommendation for monitoring and/or remedial measures to correct the deficiency.

Proper training of those who are to inspect and take readings at the embankment dam is very important. Training will ensure that the inspection staff are familiar with the proper method to read the instruments, what other data and information from the site is necessary, what anomalous behavior might look like, how to report normal and unusual conditions, and what steps need to be taken in an emergency.

9.8.6 Reservoir water level and flows

The reservoir water surface level is a key item to record when measuring other instrumentation at a damsite and should be measured and recorded each time the embankment dam is visually inspected, and when other instrumentation is observed or read. The reservoir water level is also used when evaluating the information provided by the other instruments at the site. For instance, the amount of seepage exiting the embankment dam as it relates to reservoir water level is often crucial. A pattern of increasing seepage at the same reservoir level is cause for concern. Water levels may be measured by simple elevation gauges, such as a staff gauge or numbers painted on permanent, fixed structures in the reservoir, or by complex water-sensing devices. Reservoir flow release quantities are often computed from the depth of flow in the conduit or exit channel or by predetermined conduit discharge rating tables/curves. During an emergency, it is important to monitor the water level in the reservoir and the downstream pool regularly, along with the quantity of water being released from the embankment dam's outlet works and spillway.

Chapter 10

Evaluation by Geophysical and Nondestructive Testing

Geophysical and nondestructive testing (NDT) techniques can be used to investigate the condition of a conduit directly or indirectly by providing data on the condition of the conduit and the surrounding embankment dam. These techniques are used to detect flaws, defects, deterioration, and other anomalies that could lead to a failure and do not disturb the feature being evaluated or tested. The most common techniques used include:

- Seismic tomography
- Self potential (SP)
- Electrical resistivity
- Ground-penetrating radar (GPR)
- Sonar
- Ultrasonic pulse echo and ultrasonic velocity
- Mechanical and sonic caliper
- Radiography
- Surface hardness

Depending on the particular situation, some techniques are more effective than others. The selection of the applicable technique(s) requires evaluation of the type of information needed, the size and the nature of the project, the conditions existing at the site, and any impacts that may result to the structure from performing the technique. These techniques require trained and experienced personnel to perform and interpret the results. The various applications for these techniques are summarized in table 10.1. The following sections briefly discuss these techniques.

Table 10.1.—Geophysical and NDT techniques

Investigation methods	Problem identification	Comments
Seismic tomography	Voids along outside of the conduit	Best results when inside of conduit is accessible. Good results may or may not be obtained, if sources and receivers are on outside. Target resolution is very frequency-dependent, and will strongly depend on composition of zones in a zoned embankment. Air-filled voids easier to detect than water-filled voids.
Self-potential	Seepage along outside of the conduit	Provides direct detection of seepage. Data interpretation may be difficult. Data are generally acquired at high- and low-pool conditions, for comparison.
Electrical resistivity	Locations of large buried metallic objects, possibly indicating seepage zones	Available equipment can acquire large volumes of data, interpretation and nonuniqueness may be an issue. Independent ground truthing is advisable.
Ground-penetrating radar	Locations of suspected voids, and delaminations	Depth of penetration limited in clay soils; good technique for concrete structures; can be used to image from inside of the conduit outwards. Air-filled voids easier to detect than water-filled voids. Independent ground truthing is advisable.
Sonar	Displacement and delaminations within conduit	Provides a direct measure of the interior condition of the conduit.
Ultrasonic pulse velocity	Concrete quality, thickness, and/or delamination	Limited to about 1.5 ft of thickness when access is limited to one side. With access on both sides, concrete quality can be evaluated for much thicker sections.
Ultrasonic pulse echo	Concrete quality, thickness, and/or delamination	Depth of investigation limited to 1 ft. Requires access to only one side of surface to be investigated. Can be used underwater (with waterproof transducers). Considerable judgement/experience required.
Ultrasonic pulse echo	Steel pipe wall thickness	Requires access to only one side of surface to be investigated. Can be used underwater (with waterproof transducers).
Mechanical caliper, sonic caliper	Inside dimensions of conduit	Typically used in conduits 18 inches or larger to detect changes or defects within the conduit.
Radiography (x-ray)	Steel weld integrity	Access to both sides of conduit wall is required.
Surface hardness	Concrete quality	Imprecise measurements of concrete strength.

Additional information on nondestructive testing is available in Molhotra and Carino (2004), USACE's *Evaluation and Repair of Concrete Structures* (1995b), and ACI (1998a).

10.1 Seismic tomography

The seismic tomography method (figure 138) is a noninvasive geophysical method similar to methods applied in medicine, such as ultrasound, computerized axial tomography (CAT) scans, and magnetic resonance imaging (MRI). Seismic tomography uses elastic waves produced by seismic sources implanted around or within boreholes in the embankment dam. Receivers (geophones or accelerometers) installed at other locations on the structure record the generated waves.

Seismic tomography uses the same processing technique as in the medical field, but the image is not as detailed, since sources and receivers cannot be placed at all sides of the embankment dam, and because the frequencies propagated are much lower than those used in medical imaging. However, surface-mounted sources and receivers may be sufficient to discover potential problems within the structure of the embankment dam. Target detection depends strongly on the ability to transmit and receive high frequency seismic energy through the embankment dam, the dimensions of the suspected target, the location of the phreatic surface, and whether the

Figure 138.—Seismic tomography being used on an embankment dam. Photo courtesy of URS Corporation.

suspected voids or stopes are air- or water-filled. Placement of sources and receivers inside the conduit, when accessible, can improve the technique.

The parameters recorded can provide important information about different features that may damage the embankment dam's structure, such as fractures, low density regions, saturated zones, and high stress regions. The results may be presented as cross-section images (figures 139 and 140) of compression (P- [primary]) wave velocity, or of seismic wave attenuation. These properties may be correlated to other engineering parameters of interest, such as possible fractured zones, and potential void areas. For an embankment dam in Maryland (Schaub, 1996, p. 3), the tomographic investigation interpretations revealed that the relative compaction of the earthfill around a CMP spillway conduit ranged from 65 percent to nearly 100 percent. The areas with the lowest interpreted densities were found to be under, along, and above the conduit.

For concrete, high compression (P-) and shear (S- [secondary]) wave velocities indicate competent concrete. Lower velocity values may indicate cracking, deterioration caused by ice and other weathering, alkali reaction, or defects.

10.1.1 Spectral analysis of surface waves

Recently developed geophysical procedures called Spectral Analysis of Surface Waves (SASW) and Multichannel Analysis of Surface Waves (MASW) measure the "dispersion" of surface wave velocities to evaluate material properties. (Billington and Basinger, 2004, p. 4; Park et al., 2001; and Miller et al., 1999). These techniques, termed "indirect methods," since the measurements can be made from one side of a structure, provide estimates of material properties averaged over relatively large distances.

The SASW/MASW techniques can be used on a large scale to evaluate embankment dams, such as for locating possible voids or potential seepage zones along a conduit (Stokoe, 1999, p. 3). On a smaller scale, these techniques can be used to evaluate the quality of conduit materials, such as concrete deterioration and loss of wall thickness due to corrosion.

Use of surface wave data is a powerful technique that allows measurement of soft layers beneath harder layers. This means that SASW/MASW may be able to detect possible voids in the backfill adjacent to a conduit by making measurements from inside of the conduit. (In comparison, the seismic refraction technique generally cannot be used to locate softer layers under harder layers.)

The basis of the methods is measurement of the "dispersion" of Rayleigh type surface waves (USACE, 1995c, p. 3-24). Essentially, surface waves of different wave

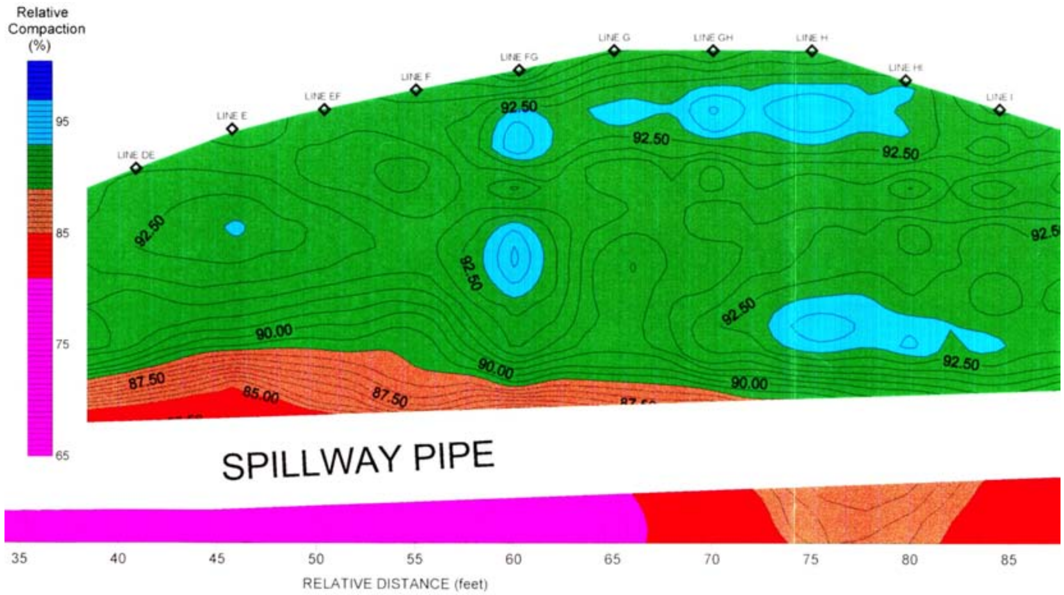

Figure 139.—Seismic tomography profile along failed CMP spillway conduit in an embankment dam.

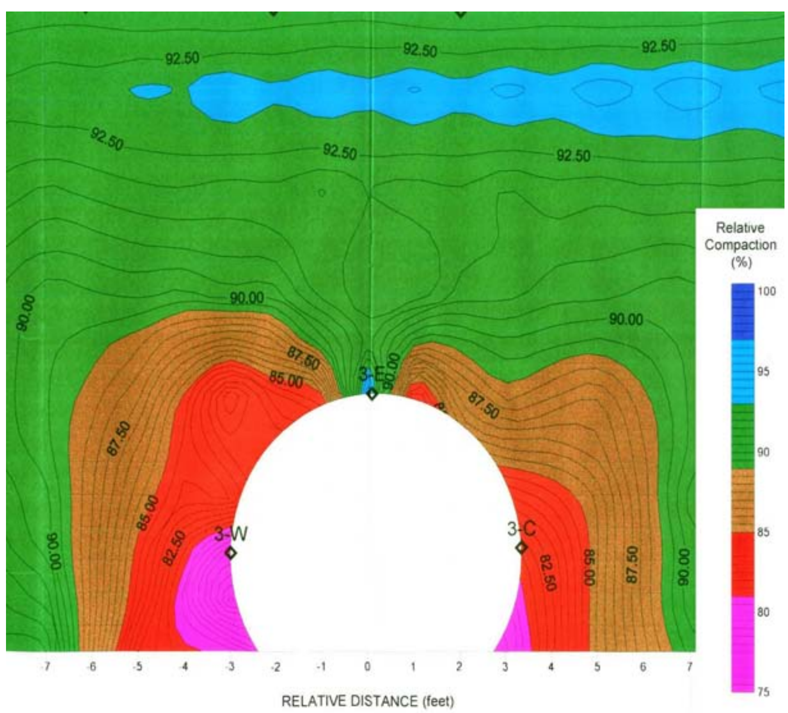

Figure 140.—Typical section from seismic tomography used to identify voids along the outside of a CMP spillway conduit in an embankment dam.

lengths (frequencies) propagate at different velocities through nonhomogeneous materials. This variation in velocity is related to the shear wave velocity and thus shear modulus (Shaw, 2003).

Different equipment is used depending on whether the SASW/MASW technique is to be used for geotechnical analysis of soil (to depths of 3 feet to 300 feet), or for structural evaluation of concrete. The equipment generally consists of two or more geophones, a hammer or other impact device for generating vibrations, and a seismograph or other data collection unit.

Higher wave frequencies and close geophone spacing are used for shallow investigations, and lower frequencies with wider spacing are used for deeper investigations. The field data are later processed with specialized software, such as WinSASW, developed by the Geotechnical Engineering Center of the University of Texas at Austin, or SurfSeis, developed by the Kansas Geological Survey.

10.2 Self potential

Self potential (sometimes referred to as streaming potential or SP) (figure 141), measures the electrical potentials (or voltages) that exist in the ground or within an embankment dam. Flowing water naturally generates these potentials as a consequence of the separation of ions in the seepage water itself. SP is considered to be the only geophysical method capable of direct detection of seepage (Corwin, 2002). Other geophysical methods, such as resistivity, infer the existence of seepage based on other measured parameters.

Theoretically, it is possible to measure these potentials and predict seepage anomalies, such as along a conduit within the embankment dam, up to several hundred feet deep. However, the technique is not widely applied, and few people or contractors can expertly interpret the data. In addition, the measured potential (usually on the order of tens of millivolts) in any area can vary with other in-situ parameters, and with man-induced voltages.

Existing procedures were developed for the USACE's Waterways Experiment Station and published in 1989 (USACE, 1989), and also for the Canadian Electricity Association Dam Safety Interest Group (Corwin, 2002). SP interpretation and modeling computer programs are beginning to be developed along the lines of existing programs available in other geophysical disciplines, such as resistivity and seismic methods.

Canadian Electricity Association Technology, Inc. has published a series of DOS program codes (Corwin, 2002) and the University of British Columbia has available a modeling procedure that runs under Visual ModFlow, and models the SP response

Figure 141.—Collecting self potential (SP) data on the crest of an embankment dam in Virginia to trace the source of observed seepage. The 75-ft high embankment dam had a sinkhole, a sand boil, and several seepage points on the downstream face. This information was used in complement with electrical resistivity imaging data. Photo courtesy Schnabel Engineering.

for a user input distribution of permeability and electrical resistivity parameters (Sheffer, 2002).

Self potential measurements are affected by soil moisture, resistivity, temperature, and other in situ parameters. Therefore, the SP technique should be combined with other methods, such as resistivity or temperature measurement.

10.3 Electrical resistivity

Electrical resistivity technology is relatively well developed and can be a very effective tool for locating large buried metal targets, and other highly electrically resistive or highly conductive targets (Ward, 1990) . The technique, involving an array of electrodes that measure the distribution of voltage applied to the ground, has been used to investigate some embankment dams. However, small changes in measured data can result in very different interpretations. Resistivity interpretations are nonunique, and should be constrained by independent data. Other field parameters (permeability, dissolved minerals, temperature) may need to be measured at the same time. The method is sensitive to interference from nearby metal objects (such as pipes and wires within the embankment dam, or overhead wires and fences).

One- or two-dimensional, or tomographic software can be used to process the data, which can be displayed as color plots of resistivity versus depth. Currently, available field equipment is capable of obtaining and automatically processing large numbers of resistivity measurements, regardless of data quality. Automatic processing can lead to misinterpretation of the data, if the operator does not recognize the problem, or is not familiar with nonuniqueness effects in conducting resistivity surveys.

10.4 Ground-penetrating radar

Ground-penetrating radar (GPR) uses high frequency electromagnetic energy to penetrate below the ground surface (figure 142). An antenna is used to transmit a short duration pulse, which travels through the air and the subsurface until it is reflected back by a change in the dielectrical properties of the material being imaged. The resulting reflections are displayed in sounding or section format, with sections being the far more common display mode. If the GPR profiles are conducted on a close spacing, the resulting data can be treated in a data volume manner, allowing arbitrary slices through a three-dimensional data mass. This three-dimensional technique can be labor intensive to acquire and process.

GPR can be used to locate possible void, stope, or incipient sinkhole areas. However, the depth of penetration of radar waves in soils and concrete depends strongly upon the electrical resistivity of the material in question. Saline pore water and clay-rich soils can severely limit this depth of penetration. Metals are opaque to radar energy, so complete radar wave reflection occurs at metal surfaces, such as steel conduit and rebar. Soils or concrete behind such metallic objects will have shadow zones or other absence of data.

Figure 142.—Conducting a ground-penetrating radar survey across dam crest to locate voids beneath roadway and spillway. Photo courtesy Schnabel Engineering.

Known void areas are extremely useful in "calibrating" a GPR survey at a particular site. Lacking known void areas, core drilling or other direct inspection methods are highly desirable to aid in the GPR data interpretation. GPR data profiles can be difficult to interpret properly, if no site "ground truth" is available. Figure 143 shows a core hole being drilled to reveal voids behind the concrete, and figure 144 shows an example of GPR profiles along a conduit invert.

Because the radar waves travel equally well in all directions, GPR may be used to image from the inside of a (nonmetallic) conduit outwards, along crown, springlines, and invert. Modern GPR equipment is commonly mounted on a cart or pole to allow imaging in the required direction. Note that steel well casings, communication cables, metal buildings, overhead wires, and other cultural features can cause anomalous-looking radar profiles. The GPR interpreter must be aware of the locations of such features at the site.

10.5 Sonar

For inundated conduits with a heavy suspended sediment load and very poor visibility, three-dimensional real-time imaging sonar is advantageous. A rotating sonar transducer mounted on a sled, crawler vehicle, or ROV can be used to scan and record the condition of a conduit (Sonex Corporation, 2002). Since the times

Figure 143.—A core hole is being drilled to reveal voids behind the concrete in this conduit.

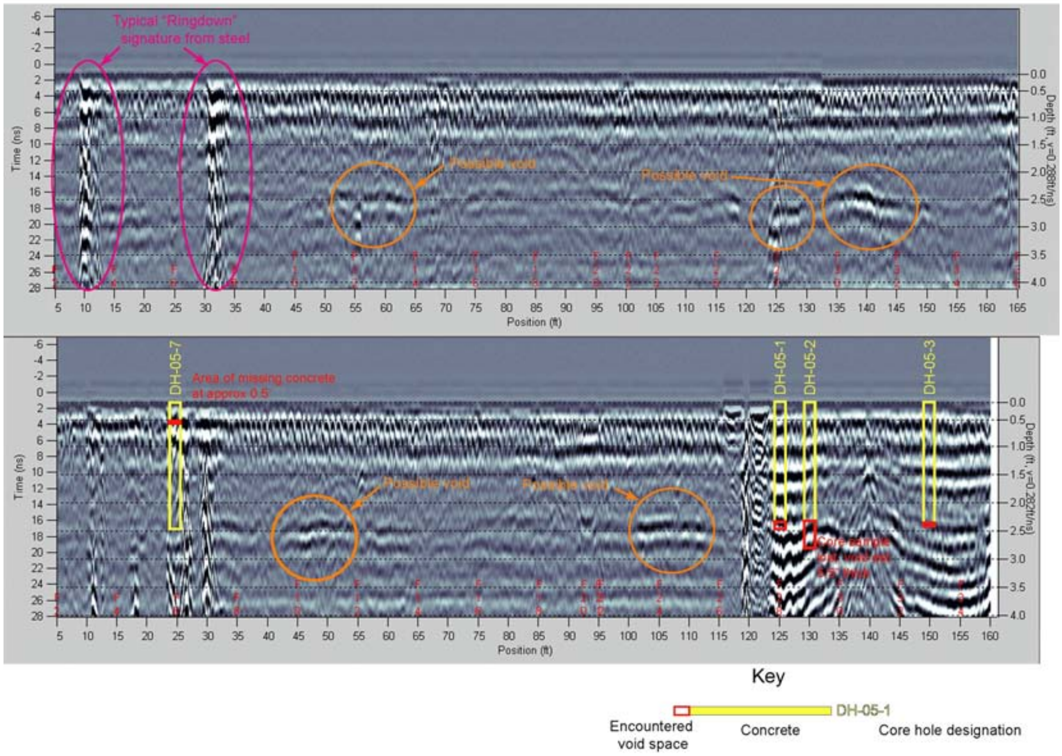

Figure 144.—GPR profiles along a conduit invert. The large amplitude white-black reflection patterns are associated with concrete underlain by voids in drill holes (DH) DH-05-01 through -03.

for generated sonar pulses to return to the transducer (through air or water) are a function of the distance that the pulse travels, it is possible to precisely measure the distance to the conduit surface. When processed by a computer, thousands of these measurements can be used to generate a map (figure 145) of the condition (deflection, joint size, depth of deterioration, etc.) of the conduit. The precision of the system is such that measurements of fractions of an inch can be made. The sonic velocity can vary with temperature and humidity, and calibration is required prior to commencing with the inspection. The accuracy of the measurements depends upon the calibration. The sonar can also be used in navigating the transporting unit within the conduit.

Recent improvements in this technology allowed the USACE to investigate the condition of a conduit without unwatering it. A computer processed signals received from sensors mounted on an ROV to generate three-dimensional images. The user can manipulate the images on the screen in real time to change the observational point of view via graphics software. Thus, the graphics software allows the user to

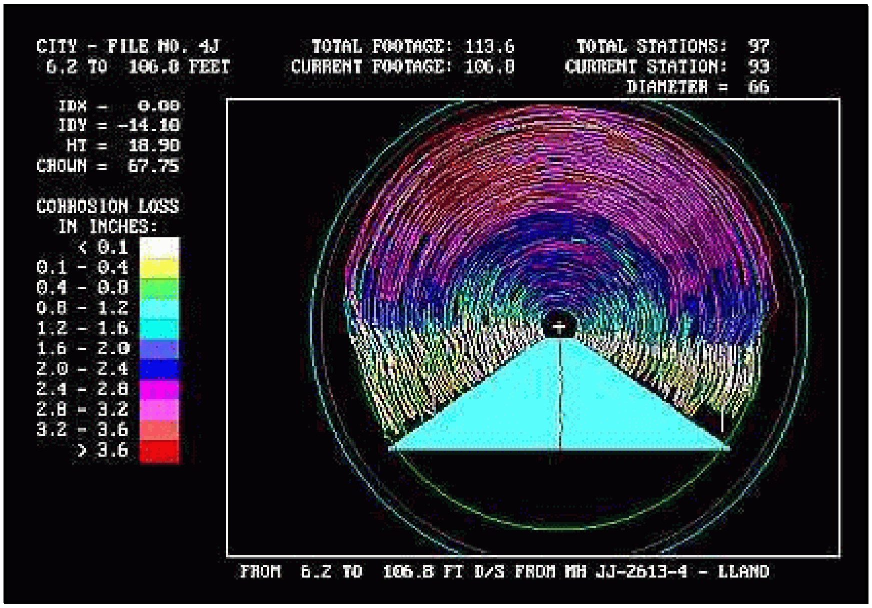

Figure 145.—A rotating sonar transducer mounted on a sled can be pulled through a conduit to measure and record changes in dimensions along the conduit. This can be very useful when determining the size for a proposed slipliner. Here, the measured corrosion loss of the crown of a steel pipe is shown at many locations on one image. The bottom of the conduit was submerged. The sonar device can work in water or air, but requires different settings for each. Thus, both above water and underwater sonar measurements cannot be made at the same time, although a mechanical device on the sonar sled can simultaneously provide information for the bottom of the pipe. Figure courtesy Sonex Corporation.

position the virtual observation point anywhere in space. Features that are not visible from the camera's view angle can become readily apparent (Britzman and Hansen, 2002, p. 3).

10.6 Ultrasonic pulse-echo and ultrasonic pulse-velocity

These methods measure the velocity and frequency content of acoustic pulses of energy through metallic (i.e., CMP) and nonmetallic (e.g., concrete) materials. Piezoelectric transducers are passed through the structure using a "smart pig" device or are attached to the structure to transmit and receive the pulses.

These methods generally use a source with known impulse characteristics, so that a transfer function can be computed between the input and the measured output (receiver) signal. By examining typical and nontypical (anomalous) velocity and frequency information, correlations may be established between sound and poor concrete conditions, and the corresponding acoustic signatures.

Generally, higher pulse velocities indicate good quality material, while decreased velocities or poor return signals (decreased high frequencies) indicate poor quality, such as voids, cracks, or deterioration. A large number of transducers used around the circumference of the conduit will improve the accuracy and the resulting image resolution.

The pulse-velocity method is widely used and has provided reliable in situ delineations of the extent and severity of cracks, areas of deterioration, and general assessments of the condition of concrete structures for many years. The equipment can penetrate thick concrete sections with the aid of amplifiers, is easily portable, and has a high data acquisition-to-cost ratio. Although most applications of the pulse-velocity method have been under dry conditions, the transducers can be waterproofed for underwater surveys. Tests have shown that the pulse-echo system is capable of delineating sound concrete, concrete of questionable quality, and deteriorated concrete, as well as delaminations, voids, reinforcing steel, and other objects within concrete. Also, the system can be used to determine the thickness of a concrete section in which only one surface is accessible. The system will work on vertical or horizontal surfaces. However, the present system is limited to a thickness of about 1.5 feet with only one side accessible. For maximum use of this system, the operator should have had considerable experience using the system and interpreting the results (USACE, 1995b, p. 2-17).

The pulse echo is limited to about a 1 foot thickness. The pulse-echo method is a variation of the pulse-velocity method and is best suited for characterizing voids and cracks parallel to the conduit surface. The pulse-velocity method is suitable for detecting cracks and voids in other directions. The pulse-echo method can be used from a single face of the conduit, whereas the pulse-velocity method requires access to both faces of the conduit. Both methods are commonly used concurrently to get a complete evaluation of the conduit (Promboon, Olson, and Lund, 2002).

Note that recent MASW (section 10.1.1) and GPR (section 10.4) advances have allowed multiple different imaging methods to be used on a given site, allowing greater interpretation confidence in difficult problem areas.

10.6.1 Ultrasonic thickness survey

The thickness of an existing metal or steel pipe can be measured using an ultrasonic thickness survey. The survey is conducted by using a pulse-echo ultrasonic thickness gauge (figure 146). An ultrasonic gauge determines the thickness of metal or steel pipe by accurately measuring the time required for a short ultrasonic pulse generated by a transducer to travel through the thickness of the pipe, reflect from the back or inside surface, and be returned to the transducer. Different types of materials have different acoustic velocities. The advantages of performing an ultrasonic survey are:

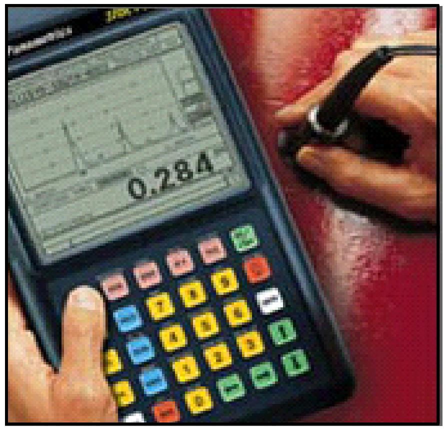

Figure 146.—Ultrasonic thickness gauge for measuring metal thickness.

- Thickness measurements can be taken without removing coating (with the exception of asphalt and concrete linings) on the interior of the pipe.
- Measurements can be taken on the exterior surfaces for exposed pipe while the pipe is in operation (i.e., full of water).
- Thickness measurements taken on the interior of the pipe help determine whether corrosion is occurring on the outside of the pipe shell for buried pipe or pipe encased in concrete.
- Measurements of paint thickness and oxidation/rust thickness can also be taken simultaneously during the plate thickness survey.

However, there are some limitations in performing a survey of this type. Factors that may prevent obtaining readings for specific sections of the metal or steel are:

- Water on the invert.
- Measurements cannot be taken for certain interior coatings, such as asphalt and cement-mortar lining.
- Rough, uneven, corroded surfaces can limit the bond between the transducer and the steel pipe, thus preventing thickness readings.
- Normally, the ultrasonic thickness survey is performed by man-entry into the pipe. However, for inaccessible pipes the thickness survey can be completed using a specially equipped pig that moves through the pipe. In some cases, if the exterior surface of a pipe is accessible, the survey can be completed without requiring man-entry into the pipe. This situation is applicable when the pipe is

located within a larger access conduit. This type of arrangement is discussed in section 3.1.1.1.

- The valleys of the pipe corrugations may be too tight to get a good bond between the transducer and the steel. Transducers vary in size, but 1/8 inch is about the smallest diameter transducer.

In performing an ultrasonic thickness survey, measurements should be taken at intervals sufficient to gather an adequate number of thickness measurement data points to help ascertain the true wall thickness of the pipe. Measurements should be taken circumferentially about the pipe. Particular emphasis should be placed on taking measurements near the invert of the pipe, as this area is more susceptible to wall thinning (metal loss) due to abrasion and corrosion. Factors to consider in determining the amount of measured wall thinning are:

- Pipe manufacturers tend to make the pipe wall slightly larger than the thickness specified to ensure they meet minimum wall thickness requirements.

- Testing accuracies depend upon the test instrument and the transducers. Most ultrasonic thickness equipment has an accuracy of at least 2 percent. Accuracy is also dependent upon proper calibration of the instrument.

If wall thinning is encountered, additional ultrasonic thickness surveys should be considered periodically (i.e., every 5 years, or more frequently, depending upon the severity of the wall thinning, corrosion damage, etc.). A history of these wall thickness surveys may indicate the expected yearly decrease in wall thickness, if uniform corrosion damage is occurring.

A stress analysis of the pipe to determine structural adequacy is recommended, if the ultrasonic thickness survey indicates a wall loss greater than 10 percent of its original specified value (see ASTM A 796 and the USACE's *Culverts, Conduits, and Pipes* [1998a, pp. 4-4 and 4-5] for guidance on performing stress analysis for CMP). A detailed wall thickness survey allows an accurate structural assessment of the pipe to be performed. The results of the thickness surveys can be compared to the minimum acceptable plate thickness specified by design criteria to determine if the pipe has a sufficient safety factor and if corrective action may be required.

Inspection of welds can also be performed by ultrasonic techniques. This process requires the use of an angle beam transducer to detect flaws in the weld metal. Ultrasonic sound waves are transmitted through a transducer, which reflects them into the weld area at an angle of 30 or 45 degrees. The angled reflection of the sound waves allows the flaw area to be detected and accurately sized. The interpretation of the results requires a great deal of experience, and should be performed by someone with a level 2 certification in ultrasonic testing.

10.7 Mechanical and sonic caliper

Calipers can provide information about the internal diameter of the conduit (USACE, 2001c, p. 32). Calipers are used for detecting any changes or defects that cause changes in interior dimensions, such as pits, holes, cracks, deformations, damage, or corrosion. Caliper measurements are made using mechanical, sonic, or ultrasonic methods. For the mechanical method, metal "feeler" instruments contact the inside wall of the conduit. The positions of the feelers are sensed electronically and recorded on a printout. The sonic or ultra sonic calipers use transducers that beam a pulse to the conduit wall. The pulse is reflected by the wall back to the transducer and interpreted based on its time of transit. Typically, calipers are used in conduits with diameters 18 inches or larger. Calipers are deployed using a wire cable or smart pig. Some caliper tools can be used underwater.

10.8 Radiography

The radiography method encompasses any type of penetrating radiation, such as X-rays, gamma rays, beta particles, neutron beams, or proton beams. Radiography is useful for detecting cracks, voids, and defects, or for viewing the internal composition of a conduit. Differences in thickness and density are easily measured and can be seen on a screen or recorded onto film. Although radiography is generally quick, efficient, and accurate, the complex nature of the equipment, costs involved, training, and certification requirements often limit its use.

10.9 Surface hardness

The rebound hammer and penetration resistance methods are quick, simple to use, and inexpensive to perform. These methods can be performed by field personnel with a limited amount of training and instruction. These methods are useful in assessing the general quality of concrete and locating areas of poor quality concrete in a conduit. However, these methods do have a number of limitations, including imprecise measurements of the in-situ strength of concrete. The rebound hammer and penetration resistance methods require man-entry access into the conduit.

10.10 Conduit evaluation by destructive testing

Although not utilized often in conduits, destructive testing can be used to gather more data. Concrete cores can be cut from selected locations to obtain representative samples. Samples are often taken from deteriorated areas and from good quality concrete for comparative purposes. A petrographer can examine the concrete cores, or strength tests can be performed on the cores. By using microscopic analysis and various chemical tests, a petrographer can determine the air content of hardened concrete, estimate the cement content, find evidence of carbonation or other reactions, and detect admixtures or contaminating substances that may have been present during construction. A petrographer can also make general observations about water-cement ratio, degree of cement hydration, early frost damage, excessive bleeding, and similar phenomena. Strength testing on the cores may include tensile and compressive. Drilling of concrete cores in conduits is expensive and should only be used when sampling and testing of the concrete is necessary.

Chapter 11

Appropriate Emergency Actions

Embankment dams are owned and operated by individuals, private and public organizations, and the government. An embankment dam failure resulting in an uncontrolled release of the reservoir can have a devastating effect on persons and property downstream. Even though an embankment dam may be well maintained, the potential for development of conditions that could lead to failure of the embankment dam always exists. Chapter 11 provides guidance on the appropriate actions needed during an emergency situation involving a conduit through an embankment dam. Often emergency situations involving the conduit lead to related problems with the embankment dam. Therefore, this chapter considers not only emergency actions for conduits, but also actions required for the embankment dam.

11.1 Implementation of an Emergency Action Plan

Many types of emergency events could jeopardize the safety and structural integrity of an embankment dam and threaten the safety of the general public downstream of the dam. Whenever people live in areas that could be inundated as a result of a failure of or misoperation at a embankment dam, the potential exists for loss of life and significant damage to downstream property. Developing thorough and consistent Emergency Action Plans (EAPs) in an effort to help save lives and reduce property damage in areas that would be affected by the failure or misoperation of a specific embankment dam and maintaining up-to-date points of contact and phone numbers are important. Copies of the EAP should be provided to emergency management agencies and personnel and periodically discussed with them. If no EAP exists, contact the State dam safety office.

Emergencies involving conduit related issues are only one aspect of potential embankment dam failures. An EAP is a formal document that:

- Identifies potential failure conditions at an embankment dam and specifies preplanned actions to be followed to attempt to prevent a dam failure and to possibly minimize downstream property damage and potential loss of life.

- Specifies actions the embankment dam owner and others should take to mitigate or alleviate a potential dam failure during an emergency.

- Contains procedures and information to assist the embankment dam owner in issuing early warning and notification messages to responsible downstream emergency management authorities of the emergency.

- Includes notification of the State dam safety officials and designates a qualified professional engineer who is experienced in embankment dam design and construction and should also be notified to assist in the emergency response.

- Contains inundation maps to show the emergency management authorities the critical areas for evacuation in case of an emergency.

An EAP is needed for two main reasons:

- To plan the coordination of necessary actions by the embankment dam owner and the responsible local, State and/or federal officials to provide for timely notification, warning and evacuation in the event of an embankment dam failure or release (controlled or uncontrolled).

- To reduce the risk of loss of life and property damage, particularly in the downstream areas, resulting from an emergency.

An effective EAP is generally comprised of six basic elements as follows (FEMA, 2004, pp. 4-5):

1. *Notification flowchart.*—A notification flowchart shows who is to be notified, by whom, and in what priority. The information on the notification flowchart is necessary for the timely notification of persons responsible for taking emergency actions. The flowchart should contain a primary and alternate telephone number (including cell phone numbers) for each person to be contacted. Instructions should state that if the person being contacted does not answer the telephone, a message should be left. But the next person on the list must be called and given the information. The caller must continue the alert process in each alert level until a person has physically been talked to in that level. This person then continues the alert process for the level in the same manner.

2. *Detection, decisionmaking/classification and notification.*—Early detection and evaluation of the situation(s) or triggering event(s) that initiate or require an emergency action are critical. The establishment of procedures for the reliable and timely classification of an emergency situation is imperative to ensure that the appropriate course of action is taken based on the urgency of the situation.

3. *Responsibilities.*—The responsibility for emergency-action-related tasks should be assigned during the development of the plan. Embankment dam owners are responsible for developing, maintaining and implementing the EAP. State and local emergency management officials having statutory obligation are responsible for the warning and evacuation of the general public within the affected areas. The EAP should clearly specify the embankment dam owner's responsibilities, to ensure effective, timely action is taken should an emergency occur at the dam. The EAP must be site specific, because all embankment dams are different.

4. *Preparedness.*—Preparedness actions are taken to mitigate or alleviate the effects of an embankment dam failure or operational reservoir release and to facilitate the appropriate response to emergencies. This section of the EAP identifies actions to be taken before and/or during any emergency.

5. *Inundation maps.*—Inundation maps should delineate the areas that would be flooded as a result of a embankment dam failure for normal and flood conditions or uncontrolled release. Inundation maps are used by both the embankment dam owner and emergency management officials to facilitate timely notification and evacuation of areas affected by an embankment dam failure or flooding. These maps greatly facilitate notification, by graphically displaying flooded areas and showing travel times for wave fronts and flood peaks at critical locations. These maps should be used in advance to develop warning and evacuation plans, but should only be used for guidance.

6. *Appendices.*—The appendices contain information that supports and supplements the material used in the development and maintenance of the EAP.

Once the EAP has been developed, approved and distributed to the proper authorities, the plan still needs to be properly maintained and exercised. Without periodic maintenance, the EAP will become outdated, lose its effectiveness, and no longer be useable. If the plan is not exercised (validated), those involved in its implementation may not be aware of their roles and responsibilities, particularly if emergency response personnel change over time. If the plan is not updated periodically, the information contained in it may become outdated, incorrect, and useless.

An EAP should be developed for site-specific conditions and to the requirements of the agency/organization that owns or regulates the use of the specific embankment dam. The intent of this document is not to provide every detail necessary to develop an effective and useful EAP. The requirements of an EAP vary from State/federal agencies as to the format, level of detail, and information presented in the EAP. For further guidance on developing an effective EAP, see the references from the

Canadian Dam Safety Association (1997); FEMA (1987, 1998, 2004); Colorado Division of Water Resources (1997); FERC (1998); Reclamation (1989a, 1995); and USACE (1996).

Once the EAP has been developed and approved, the appropriate implementation of the EAP is essential and critical to the safety of the general public living downstream of the dam. The EAP should list the proper procedures for the timely and reliable detection, evaluation, and classification of an existing, developing, or potential emergency. The conditions, events or measures for detection of an existing or potential emergency should be listed. Data and information collection systems should be discussed, such as inspection procedures, rule curves, and instrumentation plans. The process that will be used to analyze incoming data should also be discussed. Additionally, procedures, aids, instructions and provisions for evaluation of the collected information and data to assess the severity and magnitude of any existing or potential emergency situation should be discussed.

Emergencies are classified according to their severity and urgency. An emergency classification system is one means of classifying emergency events according to the different times at which they occur and to their varying levels of severity. The classification system should indicate the urgency of the emergency condition or response. Emergency classifications should use terms agreed to by the embankment dam owner and emergency management officials during the planning process, in order for the system to work and to ensure that organizations understand terminology and respond appropriately to the event.

The organizations that will use titles for emergency classifications should choose them carefully, so that everyone will understand what each classification level means when notifications are issued and received. Declaration of an emergency can be a very controversial decision. The issue should not be debated too long. An early decision and declaration are critical to maximize available response time.

Depending on the type of embankment dam, possible emergency events and the potential hazard zone downstream of the particular dam, two or more emergency classifications may be required to ensure the proper and effective response to emergencies at the dam. Coordination is required with State dam safety offices. Three embankment dam failure emergency classifications are suggested:

- Embankment dam failure is imminent or has occurred.

- A potential embankment dam failure situation is developing.

- Nonemergency or unusual condition.

The emergency classification that a failure is imminent or has occurred should convey the impression that the embankment dam is failing, and appropriate evacuation procedures should be employed. This is a situation where a failure has occurred, is occurring, or obviously is just about to occur. Therefore, once an embankment dam owner determines that there is no longer any time available to attempt corrective measures to prevent failure, the "failure is imminent or has occurred" warning should be issued. Emergency management agencies, for evacuation purposes, should conservatively interpret the phrase "failure is imminent" to mean that the embankment dam is failing, and all appropriate parties should be notified to commence emergency operations and evacuation.

The emergency classification that a potential embankment dam failure situation is developing should convey the impression that some time still remains for further analyses/decisions and remedial actions to be made before an embankment dam failure is considered to be imminent. This is a situation where the condition of the embankment dam is deteriorating rapidly and failure may eventually occur; however, preplanned actions taken during certain events could mitigate or alleviate failure of the embankment dam. Even if failure is inevitable, more time is generally available than the "failure is imminent" condition to issue warnings and/or take preparedness actions. All appropriate parties should be on standby-alert status and should be notified to commence their emergency operations and evacuation, if required.

The "nonemergency or unusual condition" classification applies where an unusual problem or condition has occurred, but a failure of the embankment dam is not considered imminent. This is a situation or circumstance that may affect the integrity of the embankment dam but is considered controllable. This condition could lead to a failure of the embankment dam, if appropriate actions or repairs are not employed. All appropriate parties should be notified periodically with regard to the status of the condition of the dam and should be on standby-alert status for emergency actions, should conditions deteriorate.

Table 11.1 provides a guide for determining the level of urgency and the emergency classification associated with emergency conditions attributed to the internal erosion or backward erosion piping of earthfill materials (Colorado Division of Water Resources, 1997, p. 11; FEMA, 1998).

Prompt and effective response to an emergency at a particular damsite could result in the mitigation or avoidance of a embankment dam failure incident, or help reduce the effects of a dam failure or operational spillway release, and facilitate response to the emergency. The preventive actions that an embankment dam owner may take include providing emergency flooding operating instructions, and arranging for equipment, labor, and the stockpiling of materials for use in an emergency situation. An effective EAP should describe preventive actions to be taken during the development of emergency conditions.

Table 11.1.—Assessing emergency classification and urgency

Incident	Level of urgency and response		
	Non emergency condition.—New or increased problem. Change in existing condition.	**Potential dam failure developing**.—Possible embankment dam failure is developing. Condition of embankment dam is deteriorating rapidly.	**Embankment dam failure is imminent**.—Embankment dam failure has occurred, is occurring, or is about to occur.
Response and notification priority	Monitor condition and call for assistance. Contact the State dam safety officials and/or design consultant.	Monitor condition; take appropriate remedial actions; emergency response personnel on standby-alert status; and begin mobilizing for failure, if required. Contact design consultant, general construction contractor, the State dam safety officials, and emergency manager and response personnel.	Commence the appropriate emergency operations and response and evacuation of affected downstream residents. Contact emergency manager and response personnel, design consultant, general construction contractor, and the State dam safety officials.
Problem or condition	Examples of possible observations		
Internal erosion and backward erosion piping	Small amount of sediment in seepage or drains.	New, stable or slowly increasing seepage rates transporting some sediment. Significant amount of sediment in seepage, drains muddy water. Reservoir level is falling without apparent cause (such as outlet works or spillway releases).	Rapidly increasing seepage transporting large amounts of sediments. Sinkholes on embankment dam or abutments, whirlpool in reservoir, significant settlement of embankment dam, significant muddy water. Whirlpool or other signs of the reservoir draining rapidly through the embankment dam or foundation.
Seepage	Downstream slope of embankment dam is wet, soft; minor sloughing; water running down dam face or abutment groins.	Significant new or increasing seepage or sand boils downstream from the embankment dam. Seepage is causing slides, which narrows the embankment dam cross section, or settlement of dam crest and loss of freeboard.	Rapidly increasing seepage and/or transporting large quantities of materials. Sand boils rapidly increasing in size or number and/or rapidly increasing flows. Seepage has caused large slides, which have reduced freeboard to the reservoir level, and/or embankment dam is overtopping due to loss of freeboard.
Sinkholes	Small depressions on embankment dam, abutment or foundation.	Significant new or larger sinkhole(s) or crest settlement. Large sinkhole over outlet works, or on embankment dam, abutment or foundation (larger than 8 in. in diameter). Stable or not increasing in size.	Sinkhole(s) or settlement rapidly increasing in size or number. Unstable or increasing sinkhole over outlet works, or on embankment dam, abutment or foundation. Whirlpool in reservoir.
Settlement	Minor settlement or depressions (less than 1 ft).	Moderate settlement of embankment dam crest or embankment slope (one-half of normal freeboard).	Significant settlement of the embankment dam crest, reservoir is overtopping the dam.
Conduit failure	Broken gate or operator, minor conduit deterioration, seepage adjacent to conduit.	Cracked or perforated conduit, sediment in seepage, deeply scoured or undermined conduit.	Significant, muddy seepage from or adjacent to conduit; sinkholes in embankment over outlet conduit.

Preventive actions involve the installation of equipment or the establishment of procedures for one or more of the following purposes:

- Preventing emergencies from developing, if possible, or warning of the development of an emergency.
- Facilitating the operation of the embankment dam in an emergency through dam operator training.
- Minimizing the extent of damage resulting from emergencies that do develop.

Timely implementation of the EAP and coordination and communication with downstream local authorities are crucial elements in the effectiveness of emergency response. The EAP should contain a discussion of provisions for surveillance and evaluation of an emergency and should clearly indicate that emergency response procedures can be implemented in a timely manner. An important factor in the effectiveness of an EAP is the prompt detection and evaluation of information obtained from instrumentation and/or physical inspection procedures.

Certain planning and organizational measures can help the embankment dam owner and local emergency response personnel manage the emergency more safely and effectively. These measures include stockpiling materials and equipment for emergency use, and coordinating information. Alternative sources of power for spillway or outlet works gate/valve operation and other emergency uses should also be provided. The EAP should list the location of each power source, its mode of operation and, if it is a portable source, the means of transportation and routes to be followed to the damsite.

The EAP should document the following items as they pertain to stockpiling materials, obtaining equipment, and contacting personnel for use in the event of an emergency. Not all embankment dams lend themselves to a need to have stockpiled materials and equipment. The materials and equipment can be stockpiled at the damsite or an accessible site within close proximity to the damsite. Resources needed may include:

- Materials needed for emergency repair and their location, source, and intended use. Materials should be as close to the damsite as possible.
- Equipment to be used, its location, and who will operate the equipment.
- How the equipment operator or construction contractor is to be contacted.

- Any other personnel who may be needed, like laborers and the design engineer, and how they are to be contacted. If there is no designer of record, a list of two to three qualified professional engineers should be available for contact.

The EAP should also document the following items as they pertain to coordination of information and communicating with the emergency response personnel:

- The need for coordination of information on flows based on weather conditions and runoff forecasts and embankment dam failure and other emergency conditions. Describe how the coordination is achieved and the chain of communications, including names and telephone numbers of responsible people.

- Additional actions contemplated to respond to an emergency situation or embankment dam failure at an unattended dam.

- Actions to be taken to lower the reservoir. Describe when and how (maximum drawdown rate) this action should be taken. Also, alternative means of evacuating the reservoir should be specified in the event the outlet works is inoperable, releases through the outlet conduit are not recommended due to a internal erosion situation, or the outlet capacity has been reduced for some reason.

- Actions to be taken to reduce inflow into the reservoir from upstream dams or control structures. The inflows should be stopped or diverted around the reservoir, if possible.

- Actions to be taken to reduce downstream flows, such as increasing or decreasing outflows from downstream dams or control structures on the waterway on which the embankment dam is located or its tributaries.

The EAP should also describe other site-specific or emergency repair actions that can be devised to moderate or alleviate the extent of the potential emergency and possible failure of the embankment dam. The EAP will recommend actions, but serves only as a guide, since there are typically many variables. A trained dam safety official will, in most cases, need to determine the type of action required.

Table 11.2 provides a list of potential problems and immediate response or emergency repair actions that can be undertaken. This is a relatively comprehensive list and includes problems and the associated emergency response as they relate to conduits through embankment dams (FEMA, 1987; Colorado Division of Resources, 2002). Caution is advised in using table 11.2, since many variables are involved, and each damsite is different.

Table 11.2.—Potential problems and immediate response or emergency repair actions

Problem or conditions	Cause	Response or emergency repair actions
Internal erosion and backward erosion piping through the embankment dam, foundation or abutments	Water has created an open pathway, channel or pipe through the embankment dam. The seepage water is eroding and carrying embankment materials. Large amounts of water have accumulated on the downstream slope. Water and embankment materials are exiting at one point. Surface agitation may be causing the muddy water. A break in the conduit could be allowing water to discharge out of the conduit, in the case of a pressurized conduit beneath the embankment dam. A flow path has developed along the outside of the conduit.	• Begin monitoring the outflow quantity and establishing whether water is getting muddier, staying the same, or clearing up. • If the quantity of flow is increasing, the reservoir should be lowered until the flow stabilizes or stops. • Search for a possible opening on the upstream side of the embankment dam and plug, if possible, as noted in the sinkhole section above. • Place a protective filter of sand and gravel over the exit point(s) to prevent further migration of fine embankment materials. • Continue operating the reservoir at a reduced reservoir level until repairs can be made. • Engage a qualified professional engineer to inspect the conditions and recommend further corrective actions to be taken.
Seepage water exiting from a point adjacent to the conduit	Break in the conduit allowing water to discharge out of the conduit, in the case of a pressurized conduit beneath the embankment dam. A flow path has developed along the outside of the conduit or a saturated area on the embankment above the conduit has developed.	• Thoroughly investigate the area by probing and/or shoveling to see if the cause can be determined. Caution should be used when shoveling in the embankment where seepage is occurring, so as to not aggravate the situation. As a precaution, a supply of sand and gravel may be needed to prevent un controlled seepage. • Determine if the leakage is carrying soil particles or sediments. • Construct a measuring device and channel the seepage to the measuring device, to monitor and determine the quantity of flow. • Stake out the saturated area and monitor for growth or shrinkage. • Continue frequent monitoring of the seepage area for signs of slides, cracking or increase or changes in the seepage condition. • If the seepage flow increases or is carrying embankment materials, the reservoir should be lowered until the leakage stops or is stabilized. • Engage a qualified professional engineer to inspect the conditions and recommend further corrective actions to be taken.
Large increase in flow or sediment in seepage	A shortened seepage path or increased storage levels	• Accurately measure outflow quantity and determine amount of increase over previous flow rates. • Collect jar samples of the seepage to compare the turbidity of the water with time. • If either quantity or turbidity has increased by 25%, a qualified professional engineer should be engaged to inspect the conditions and recommend further corrective actions to be taken.
Sinkholes	Backward erosion piping of embankment materials or foundation causes a sinkhole. A sinkhole can develop when a subterranean erosion feature	• Inspect other parts of the dam for seepage or more sinkholes. • Identify actual cause of the sinkhole(s). • Check seepage and leakage outflows for dirty/muddy water.

Problem or conditions	Cause	Response or emergency repair actions
	collapses. A small hole in the wall of a conduit can allow backward erosion piping of materials and develop a sinkhole. Dirty water at the exit portal indicates erosion of the embankment dam materials.	• Carefully inspect and record location and dimensions (depth, width, length) of the sinkhole. Stake out the sinkhole to monitor any growth and development of the sinkhole. Frequent monitoring of sinkholes and seepage. • Lower the reservoir level to a safe level or until the seepage stops. If the sinkhole results from backward erosion piping of embankment materials into the conduit, alternative means to evacuate the reservoir may be required, such as siphoning, pumping, or controlled breach. • Excavate the sinkhole and plug the flow with whatever material is available (e.g., hay bales, bentonite, or plastic sheeting), if the entrance to the internal erosion can be located. • Place a protective filter of sand and gravel over the exit point(s) to prevent further migration of fine embankment materials. • Engage a qualified professional engineer to inspect the conditions and recommend further corrective actions to be taken.
Excessive settlement of the embankment or dam crest	Lack of or loss of strength of embankment materials. Loss of strength can be attributed to infiltration of water into the embankment materials from a crack in the conduit or loss of support by the dam foundation, causing a settlement or collapse of a conduit. Internal erosion or backward erosion piping of the embankment dam materials along the conduit.	• Establish monuments along length of crest and selected locations on the embankment dam to determine exact amount, location, and extent of the settlement. • Engage a qualified professional engineer to determine the cause of the settlement and to supervise all steps necessary to reduce possible threat to the dam and correct the condition. • Re-establish lost freeboard, if required, by placing sandbags or backfilling in the top of the slide with suitable embankment materials. Caution should be exercised not to further increase slide potential. • Re-establish monuments across the crest and selected locations on the embankment dam and monitor monuments on a routine basis to detect possible future settlement. • If continued movement of the settlement of the embankment dam is seen, begin lowering the reservoir at a rate and to an elevation considered safe given the settlement condition. • Continue operating the reservoir at a reduced reservoir level until repairs can be made.
Conduit failure	Cracks, holes or joint offsets in the conduit caused by settlement, rust, erosion, cavitation and poor construction. Broken/bent support block or control stem and broken/missing stem guides due to concrete deterioration, rust, excessive force exerted when operating the outlet gate/valve, and poor maintenance. Damage due to rust, cavitation, erosion, vibration, wear, ice action, or excessive stresses from forcing gate/valve closed when it is jammed.	• If internal erosion or backward erosion piping of the embankment materials through the conduit is the problem, close the outlet gate/valve to protect the embankment dam from further erosional damage. • Lower the reservoir to a safe level. If the outlet works is inoperable or cannot be operable for some reason, alternative means to evacuate the reservoir may be required, such siphoning, pumping, or controlled breach. • Monitor the conduit for settlement, development of sinkholes, and muddy leakage. • Implement temporary measures to protect the damaged structure, such as closing the outlet gate/valve. • Employ experienced professional divers, if necessary to assess the problem and possibly implement repairs. • Engage a qualified professional engineer to inspect the conditions and recommend further corrective actions to be taken.

11.2 Obtaining the services of a qualified professional engineer

Tens of thousands of embankment dam owners in the United States have exposure to liability for the water stored behind their dams. The responsibility for maintaining a safe embankment dam rests with the owner. For many owners, the proper operation and maintenance of the embankment dam is only one aspect of their organization's activities. Safely maintaining the embankment dam is a key element in preventing a failure and limiting the liability that an owner could face. An important way to help reduce an owner's exposure to the potential for an embankment dam failure is to have a qualified dam engineer periodically inspect and assess the dam for the development of problems that could lead to the dam's failure. The engineer should provide a written inspection report with recommendations for repairs for any potential problems found.

11.2.1 The need for an engineer

Embankment dams, like any other natural or constructed structures, will deteriorate with time. Failure of a embankment dam, whether due to conduit deterioration, inadequate spillway capacity, seismic inadequacies, or other reasons could leave the dam owner liable for lives lost and property damage that occur downstream as a result of the failure. For these reasons, the owner needs to be sure that the embankment dam and any appurtenant water-retaining structures have been designed, constructed, and maintained to withstand each of the probable loadings that these structures could be subject to during their lifetime. To maintain a safe embankment dam and minimize the possibility of a dam failure, regular periodic inspections, proper maintenance, and occasional repair and rehabilitation of the structures are inevitable. To perform these tasks, an owner needs the expertise of a qualified professional engineer (ADSO, undated), experienced in the design and construction of embankment dams and appurtenant structures. If no design and construction drawings and records exist for the engineer to work with, it may be necessary for the engineer to develop basic plans and calculations. These will help the owner and engineer better understand the structures, evaluate them for stability conditions, and understand the consequences of a embankment dam failure. An engineer can also provide the owner with assistance in selecting a contractor to perform repair or remediation work if necessary and can provide construction quality control if needed.

11.2.2 The type of engineer needed

Choosing a registered professional engineer (P.E.) with a civil and geotechnical engineering background, who is competent and experienced in the field of dam safety is important. Criteria to look for in the prospective dam engineer include:

- A licensed professional engineer, P.E., with a civil engineering degree

- A minimum of 10 years of experience with embankment dam design, construction, and inspections

- A knowledge of the rules and regulations governing embankment dam design and construction in the State where the dam is located

- Specific experience in the problem areas, such as hydrology, hydraulics, structural, or geotechnical engineering

11.2.3 Finding a qualified dam engineer

A good way to locate a qualified professional engineer is to contact your State dam safety office for recommendations. If the State dam safety office is not listed in your local telephone directory, you may find this information on the Internet under the (name of State) dam safety office or on the (name of State) government home page. Another source for obtaining the telephone number of your State dam safety office and/or the names of experienced dam engineers within your State is the Association of State Dam Safety Officials (ASDSO) at 859-257-5140 in Lexington, Kentucky.

11.2.4 Choosing an engineer who is best for your needs

Consultants are typically selected for engineering consulting services using one of three basic processes:

- *Qualification-based*.—Qualification-based selection means that the knowledge, experience, and ingenuity of the engineer are the critical factors in making the selection. This strategy is used when the owner is uncertain about the exact problem or the best solution to the problem. Typically, several engineering firms submit their technical qualifications, experience with similar projects, reputation with existing clients, and any other factors pertaining to the specific project. The owner selects the three most qualified firms to make brief presentations outlining cost-effective and innovative approaches to solve the problem. Based upon these presentations, the owner chooses the most qualified engineer to develop a scope of work. When agreement on the scope of work is achieved, the engineer and the owner negotiate a price that is fair and reasonable to both parties.

- *Fee-based*.—Fee-based selection means that the determining factor in choosing the engineer is the engineer's fee. This approach can be used if the owner knows exactly what work is needed and can clearly define the scope of work. This process has the disadvantage that the engineer best qualified to perform the work may not get the job.

- *Intermediate.*—The intermediate option is a cross between the qualification-based selection and fee-based selection processes. In the intermediate process, the owner prequalifies engineers based on their experience and qualifications, who are then asked to submit a fee-based proposal for a defined scope of work. This process ensures a higher level of certainty that the work will be of superior quality, but requires the owner to clearly define the scope of the work to be done.

11.3 Sinkholes and subsidence

Sinkholes or subsidence of the embankment surface in the immediate area of the conduit are usually the result of erosion of the embankment material. They usually indicate a very serious problem that needs immediate attention. Figure 147 shows an example of a sinkhole that occurred over an spillway conduit.

Seepage from the area around the conduit at the downstream end is also a serious problem, especially if it is a new occurrence. Seepage that is carrying embankment material, viewed as muddy water, is of immediate concern. Seepage of this type in conjunction with active subsidence or sinkholes is cause for immediate alarm and emergency action. Sinkholes can also develop around or adjacent to air shafts constructed to supply air to slide gates within an outlet works conduit.

The following section will discuss sinkholes and subsidence associated with conduits through embankment dams. These types of phenomena may occur on embankment dams for other reasons, but that is outside the scope of this document. For an example of a sinkhole that developed over a conduit, see the Sardis Dam case history in appendix B.

11.3.1 Initial response

The first response to the observance of new sinkholes or areas of subsidence is to initiate appropriate emergency actions. Unless it is determined conclusively that the conditions on the embankment dam are stable and not deteriorating, then it should be assumed that an emergency exists. The emergency action plan should be implemented. The reservoir should be drawn down as soon as possible, but not necessarily through the existing conduit. Section 11.4 discusses alternative means of reservoir evacuation. New seepage or cloudy seepage as discussed in chapter 9 is also of concern.

Figure 147.—Sinkhole over a spillway conduit.

11.3.2 Initial remediation

If the sinkholes are active and it appears that immediate remediation is needed to stabilize the situation, the placement of a well graded sand and gravel mix with nonplastic fines into the sinkhole can be attempted. The concept is that placement of these materials directly into the hole will cause the sand and gravel to be transported directly to the defect in the conduit. A well graded mix will hopefully contain some particles that are larger than the defect and these will thus get trapped. Once this occurs, then other, smaller particles will be trapped. Eventually the process is capable of filtering the embankment dam's core material, causing a seal to form, arresting the erosion completely. This type of solution should only be considered a temporary one, to be followed by a full investigation of the problem.

11.3.3 Investigation

A full investigation should be conducted to determine the root cause of the sinkhole or subsidence area. This is absolutely necessary. No permanent solution can be designed until the problem is pinpointed.

Should a sinkhole become visible on the surface of an embankment dam, it is likely that an erosional failure mode is well underway. Emergency measures should be instituted as described in this chapter. After the emergency conditions have been stabilized, probably by lowering the reservoir level, a forensic investigation of the sinkhole is warranted. A carefully planned and executed investigation can provide important information that will help determine what type of repair is most appropriate.

The surface expression of a sinkhole is most often a small indicator of a much larger cavity beneath the surface. Any investigation of a sinkhole should assume that the subsurface conditions are much worse than they appear to be. Case histories have demonstrated that sinkholes at depth can be much larger than what appears on the surface.

Most often, a sinkhole that was caused by erosion of embankment material into a conduit will be located immediately above the alignment of the conduit, and the following discussions apply to this situation. Figure 148 illustrates a typical sequence of the formation of a sinkhole located above a conduit. Figure 149 shows an example of a sinkhole where the continued removal of soil would have caused the roof of the cavern to migrate to the surface of the embankment dam. Sinkholes that are not associated with a conduit may have different considerations and are not further discussed here.

Investigations of sinkholes above conduits should be preceded by a review of the conduit and embankment design to ensure that the investigation does not increase the amount of damage. Most often, the sinkhole is investigated initially by a backhoe excavation conducted from the surface. This is performed to initially determine the magnitude of the problem and to see if the cause can be readily established. Also, the interior of the conduit below the sinkhole area should be inspected to determine if there are holes or other damaged areas that could be the point where embankment material has entered the conduit.

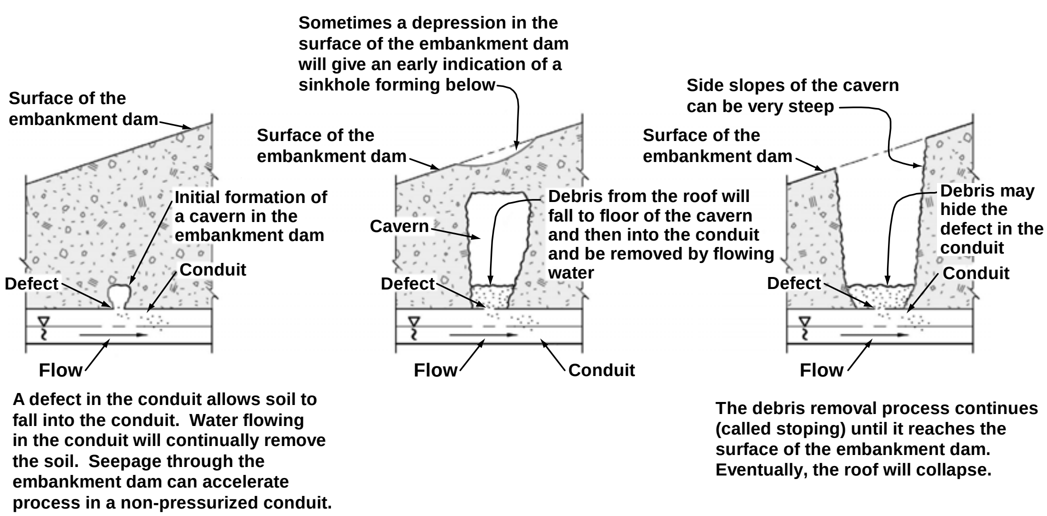

Figure 148.—Typical sequence of the formation of a sinkhole.

Figure 149.—Incipient sinkhole in an embankment dam. Eventually, the continued removal of the soil at the bottom of the cavern would have caused the roof to migrate to the surface of the embankment dam.

If the backhoe investigation results in limited information, it may be necessary to perform a major excavation of the embankment dam to ascertain that the entire sinkhole has been found. This investigation may be combined with the actual repair, as long as the excavation plans are sufficiently flexible to allow for complete removal of the sinkhole wherever it is found.

In-situ testing has been successful at some sites to look for soft areas or voids. A cone penetrometer testing program has been used. A closely spaced series of tests performed on a grid pattern can help discern the limits of any soft areas. At other sites, ground penetrating radar has been somewhat successful to help locate some incipient sinkholes that were near the embankment dam crest, but had not yet broken through to the surface.

11.3.4 Repair

If complete replacement of the conduit is chosen as the repair method, then it is much easier to repair the embankment dam. If the conduit is repaired by some in-situ method, then the repair of the sinkhole is made more difficult. In both cases, the basic concept is to repair the embankment dam with a material that is as good or better than the original material. The material to be used should be selected to perform the same function as the surrounding material. If the repair area is within the impervious core portion of the embankment dam, then similar material should be used. Similarly, shell material should also be used in areas outside of the core. If existing filters and drains were impacted, then these too should be replaced. New filters and drains should be added as needed.

Several factors determine the extent of the excavation required to repair a sinkhole that was caused from embankment soil being eroded into a conduit defect. One factor is the method used to repair the defect in the conduit. In some cases, the conduit will be not be replaced, but will be repaired by one of the sliplining methods. In that case, the embankment does not have to be excavated to gain access to the conduit from the outside for repair operations. The extent and configuration of the required excavation will then depend on how much embankment was damaged by the sinkhole, and how the excavation must be prepared before subsequent replacement of the sinkhole and excavated embankment can proceed.

The configuration for the excavation made to repair the sinkhole must consider differential settlement that will occur between the excavation backfill and adjacent embankment and foundation soils that have already consolidated. The shape of the excavation must also allow efficient operation of compaction equipment used in the reconstruction. Section 5.2 discusses the dangers of arching that can occur in backfilled trenches that are overly steep, particularly when the trench is transverse to the embankment centerline. Recommendations in that section suggest that any excavation made to repair sinkholes should probably be no steeper than 2H:1V for this reason, and only that steep, if the embankment soils have favorable properties. Flatter slopes are recommended for less favorable conditions. The excavation must also be configured for use of appropriate compaction equipment. The slopes of the excavation must be flat enough to operate equipment safely as the backfill of the sinkhole proceeds.

If the repair of the sinkhole involves excavation and replacement of the damaged conduit, consider the recommendations for conduit replacement provided in chapter 13 together with the recommendations above. In most cases, the excavation required to replace the conduit will also remove the portion of the embankment dam that was damaged by the sinkhole.

11.4 Alternative means of reservoir evacuation

Alternative means of evacuating the reservoir should be specified in the event that the outlet works is inoperable, releases through the conduit are not possible/recommended due to internal erosion or backward erosion piping, or the outlet capacity has been reduced for some reason. The selection of a means that is appropriate depends on the size of the reservoir, the physical features of the particular damsite, the availability of equipment and materials, the volume of water that could be released, and the required rate of release. Care should be employed in determining the means of reservoir evacuation during a specific emergency, to ensure that the reservoir releases do not cause loss of life or significant property damage downstream.

11.4.1 Siphoning

A siphon is a closed conduit system formed in the shape of an inverted U. A siphon (figure 150) can be used to partially drain a small reservoir. A single or a series of siphons can be constructed. Typically, siphons are placed up and over the embankment dam and extended to the toe of the dam. The downstream portion of the siphon can be charged with water and then released to create the siphonic action to start the siphoning of water. The downstream end of the siphon should be equipped with a gate or valve to facilitate creating the siphonic action. Multiple methods should be considered for priming siphons, such as a vacuum pump, water pump, or hand pump. Provisions for breaking the siphon (siphon breaker vent) should be provided at the crest of the embankment dam, should the need arise. At the discharge end of the siphon, the area should be properly protected to ensure that the discharging water does not cause erosional damage. A siphon over an embankment dam is illustrated in figure 151. The siphon in figure 151 is shown extending over the dam crest to avoid excavation into the embankment dam. If the siphon must be excavated through the embankment dam crest, the guidance provided in chapter 5 should be utilized.

Siphons are typically constructed of either PVC, HDPE, steel pipe and typically do not exceed 12 inches in diameter; however, in some instances, siphons as large as 15 to 18 inches in diameter have been successfully utilized for small embankment dams. Because of the negative pressures prevalent in the siphon, the pipe should be sufficiently rigid to withstand the collapsing forces. Pipe joints must be watertight, and the designer must take measures to avoid cracking of the pipe caused by movement or settlement of the embankment dam. In order to prevent absolute pressures within the pipe from approaching cavitation or collapsing presssures, the total drop of the siphon should be limited to a maximum of 20 feet. During emergencies, some cavitation damage may be an acceptable tradeoff.

Embankment dam owners and surrounding property owners should be aware that the use of siphons results in more frequent fluctuation in reservoir level when compared to more traditional pipe-and-riser spillway systems. This is a result of the inherent inefficiency of the siphon prior to priming of the system. Siphons prime with a head between 1 and 1¼ times the diameter of the siphon above the siphon invert. For example, the water surface in the reservoir will need to rise about 12 to 14 inches before a 12-inch diameter siphon becomes most efficient. Once the siphon primes, outflow increases very little with increases in head (reservoir level) (Monroe, Wilson, and Bendel 2002, p. 20). If a series of siphons is used at a site, they must be properly spaced to avoid close proximity. To close of proximity to

Figure 150.—Siphon used to lower the reservoir water surface through the upper entrance of an outlet works intake structure.

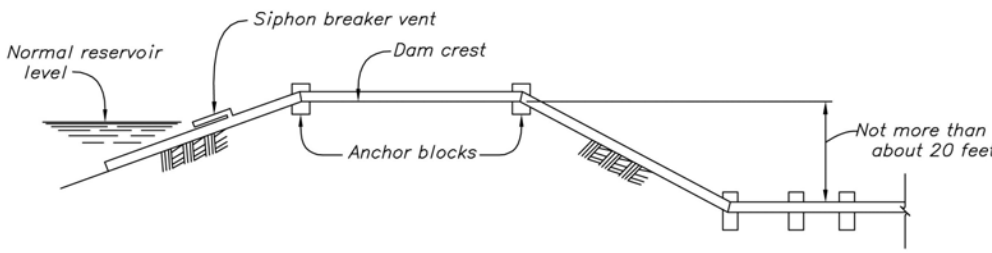

Figure 151.—A simple siphon constructed over the crest of an embankment dam.

each other may cause problems at the intakes of the siphons. Air may be sucked into the siphon pipe, causing the vacuum to be lost.

The advantages of a siphon include:

- The installation of a siphon can be performed in a relatively short amount of time.
- A siphon can be constructed with automatic operation to eliminate the need for frequent manual manipulation.
- The reservoir does not have to be completely drained. Maintaining a partial pool allows for the maintenance of some of the aquatic habitat.
- A siphon allows for the removal of cool water from relatively deep areas within the reservoir to promote cold water fish habitat downstream. In areas where trout populations are threatened by high water temperatures, a siphon can be used to combat the rise in stream temperature.

- Specialty contractors are not required, if quality engineering oversight is available during construction.

The disadvantages of a siphon include:

- Inability to handle flows greater than the designed capacity even though design head exceeds the design level.

- Inefficient flow at heads below 1 to 1¼ times the diameter of the pipe, causing more frequent fluctuation in the water surface when compared to pipe-and-riser spillways.

- Not cost effective (or, in many cases, feasible) for large watersheds. Generally limited to small drainage basins with relatively small peak inflows.

- A siphon designed for automatic operation may require excavation into the embankment dam to locate it below the anticipated reservoir water level.

- If the siphon extends above the reservoir water level, it will require some means of initiating the siphonic action.

- Can be susceptible to vandalism unless protective measures are taken.

- Inability to drain the entire reservoir and limited ability to drain reservoir deeper than about 20 to 25 feet.

- Some underwater work may be required for construction of the siphon.

- A siphon is not recommended in colder climates. Siphons are susceptible to blockage with ice unless special provisions are implemented during design and construction. Siphon piping may require removal from the reservoir during winter to avoid damage from ice loadings. Otherwise, the ice surrounding the siphon may have to be broken up daily.

- The theoretical lift on the upstream side of the siphon is less for sites at higher elevations compared to those at sea level.

- Best suited for low head operations to avoid cavitation potential.

- Storms or snow may prevent site access for personnel to operate the siphon.

For examples of siphons constructed at embankment dams, see the Crossgate and Sugar Mill Dams case histories in appendix B.

11.4.2 Pumping

Pumping of the reservoir water can be used to drain relatively small reservoirs. A single or a series of high capacity (e.g., 3,000-gal/min) portable pumps can be delivered to the damsite to assist in draining of the reservoir. The pumps can be positioned in the spillway entrance or on the embankment dam crest (figure 152) and allowed to discharge into the spillway or outlet works, or the pump can be placed over and down the downstream face of the embankment dam to the downstream toe. At the discharge end of the pumps, the area should be properly protected to ensure that the discharging does not cause erosional damage.

Pumps are usually self-contained and trailer or skid mounted. They can usually be rented in nearby major metropolitan communities, or they can be delivered and set up by the supplier. State or local flood control agencies may be another readily available source for high capacity pumps. Pumps are typically gasoline, diesel, or electrically powered. If electrically powered, a reliable electrical source should be available at the damsite. Purchasing pumps requires continuing maintenance on the pump and has a greater one-time expense compared to renting. Purchase may be justified for remote locations or to fulfill other needs for a high capacity pump by the dam owner, especially if one pump can service multiple dams. See the Balman Reservoir Dam case history in appendix B for an example of using a pump to evacuate a reservoir.

Another consideration with pumps is the limited net positive suction head available, (NPSHA) which essentially is the atmospheric pressure less any suction line friction losses and the height of the lift. To avoid cavitation, all pumps are rated with a net positive suction head required (NPSHR). If the NPSHA does not exceed the NPSHR, the pump will not operate. Also, as the NPSHA approaches the NPSHR, the pump capacity decreases. Placing the pump as close to the reservoir water surface as possible and using large diameter suction lines to minimize friction loss maximizes pump capacity. For reservoirs deeper than about 15 to 20 feet, pumps located on the dam crest or spillway crest may not be able to totally drain the reservoir because the height of the lift from the water surface to the pump by itself exceeds the NPSHA when reservoir levels are down, unless the pump can be moved along with the receding reservoir water surface. Because of the potential for complications caused by terrain and reservoir sediments, the frequent movement of trailer-mounted pumps may not be practical. However, floating pumps are available for these situations but are usually more expensive and probably not as available for rent as trailer-mounted units.

Totally draining the reservoir is usually not necessary to successfully evacuate the reservoir to levels that mitigate embankment dam failure. Usually, only a portion of the total reservoir requires evacuation to stop or control the erosion processes occurring within the embankment dam.

Figure 152.—All available resources may need to be utilized to drain the reservoir in an emergency. Here, the local fire department assists in draining a lake during a thunderstorm after the 45-year old CMP spillway collapsed.

11.4.3 Removal of the inlet structure (tower or riser pipe) of a drop inlet spillway conduit

If the existing spillway conduit can safely accommodate flowing water (e.g., it is in good condition), it may be possible to provide a limited amount of reservoir drawdown by carefully removing a section of the upper portion of the inlet structure and allowing the reservoir to drain out through the existing spillway. This method is best suited for low hazard embankment dams with small diameter riser pipes. To accomplish this method, the riser is removed in stages. For example, a CMP riser pipe can be removed in sections using an abrasive "cutoff saw" or hydraulic shears to cut vertical slots in the upper few inches of the riser followed by bending the metal wall downward. The reservoir should be allowed to drop to a safe level prior to removing additional sections of the riser.

If the riser can be temporarily isolated from the reservoir with a portable cofferdam, a torch can be used to remove the upper portion of the riser. This procedure was used to lower the pool of an embankment dam in Maryland by about 1.5 feet (figure 153). Care must be taken when removing the inlet structure to ensure that the materials being removed do not fall into and plug the spillway conduit, and debris does not enter the conduit (figure 154).

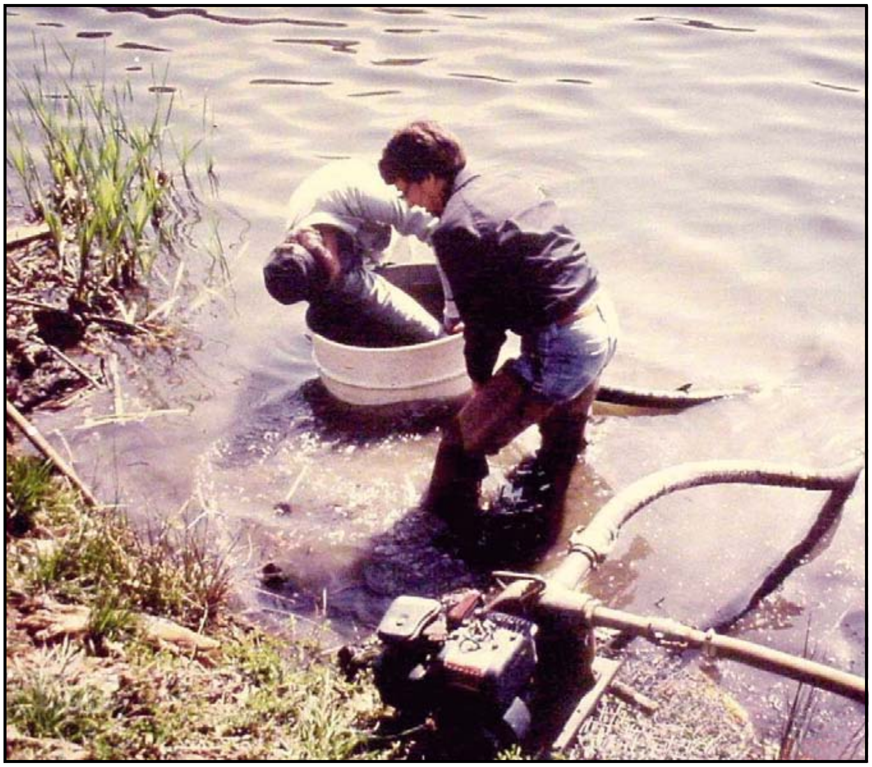

Figure 153.—The upper portion of a small metal pipe riser structure was removed with an acetylene torch. The riser was isolated from the pool by surrounding it with a large drum pressed into the soil. A pump was used to remove the water between the drum and the riser so that the work was completed in the dry.

Figure 154.—Care must be taken to ensure that debris does not clog spillway after removal of the riser.

11.4.4 Removal of the control structure of the spillway (concrete spillway)

The concrete control structure/weir of an existing spillway can be partially or fully removed to facilitate the lowering of the reservoir in the event of an emergency. The concrete control structure is typically used in earth cut spillways as a grade fill. If blasting is employed to remove the concrete, caution should be taken to ensure that the blasting does not cause additional damage to the embankment dam or foundation. Controlled blasting techniques with minimum particle velocities can be used effectively to remove the concrete structure. Figure 155 shows a concrete control structure being partially breached to allow for lowering of the reservoir.

11.4.5 Excavation of a trench through an earthcut spillway

For embankment dams with an earthcut spillway channel or emergency spillway at one of the abutments, a trench can be excavated through the discharge channel to deepen and/or widen the existing spillway discharge channel. Care should be employed to ensure that water being released through the new channel does not cause erosional headcutting of the channel, resulting in an uncontrolled larger-than-planned reservoir release. The trench should be excavated down to or into erosion-resistant materials, if possible. If not, the excavated channel should be protected by placing erosion-resistant materials in the channel, such as rock riprap, concrete rubble (if temporary), sandbagging, plastic sheeting, or geotextiles. The materials should be properly placed in the channel and stabilized to prevent the materials from being washed away. If sandbagging is used in high velocity flows, the sandbags should be placed beginning near the edge of the flow, where the velocities are low, and working toward the high velocity area. The largest sandbag possible should be used, and the ends of the bags should be securely fastened so that material is not washed out.

11.4.6 Excavation of a spillway through the embankment dam abutment

Similarly to excavating a channel through an existing earthcut spillway, a trench can be excavated through the abutment of the embankment dam (figure 156) to provide emergency release of the reservoir, if required. The trench should be properly located to ensure that uncontrolled releases of the reservoir through the channel do not encroach upon the embankment dam and cause an unanticipated breaching of the embankment dam. Care should also be employed to ensure that the excavated channel does not cause larger-than-anticipated flooding downstream of the embankment dam. Construction and cautions similar to those mentioned in the previous section for the excavation of a trench through an existing earthcut spillway should be employed.

Figure 155.—Spillway being partially breached to lower the reservoir.

Figure 156.—The owner of this embankment dam excavated a channel around the dam to prevent its overtopping during a hurricane.

11.4.7 Controlled breach of the embankment dam

A controlled breach of the embankment dam is an alternative means that can be considered to lower the reservoir to a safe level in the event of an emergency at a dam. The embankment dam can be partially or fully breached, depending on the situation or configuration of the dam. The breach location should be carefully selected. Consideration should be made to locate the breach where it can be controlled, the height of the embankment dam is the shortest, and the downstream consequences will be low. Local emergency responders should be involved with all planning for the breach, including any evacuation of the downstream population.

The breaching of the embankment dam should be done in stages and in a controlled manner to ensure that a catastrophic failure of the embankment dam does not occur, causing unanticipated and unwanted downstream property damage or loss of life (figure 157). First, a discharge channel should be excavated down the embankment downstream face or abutment area to convey the discharging water safely to the downstream channel. This channel should be excavated down to or into erosive-resistant materials, if possible. If not, the excavated channel should be protected by lining the channel with erosion-resistant materials, such as rock riprap, concrete rubble (if temporary), sandbagging, plastic sheeting, or geotextiles. The embankment dam can then be breached in a slow and staged operation. The embankment dam is first excavated down to a point that will allow a predetermined maximum amount of water to flow through the breach. The initial flow of water through the breach should be as minimal as possible and allowed to stabilize and diminish before removing another small portion of the embankment dam. The excavation of the embankment material should be kept at a minimal amount to limit the quantity of water discharging through the breach section at any time. This process can then be repeated until the desired breach dimensions have been obtained. If possible, a cofferdam should be upstream of the area to be breached, which serves to prevent a catastrophic failure, if the breached section begins to erode in an un controlled manner. See the Balman Reservoir Dam and Empire Dam case histories in appendix B for examples of controlled dam breaches to draw down reservoirs.

11.5 Gate or valve operational restriction

A gate or valve operational restriction is an emergency action used to lessen the risk associated with potential failure modes resulting from internal erosion or backward erosion piping, as discussed in chapter 7. The restriction is normally kept in force until the entire conduit is restored to a serviceable condition. The gate or valve operational restriction may require that the gate or valve not be operated at all or only be operated to such an opening as to keep the downstream conduit from pressurizing. The restriction typically applies to normal operating conditions. If an emergency arises requiring reservoir evacuation, the restriction could be removed.

Figure 157.—A controlled breach of an embankment dam begins after the 45-year old CMP spillway conduit collapsed and the lake level began to rise.

In addition to the gate or valve operational restriction, other supplemental actions should be considered, such as:

- The reservoir water level may need to be restricted below a certain water level. See section 11.6 for further guidance on implementing a reservoir restriction.

- A periodic monitoring program (i.e., weekly) may need to be implemented, which includes observation and documentation of the seepage outflow from the conduit. The upstream and downstream faces and the embankment dam crest above the conduit alignment should also be visually inspected.

- Periodic man-entry or CCTV inspections (i.e., annual, semi-annual) may need to be implemented to evaluate changing conditions with the conduit.

- The EAP may need to be implemented.

- The EAP may need to be updated to include specific discussion of the operational restriction.

11.6 Reservoir operating restriction

A reservoir operating restriction is not an emergency action, only an interim measure. A reservoir operating restriction requires that the reservoir be operated to maintain a water level below a certain elevation to reduce the risk of internal erosion or backward erosion piping of the embankment dam to an acceptable level of risk. The water level selected is typically lower than the normal water surface. The establishment of a reservoir operating restriction should consider not only the reduction of risk, but also potentially significant adverse impacts, such as:

- Limiting the operational flexibility of the reservoir.
- Reducing or severely curtailing water storage availability for project purposes.
- Severely compromising flood control operations.
- Endangering reservoir habitat.
- Sensitive and significant cultural resource sites may be exposed more frequently, as the reservoir is lowered, and subject to vandalism.
- During droughts, the reservoir could be severely reduced because of a lack of opportunity to store water as a buffer against drought.

A reservoir restriction should remain in place until dam safety modifications have been completed or until a review of additional performance data (i.e., seepage [weir] flows, piezometer data, settlement point data, and visual inspections) leads to other conclusions.

Chapter 12

Renovation of Conduits

The selection of the proper method for renovation, replacement, repair, or abandonment of a conduit is very site specific. Many factors go into the selection of the method to be used. This chapter will address design and construction considerations for the renovation methods. Chapter 13 discusses replacement of conduits, and chapter 14 discusses repairs and abandonment of conduits.

When evaluating older structures for renovation, the designer should proceed with caution. Previous designs may have utilized differing criterion or loadings compared to what is used in modern conduit design. The designer should consider materials available at the time of construction, and changes in material properties, design practices, and construction methods. For example, reinforcing bars used in reinforced concrete have undergone significant changes in the last 100 years. Yield strengths, allowable stresses, bar shapes (e.g., plain round, old-style deformed, twisted square), and splice lengths all have changed, compared to what is used today for modern structures. If original design information is not available, the designer will need to make conservative assumptions. The designer may find it beneficial to consult references that contain information on old design and construction methods. An example of this type of reference would be the ACI Detailing Manuals (available since 1947) (Concrete Reinforcing Steel Institute, 2001, p. 2). The designer could utilize these manuals to determine typical reinforcement details commonly used during the period of design.

The understanding of a historical timeline can often assist the designer with identifying conduits that may remain relatively free of long term deterioration and those that may require actions for renovation, replacement, or repair. Typically, timelines cannot specifically identify exact dates or structures when changes in methods or materials may have occurred because most of the available information is based upon a collective understanding that evolved over a period of time. Available timelines in many cases may be agency specific. An example of a historical timeline developed by Reclamation is illustrated in figure 158. This timeline was developed based on significant events that have occurred in regards to their experience with concrete technology. Using a timeline such as this, if the designer knows the approximate date of construction for a particular conduit, a preliminary assessment of its likely condition can be made.

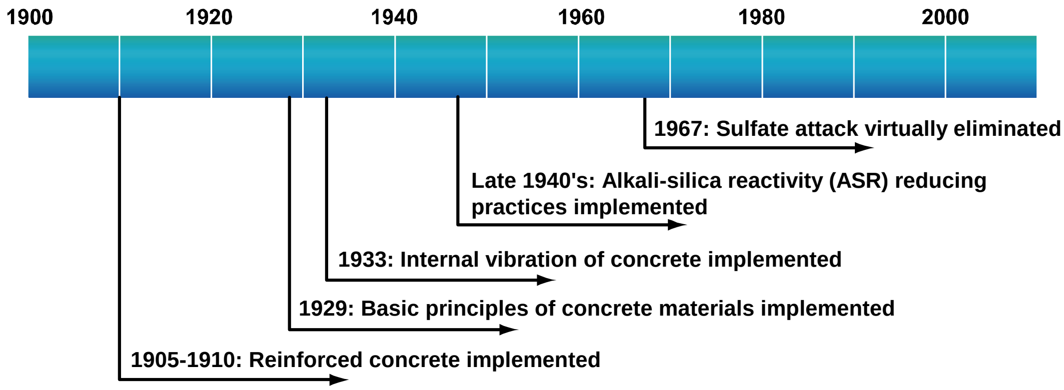

Figure 158.—An example of a historical timeline that can assist the designer in evaluating the condition of a conduit.

The nation's inventory of embankment dams and the conduits within them are aging and deteriorating. Many conduits require renovation to avoid potential embankment dam failures. Many of these conduits are too small to enter for construction activities while renovating them to address this deterioration issue. Traditionally, removal and replacement of the entire conduit has been one of the most frequently pursued alternatives, but one which can be very costly and time consuming. Removing and replacing the entire conduit requires excavation of a large portion of an existing embankment dam. Removal and replacement typically requires draining of the existing reservoir resulting in significant economic impacts. The large excavation of the embankment dam leads to safety concerns for the downstream population while the dam is in the breached condition, and concerns for the development of seepage and erosion within the recompacted earthfill en closure section (Cooper, Hall, and Heyder, 2001, p. 2).

In recent years, renovation has become a popular means of avoiding the traditional removal and replacement method. Methods for renovation include a variety of "trenchless technologies." The term trenchless technology applies to the renovation of existing conduits without requiring complete excavation (open-cut) over the alignment of the conduit. Trenchless technology is rapidly evolving in response to the introduction of new materials, products, and installation systems (USACE, 2001d, p. 3). The users of this document are urged to always refer to the latest manufacturers' recommendations when considering trenchless technology.

The most common renovation method is sliplining. Sliplining involves pulling or pushing a pipe of smaller diameter into the existing conduit and grouting the annulus. Flexible plastic and steel pipe has been successfully utilized for sliplining. Another method that has been used in limited applications is plastic cured-in-place pipe lining. This involves the insertion of a membrane into the existing conduit, which is then cured in place, forming a closely fitting plastic pipe within the existing conduit. See chapter 2 for discussion of materials used in the design and construction of conduits.

12.1 Sliplining

Sliplining an existing conduit through an embankment dam generally consists of installing a new, smaller-diameter pipe into the conduit. The annulus between the new pipe and the existing conduit is grouted. New entrance and terminal structures are sometimes constructed if the existing structures were deteriorated or were required for removal to facilitate installation of the slipliner. Also, a filter diaphragm or collar is constructed around the downstream portion of the existing conduit.

The advantages of sliplining an existing conduit through an embankment dam include:

- *Excavation.*—Excavation of the embankment dam is minimized. However, some excavation may still be required on the upstream and/or downstream face of the embankment dam for removal and replacement of the entrance and terminal structures and for installation of the filter diaphragm or collar.

- *Maintain reservoir level.*—In some situations (i.e., conduit has upstream control), the reservoir can be maintained at its normal water surface, if the slipliner can be installed from the downstream end of the conduit.

- *Construction.*—The construction time is usually less, reducing impacts to downstream users.

- *Costs.*—Construction costs for sliplining are generally less than for other conduit renovation or replacement methods.

The disadvantages of sliplining an existing conduit through an embankment dam include:

- *Deteriorated conditions.*—Sliplining is not appropriate for existing conduits in a significantly deteriorated condition or where the surrounding embankment has been damaged by internal erosion or backward erosion piping.

- *Alignment limitations.*—For inaccessible conduits, sliplining is generally limited to straight conduits. However, in certain situations, sliplining may be applicable for conduits with minor changes in alignment. If the conduit is accessible by man-entry, bends can usually be accommodated by using short sections of pipe.

- *Specialized contractors.*—Specialized contractors are needed sometimes for installation of the sliplining and grouting of the annular space.

- *Loss of reservoir.*—The reservoir is typically drained to provide upstream and downstream access to the conduit.

If the existing conduit has experienced significant deterioration or damage, further consideration is required before proceeding with sliplining. Further consideration should include:

- *Collapse.*—If the existing conduit appears to be on the verge of collapse, this may be an indication of considerable disturbance or movement of material outside of the conduit.

- *Seepage.*—The greater the seepage flow, the more concern that the flow regime could change considerably after sliplining, which could affect safety of the embankment dam. A change in the flow regime could force seepage to flow along the exterior of the conduit. Any evidence that the seepage is under pressure from the reservoir head should be a consideration for replacement of the existing conduit in lieu of sliplining.

- *Location.*—The location of any deterioration or damage within the existing conduit should be evaluated. Seepage upstream of a filter or impervious core may not be as much of a concern as seepage downstream of these features.

- *Void.*—If a void exists behind an opening in the existing conduit, the conduit should probably be considered for removal and replacement. However, some consideration should be given to where the void is located (near the intake structure is less problematic than near the embankment dam centerline). If the void does not seem to be associated with much seepage flow, this could be more of an indication that the void could be the result of erosive forces from the discharges through the existing conduit and that sliplining may be an option. In some cases, it may be possible to fill the voids with grout. Sliplining of the existing conduit may not be economical, if extensive grouting of large voids along the outside of the conduit will be required prior to inserting the HDPE slipliner. The costs involved should be compared to those required for removal and replacement of the conduit.

- *Deterioration process.*—If deterioration of the existing conduit is caused by a corrosive process, then the useful life expectancy of the existing conduit should be evaluated based on the knowledge that deterioration will continue. Generally, the slipliner should be designed to accommodate all internal and external loadings without any support provided from the deteriorating conduit. If the existing conduit is expected to provide support for the new slipliner, and the life expectancy of the existing conduit is less than the life of the project, then removal and replacement should be considered (figure 159).

Figure 159.—The holes in this CMP conduit were clearly visible after removal. The conduit was considered to be so severely corroded that sliplining was not an option and it was removed and replaced. Photo courtesy of Maryland Dam Safety Division.

12.1.1 Thermoplastics

The guidance provided in section 12.1.1 mainly pertains to sliplining of inaccessible conduits. The reader should understand that if the conduit is accessible by man-entry, variance from this guidance will be required.

The most commonly used thermoplastic for sliplining is smooth walled HDPE pipe. PVC pipe has been used in limited applications for sliplining, but has a number of disadvantages as discussed in section 2.2.1. For this reason, only HDPE pipe will be discussed in this chapter. HDPE pipe used for sliplining should meet the requirements of ASTM D 2447, D 3035, and F 714.

Additional design and construction guidance is available from other sources, such as CPChem's *The Performance Pipe Engineering Manual* (2003), NRCS's *Structural Design of Flexible Conduits* (2005), Plastic Pipe Institute (PPI) *Handbook of Polyethylene Pipe*, and the upcoming FEMA-sponsored "best practices" guidance document for plastic pipe used in dams (expected publication date, 2006).

12.1.1.1 Design considerations

The designer must evaluate a number of design parameters when considering HDPE pipe for use in sliplining. A few of the most significant design parameters include:

- Seepage paths

- Service life
- Initial inspection of the existing conduit
- Selection of the diameter and thickness
- Thermal expansion/contraction
- Stress cracking
- Joints
- Flotation
- Entrance and terminal structures

These parameters are further discussed in the following paragraphs. On a case-by-case basis, the designer may need to consider additional parameters depending on the performance criteria and design requirements of the specific application.

Seepage paths.—When an HDPE slipliner installation eliminates seepage into the conduit, the flow patterns within the surrounding embankment are changed and other undesirable seepage paths may develop. Any existing seepage paths may experience an increase in flow. For instance, if an existing deteriorated conduit has acted as a drain and reduced the phreatic surface within the embankment dam, the phreatic surface may increase and force flow through the dam, (along the exterior of the existing conduit) after slipliner installation. This seepage and the potential for internal erosion or backward erosion piping along the conduit must be addressed by installing a filter diaphragm or collar at the downstream end of the existing conduit. The filter diaphragm or collar should be designed to prevent migration of the fines in the embankment dam and should be placed around the entire circumference of the existing conduit. For guidance on the design and construction of the filters, see chapter 6.

Service life.—The service life for HDPE pipe is a function of the stress history of the pipe. A typical design calls for a 50- to 100-year service life.

Initial inspection of the existing conduit.—A thorough inspection of the existing conduit is required prior to selecting the diameter of the HDPE slipliner. Depending on the diameter of the existing conduit, man-entry or CCTV inspection methods should be used; see section 9.5.2 for guidance on inspection of conduits. The condition of the existing conduit, existence of any protrusions or obstructions, joint offsets, amount of deflection, and evidence of leakage or movement of embankment materials should be determined. The existence of any deflections, protrusions, or irregularities in the existing conduit will control the selection of the slipliner diameter. The pulling or pushing of a template (figures 160 and 161), inflatable pipeline sphere, or soft (typically open cell polyurethane foam) pig (figure 162) through the existing conduit is recommended. This will also ensure that sliplining can be done without difficulty.

<u>Selection of the diameter and thickness</u>.—The selection of the diameter and thickness of the HDPE slipliner should consider the following factors:

- *Size and condition of the existing conduit*.—The size of the existing conduit limits the diameter of the HDPE slipliner. Further, if the existing conduit has any protrusions or obstructions (i.e., deflection, joint offsets), the diameter of the HDPE slipliner may need to be made smaller to accommodate these restrictions.

- *Discharge requirements*.—At maximum full open operation, the HDPE slipliner should not flow greater than 75 percent full (i.e., 75 percent of the inside diameter) at the downstream end, to minimize the risk of surging or pressure flow developing in the conduit. Pressurized HDPE slipliners are not recommended for significant and high hazard embankment dams. However, an alternative to single wall HDPE pipe is available. This involves the use of a dual wall containment pipe; see the discussion later in this section for further information.

  HDPE pipe is very smooth. While the insertion of a new HDPE slipliner results in a smaller flow area, the reduced friction of the water passing through the slipliner results in only minimal losses of hydraulic capacity, if any. Typically, a new, smaller diameter HDPE slipliner has a hydraulic capacity equal to or greater than the original conduit. For example, the Manning's "n" value for smooth walled HDPE pipe is 0.009, compared to 0.010 for steel, 0.013 for concrete, and 0.022 for CMP.

- *Clearance requirements for grouting of the annulus between the existing conduit and the HDPE slipliner*.—To maintain sufficient clearance during the sliplining process, the outside diameter of the slipliner should be at least 10 percent smaller than the inside diameter of the existing conduit (ASTM F 585). This clear dimension between the interior surface of the existing conduit and outside surface of the slipliner allows for problem-free installation and grouting of the annular space. The designer needs to verify that the clear dimension will accommodate grout and vent pipes, when selecting the outside diameter of the HDPE slipliner.

- *Internal and external loadings*.—Conservatively, the HDPE slipliner should be designed with the assumption that the existing pipe continues to deteriorate after renovation is completed and will provide no support. For this reason, the

Figure 160.—Crossbar template attached to a CCTV camera-crawler to check for irregularities in the CMP conduit.

Figure 161.—A horseshoe shaped template used for checking irregularities in a conduit. The template is attached to the CCTV camera-crawler.

Figure 162.—A styrofoam pig used for checking irregularities in a conduit.

HDPE slipliner should resist all internal and external loadings. Internal loadings consist of water pressure and vacuum. If calculations show that the HDPE slipliner is susceptible to internal vacuum pressures, provisions should be made in the design to provide a means of letting air (i.e., air vent or an air valve) into the HDPE slipliner just downstream of the gate or valve. If the HDPE slipliner will have a downstream control gate or valve and is designed to be pressurized, the designer should consider the possibility that the gate or valve can be closed rapidly and cause water hammer. Good practice requires a properly designed gate or valve to have a closure rate that prevents the development of surge pressures within the HDPE slipliner. External loadings consist of soil and hydrostatic. In some situations, construction loadings from construction traffic and grouting may need to be analyzed. The designer should evaluate potential modes of failure consisting of wall crushing, buckling, and deflection.

Thermal expansion/contraction.—HDPE pipe has a relatively high linear coefficient of thermal expansion. For the temperature range between 22 and 86 °F, the linear coefficient of thermal expansion for HDPE pipe (9.0×10^{-5} in/in x °F) is high compared to steel (6.7×10^{-6} in/in x °F). In designing an HDPE slipliner, means of addressing thermal expansion/contraction should be considered. In a buried application, such as a conduit, the temperature variation is usually small due to the insulating effect of the surrounding embankment on the conduit.

After the HDPE slipliner is placed within the existing conduit, time should be provided for its temperature to equalize prior to grouting. Typically, 24 to 48 hours should be adequate. However, the designer should use their own judgement and allow for additional equalizing time when installations occur during periods of extreme temperatures. Circulating water in the slipliner can assist with the temperature equalization process. Nighttime installation may be another option to consider to reduce the effects of extreme temperature.

The use of upstream and downstream end restraints on the HDPE slipliner will limit expansion/contraction. Once the HDPE slipliner is grouted in place, it should undergo little expansion/contraction. This is largely due to the low modulus of elasticity of the HDPE pipe. The HDPE slipliner may try to expand, but in a restrained condition it cannot mobilize forces of the magnitudes required to cause expansion movement. Since HDPE pipe does not bond with the grout, the resistive forces are largely from the friction along the HDPE pipe/grout interface. Therefore, thrust-accommodating end structures are generally not required for HDPE sliplined conduits.

Stress cracking.—HDPE pipe failures are often attributed to the effects of environmental stress cracking (also called slow crack growth). This phenomenon can occur during the handling or installation of HDPE pipe. The HDPE pipe could be gouged, scratched, or kinked, resulting in a weak spot on the pipe wall. Subsequent operations of grouting the HDPE pipe or pressurizing the conduit result cracking of the weakened section. Specifying HDPE pipe made with ASTM D 3350 cell classification 345464C grade resin provides the highest level of resistance to slow growth cracking and can negate the possibility of this type of failure. This ensures a virgin, high-grade, very stiff resin which has been found highly resistant to environmental stress cracking. Other grades of resin often contain some percentage of low-grade recycled resins.

Joints.—The most common method used to join HDPE pipe is heat fusion (ASTM D 2657). This method is also known as butt fusion. The butt fusion technique is a widely used and industry-accepted heat fusion method for joining sections of smooth solid walled HDPE pipe. This method produces a joint that is watertight and is as strong or stronger than the HDPE pipe material itself, if performed correctly. A special machine (figure 163) is used to trim the ends of the pipes (facing or squaring off), align the ends of the pipe, heat both ends of the pipes to about 400 to 450 °F, and force the ends together under pressure. The melt bead size required for the thickness and diameter of the HDPE pipe determines how much pressure and time is needed for fusion of the joint. About 1 hour should be allowed for the joint to adequately cool after completion of the fusion process (figure 164). Trial fusions should be considered at the beginning of the day, so the fusion procedure and equipment settings can be verified for the actual jobsite conditions. Manufacturer's recommended procedures should always be observed for heat fusion.

Figure 163.—HDPE pipe joint being fusion welded.

A small bead (figure 165) is formed where the melted material is extruded from the joint. Beads appear on both the inside and the outside of the HDPE pipe. The need for bead removal is uncommon, but can be accomplished using special tools after the joint has throughly cooled to ambient temperature. If removal is necessary, the personnel using the debeading tool should be properly trained, so the HDPE pipe is not needlessly gouged. The existence of the interior bead has a negligible impact on the hydraulic performance of the slipliner. A bead exists on the exterior surface of the joint. If proper annulus clearance is provided by the designer, this should not affect slipliner insertion or the grouting process. The beads should be throughly inspected for uniformity and proper size around the entire joint. Visual inspection is usually adequate; however other methods, such as radiographic or ultrasonic methods, can be used. The use of fusion machine operators who are skilled, knowledgeable, and certified by the manufacturer will produce a good quality joint. Improperly heat fused joints cannot be repaired and must be cut out, and the ends

Figure 164.—Finished fusion welded joint.

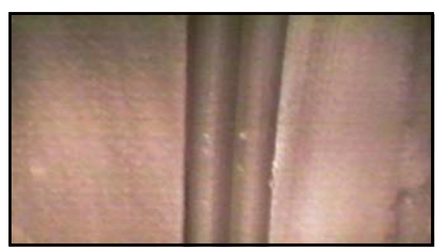

Figure 165.—Interior view of finished joint bead.

must be properly joined (ASTM D 2657). Upon completion of the repair, the HDPE slipliner should be retested for leaks.

Unlike bell and spigot pipe, such as PVC, heat fusion creates a continuous joint-free pipe of nearly constant outside diameter. Bell and spigot joints are susceptible to separation as the embankment dam settles. Because the HDPE slipliner joint does not take up a large part of the original conduit, a larger inside diameter slipliner can be used. This is an advantage when compared to flanged joints.

During cold weather, additional time is required to warm up the fusion machine and to heat the ends of the HDPE pipe. A shelter (figure 166) may need to be constructed for joining the sections of HDPE pipe in case of inclement weather. For additional cold weather procedures, see ASTM D 2657.

Other joining methods for HDPE pipe include:

- *Joints made by extrusion welding.*—Many prefabricated fittings (i.e., elbows, bends, and tees) can be joined to the HDPE slipliner with heat fusion (ASTM D 3261) in the field using an extrusion gun. The extrusion gun (figure 167) is a hand held extruder that preheats the surface of the HDPE pipe and feeds a molten bead of polyethylene into the joint. Extrusion-welded joints are not as strong as butt fusion joints. Proper training is required for using the extrusion gun. Extrusion welding has been successfully used for connecting HDPE grout

Figure 166.—Cold weather shelter constructed for joining sections of HDPE pipe.

Figure 167.—Hand held extrusion gun.

Figure 168.—HDPE grout pipe attached to HDPE slipliner.

and air vent pipes to the slipliner (figure 168). Extrusion welding cannot be used to repair damaged HDPE pipe.

- *Mechanical joints*.—The most common mechanical joint is the flange adapter. Flanged connections are often used to connect HDPE pipe to steel pipe. The flange adaptor consists of a stub end, which is typically butt fused to the HDPE pipe, and a flanged end, which is joined with bolts and nuts to the flanged end of another pipe. Flanged connections allow for easy assembly and disassembly of the joint. Flange joints tend to require more annular space than butt fusion joints.

- *Other joints*.—Some HDPE pipe products have integral threads or snap joints that allow sections to be easily joined without special equipment. However, these should only be used for nonpressurized applications in low hazard embankment dams due to the potential for pullout. Some types of plastic pipe use gasketed or glued bell and spigot joints. HDPE pipe cannot be joined by threading or solvent bonding.

Flotation.—When grouting an HDPE slipliner within an existing conduit, it is likely that the slipliner will "float" or be displaced upward by the fluid pressure of the grout in the annulus between the existing conduit and slipliner. Due to the relatively light weight of HDPE pipe, floatation can be more pronounced with this material. Floating of the slipliner may not allow for grout to completely encase the HDPE pipe and therefore reduce the overall strength of the structure. Floatation can also result in vertical misalignments, which may alter the hydraulics of a conduit, especially one that would flow under open channel conditions.

Steps should be taken to address this floating potential, such as using spacers or blocking between the existing conduit and the slipliner. Figure 169 shows an example of spacers being attached to the HDPE slipliner by extrusion welding. Some manufacturers have recommended that the HDPE pipe be filled with water to reduce flotation of the pipe during grouting. However, this does not always prevent flotation, because water is not as dense as the surrounding grout, and blocking is still necessary. Other manufacturers strongly advise against filling the HDPE pipe with water and instead recommend properly installed blocking and staged grouting.

HDPE pipe is flexible and can conform to alignment changes; therefore, a larger HDPE section is more applicable than a rigid slipliner section, such as steel. This tends to greatly minimize the potential distance an HDPE slipliner can float (i.e., by reducing the size of the annulus) and reduces the potential adverse effects of any displacement. One caution is that since the HDPE slipliner is more flexible, it may require more spacers than a rigid liner for the same span lengths, to control floatation, and provide sufficient room to fully encase the liner. However, if the alignment in the existing conduit varies, then the flexible liner will adapt more easily to the alignment, but will still require sufficient spacers to ensure adequate encasement. Spacers extending the full length of the HDPE slipliner are not recommended. Spacers should be designed to allow grout to fill the annulus between the existing conduit and the HDPE slipliner. The type and spacing of spacers will vary depending on the standard dimension ratio (SDR) of the HDPE slipliner and should be based on the recommendations of the HDPE pipe manufacturer.

Entrance and terminal structures.—The sliplining of an existing conduit may require partial or full removal and replacement of certain structures to improve release capabilities or to facilitate construction. Figure 170 shows an intake structure

Figure 169.—Spacers being attached to an HDPE slipliner using an extrusion gun.

Figure 170.—The intake structure has been removed as part of a conduit renovation.

that has been removed as part of outlet works renovation involving sliplining. For guidance on the design and construction of entrance and terminal structures, see section 3.4. Specially fabricated steel transitions are sometimes used at the critical upstream end of a conduit being sliplined with HDPE pipe. The transition and HDPE pipe are connected using a flanged joint.

12.1.1.2 Construction considerations

The designer must evaluate a number of construction parameters when considering HDPE pipe for use in sliplining. A few of the most significant parameters include:

- Sample testing and certification
- Handling and storage
- Installation
- Repairs to the HDPE slipliner prior to or during the insertion process
- Grouting
- Postinspection and acceptance
- Maintenance and repair of the completed HDPE slipliner
- Alternatives to sliplining existing conduits with solid walled HDPE pipe

These parameters are further discussed in the following paragraphs. On a case-by-case basis, the designer may need to consider additional parameters, depending on the construction requirements of the specific application.

Sample testing and certification.— Manufacturer's certification should be furnished prior to any shipment of HDPE pipe to the worksite. The certification provides proof that the HDPE pipe was manufactured, sampled, tested, and inspected in accordance with ASTM F 714 and meets the requirements. More details can be obtained by requesting the actual test data from the manufacturer. Not all HDPE pipe is tested; manufacturers may only test certain lots of pipe or perform testing at regular, scheduled intervals.

Handling and storage.—HDPE pipe is much lighter than steel or concrete pipe and generally does not require heavy lifting equipment. HDPE pipe is shipped in longer lengths than steel or reinforced concrete pipe due to its lighter weight.

HDPE pipe should be carefully handled and stored according to all of the manufacturer's recommendations. The manufacturer often ships handling instructions with the HDPE pipe. Cold weather handling precautions should be used to eliminate any impacts on HDPE pipe when temperatures are at or below freezing to avoid fracturing of the pipe. Handling of HDPE pipe when the temperature is below -10 °F is not recommended. The pipe should not be dropped or allowed to be dumped when off-loading. Strap slings should be utilized for straight HDPE pipe and the use of chains and hooks should be avoided. Lifting points should be well spread and evenly spaced. The HDPE pipe should be fully inspected at the time of delivery, with any defects noted. The HDPE pipe should be stacked on firm, flat ground, adequately supported, kept away from heat sources, and kept in original protective packaging until used. Pipes should not be stacked higher than five units or 10 feet, whichever is less. Stacking pipe with differing wall thicknesses and pressure ratings should be avoided. Testing has shown that unlike PVC, HDPE pipe does not become brittle under exposure to ultraviolet (UV) radiation. This resistance to UV radiation is the result of the small percentage of carbon-black which is added to the HDPE pipe material during the manufacturing process. Since virtually all conduits are buried, such exposure is generally minimal.

Installation.—HDPE pipe is a flexible material and, as such, can easily accommodate minor changes in vertical and horizontal alignment of the existing conduit being lined. Guidance on sliplining installation includes:

- *Preparation of existing surfaces*.—The existing conduit surfaces that grout will be placed against need to be free of roots, sediments, mineral deposits, dust, latence, loose or defective concrete, curing compound, coatings, and other foreign materials. Any sediments or debris should be removed from the invert of the existing conduit. Any bolts or other projections should be cut off flush and/or ground smooth with the interior surface of the existing conduit. See section 9.6 for guidance on cleaning conduits.

  A thorough inspection of the existing conduit is required prior to installing the HDPE slipliner to ensure that no obstructions remain that may hinder slipliner insertion. Prior to slipliner insertion, a soft pig or inflatable pipeline sphere of the same diameter as the HDPE pipe should be pulled through the existing conduit to check for proper clearance. Consideration should be given for adequate spacers and grout pipes to be attached to the HDPE slipliner.

- *Leak testing of joints*.—Hydrostatic testing of the joints is required and should be done prior to installation, using a sustained pressure test to find leaks in the HDPE slipliner. Prior to performing the hydrostatic test, the slipliner should be properly restrained from movement. Depending on the limits of the testing equipment, the entire length of HDPE slipliner can be tested at one time or the test can be separated into shorter sections. If a leaking joint is found, it will need to be cut out and a new section of HDPE pipe installed and the ends of the pipe heat fused together. Further guidance on leak testing is provided in CPChem's Performance Pipe Technical Note 802 (2002) and ASTM F 905.

- *Access and insertion*.—HDPE pipe is light in weight relative to more traditional pipe materials, and as such is easier to insert into existing conduits. Pulling, pushing, or a combination of both are the typical methods for slipliner insertion. Backhoes, bulldozers, and winches have been used to assist with the slipliner insertion (figure 171).

  The HDPE slipliner should be inserted following an approved installation plan, manufacturer's recommendations, and ASTM F 585. Sufficient work area must be available at the downstream toe of the embankment dam for insertion of the slipliner. For small embankment dams, smooth walled HDPE pipe sections can be fused into one long section on the crest of the dam and transported to the downstream toe. The HDPE slipliner is then inserted into the downstream end of the conduit and simply pushed upstream. For larger embankment dams, access to both the upstream and downstream portals should be obtained, so the fused sections of HDPE pipe can be pulled through the existing conduit with the use of a special pulling head attached to the slipliner.

  The pulling head design is based upon the axial pulling (tensile) stress. The axial pulling stress can be estimated by dividing the force of the pull load by the

Figure 171.—Insertion of an HDPE slipliner into an existing concrete conduit. A backhoe is being used to assist with slipliner insertion.

cross-sectional area of the pipe wall thickness. The pulling load is a function of many variables, such as the weight of the slipliner and frictional drag. A variety of pulling head configurations are possible, depending upon the application. Approved manufacturer pulling head recommendations should be followed.

The nose cone pulling head (also known as the banana nose or soft nose) is a simple and cost effective configuration to use where the pulling stress on the HDPE slipliner is less than 700 lb/in^2. The nose cone pulling head is made from a few extra feet of HDPE pipe that has been fused onto the slipliner. Evenly spaced wedges are cut into the leading edge of the pulling head. A couple of alternatives exist for the nose cone configuration: (1) the wedges are collapsed towards the center to form a cone and fastened together with bolts. A pulling cable is attached to secondary bolts that extend across the collapsed nose (figure 172) and (2) holes are drilled through the wedges and cables or narrow plates are attached to the wedges. The cables or plates are attached to a pull ring (figure 173).

If the sliplining application requires higher pulling stresses, the manufacturer should be consulted for specialty pulling head configurations. Fabricated mechanical pulling heads are available.

Blocks of wood or other material (called blocking or bridging) should be attached to the top of the HDPE slipliner, so that the slipliner will not contact the top of the existing conduit. Once the slipliner insertion (figure 174) begins, it should continue without any stoppage until completion. The pulling method will result in some stretching of the HDPE slipliner (1 percent of the total length). The slipliner will also experience differential temperatures before and

Figure 172.—Nose of HDPE slipliner modified for pulling into an existing conduit.

Figure 173.—A nose cone configuration utilizing a pull ring.

Figure 174.—Insertion of an HDPE slipliner into an existing CMP outlet works conduit.

after insertion, which will affect the length of the slipliner (1 in./100 ft/10 °F). Allowances for these changes in HDPE slipliner length need to be considered during insertion (figure 175). A 24-hour relaxation period is recommended to allow the slipliner to recover its length.

In some instances, a vertical riser pipe that is connected to the horizontal conduit may be required (typically associated with service or auxiliary spillways). The connection of the HDPE slipliner to a riser pipe can be somewhat difficult. A custom transition piece may need to be fabricated. Sometimes the riser can be removed and replaced with a new structure that facilitates a mechanical connection to the conduit slipliner.

Repairs to the HDPE slipliner prior to or during the insertion process.—Damage to the HDPE slipliner may occur from improper shipping and handling or from poor insertion technique. Damage can be in the form of kinks, punctures, breaks, or abrasion. HDPE pipe that undergoes this type of damage cannot be repaired, and the damaged section of pipe should be removed and replaced. The damaged section of HDPE slipliner should be cut out and a new section of pipe installed. The HDPE slipliner should be cut one pipe diameter on each side of the damaged area. The ends of the existing pipe and replacement pipe should be heat fused together.

Figure 175.—This gap shows that the designer did not adequately consider the potential for thermal expansion/contraction during installation of this HDPE slipliner. To avoid this problem, the HDPE slipliner should have been designed to extend beyond the end of the existing conduit.

Grouting.—Careful grouting of the annular space between the existing conduit and the HDPE slipliner is essential. This can be a complex process, requiring the experience of a qualified contractor. Full encapsulation for the entire length of the annulus rarely is achievable, and the HDPE slipliner is typically designed to withstand all internal and external loadings independently from exterior conditions. A lightweight, low density grout containing no aggregate will ensure the best result. The following guidance discusses the grout plan, mix design, sequencing, and injection:

- *Grout plan*.—A grouting plan detailing the contractor's grout mix equipment, setup, procedures, sequencing, plan for handling waste, method for communication, and method for sealing and bulkheading the upstream and downstream ends should be submitted for review and approval prior to initiation of grouting operations.

- *Grout mix*.—The grout mix, consisting of water, cement, flyash, and chemical admixture must remain fluid and not thicken for at least 2 hours. Premature thickening of the grout will result in high injection pressures, and inadequate support of the HDPE slipliner.

  Cement meeting ASTM C 150, type II is generally considered acceptable for use in grout mixes for injection. Other types may be considered based on particular applications. The chemical admixture used will depend upon the type of injection application. High-range, water-reducing, shrinkage-compensating, plasticizing admixtures may be beneficial.

The grout should be tested in accordance with ASTM C 939. The design should specify mix design, density, viscosity, maximum injection pressure, initial set time, 24-hour and 28-day compressive strength, shrinkage, stability, and "bleed" or fluid loss. A minimum design compressive strength of 4,000 lb/in^2 at 28 days is generally acceptable.

The grouting contractor must have dependable equipment of a size that will allow the grouting to be done quickly. The contractor must also have backup equipment available and ready at the site. Any grout not used after 20 minutes should be wasted. Grouts are susceptible to degradation by excessive water infiltration before the grout sets. Extensive use of flyash aggravates this problem. Therefore, flyash-based lightweight grouts are not recommended.

- *Sequencing and injection.*—The grouting equipment should be capable of mixing and delivering the grout at a rate that will allow the annular space to be entirely filled in a continuous operation, unless staged grouting is being used. The contractor should monitor the grout pressure. If the existing conduit has deflected vertically from a straight alignment, trapped air could result in a void in the grout.

  Grout injection can be accomplished by a number of methods, including gravity and pressure:

  1. *Gravity.*—Injection of grout into the annular space starts at the upstream end of the HDPE slipliner and progresses toward the downstream end, so as to more easily displace water and debris. Suitable injection tubes must be inserted at the upstream end. Vent pipes installed at the downstream end should be 150 percent larger than injection tubes, to minimize the potential for clogging. Dirty water and excess grout discharged from the downstream vent tube should be collected and disposed of properly. Grouting should continue until heavy grout exits from the downstream vent tube.

  2. *Pressure.*—HDPE grout pipes (typically 1 to 1½-inch diameter) are extrusion welded to the crown of the slipliner prior to installation. The designer will need to determine the required number and length of each individual grout pipe. The number of grout pipes required for a particular sliplining application is a function of the diameter of the pipe and the expected length of grout travel, once it leaves the end of the grout pipe. A rule of thumb used by Reclamation assumes about 25 to 30 feet of grout travel from the end of the grout pipe. For example, if a deteriorated outlet

works conduit (150 feet in length) is to be sliplined, 4 grout pipes (120, 90, 60 and 30 feet in length) would be needed to grout the annulus.

Bulkheads are placed around the annulus of the slipliner at both ends of the conduit to contain the grout. The bulkheads must be secured in place and sealed, so no leakage of grout will occur during grouting operations. Air vent bleeder systems are installed through the bulkheads, near the crown of the existing conduit at both ends of the conduit to prevent air and bleed water from being trapped within the annular space.

Injection of grout into the annular space starts at the downstream end of the HDPE slipliner and progresses upstream through the longest grout pipe first, while low pressure air (5 lb/in^2) is pumped through the downstream air vent. The air pressure assists in holding the grout in the annulus space. Grout pressures should be kept as low as possible to avoid collapsing the HDPE slipliner. As grouting begins, the upstream air vent through the bulkhead remains open until grout begins to flow from the vent, and then the vent is closed.

Grouting continues in the longest pipe until no air returns from the next longest pipe, or grout no longer flows through the longest pipe. The longest pipe is plugged and grouting is initiated on the next longest grout pipe and the sequence is repeated for this pipe. Pumping of air in the air vent is discontinued when the shortest grout pipe on the conduit is being grouted. Grouting of the last grout pipe is continued until the annular space is fully grouted. When the annular space is fully grouted and heavy grout returns from the downstream air vent, a grout pressure of 10 lb/in^2 is maintained for 10 minutes to ensure all voids are filled.

Postinspection and acceptance.—The completed HDPE slipliner (figure 176) should be visually inspected by trained personnel to evaluate the conditions within the renovated conduit. If the sliplined conduit is too small for man-entry inspection, CCTV inspection methods should be used. Some designers may want to consider the use of white or gray HDPE pipe to reduce glare using CCTV equipment, figure 177 shows an example of this type of pipe. See section 9.5.2 for guidance on inspection of conduits. No localized dimpling or distortion of the HDPE slipliner wall or infiltration of groundwater or grout should be present.

Maintenance and repair of the completed HDPE slipliner.—No maintenance is typically required for the HDPE sliplined conduit, unless the conduit requires some type of cleaning. Periodic operation of the conduit usually is sufficient to flush sediments through the system. HDPE pipe is smooth and generally resists the adherence of sediment deposits. See section 9.6 for guidance on cleaning of conduits.

Figure 176.—Completed HDPE slipliner in existing CMP spillway conduit. Photo courtesy of Maryland Dam Safety Division.

If the HDPE slipliner experiences some type of damage over the long term, the damage should be assessed by trained personnel using man-entry or CCTV inspection methods as discussed in section 9.5. Repair of HDPE pipe after installation and grouting is completed is not practicable for the buried sections of pipe. Very little can be done to effectively repair the existing HDPE slipliner in buried sections of pipe within the embankment dam. However, another HDPE slipliner of smaller diameter can usually be inserted and grouted in place. Sections of HDPE pipe that are exposed and accessible may be repaired by cutting out the damaged section and replacing the entire section of pipe. Further guidance on replacement and methods available for joining pipe are provided in CPChem's *The Performance Pipe Engineering Manual* (2003).

For examples of projects that have utilized HDPE sliplining, see the case histories in appendix B for Round Rock and Twin (Turtle) Dams.

Alternatives to sliplining existing conduits with solid walled HDPE pipe.—A newer application of HDPE pipe for sliplining existing conduits involves the use of dual containment HDPE pipe. This application is recommended, if the HDPE slipliner is to be pressurized. Dual containment HDPE pipe is manufactured as two separate HDPE pipes and assembled by placing one inside the other. The inside pipe (containment pipe) is centered within the outer pipe (carrier pipe) with end spacers (centralizers) located at each end of a section of pipe. The end spacers are fabricated for a tight fit and are extrusion welded to the dual wall containment pipe.

Figure 177.—White HDPE pipe can reduce glare when using CCTV inspection equipment.

Intermediate spacers (known as spiders) are placed at intermediate points between the end spacers to provide additional support. The distance between the spiders is a function of the pipe diameter and wall thickness. Large diameter, thick walled pipe does not require the spiders to be as close as small diameter, thin walled pipe. The joints of the dual containment pipe are joined using the same heat fusion method, as used for joining single walled HDPE pipe.

The annulus between the existing conduit and the outside pipe is grouted. However, the annulus between the inside and outside pipe of the dual containment pipe remains open, even after installation of the end spacers and spiders, and upon completion of the heat fusion of the joints. This is one of the most desirable aspects of this type of pipe. The open annulus allows downstream detection of any leakage from the inside pipe, while still having full containment protection provided by the outside pipe. The end spacers and spiders are designed with openings to allow any leakage to pass through the annulus between the two pipes and exit at a downstream location. The dual containment pipe provides a redundancy and additional safety factors to the system. The inside pipe is rated at 75 percent of the normal bursting pressure due only to the inability to inspect the outside surface of the heat fusion joints. For this reason, the inside pipe is typically designed for a 33 percent higher pressure than the outside pipe.

When sliplining an existing CMP, the CMP is typically assumed to be corroded to the point that it cannot be relied upon to provide any strength. With the dual containment HDPE pipe, both pipes can be designed for the full expected loading. This allows a factor of safety of at least 2 without relying on exterior conditions, such as the existing CMP or the annulus grouting.

An additional benefit of the dual containment pipe is that the inside pipe is not subjected to any outside protrusions within the existing conduit that could possibly damage the pipe. The inside pipe is protected by the outside pipe, so if there is any damage the outside pipe will protect the inside pipe. Figure 178 shows an example of dual containment pipe.

Dual containment HDPE pipe weighs approximately twice as much as a solid walled pipe with a diameter equal to the outside pipe. The cost for materials and installation of the dual containment pipe is typically slightly more than twice what a solid walled pipe might cost. The higher cost is mainly due to increased labor required for heat fusion and installation of the dual containment pipe.

When sliplining an existing conduit with a solid walled HDPE pipe, the discharge capacity is normally not reduced because any loss in flow area is compensated by the smoother surface of the HDPE pipe in comparison to CMP, concrete, etc. This is due to the extremely low hydraulic friction in the HDPE pipe. With the dual containment pipe, the cross sectional area is further reduced by the smaller diameter of the inside pipe. The smaller cross sectional flow area may result in a net loss in discharge capacity compared to the original capacity of the existing conduit. The loss in capacity is dependent on the diameter of the inside pipe of the dual containment pipe. The loss in discharge capacity will be a smaller percentage of the original capacity as the existing conduit diameter gets larger.

Figure 178.—A 14-inch diameter interior HDPE pipe is being inserted into a 20-inch diameter outside pipe. Intermediate spacers are used to keep the interior HDPE pipe centered and supported. The annulus grouting between the existing CMP and exterior HDPE pipe has been completed.

HDPE joint fusion machines require specialized contractors, and mobilizing them to a construction site can add to the cost of a sliplining project, especially for smaller embankment dams. For some nonpressurized projects in low hazard embankment dams, an alternative product, such as "Snap-Tite®" may be used. This proprietary product consists of gasketed joint grooves machined onto lengths of standard HDPE pipe. This alternative is not appropriate for use with pressurized conduits, due the possibility of leakage through the gasketed joints. This type of pipe is lightweight, and sections of the pipe can be easily handled by three or four workers and a backhoe (figure 179). After the first section of the pipe is inserted into the existing conduit, the next section is aligned with the first section and the joint is lubricated and pulled together with "come-along hoists" and chains wrapped around the liner. As in the usual method of installing HDPE slipliner, the first section of the pipe is tapered to allow the leading edge to ride over irregularities in the existing conduit. The joined sections of pipe can be pulled or pushed into the existing conduit. Pulling from the upstream end is preferred, because in some cases, excessive force used to push the liner from the downstream end has damaged some joints. The blocking and grouting procedure would be the same as that for fused HDPE pipe. For further design and installation details, see the manufacture's data.

The primary advantage of using Snap-Tite instead of standard heat-fused HDPE pipe is reduced installation costs, since a fusion machine is not needed. Disadvantages include the high cost of the proprietary pipe (due to the cost of machining of the joint grooves) and that the product has a relatively thin wall, so it may not be suitable under high embankment dams. The product usually consists of

Figure 179.—Sections of proprietary Snap-Tite® pipe can be easily handled with small equipment.

pipe with an SDR of 26 or 32.5, although heavier pipes may be available for custom applications. The designer will need to evaluate the suitability of this type of product for their project. For an example of a conduit renovation at a low hazard embankment dam using Snap-Tite, see the Rolling Green Community Lake Dam case history in appendix B.

12.1.2 Steel pipe

In most applications, steel pipe slipliners are used within accessible conduits. Section 12.1.2 mainly pertains to conduits accessible by man-entry. However, some discussion of steel pipe slipliners used within inaccessible conduits is provided.

Steel pipe slipliners (figure 180) are generally applicable, if the existing conduit is straight and does not have bends or significant invert slope changes. Steel pipe slipliners can be installed in conduits with bends or slope changes, if adequate clearance will allow for the insertion of the fabricated pipe sections.

12.1.2.1 Design considerations

The designer must evaluate a number of design parameters when considering steel pipe for use in sliplining. A few of the most significant design parameters include:

- Seepage paths
- Service life
- Initial inspection of the existing conduit
- Selection of the diameter and thickness
- Thermal expansion/contraction
- Stress cracking
- Fabrication
- Joints
- Flotation
- Entrance and terminal structures

Figure 180.—The Lake Linganore Dam outlet works consists of a 48-in diameter RCP conduit with a sluice gate located at the downstream end, which places the conduit under full reservoir head at all times. Concern about the integrity of the RCP joints led to installation of a steel slipliner when the inoperable sluice gate was replaced.

These parameters are further discussed in the following paragraphs. On a case-by-case basis, the designer may need to consider additional parameters, depending on the performance criteria and design requirements of the specific application.

Seepage paths.—For a discussion of seepage paths refer to section 12.1.1.

Service life.—The service life of a steel pipe slipliner is considered to be indefinite, if the coatings on the interior surface are properly maintained. However, improper maintenance of the interior surface could result in deterioration of a steel pipe slipliner in less than 25 years. Protective coatings are rarely completely effective, because even on application, they contain discontinuities, such as pinholes, flaws, scratches, and connected porosity. The use of a cathodic protection system (CPS) may be applicable in certain situations. A CPS has been used in conjunction with protective coatings, have been effective in controlling corrosion. A CPS consists of anodes that pass a direct current to the steel pipe liner through the electrolyte environment. With a CPS, the whole steel pipe slipliner becomes a cathode and does not corrode. CPSs can be one of two types, galvanic anode or impressed current anode. Hybrid CPSs can contain both types of anodes to provide protective current to all surfaces of the protected steel pipe slipliner (USACE, 1999, p. 1-1).

Initial inspection of the existing conduit.—For a discussion of the initial inspection of an existing conduit refer to section 12.1.1.

Selection of the diameter and thickness.—The selection of the diameter and thickness of the steel pipe slipliner should consider the following factors:

- *Size and condition of the existing conduit*.—Similar size and condition requirements are needed for steel pipe slipliners as are used for the design of HDPE slipliners, see section 12.1.1 for further details.

- *Discharge requirements*.—Similar discharge requirements are needed for steel pipe slipliners as are used for the design of HDPE slipliners, see section 12.1.1 for further details.

- *Clearance requirements for grouting of the annulus between the existing conduit and the steel pipe slipliner*.—Similar clearance requirements are needed for steel pipe slipliners as are used for the design of HDPE slipliners, see section 12.1.1 for further details. However, it should be noted that flanged joints generally require more clearance than heat fused HDPE joints or welded steel joints.

- *Internal and external loadings*.—Similar loadings can be expected on steel pipe slipliners as are used for the design of HDPE slipliners; see section 12.1.1 for further details. Steel pipe slipliner thickness requirements are generally governed by external pressures and the potential for buckling during future unwatering of the slipliner. Steel pipe should be designed in accordance with industry-accepted methods, such as those found in AWWA M11 (AWWA, 2004c), Amstutz (1970), and Jacobsen (1974).

Thermal expansion/contraction.—Thermal expansion/contraction is generally not a concern with steel pipe slipliners, as long as it does not have any portion exposed to the environment.

Stress cracking.—Stress cracking is generally not a concern with steel pipe slipliners.

Fabrication.—Steel pipe used for sliplining should be fabricated in accordance with AWWA C200 (1997) and ASTM A 36 and A 53. The steel pipe slipliner should be hydrostatically tested at the factory based on the design pressures. The steel pipe slipliner should be coated as specified by the designer.

To avoid delays to the construction schedule, the steel pipe slipliner should be shop fabricated while other site preparations for installation of the slipliner are being performed (e.g., construction access, or concrete removal). Depending on the diameter and length of the steel pipe slipliner, it may be advantageous to shop fabricate the slipliner in sections, so finished sections can be installed while other segments are being fabricated. Sometimes a separate procurement contract for the steel pipe slipliner is issued early, so fabrication can begin sooner.

Joints. - Steel pipe slipliner joints should be welded in accordance with AWWA C206 (2005). Flanged steel joint rings should be fabricated in accordance with AWWA C207 (2002). "Full face" rubber gaskets between the flanges are required in order to ensure a watertight joint. Fittings should have a factory-applied coating to protect against corrosion. Bolts should be of stainless steel or low allow steel and should be field coated after installation.

Flotation.—Flotation is a concern for steel pipe slipliners, but not as much as it is for thermoplastic slipliners. The weight of the steel pipe slipliner helps to prevent it from being displaced upward by the fluid pressure of the grout in the annulus between the existing conduit and steel pipe slipliner. Spacers or jack screws are always required to secure the steel pipe slipliner from movement during grout placement.

Entrance and terminal structures.—The steel pipe sliplining of an existing conduit may require partial or full removal and replacement of certain structures to improve release capabilities or to facilitate construction. For guidance on the design and construction of entrance and terminal structures, see section 3.4.

12.1.2.2 Construction considerations

The designer must evaluate a number of construction parameters when considering steel pipe for use in sliplining. A few of the most significant parameters include:

- Handling and storage
- Installation
- Repairs to the steel pipe slipliner prior to or during the insertion process
- Grouting
- Postinspection and acceptance
- Maintenance and repair of the completed steel pipe slipliner

These parameters are further discussed in the following paragraphs. On a case-by-case basis, the designer may need to consider additional parameters, depending on the construction requirements of the specific application.

Handling and storage.—During loading, transportation, unloading, storage, and laying, every precaution should be taken to prevent damage to the steel pipe slipliner sections, linings, coatings, and flanges. Steel pipe slipliner sections should be stored on timber blocking and adequately protected from weather and damage. Any damage to lining or coating will need to be properly repaired. If proper repair cannot be made, the damaged section will need to be replaced.

Installation.—The steel pipe slipliner should be supported and braced to prevent distortion during installation and grouting. Spacers are required to maintain separation between the existing conduit and the steel slipliner during grouting. Figure 181 shows spacers being installed on a steel pipe slipliner. Guidance on installation includes the following. Additional guidance on steel pipe installation is available in AWWA M11 (2004c).

- *Preparation of existing surfaces*.—The existing conduit surfaces, against which grout will be placed, should be free of latence, dirt, dust, grease, oil, loose or defective concrete, curing compound, coatings, and other foreign materials. Any sediments or debris should be removed from the invert of the existing conduit. Any bolts or other projections should be cut off flush and/or ground smooth with the interior surface of the existing conduit.

  If the existing conduit is full of water, considerations should be made for unwatering. A thorough inspection of the existing conduit is required prior to installing the steel pipe slipliner to ensure no obstructions remain that may hinder slipliner insertion.

- *Leak testing joints* .—Steel pipe slipliner joints should be welded and tested in accordance with AWWA C206 (2005), except testing of field welds should be by the ultrasonic method. Ultrasonic examination of field welds should conform to the requirements of American Welding Society (AWS) D1.1 (2004). Any offsets that could reduce the hydraulic capacity of the steel slipliner should be ground flush and flared into the adjacent surfaces. In lieu of ultrasonic examination, the field welded joints can be tested by the liquid penetrant method in accordance with ASTM E 165. The liquid penetrant method provides an indication of the presence, location, and to a limited extent, the nature and magnitude of any discontinuities. Welds can also be tested using radiographing and magnetic-particle methods.

- *Access and insertion*.—The steel pipe slipliner is typically installed by pulling, pushing, or a combination of both similar to an HDPE pipe as discussed in section 12.1.1.

Repairs to the steel pipe slipliner prior to or during the insertion process.—
Damage to the steel pipe slipliner may occur from improper shipping and handling

Figure 181.—Spacers being installed on a steel pipe slipliner.

or from poor insertion technique. Damage can be in the form of kinks, punctures, breaks, or abrasion. Steel pipe slipliners that undergo this type of damage can be easily repaired, if the pipe has not been inserted into the existing conduit. The repair would typically involve cutting out the damaged area of steel pipe slipliner and welding a new piece of steel plate in place. An installed slipliner with a damaged area can only be repaired in place if it is of sufficient diameter to allow man-entry. If the steel pipe slipliner diameter is too small for man-entry, the section of pipe will need to be removed to perform the proper repair.

Grouting.—Careful grouting of the annular space between the existing conduit and the steel pipe slipliner is essential. This can be a complex process, requiring the experience of a qualified contractor. A lightweight, low density grout containing no aggregate will ensure the best result. The following guidance discusses the grout plan, mix design, sequencing, and injection:

- *Grouting plan*.—A grouting plan detailing the contractor's grout mix equipment, setup, procedures, sequencing, plan for handling waste, method for communication, and method for sealing and bulkheading the upstream and downstream should be submitted for review and approval prior to initiation of grouting operations. The contractor should also provide the method to be used to remove trapped air from any high points in the existing conduit during the grouting operation. All welding and inspection of the steel pipe slipliner should be completed before grouting operations commence.

- *Grout mix*.—For guidance on grout mix, see section 12.1.1. The grout mix used for grouting of the annulus for a steel pipe slipliner will be similar to that used for grouting of the annulus of an HDPE slipliner.

- *Sequencing and injection*.—For guidance on sequencing and injection of grout, see section 12.1.1.

  Grout injection can be accomplished by a number of methods, including gravity and pressure:

  1. *Grouting for accessible existing conduits*.— Threaded couplings are installed through the steel pipe slipliner, from which grouting operations can be performed. After grouting operations have been completed, a pipe plug is installed in the threaded coupling, tightened, seal welded, and ground flush with the interior surface of the steel pipe slipliner.

     Grouting is best accomplished in lifts (four lifts are recommended for large diameter steel pipe slipliners). For each lift, grouting should begin at the downstream end of the conduit and proceed upstream.

     Grout injection should begin at the lower ports in the steel pipe slipliner and progress to the next higher ports. Grouting should begin by injecting through the lower ports and continue until grout return is observed at the next higher ports. Recommended port locations are at the invert, 45 degrees each side of the invert, both springlines, 45 degrees each side of the pipe crown, and the pipe crown.

     Grout should be pumped at pressures not exceeding 10 lb/in^2 at the injection ports. Any trapped air along the high points in the annular space should be expelled through air vents located on the crown. Rings of grout ports should be spaced at about 40-foot intervals. The designer should determine the required number and location of all ports and actual grouting pressures to be used. The steel pipe slipliner position, circularity, and shape should be monitored during grouting operations. Upon completion of grouting, grout plugs should be installed and ground flush with the steel liner surface.

  2. *Grouting for inaccessible existing conduits*.— Grouting of steel slipliners will be very similar to that used for grouting of HDPE slipliners. The main difference being the use of steel grout and vent pipe, welded to the steel slipliner. Full encapsulation for the entire length of the annulus rarely is

achievable, and the steel pipe slipliner is typically designed to withstand all internal and external loadings independently from exterior conditions.

Grout injection can be accomplished by a number of methods, including gravity and pressure:

- *Gravity.*—For guidance on gravity grouting, see section 12.1.1.

- *Pressure.*—Steel pipe slipliner grout pipes (typically 1 to 1½-inch diameter) are welded to the crown of the slipliner prior to installation. For guidance on pressure grouting, see section 12.1.1.

Postinspection and acceptance.—Trained personnel should visually inspect the completed steel pipe slipliner installation to evaluate the conditions within the renovated conduit. If the sliplined conduit is too small for man-entry inspection, CCTV inspection methods should be used. See section 9.5.2 for guidance on inspection of conduits. No damage to linings or coatings, or infiltration of groundwater should be present.

Maintenance and repair of the completed steel pipe slipliner.—Maintenance typically performed on steel pipe slipliners involves coatings on the interior of the slipliner. The interior surface of the pipe may require periodic recoating, if it has been subjected to abrasion or corrosion. The only other maintenance required for steel pipe slipliners would involve cleaning of the conduit. Periodic operation of the conduit usually is sufficient to flush sediments through the system. The steel slipliner is smooth and generally resists the adherence of sediment. See section 9.6 for guidance on cleaning of conduits.

If the steel slipliner experiences some type of localized damage (abrasion, buckling, cavitation, corrosion, etc.) over the long term, the damage should be assessed by trained personnel using man-entry or CCTV inspection methods as discussed in section 9.5. Repair of steel pipe slipliners after installation and grouting is completed can be accomplished, if the pipe is of sufficient diameter to allow man-entry. If the damaged area of steel pipe is not extensive, the repair would typically involve cutting out the damaged area and welding a new piece of steel plate in place or applying weld material in the damaged area. The welded material would be ground to a smooth finish. If the steel pipe slipliner diameter is too small for man-entry, a steel pipe slipliner of smaller diameter can usually be inserted and grouted in place.

For examples of projects that utilized steel pipe sliplining, see the case histories for Como and McDonald Dams in appendix B.

12.2 Cured-in-place pipe

Plastic cured-in-place pipe (CIPP) liners are typically used within inaccessible conduits. However, guidance provided in section 12.2 basically applies to lining of accessible or inaccessible conduits. CIPP liners are best suited for existing conduits that are not severely damaged or deformed and have constant diameters and no sharp bends. The designer should consider the method of CIPP liner installation as part of the design process. CIPP liners can be inserted into the existing conduit by either the inversion method (ASTM F 1216) or the pulled-in-place method (ASTM F 1743).

Additional information on CIPP liners used in sewer and pipeline application is available in USACE's *Guidelines for Trenchless Technology: Cured-in-Place Pipe (CIPP), Fold-and-Formed Pipe (FFP), Mini-Horizontal Directional Drilling (Mini-HDD), and Microtunneling* (1995d) and ASTM D 5813.

12.2.1 Design considerations

The designer must evaluate a number of design parameters when considering CIPP lining for conduits. A few of the most significant design parameters include:

- Seepage paths
- Service life
- Initial inspection of the existing conduit
- Selection of the diameter and thickness
- Thermal expansion/contraction
- Stress cracking
- Fabrication
- Joints
- Flotation
- Entrance and terminal structures

These parameters are further discussed in the following paragraphs. On a case-by-case basis, the designer may need to consider additional parameters, depending on the performance criteria and design requirements of the specific application.

Seepage paths.—For a discussion of seepage paths, refer to section 12.1.1.

Service life.—Research conducted by the Trenchless Technology Center at Louisiana Tech University found that the service design life of CIPP liners generally exceeds 50 years. The inversion tube processes, in which a resin-impregnated tube is cured in place, may not be suitable for lining bituminous coated CMP conduits unless they are prelined to prevent contamination of the resin by chemicals present in the asphalt coating (USACE, 1990, p. 3).

Initial inspection of the existing conduit.—For a discussion of the initial inspection of an existing conduit refer to section 12.1.1.

Selection of the diameter and thickness.—The selection of the diameter and thickness of the CIPP liner should consider the following factors:

- *Size and condition of the existing conduit*.—The size of the existing conduit will limit the diameter of the CIPP liner. A determination of the condition of the existing conduit is required to estimate the contributing support. A conservative assumption would be that the existing conduit is "fully deteriorated." For this assumption, the existing conduit provides no contributing support, and the CIPP lining needs to carry the external and internal loads resulting from the embankment dam and hydrostatic water pressures. A less conservative assumption would be an existing conduit in a partially deteriorated condition. For this condition, the existing conduit is assumed to be able to accommodate all internal and external loads for the life of the renovated conduit. If the existing conduit is large enough for man-entry, spot repairs of deteriorated areas may be considered prior to CIPP liner placement.

- *Discharge requirements*.—Similar discharge requirements are needed for CIPP liners as are used for the design of HDPE slipliners; see section 12.1.1 for further details.

- *Clearance requirements*.—Grouting is not normally required, since the CIPP liner fits tightly against the interior surface of the existing conduit. However, the designer needs to closely evaluate the applicability of a CIPP liner used for lining of a CMP conduit. The CIPP liner cannot tightly fit within the corrugations, and the ability to grout this annulus would be difficult.

- *Internal and external loadings*.—The CIPP liner design is based on the condition (partially or fully deteriorated) of the existing conduit, the type of application (pressurized or nonpressurized), and the resin and fabric tube material construction. If the existing conduit can support the soil and surcharge loads throughout the design life of the rehabilitated conduit, it is considered to be partially deteriorated for use in computing the required design thickness. Typically, If the existing conduit is not structurally sound and cannot support soil and live loads or is expected to reach this condition over the design life of the rehabilitated conduit, it is considered to be fully deteriorated for use in computing the design thickness. Other factors affecting the thickness of the CIPP liner are groundwater, soil types, and loadings on the existing conduit. Guidance for design should be in accordance with the manufacturer's recommendations and ASTM F 1216. Proper precautions are required to provide required venting (i.e., air vent or an air valve) to avoid collapse by internal vacuum pressures.

Thermal expansion/contraction.—Thermal expansion/contraction are generally not a significant concern with CIPP.

Stress cracking.—Stress cracking is generally not a concern with CIPP.

Fabrication.—The CIPP liner should be fabricated in a diameter size which will tightly fit the internal circumference of the existing conduit after installation. Allowance should be made for any circumferential stretching during the inversion process. The volume of resin should be sufficient to fill all voids in the tube material at nominal thickness and diameter. The resin volume may need to be adjusted by adding 5 to 10 percent excess resin to account for the change in volume due to polymerization and to allow for migration of resin into open cracks or joints in the existing conduit.

Joints.—Typically, CIPP liners are installed as one continuous length, and no joints are required.

Flotation.—Flotation is not an issue with CIPP, since the interior of the CIPP liner is filled with water during the curing process, and grouting is typically not required.

Entrance and terminal structures.—The installation of a CIPP liner into a existing conduit may require partial or full removal and replacement of certain structures to improve release capabilities or to facilitate construction. For guidance on the design and construction of entrance and terminal structures, see section 3.4.

12.2.2 Construction considerations

The designer must evaluate a number of construction parameters when considering CIPP lining for conduits. A few of the most significant parameters include:

- Installation
- Repairs to the CIPP liner prior to or during the insertion process
- Curing
- Grouting
- Postinspection, testing, and acceptance
- Maintenance and repair of the completed CIPP-lined conduit

These parameters are further discussed in the following paragraphs. On a case-by-case basis, the designer may need to consider additional parameters, depending on the construction requirements of the specific application.

Installation.—Upstream and downstream access as typically required for installation. Installation should be in accordance with the manufacturer's recommendations and ASTM F 1216 (inversion method) and ASTM F 1743 (pulled-in-place method). The manufacturer should provide the minimum and maximum allowable hydrostatic pressures. The use of a nontoxic lubricant is recommended to reduce friction during inversion. The use of an experienced CIPP liner installer is highly recommended.

- *Preparation of existing surfaces*.—The existing conduit surfaces should be free of roots, sediments, mineral deposits, and loose or defective concrete. Any sediments or debris should be removed from the invert of the existing conduit. Any bolts or other projections should be cut off flush and/or ground smooth with the interior surface of the existing conduit. See section 9.6 for guidance on cleaning of conduits. If the existing conduit is full of water considerations will be required for unwatering. A thorough inspection of the existing conduit is required prior to installing the CIPP liner to ensure no obstructions remain that may hinder CIPP insertion.

- *Access and insertion*.—Two methods of installation are available for insertion of the CIPP liner:

    1. *Inversion method*.—The inversion method (figures 182 through 184) consists of utilizing air or water (hydrostatic head) to push the CIPP liner inside-

out as it advances along the conduit. The inversion method is used for conduits with diameters ranging from 4 to 108 inches.

2. *Pulled-in-place method*.—The pulled-in-place method consists of a winch attached to a cable, which is attached to the CIPP liner and used to pull the liner into position. The liner is then inflated through the inversion of a calibration hose by the use of hydrostatic head or air pressure. The pulled-in-place method is usually only done where sufficient water pressures are not available or the scaffold towers required for the inversion process are not practical or where a particular lining is required. Insertion by the pulled-in-place method has some limitations due to the size and weight of the liner and possible resulting damage by moving the liner through the existing conduit. The pulled-in-place method is used for conduits with diameters ranging from 4 to 96 inches.

<u>Repairs to the CIPP liner prior to or during the installation process</u>.—Damage to the CIPP liner may occur from improper shipping and handling or from poor installation technique. Damage can be in the form of punctures, breaks, or abrasion to the CIPP liner or improper injection and care of the liquid resin. CIPP liner that experiences this type of damage must be replaced.

<u>Curing</u>.—After the CIPP liner is in place, a suitable heat source, water recirculating equipment, and temperature gauges are required to circulate heated water throughout. Curing of the CIPP must consider the existing conduit, resin system, and the surrounding embankment conditions, including temperatures, moisture levels, and thermal properties. Curing should be in accordance with the manufacturer's recommendations and ASTM F 1216. The cured CIPP liner will take the shape of the existing conduit, including any deformities. CIPP is usually thermally cured by the circulation of heated water (up to 82.2 °C or 180 °F). Alternative resins can be used that require lower curing temperatures, if thermal stresses are a design concern. However, once the curing process is complete, the CIPP liner is stable to heat and cannot be made to flow or melt again. Ultraviolet and ambient cure methods are alternatives to the thermal curing method, but these methods may have installation limitations or properties not equal to those of a thermally cured product. When completed, the CIPP liner acts as a new watertight lining within an existing conduit. After the curing process is completed, the ends of the CIPP liner can be trimmed flush as needed. If a service connection or air vent opening is required, this can be done by personnel using a special cutting device for conduits large enough for man-entry. If the conduit is too small for man-entry, a robotic crawler with a special cutter can be used to cut the required opening in the CIPP liner.

<u>Grouting</u>.—Grouting is not normally required, since the CIPP liner fits tightly against the interior surface of the existing conduit.

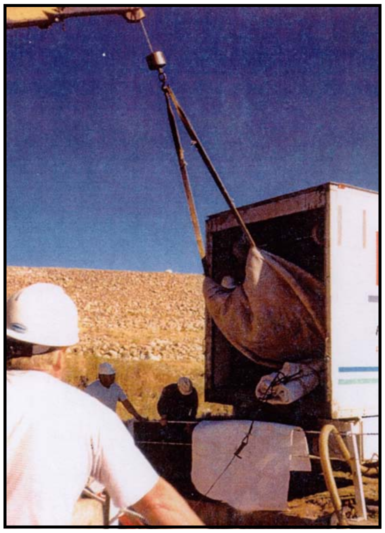

Figure 182.—Unloading the CIPP liner prior to installation.

Figure 183.—CIPP liner being positioned for installation.

Figure 184.—The hydrostatic inversion method is being used for CIPP liner installation into an existing conduit.

Postinspection, testing, and acceptance.— Trained personnel should visually inspect the completed CIPP installation. No dry spots, lifts, delamination, pinholes, or infiltration of groundwater should be present and the CIPP liner should be in a fully expanded condition. Wrinkles (figure 185) that could reduce the hydraulic capacity of the CIPP liner should not be allowed. If the CIPP-lined conduit is too small for man-entry inspection, CCTV inspection methods should be used. See section 9.5.2 for guidance on inspection of conduits.

Two samples (cut from the end of the cured CIPP liner) should be prepared and submitted for the purpose of acceptance testing. The samples should be prepared in accordance with ASTM F 1216 or ASTM F 1743 for flexural and tensile testing. Testing should be used to verify flexural properties in accordance with ASTM D 790 and tensile properties in accordance with ASTM D 638.

Maintenance and repair of the completed CIPP-lined conduit.—No maintenance is typically required for the CIPP-lined conduit, unless the conduit requires some type of cleaning. Periodic operation of the CIPP-lined conduit usually is sufficient to flush sediments through the system. The CIPP lining is smooth and

Figure 185.—The calibration hose of this pulled-in-place CIPP developed "fins." These fins were considered to have an insignificant effect on hydraulic capacity for this particular conduit application and were not removed.

generally resists the adherence of sediment. See section 9.6 for guidance on cleaning of conduits.

If the CIPP liner experiences some type of damage over the long term, the damage should be assessed by trained personnel using man-entry or CCTV inspection methods as discussed in section 9.5. Repair of the CIPP lining after installation and curing are completed is possible. Repair kits are available from the manufacturer.

For an example of a project that utilized a CIPP liner see the case history for Willow Creek in appendix B.

12.3 Spray lining

Spray lining is a conduit renovation method that has been used since the 1920s mainly for small diameter water mains (USACE, 2001d, p. 10). Spray lining typically involves the spraying of a cement mortar mixture or epoxy resin against the inside walls of the existing conduit. Trowels that trail the rotating sprayer head smooth the sprayed cement mortar. Spray lining has been used to retard iron pipe corrosion and reduce the rate of deterioration of the existing walls of water mains.

While this renovation method may have some limited applicability for low hazard embankment dams, it is not recommended for significant or high hazard embankment dams. The spray lining technique cannot ensure a long term watertight barrier and has a limited life expectancy.

Chapter 13

Replacement of Conduits

Generally, removal and replacement of an existing conduit through an embankment dam consists of excavating the dam down to the existing conduit, stockpiling the material, removing the existing conduit, constructing a new conduit and possibly new entrance and terminal structures, installing a filter diaphragm or collar around the downstream portion of the conduit, and replacing the embankment material. A cofferdam may also be required if the reservoir cannot be drained during construction.

Removal and replacement of a deteriorating conduit can be time consuming and expensive compared to other renovation methods. Typically, construction costs for removal and replacement may be 5 to 10 times higher than for sliplining or cured-in-place conduit renovation methods. This cost difference depends upon the height of the embankment dam. However, if the embankment dam is small and the downstream impacts to users are acceptable; this method may be more advantageous than renovation. Often, removal and replacement is the alternative of choice for low hazard embankment dams, since it generally is less expensive. This is especially true on older low hazard embankment dams, where they may have been built without adequate engineering. Few designers will want to try and guess how the embankment dam was built. The safer and more efficient solution would be to remove and replace the conduit and possibly the entire embankment dam..

The advantages of removal and replacement of an existing conduit through an embankment dam include:

- *Evaluation.*—The exposed foundation of the conduit can be fully evaluated.
- *Repairs.*—Areas along the existing conduit that may have been damaged by internal erosion or backward erosion piping can be repaired.
- *Seepage.*—Extensive seepage control measures along the conduit can be installed.

- *Design modifications.*—The new conduit can be designed to provide increased discharge capacity to meet current or future operational and emergency release requirements.

The disadvantages of removal and replacement of an existing conduit through a high embankment dam include:

- *Cofferdam.*—Unless the reservoir can be drained, the construction of a cofferdam is generally required. Inflows into the reservoir will need to be diverted. In some special cases a downstream cofferdam may also be required.

- *Costs.*—Construction costs for removal and replacement are generally higher than for other renovation methods. Construction costs rapidly rise as the height of the embankment dam increases.

- *Reservoir operations.*—Construction may impact reservoir operations and add risk to the downstream community.

- *Seepage paths.*—If proper compaction of the embankment closure section is not obtained, potential seepage paths may exist along the junction of the closure section and existing embankment

13.1 Embankment excavation slopes.

An excavation transverse to an existing embankment dam centerline increases the potential for hydraulic fracture of the replacement embankment material from arching. Because hydraulic fracture poses special hazards when the reservoir is subsequently refilled, special care is required for designs that involve excavation transverse to the existing embankment dam. The excavation should be wide enough to accommodate motorized compaction equipment, and the side slopes should be flat to reduce differential strain.

The guidance for excavation discussed in chapter 5 applies equally to construction involving removal and replacement of existing conduits. Excavations for conduits in soil foundations should be wide enough to allow for backfill compaction parallel to the conduit using heavy rolling compaction equipment. Equipment used to compact along the conduit should be free of framing that prevents its load-transferring wheels or drum from working against the structure (USACE, 2004a, p. 6-6).

13.2 Removal of the existing conduit

The first step in removal and replacement of the existing conduit is usually to excavate the embankment dam to the invert of the conduit and remove it. Removal of the entrance structure (figure 186), terminal structure, or other structures may be required due to age or deterioration, or to ease construction of the replacement structures. Occasionally, where removal of the existing conduit is difficult and expensive, the existing conduit may not be removed, but will be abandoned by backfilling the conduit with grout and installing a new conduit at a separate location. See section 14.3 for guidance on abandonment of conduits. Excavations should be wide enough at the bottom to ensure adequate working room for removal of the existing conduit and replacement with the new conduit, and compaction of earthfill materials.

A qualified professional engineer or engineering geologist should carefully observe and document the excavation required for the removal of the existing conduit to verify that any damaged embankment or foundation materials have been fully removed and/or treated prior to construction of the new conduit and replacement of embankment materials. For an example of the replacement of a conduit, see the Pablo Dam case history in appendix B.

13.3 Design and construction of the conduit

For guidance on design and construction of the conduit, see chapters 3 and 4.

13.4 Design and construction of the filter

For guidance on the design and construction of the filter, see chapter 6. The new filter should be designed to extend upstream into the embankment dam. Frequently, the filter installed in this situation is larger than that used for first time construction of an embankment dam. The filter should extend to both sides of the new conduit and key into the existing embankment dam. If the existing embankment dam has a chimney filter, the filter should be designed to be a part of that system where feasible.

13.5 Design and construction of the replacement embankment dam

The following sections discuss aspects of design and construction that the designer should consider for the replacement portion of the embankment dam.

Figure 186.—Excavator removing the intake structure for an existing outlet works.

13.5.1 Zoning

If the conduit is being replaced in a zoned earthfill embankment dam where a central core is substantially different in properties than the outside embankment shells, backfill for the conduit should coincide with the zoning for the embankment dam. Core zone backfill should only be used around the conduit through the core section, with shell backfill soils used through those sections of the conduit. An exception to this recommendation is where rock shell zones include large angular rocks that could impose point loads on the conduit that exceed its strength. For that condition, cushioning soil with small sand and gravel should encircle the conduit to prevent this problem.

13.5.2 Compaction considerations for backfill used in rebuilding the embankment dam

The soil removed from the embankment dam as the existing conduit is excavated is frequently reused to backfill the notch in the dam. Designers should carefully evaluate the water content of these soils and determine if drying or wetting is required for satisfactory reuse. The excavated slopes in the existing embankment dam may remain exposed for a period of time before they are backfilled. The time over which the excavation made to replace the conduit is left exposed may be hot, dry weather. In this case, the exposed soils on the face of the excavation may desiccate to considerable depths. Before commencing backfilling of the excavation in the embankment dam, any desiccation cracks in the existing dam must be removed, and the earthfill surface disked and moistened. This process will probably have to be delayed until immediately before backfill of an interval of the embankment dam is ready to commence. If backfilling of the excavation is interrupted during hot weather, the surface of the reconstruction backfill also should be closely inspected for desiccation features before placing new fill. Poorly bonded lifts can occur during interruptions of fill placement. They provide an avenue for possible internal erosion.

Designers should consider these important points:

- *Testing.*—Soils used to rebuild the embankment dam should be evaluated by the same tests that would be used to evaluate soils for a new embankment dam. The water content, plasticity, gradation, compaction properties, and dispersivity of clay fines are important evaluations. If the replacement fill is in a zoned embankment dam, similar zoning should be used.

- *Water content.*—Soils used to rebuild the embankment dam should usually be placed wet of Standard Proctor optimum water content to improve their flexibility and resistance to cracking and arching. Compacting soils at water contents that are 1 to 3 percent wet of optimum significantly improves their flexibility. At the same time, the likelihood that pore pressures could be generated in medium to high plasticity clays in fills of significant height should also be evaluated. Designers must weigh the advantages of compacting soils wet of optimum against the disadvantages of this wetter compaction water content. The lower shear strength and potential pore pressures generated by wetter compaction water contents must be considered in the design stability evaluations. Many designers consider excessive pore pressures to be a lesser long term danger to the successful performance of an embankment dam than the danger of arching and hydraulic fracture if the soils are placed dry.

- *Exposed filler.*—Special care to remove desiccation cracks in exposed fill surfaces is important. This applies to the exposed excavation slopes and to layers of fill used in reconstructing the embankment dam.

For additional guidance on hydraulic fracture and closure sections, see section 5.2.

13.6 Construction impacts

Generally, the construction period for a complete removal and replacement of a conduit will require more time than other renovation methods. Mitigating the impacts of a longer construction period may require consideration of the following:

- *Water requirements.*—Providing for diversion and downstream water requirements (irrigation, etc.).

- *Traffic control.*—Traffic control measures (lighting, signs, etc), road closures, construction of detours (such as detouring dam crest traffic).

- *Disturbance.*—Larger disturbance areas and potential environmental issues.

- *Draining or drawing down of the reservoir.*—Caution is required if the reservoir is drained, since the existence of a heavy bed load could move and block the intake structure.

13.7 Construction of a cofferdam or temporary diversion channel.

A cofferdam (figure 187) may be needed to act as a temporary barrier to protect the construction area from flooding (Reclamation, 1987a, pp. 499-500). If construction for the removal and replacement of the existing conduit can be performed during the low water season, the use of a cofferdam may be minimal. However, where the reservoir inflow characteristics are such that construction cannot be done during a low water season, the cofferdam must be designed for safety and for optimal height to accommodate the full range of expected inflows during the construction period. Often the flood selected for sizing diversion requirements is based on the projected length of the construction period. For instance, if the projected length of the construction period is 1 year, the cofferdam would need to be able to accommodate a flood with a return interval of 5 years. If the projected length of the construction period is 2 years, a 10-year diversion flood would be used.

At some sites, a diversion channel in lieu of a cofferdam may be more practical. Diversions or cofferdams may not be justified for smaller embankment dam projects with limited working room for the removal and replacement of conduits. For these projects, certain hydrologic events during construction may impede progress and cause delays resulting in added costs for clean up of debris caused by flooding. For this situation, construction should be scheduled during periods when the lowest rainfall is probable to avoid these problems. Figure 188 shows an embankment dam site where rainfall during construction has partially flooded the exposed excavation and conduit removal. This flooding resulted in a delay and cleanup before construction could resume.

13.8 Design of entrance and terminal structures

The removal and replacement of an existing conduit may require partial or full removal and replacement of certain structures to improve release capabilities or to

Figure 187.—Cofferdam constructed around the construction area for a new outlet works.

facilitate construction. For guidance on the design and construction of entrance and terminal structures see section 3.4.

13.9 Microtunneling

Microtunneling (also called "pipe jacking" or "boring and jacking") is a remotely controlled process for installing conduits underground without the need for excavation of a trench.

Microtunneling techniques should not be used for installation of conduits through embankment dams. Difficulties exist with obtaining a watertight seal along the conduit and potentially with disturbing the embankment dam during installation. Until emerging technology and procedures are significantly improved and shown to be reliable, it is recommended that this renovation method be restricted to installation of conduits in abutments and foundations. Installation of conduits in the abutments and foundations of embankment dams has been successfully performed. The discussion in this section only applies to conduit installation in abutments or foundations.

A successful microtunneling project requires detailed site investigation, appropriate consideration of design criteria, preparation of comprehensive bid documents, accurate contractor submittal information, careful execution by a highly skilled operator and crew, and a knowledgeable, experienced contractor (USACE, 1995d, p. xviii).

Figure 188.—During the removal of this outlet works conduit a flood occurred. No stream diversion or cofferdam was used at this site.

Microtunneling utilizes a two-step process. The first step is installation of and grouting around a liner pipe, also called a carrier pipe or shield. The second step involves installation of the permanent conduit and grouting of the annular space between the two pipes.

Microtunneling has been used to construct conduits up to about 84 inches in diameter. The tunneling technique uses a tunnel boring machine with the tunnel lining being jacked into place as the boring machine advances. The tunneling procedure is remotely controlled and operated and requires limited entry into the tunnel by construction personnel. Tunnel boring machines use laser guidance control systems. Machines are available to drive 300 feet or more in length in soft ground.

Microtunneling consists of a tunnel boring machine that is equipped with a cutting head slightly larger than the tunnel liner. The cutting head has to be carefully selected to deal with the expected ground conditions. Sections of the tunnel liner are assembled outside of the tunnel, and as the boring machine excavates ahead of the conduit liner, the tunnel liner is jacked into place. The cuttings from the tunneling machine are ground into small particles and removed with a bentonite slurry pumped to a location outside of the tunnel. A lubricating fluid is generally pumped into the annular space between the excavation and the tunnel liner to help facilitate the jacking of the liner. At the high slurry flow rates and velocities typically used in microtunneling, only a very short time is required to erode soil at the face and cause large settlements. The operator must vigilantly control slurry circulation and machine torque to avoid such unacceptable events (USACE, 1995d, p. 4-44). Figure 189 shows a microtunneling installation when an operator stopped advancement.

The selection of the microtunnel liner material is critical. The liner material must be strong enough to withstand the jacking forces and other anticipated loads that the liner could be subjected to during the installation and life of the liner. The liner should be watertight and able to withstand internal and external water pressures. The liner should also be able to withstand any external earth and grouting pressures exerted on it. Once installed, the liner should be strong and durable enough to provide dependable service for the life of the project. Liner material considerations consist of welded steel pipe, reinforced concrete steel cylinder pipe, plain reinforced concrete pipe, HOBAS pipe (a composite fiberglass pipe) and HDPE pipe. Welded steel pipe is the liner material used on most projects. Steel pipe is very suitable for jacking and generally requires a relatively thin-walled section compared to concrete pipe, which reduces the required tunnel excavation to install the steel pipe liner. Welding can be used to repair any damage to the steel pipe and for attaching fittings. The steel pipe has a relatively smooth exterior, which helps reduce the friction between the excavation and the liner as it is being jacked into place. The steel can also be designed to withstand all anticipated internal and external loads. If the tunnel boring bends slightly, the steel liner could buckle as it is forced around the bend. If the liner does buckle, it will be very difficult to repair.

Another consideration of the microtunneling technique is the selection of the liner joints. The joints should have adequate compressive, bending and tensile strength to withstand the forces required to jack the liner into place. Once installed, the joints between each section of microtunneled liner need to be able withstand both internal and external water pressures without leakage. The joints need to be perpendicular to the centerline of the pipe, square, and snugly fit, so the conduit will be straight and easy to steer with the tunnel boring machine. Once installed, the joints of the liner are generally exposed to very minor additional stresses.

Because the pipe can “set” or “freeze” in a location if jacking operations are stopped for any length of time, a continuous installation process may be necessary. Common practice dictates the use of hydraulic jacks with a capacity greater than anticipated in order to avoid this situation.

Once the liner is installed, the annular space between the excavation and the tunnel liner is generally grouted in a continuous operation. As the grouting procedure takes place, the lubricating fluid is forced out of the annular space. The grouting is intended to reduce or eliminate potential seepage along the tunnel liner. The designer should explicitly specify the requirements for the liner installation and grouting. In addition, the contractor should be required to submit for approval by

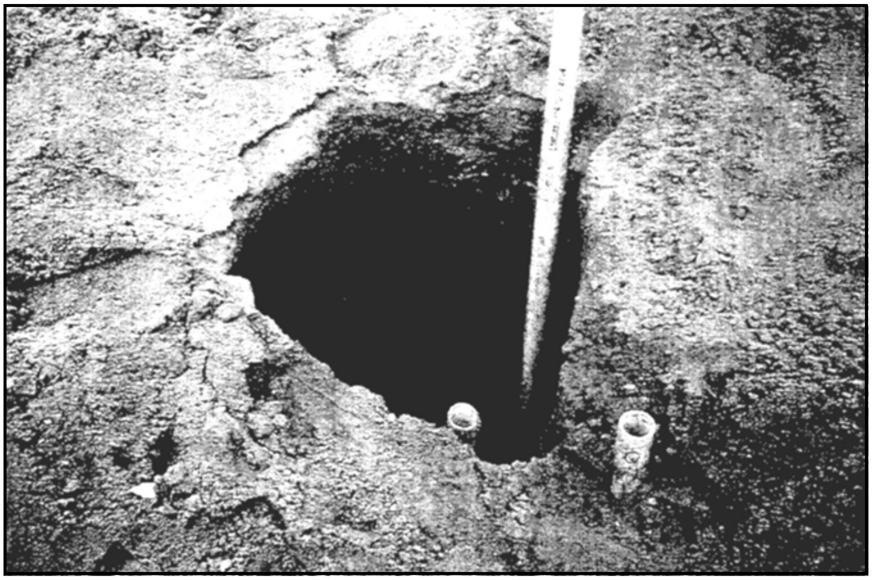

Figure 189.—A very large surface void appeared directly above a test microtunneling installation when the operator stopped advancement of the machine and briefly allowed the slurry pump to continue to circulate the drilling mud. The void appeared in backfill surrounding some test instruments, some of which appeared to support the adjacent soils.

the design engineer, the proposed procedure and materials prior to implementation. Worster, et al. (2002, p. 9) state:

> Many of the contractors doing this work may be accustomed to applications where seepage along the outside of the carrier pipe, or between the carrier and liner pipe, is not as significant of a concern. The focus on grouting and pipe placement requirements may be unusual for these contractors. For this reason, requirements for grouting the carrier pipe/embankment contact, placement of the liner pipe, and grouting the liner pipe/carrier pipe annulus should be specified in detail. A contractor submittal on the means and methods to accomplish these items should be required for approval, to ensure that all specification requirements are met and proper procedures are followed during construction.

Microtunneling with wet recovery of the tunnel boring machine is a relatively new and rapidly developing method of installing conduits into a reservoir partially or fully filled with water. For a wet recovery of the tunnel boring machine, a recovery pit is excavated into the reservoir bottom. The tunnel is advanced from the downstream side of the embankment dam in the direction of the pit. Once the tunnel has advanced to a point near the pit and where the advancing face of the tunnel is still stable, the tunneling is stopped and the hydraulic lines are disconnected from the tunneling machine. A temporary bulkhead is placed inside of the tunnel downstream of the boring machine. The boring machine and tunnel liner are then jacked for the final length of the tunnel, until the boring machine is pushed into the recovery pit.

The tunnel boring machine can then be recovered by a barge in the reservoir. An intake structure can also be installed by prefabricating it in the dry, floating it out to the required location, and sinking it into place.

If the upstream end of the pipe will be located below the reservoir elevation, then a cofferdam will be required. In one case, a jacking pit was excavated in one abutment of an embankment dam, from which pipes were extended to upstream and downstream cofferdams (Wooten, Fortin, and Walker, 1997, p. 518).

A properly designed filter diaphragm or collar should be constructed near the downstream end of the pipe to intercept and control potential seepage along the outside of the conduit. For guidance on the design and construction of the filter, see chapter 6.

13.10 Horizontal directional drilling

Horizontal directional drilling (HDD) was developed in the 1970s to install underground pipelines for the oil and gas industry without the need for trench excavation along the entire length of the pipe (USACE, 1998b, p. 1). This method is generally used on pipes smaller than about 12 inches in diameter.

Horizontal directional drilling techniques should not be used for installation of conduits through embankment dams. Difficulties exist with obtaining a watertight seal along the conduit and potentially with disturbing the embankment dam during installation. Until emerging technology and procedures are significantly improved and shown to be reliable, it is recommended that this renovation method be restricted to installation of conduits in abutments and foundations. Installation of conduits in the abutments and foundations of embankment dams has been successfully performed. The discussion in this section only applies to conduit installation in abutments or foundations.

HDD should be restricted to installation of conduits in embankment dam foundations, with the entrance and exit points located at least 300 feet from the dam (USACE, 2003a, p. 2). In controlled tests conducted by the USACE, hydraulic fracture of the embankment dam, ground subsidence, heave of the ground surface, and significant collapses have all occurred with HDD.

HDD consists of drilling a pilot hole through the soil with a fluid-powered cutting tool attached to a hollow drill stem. Drilling fluid under pressure is pumped through the drill stem to power the drill tool. The drill can be steered, both vertically and horizontally, so that HDD installations need not be straight. After the pilot hole is completed, it is often enlarged with a reamer. Then, a permanent pipeline is attached to the cutting end of the drill stem and "pulled back" through the hole as the drill stem is retrieved. The most common pipe material is HDPE, although PVC and steel can be used.

Legitimate concerns are associated with the fluid pressures used for excavation during the horizontal directional drilling process, and the potential for hydraulic fracturing of the embankment dam. Reasonable limits must be placed on maximum fluid pressures in the annular space of the bore to prevent inadvertent drilling fluid "returns" to the ground surface. However, it is equally important that drilling pressures remain sufficiently high to maintain borehole stability, since the ease with which the pipe will be inserted into the borehole depends upon borehole stability.

The drilling pressures should be measured in the borehole, not at the pump. The in-situ pressures should be compared with theoretical criteria that would cause hydraulic fracturing of the adjacent soils. In a test study by the U.S. Army Corps of Engineers, it was found that the pressure in an HDD hole drilled through a levee embankment was nearly independent of the drill rig pump pressure. Monitoring of piezometers in the embankment levee showed that while significant increases in pore pressures in the embankment occurred as drilling operations progressed, the pressures quickly dissipated to their original levels. Studies on plastic behavior showed that fluid pressure may cause hydrofracture (figure 190) when the pressure exceeds twice the value of undrained cohesion of the soil, that is, the unconfined compressive strength of the soil. Therefore, a pressure of 100 lb/in^2 would be expected to cause hydrofracture in a clay with an unconfined compressive strength less than 7.2 t/ft^2, a very stiff clay. According to these assumptions, for a compacted saturated clay with a soil unit weight of approximately 125 lb/ft^3, 8.7 lb/ft^2 for each 10 feet of depth would be required to cause hydrofracture. However, these studies did not address the propagation of hydrofracture through the soil but focused solely on the pressures required to initiate hydrofracture. Studies on the elastic behavior compared the stresses on the boundary of the hole and compared these stresses with the tensile strength of the soil. The coefficient of lateral earth pressure was varied to determine its effect on hydrofracture potential. As the ratio of horizontal soil stress to vertical soil stress approached unity, the stresses required to produce hydrofracture were comparable to those computed for the plastic deformation analysis. The fact that hydraulic fracture has not been observed in many HDD projects where the theoretical criteria have been exceeded makes it clear that important factors have been ignored (USACE, 1998b, p. 6).

A properly designed filter diaphragm or collar should be constructed near the downstream end of the pipe to intercept and control potential seepage along the outside of the conduit. For guidance on the design and construction of the filter, see chapter 6.

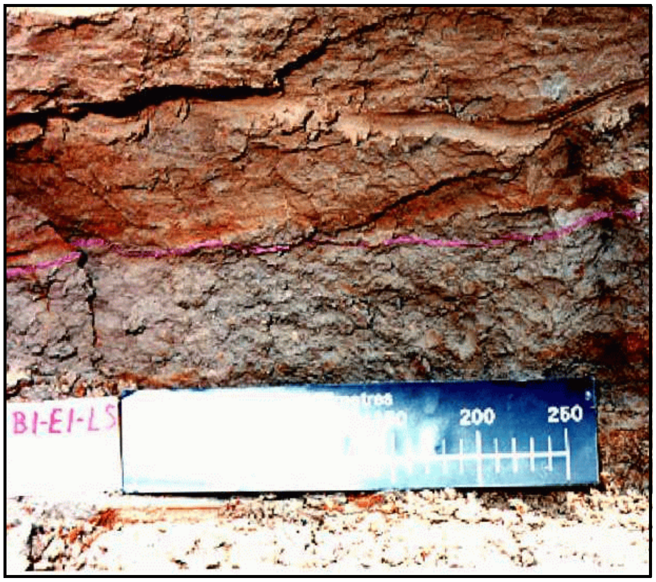

Figure 190.—Hydraulic fracture of embankment levee during HDD installation is visible in this photograph. The drilling mud, which was dyed pink, so that such fractures could be observed, followed an interface along a clay seam (USACE, 1998b, p. 29).

Chapter 14

Repair and Abandonment of Conduits

When water is flowing in an uncontrolled way through the embankment materials surrounding a conduit, some designers have attempted to grout these defects to reduce the water flow. This chapter discusses important considerations for grouting around conduits.

The chapter also discusses methods used to repair damaged conduits when complete replacement is not necessary. Finally, the factors that are important to consider if a decision is made to leave a conduit in place, but abandon it for use are discussed. Recommendations for filling the abandoned conduit and ways to protect against potential pathways for water to flow through the abandoned conduit are discussed.

Previous chapters have discussed methods for renovation and for replacement of conduits. This chapter discusses various other methods available for repairing or abandoning damaged and deteriorated conduits.

14.1 Grouting along the exterior of the conduit

Grouting around conduits is not recommended as a sole solution for prevention of internal erosion or backward erosion piping. Grouting will not likely provide 100-percent encapsulation of the conduit, and seepage gradients in the 'windows' in the grout may actually be higher than the initial gradients before grouting and should always be combined with a downstream filter diaphragm or collar. Grouting can be used to fill or partially fill voids created by internal erosion or backward erosion piping, to reduce future settlements, but filter diaphragms or collars or other positive means must be used to prevent internal erosion. Water can penetrate cracks that cannot be grouted closed.

Generally, grouting along the exterior of an existing conduit consists of injecting cement or chemical grouts into voids. Grouting can also be used to seal leaking joints in the existing conduit. Grouting can be performed from the interior of the existing conduit, if man-entry is possible. If man-entry is not possible, grouting must be performed from the surface of the embankment dam.

The advantages of grouting along an existing conduit through an embankment dam include:

- *Impacts*.—Construction impacts to downstream users are minimized.

- *Costs*.—Construction costs are generally lower (initially) than for other renovation or replacement methods. However, grouting along conduits is not always fully successful and should not be considered a permanent repair. This method does allow for additional grouting attempts to be made in the future.

The disadvantages of grouting along a conduit through an embankment dam include:

- *Not permanent*.—This method should not be considered a permanent renovation method. This method will have limited effect on corrosion prevention or improvement of structural integrity of the conduit.

- *Voids*.—The filling of all voids surrounding the existing conduit cannot be verified.

Grouting from the interior of man-entry accessible existing conduits generally provides a higher degree of success in filling voids outside the existing conduit and sealing joints. Geophysical methods can be used to identify areas of suspected voids. See chapter 10 for guidance on geophysical methods. These areas can be drilled and a small video camera inserted to determine the extent of the void. Accommodations will need to be made for removal and control of water leaking into the existing conduit as the grout injection work is performed. Partial lowering of the reservoir will reduce leakage into the existing conduit. While work is going on within the existing conduit, a pumping system to discharge reservoir inflows may be required to keep the reservoir at the desired elevation or to meet downstream requirements.

Existing conduits that are not accessible to man-entry generally have limited success in filling voids outside the conduit and sealing joints. Drilling is usually accomplished from the surface of the embankment dam. As the height of the embankment dam increases, so does the degree of difficulty for injecting grout at the desired location along the existing conduit. Grout is advanced from the surface through the drill hole to the void using pressure. The designer is cautioned that this method, unless carefully controlled, has the potential for causing hydraulic fracture within the embankment dam. Drilling from the surface of the embankment dam is not advisable for situations where the reservoir water surface cannot be lowered. If the reservoir cannot be lowered, grouting along the upstream portion of the conduit is not practicable. See section 14.3 for additional guidance on drilling in embankment dams.

In the repair of voids along the outside of the conduit at Lake Tansi Dam (Heckel and Sowers, 1995), grout was placed through borings drilled from the top of the embankment dam using only the pressure from gravity (no pump). Holes with large grout "takes" should be redrilled and grouted again to ensure that the voids are filled. The designer should strategically select the location of any additional holes to maximize the filling of voids.

In order to minimize the potential for accidentally filling the conduit with grout, any cracks or open joints in the conduit should be first repaired. For guidance on repair techniques, see section 14.2.

Any cracks or other defects should be plugged prior to grouting operations. Unless conditions indicate otherwise, grout holes should be drilled and grouted in an appropriate pattern beginning at the downstream end of the conduit at the pipe invert progressing up and around the pipe and from the downstream to the upstream end of the conduit. Figure 191 shows the drilling of a grout hole from the interior of an existing conduit.

Grouting should progress so that grout is pushed in the direction of increasing water pressures and up around the conduit to maximize grout penetration and filling of voids, and to minimize the potential for erosion of grout by flowing seepage waters. If suitably safe conditions exist, several rows of grout holes in advance of the

Figure 191.—Drilling a grout hole from the interior of a conduit.

grouting operations should be opened to monitor the progress of grouting and observe any connections between grout holes due to existing voids around the conduit. This will assist in the assessment of areas where notable erosion and embankment damage has occurred and the need for secondary and tertiary grout holes.

Prior to any grouting or other conduit remediation work, consideration should be given to the installation of appropriate instrumentation in the vicinity of the conduit in order to (1) establish baseline water pressure and seepage gradient conditions prior to repair work, and (2) detect any changes to embankment and foundation water pressures that may result from corrective actions. Depending on the nature of the problem and the corrective actions, such instrumentation will indicate whether the repairs are successful in lowering embankment water pressures, or when repairs may cause undesirable changes that require further corrective actions.

Grouts used for injection into defects or joints and along conduits are usually cementitious or chemical.

- *Cementitious.*—For guidance on cementitious grout, see section 14.2.2.

- *Chemical.*—For guidance on chemical grout, see section 14.2.2. Figure 192 shows grouting operations within a conduit. Chemical grouts have also been used for grouting voids using drill holes from the surface of the embankment dam. For additional guidance on chemical grouts, see USACE's *Chemical Grouting* (1995a).

See the Lake Darling case history in appendix B, for an example of grouting of voids existing along a conduit.

14.2 Repair techniques

Repairs to conduits are typically performed to prevent further deterioration or prior to more extensive renovation methods. Repairs often are done to stop the inflow of water into the conduit through cracks or joints. The following sections mainly pertain to the repair of concrete conduits. For repair guidance for plastic pipe, see sections 12.1.1 and 12.2; for repair guidance on steel pipe, see section 12.1.2.

14.2.1 Concrete repairs

Concrete used in modern conduits is very durable and if properly proportioned and placed will provide a very long service life under normal operating conditions (Reclamation, 1997, p. 1). However, many existing conduits were constructed years ago using early concrete technology. Neglecting to perform periodic maintenance and

Figure 192.—Injection of urethane grout to stop leakage.

repairs could result in continued deterioration and/or failure. A successful concrete repair depends on quality of workmanship, procedures followed, and materials used. A systematic approach to repair should be followed. Sources, such as the Bureau of Reclamation, U.S. Army Corps of Engineers, American Concrete Institute, Portland Cement Association, the International Concrete Repair Institute, and private authors have developed good systems and methodologies. A summary of Reclamation's concrete repair system is presented here as an example (see Reclamation's *Guide to Concrete Repair* [1997] for a detailed discussion on concrete repair):

- *Determine the cause(s) of damage.*—The cause of damage or deterioration (e.g., abrasion, cavitation, poor design and construction, etc.) to the original concrete must be assessed (figure 193), or else the repair of concrete may also become subject to the same damage or deterioration. A determination must be made as to whether the damage is the result of a one-time or a recurring event. The damage could also be the result of multiple causes, such as improper design, low quality materials, or poor construction technique.

- *Evaluate the extent of the damage.*—The extent and severity of the damage and the effects on the serviceability of the existing conduit must

Figure 193.—Determine the cause of the damage to determine the proper repair method.

be understood (figure 194). This evaluation will need to determine how quickly the damage is occurring and what the likely progression will be. The simplest and most common evaluation technique is by the use of sounding (hammer blows applied to the concrete surface). Experienced personnel using sounding and visual observation can assess indications of the extent of damage. In smaller conduits, CCTV inspection techniques may be required.

Sounding can provide an indication of delaminated or disbonded concrete by listening for drummy or hollow sounds. For deeper delaminations or for delaminations with minute separation, placing a hand close to the location of hammer blows or watching sand particles on the surface can provide information. If vibrations are felt or if the sand particles bounce on the surface this can be an indication of delamination.

Sounding can indicate concrete strength by the sounds created as the hammer hits the surface or by the rebound of the hammer. High strength concrete has a distinct ring as the hammer hits the surface, and the hammer rebounds sharply. Low strength concrete has a dull thud as the hammer hits the surface, and the hammer rebounds only slightly.

Concrete cores can be taken from damaged areas to assist with the detection of subsurface deterioration. Laboratory testing and pertrographic analysis are needed to confirm the causes of deterioration.

Non destructive testing methods can also be used to assist with the detection of damage. These include the Schmidt Rebound Hammer, ultrasonic pulse velocity, and acoustic pulse echo devices.

The extent of damage determined by the above methods should be mapped and the volume of repair concrete computed for preparation of the repair specifications. In existing conduits that are too small for man entry, CCTV should be utilized to evaluate the extent of damage.

- *Evaluate the need for repair.*—The need for immediate repair should be closely evaluated. If the safe operation of the conduit is affected, immediate repair may be necessary. However, most concrete damage progresses at a slow rate, and early detection and action may be able to slow the rate of deterioration. Early detection may also mean the difference between a repair and complete replacement. Not all deterioration will require repair if it is non safety related. Small hairline cracking on the concrete surface caused by drying and shrinkage usually does not require repair.

  An important consideration in determining the need for repair is the scheduling. Except in emergencies, many conduits cannot be removed from

Figure 194.—Evaluate the extent of the damage.

service for repair at certain times of the year without causing significant losses of water.

- *Select the repair*.—Once sufficient information has been obtained, the proper repair method can be determined. Fifteen standard repair materials can be used. If the standard repair materials cannot be utilized, consideration of nonstandard materials will be required. A detailed discussion of standard repair materials can be found in Reclamation's *Guide to Concrete Repair* (1997).

- *Prepare the old concrete for repair*.—Preparation of the old concrete is required to obtain a durable repair. Even the best repair materials depend upon proper preparation of the existing concrete. All unsound or deteriorated concrete must be removed before new repair materials are applied. The steps involved in preparation consist of:

    1. *Sawcut the perimeters*.—Sawcut the perimeter of the area of existing concrete to be repaired using a depth of 1 to 1.5 inches. Deeper sawcuts should be avoided to reduce the risk of cutting the reinforcement. The sawcuts can be tilted inward 2 to 3 degrees to act as a retaining keyway that lock in the repair to the existing surrounding concrete. The shape of the area to be sawcut should not have any sharp angles, since these are difficult to compact the repair material into. Rounded corners are preferable, but require the use of a bush hammer or jackhammer held vertically.

    2. *Concrete removal*.—Remove all deteriorated concrete to provide a sound surface for the repair materials to bond. The preferred methods of concrete removal are high pressure hydroblasting or hydrodemolition.

These methods leave a high quality concrete surface in place without leaving any microfractures in the remaining concrete. Impact concrete removal methods with jack hammers and bush hammers result in microfractures that will require subsequent removal by hydroblasting.

Shallow deterioration (less than ½ inch) can be removed with shot blasting or with dry/wet sandblasting. Some environmental precautions are required with dry blasting.

3. *Reinforcement preparation.*—All scale, rust, corrosion, and bonded concrete must be removed from the reinforcement. Methods for removal include wire brushing, high water pressure, or sand blasting. In areas where corrosion has reduced the diameter of the reinforcement to less than 75 percent of the original diameter, the reinforcement will need to be removed and replaced.

4. *Maintenance of prepared area.*—The prepared area must be maintained in a clean and protected manner until the repair materials are placed and cured. Seasonal precautions may be needed for temperature and moisture.

- *Apply the repair method.*—Each of the 15 standard repair materials has unique application requirements. The methods differ for repair of cracks and repair of damaged areas. Cracks need to be evaluated to determine if they are live (opening and closing) or dead (static). Different resins are applied or injected depending on the objective (structural bond, water leakage sealing). Figure 195 shows an example of a repair being made to a damaged area using epoxy bonded replacement concrete. A detailed discussion of application requirements for the different repair materials can be found in Reclamation's *Guide to Concrete Repair* (1997).

- *Cure the repair properly.*—Proper curing is required for all standard repair materials, with the exception of a few resinous systems. Inadequate or improper curing can reduce the service life of the concrete repair and result in significant costs for replacement of the repair. A detailed discussion of curing requirements for repair materials can be found in Reclamation's *Guide to Concrete Repair* (1997).

Limited repairs of concrete underwater are possible. However, these types of repairs are difficult and require special products and repair methods. Only highly qualified and experienced contractors should attempt this type of work (ACI, 1998b).

Figure 195.—A repair being made using epoxy-bonded replacement concrete. The use of epoxy resin ensures a strong, durable bond between the old concrete and the replacement concrete.

14.2.2 Repairs using grouts

Joint and defect repair in an existing conduit is an important part of the preparation process for many renovation methods. Leakage from joints or cracks in an existing conduit may severely hamper grouting of the annular space between a slipliner and the conduit, and every effort should be made to stop the leakage before grouting (Bendel and Basinger, 2002, p. 6). Two types of grout can be used:

- *Cement grouts*.—The traditional "neat cement" grout (cement mixed with water to make a pumpable slurry) is relatively inexpensive and can be used to fill large and small voids. Some additives are available to adjust the set time and also to reduce the tendency for "bleeding" (which occurs when the cement settles out, leaving water-filled voids.) However, cement grouts are not flexible, so if additional movement occurs, the grouted area may again develop a leak at a later date. Typically, cement grouts may be used to inexpensively fill large voids prior to injection of chemical grouts.

  The grout mix design may require adjustment in the field. Plasticizers may be used to facilitate cementitious grout injection. The amount of plasticizer may also require field adjustment.

- *Chemical grouts*.—These types of grout include sodium silicate, acrylates, polyurethanes, lignins, and resins. Of these, the polyurethanes are the most common for repair of leaking joints and cracks in conduits. Polyurethane grouts are generally viscous liquids which react with water to form a solid or semisolid material. The properties of common chemical grouts are described in the USACE's *Chemical Grouting* (1995a).

Chemical grouts are classified as hydrophobic or hydrophilic depending on how reactive they are when mixed with water. Hydrophilic grouts can incorporate a large amount of water into the cured products, and may shrink substantially if allowed to dry out completely. Therefore, hydrophilic grouts should be used in locations which are not kept wet at all times. Hydrophilic grouts cure with lower strengths than epoxy types of grout and are not considered to produce a structural bond in the area being grouted. Hydrophilic grouts form a closed-cell flexible foam barrier to stop the seepage of water through the joint. These grouts are usually injected when the ambient air temperatures in the existing conduit is above 32 °F. Figure 196 shows an example of resin injection equipment used for small repair projects.

For examples of grout cracks in an existing conduit see the case histories for Pablo and Ridgway Dams in appendix B.

14.3 Conduit abandonment

When the existing conduit deteriorates to a point where it can no longer serve its intended design purpose, a decision must be made to renovate, remove/replace, or to abandon it. In some cases, the designer may find it technically and economically more feasible to abandon the conduit by grouting it closed and leaving it in place. For instance, abandonment has some advantages over removing the conduit because a large trench is not required to be excavated transverse to the embankment dam. Backfilled excavations in an existing embankment are a source of differential settlement and potential hydraulic fracture. If abandonment is selected, a filter diaphragm or collar should be part of a design to intercept any flow that could potentially occur through defects in the grouted conduit or along the interface between the existing conduit and earthfill. For guidance on the design of filter diaphragms and collars, see sections 6.4 and 6.5, respectively. At embankment dams with small reservoirs, abandonment of a conduit may be done in conjunction with the installation of a siphon. See section 11.4.1 for guidance on the design and construction of siphons. For an example of a conduit abandonment, see the case history for St. Louis Recreation Lake Dam in appendix B.

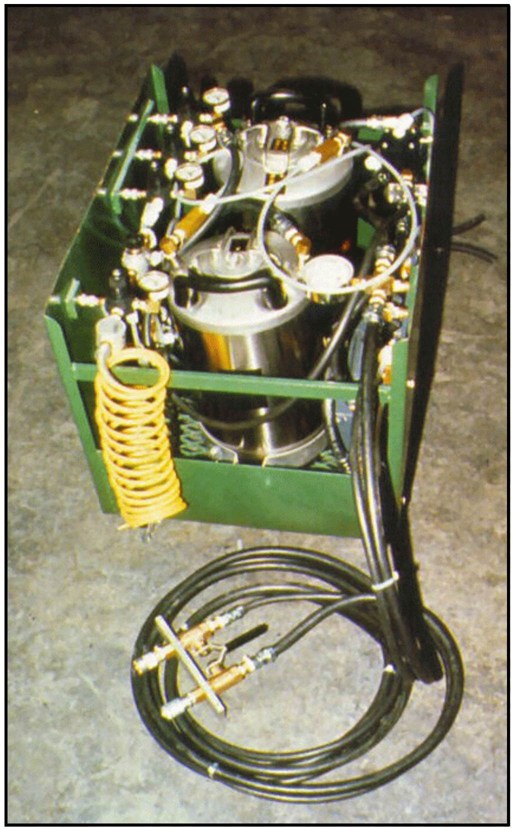

Figure 196.—An example of resin injection equipment used for small repair projects. Components are pumped independently and mixed at the nozzle for injection.

The advantages of abandoning an existing conduit through an embankment dam include:

- *Reservoir operation*.—Abandonment can in some cases be done while the reservoir is full. See section 14.3.1 for precautions involving drilling from the surface of the embankment dam.

- *Costs*.—Costs are generally less than other renovation and replacement methods.

The disadvantages of abandoning an existing conduit through an embankment dam include:

- *Grouting*.—Difficulties may exist trying to grout the existing conduit full.

- *Loss of use.*—A replacement means of providing downstream flow requirements reservoir evaluation comparability, and flood discharge capacity will be required.

The most common way to abandon an existing conduit is by the injection of grout or concrete. Two methods are usually considered for grout injection:

- *From upstream or downstream access.*—If conduit access is available from either an upstream or downstream location, this typically provides the simplest method for grout or concrete injection. Removal of a portion of the entrance or terminal structures may be required to attain sufficient access. Concrete (with a slump of about 6 or 7 inches) injection is typically more economical, since a larger diameter (about 5 inches) pump line can be used. This type of concrete is often called "backfill" concrete placed with a "slick" line. Injection of grout typically uses a pump line diameter of about 1 to 1½ inches. Also, when using concrete, a pump truck can be used. Grout injection normally requires a mixer, which will deliver grout at a slower capacity and will require more time to fill the existing conduit.

- *Through holes drilled from the surface of the embankment dam.*— When the upstream and downstream ends of the existing conduit are inaccessible, it may be possible to fill the conduit with grout or concrete through holes drilled from the surface of the embankment dam (figure 197). In order to be successful, the precise location of the existing conduit must be determined, and the driller must be experienced and proceed with caution. For an example of this type of conduit abandonment, see the Clair Peake Dam case history in appendix B.

While completely filling the existing conduit is recommended, the need for completely or partially filling the entire conduit will need to be evaluated based on safety concerns and costs. The indicated grouting and backfill procedures in this section may require modification to adapt to given site conditions. The designer is cautioned that grout injection from the surface, unless carefully controlled, has the potential for causing hydraulic fracture within the embankment dam. Drilling from the surface of the embankment dam is not advisable for situations where the reservoir water surface cannot be lowered.

Another possible reason for abandoning an existing conduit would be when a proposed embankment dam raise would result in much higher embankment loads over portions of the conduit. Structural analysis may determine that the higher embankment loads would fail the existing conduit and measures to strengthen it are not feasible. To prevent failure and to reduce the potential for internal erosion and backward piping erosion of embankment materials, the existing conduit would need to be abandoned and a new conduit constructed. The abandonment of the existing conduit may best be postponed until after the replacement conduit has been

Figure 197.—Abandonment of a conduit by injection of cement grout through holes drilled from the surface of the embankment dam to depths of up to 60 feet.

constructed and is fully operational. The existing conduit is used for diversion while the new conduit is constructed.

Any abandonment activities should also evaluate the need for partial or full demolition of the entrance and terminal structures, gate houses, plugging of gate chambers and shafts, and removal of certain mechanical equipment. Blasting for demolition should not be permitted. Shaft structures can be backfilled with compacted sand instead of concrete or grout.

14.3.1 Drilling into the existing embankment dam

Historically, drilling into an existing embankment dam has been performed for many reasons. Some of these included the perceived need to extract samples for laboratory sampling or to install instrumentation. However, as discussed in the following paragraphs, drilling into an embankment dam can cause serious damage. The need to drill into an embankment dam should therefore be carefully considered. Many properties of the soils comprising an embankment dam can be reasonably estimated from existing, published data, such as Reclamation's *Design of Small Dams* (1987a). In other cases, general conditions within the embankment dam, such as the phreatic water level, can be reasonably estimated without the need to install a piezometer. Installing an instrument within the embankment dam to develop data on a developing failure mode involving internal erosion or backward erosion piping is rarely successful. The installation of an instrument in exactly the proper place would be very fortuitous.

If drilling into an embankment dam has been determined to be necessary, drilling through any portion of an embankment dam should be performed with extreme caution. Improper drilling procedures increase the potential for hydraulic fracture. Drilling fluids, such as water or bentonite, are commonly used during drilling to enhance removal of drill cuttings, but these fluids should be avoided whenever possible when drilling in the fine grained embankment zones. Drilling fluids are typically pumped to the bottom of the hole through the drill stem, exiting through the drill bit. For many reasons, such as a small annulus space around the bit or clogging of the hole by cuttings, the fluids can be quickly pressurized by the pumping action. The drilling equipment can produce pressures in the hole that can rapidly exceed the surrounding earth pressure, causing direct hydraulic fracture of the embankment material. Hydraulic fracture has been known to occur even when extreme caution was being exercised and pressures were being constantly monitored.

Auguring is the preferred method for drilling in the core of embankment dams. Auguring uses no drilling fluid and is inherently benign with respect to hydraulic fracturing. A hollow-stem auger permits sampling in the embankment dam and allows sampling/testing of the foundation through the auger's hollow stem, which acts as casing. If fluids must be used, the risks must be understood and specific procedures should be employed to minimize the chance for hydraulic fracturing. Figure 198 shows an example of an auger being used.

With the exception of auguring, any drilling method has the potential to hydraulically fracture an embankment, if care is not taken and attention is not paid to detail. Various agencies have developed specific regulations regarding drilling with fluids in dams. For instance, Reclamation (1998b, pp. 222-223) indicates that the drilling methods that may be approved for drilling in embankment dams, if auguring is not practical (i.e., cobbly fill) are:

- Cable tool
- Direct rotary with mud (bentonite or biodegradable)
- Direct rotary with water
- Direct rotary with air-foam
- Down-hole hammer with reverse circulation

Figure 198.—An auger is being used to drill within an embankment dam.

Selection of any one of the above methods should be based on site-specific conditions, hole utilization, economic considerations, and availability of equipment and trained personnel.

Once drilling has commenced, drilling personnel are responsible for controlling and monitoring drill pressure, drill media circulation loss, and penetration rate to ensure that the drilling operation minimizes the possibility for hydraulic fracture. All aspects of the drilling operation should be closely monitored, including drill fluid pressures, drill fluid return volume, drill bit pressures, and occurrence of surface seeps (breakouts) of drill fluid. If a sudden loss of drill fluid occurs during embankment drilling within the core, drilling should be stopped immediately. Action should be taken to stop the loss of drill fluid. The personnel operating the drill and the designer should carefully evaluate the reason for the fluid loss.

Other agencies, such as the Corps of Engineers, may not allow drilling with any fluids. See USACE's *Procedures for Drilling in Earth Embankments* (1997) and *Geotechnical Investigations* (2001a).

14.3.2 Inspection

A thorough inspection of the existing conduit is required prior to beginning any abandonment activities. Depending on the diameter of the conduit, man-entry or CCTV inspection methods should be used; see section 9.5 for guidance on inspection of conduits. The condition of the existing conduit, existence of any protrusions or obstructions, joint offsets, amount of deflection, and evidence of leakage or internal erosion should be determined.

14.3.3 Preparations

The existing conduit surfaces against which grout will be placed should be free of roots, sediments, mineral deposits, dust, latence, loose or defective concrete, curing compound, coatings, and other foreign materials. See section 9.6 for guidance on cleaning conduits. Any sediments or debris should be removed from the invert of the existing conduit. Any bolts or other projections should be cut off flush and/or ground smooth with the interior surface of the existing conduit.

If water is entering the existing conduit and cannot be stopped, an inflatable bladder may be required. The pressure required to inflate the bladder and seal off inflow must be closely evaluated to avoid rupturing the conduit. This is usually a concern when high head is present in the reservoir. Also, some pipe materials, such as corroding CMP, are more susceptible to rupture. Abandonment of the existing conduit may need to be scheduled to allow grouting operations when the reservoir is at its lowest annual elevation. Siphons or pumps can be used to further reduce reservoir elevations. In some cases, the construction of a cofferdam may be more applicable, if the reservoir water level needs to remain at a constant elevation. If a new conduit is being constructed, grouting of the existing conduit can be delayed until the new conduit can be used for diversion.

For accessible existing conduits, any open or leaking joints or holes should be patched to minimize grout leakage. A bulkhead should be installed at the downstream end of the existing conduit to resist the loadings from the grout or concrete. An air return (vent) pipe or a series of pipes should be installed at the crown of the conduit and extend from the upstream end of the conduit to the bulkhead.

Grout pipes should be installed at the crown of the conduit. The longest pipes should be attached directly to the crown and the other pipes as closely to the crown as possible. Vent pipes can be used as grout pipes after grout return occurs. Grout pipes typically are Schedule 40 PVC, HDPE, or electrical mechanical tubing. Generally, grout pipes less than 100 feet in length can be ¾ inch diameter. Grout pipes longer than 100 feet should be 1½-inch diameter. The grout and vent pipes installed at the crown should be water tested.

Grouting equipment should be capable of continuously pumping grout at any pressure up to 50 lb/in^2. Injection pipes for concrete should be about 5 inches in diameter.

The abandoning of an inaccessible existing conduit is much more problematic due to the lack of access into the interior of the conduit. Stopping the flow of water into the existing conduit may be difficult, if there is an opening through the conduit. Abandonment may be possible by drilling into the conduit from the surface of the embankment dam at several locations and injecting a thick sand and grout mix (sometimes referred to as compaction grout, limited mobility grout, or LMG) to form a bulkhead (Cadden, Bruce, and Traylor, 2000, p. 8). This technique was successfully used to stop leakage in a deteriorating conduit through a 65-foot high embankment dam in southern Maryland (Traylor and Rehwoldt, 1999). In this case, the approximate location of the conduit was first established by use of several geophysical methods (magnetometer, resistivity, and self-potential). An experienced driller can detect when the drill bit enters the existing conduit, advance it to the middle of the conduit, and then pump the grout to form the bulkhead (Traylor and Rehwoldt, 1999, p. 6). Grout was tremied into the existing conduit through additional holes drilled from the surface of the embankment dam.

14.3.4 Grouting

The following paragraphs discuss the grout plan, mix design, and procedure:

- *Grouting plan.*—A grouting plan detailing the contractor's grout mix equipment, setup, procedures, sequencing, plan for handling waste, method for communication, and method for sealing and bulkheading upstream and downstream should be submitted for review and approval prior to initiation of grouting operations.

- *Grout and concrete mixes.*—Use a grout mix with a water (ASTM C 94) to cement (ASTM C 150) ratio of approximately 0.7:1 to 0.5:1. A grout fluidifier (ASTM C 937) may be needed to promote flowabilty, reduce water requirements, reduce bleeding, reduce segregation, increase strength, and eliminate grout shrinkage during setting of the grout mix. Trial mixes should be mixed at the job site prior to grouting to confirm the expected performance of the mix. For concrete injection, ¾-inch aggregate should be used. A 28-day compressive strength of 3,000 lb/in^2 is generally acceptable.

- *Procedure.*—The pressure at the crown of the conduit as measured at the vent pipe should not exceed 5 lb/in^2. Grouting is stopped when the air return pipe in the crown flows full with grout. Cap the grout and air return pipes. Remove the bulkhead upon completion of grouting operations. For grouting or backfilling of long existing conduits, the use of sections is recommended. Long grout or backfill placements could result in sufficient expansion and/or contraction to induce cracking of the existing conduit (concrete). The use of sections is also conducive to ensuring an acceptable seal. Figures 199 and 200 show grouting operations involved with the abandonment of an outlet works conduit.

Figure 199.—Grout is delivered to the site and is pumped into the conduit being abandoned.

Figure 200.—Grout being delivered to the pumping truck.

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Sound engineering judgment should always be applied when reviewing any of these references. While most of these references contain valuable information, a few may contain certain information that has become outdated in regards to design and construction aspects and/or philosophies. Users are cautioned to keep this mind when reviewing these references for design and construction purposes.

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Glossary

The terms defined in this glossary use industry-accepted definitions whenever possible. The source of the definition is indicated in parentheses.

Abandonment: Discontinuation of the use of a structure without intent to resume.

Abrasion (ASTM D 653, 2002): A rubbing and wearing away.

Abutment (FEMA, 2004): That part of the valley side against which an embankment dam is constructed. The left and right abutments of embankment dams are defined with the observer viewing the dam looking in the downstream direction, unless otherwise indicated.

Accident (ICOLD, 1974): A significant problem with an embankment dam that has been prevented from becoming a failure by remedial measures.

Acidity: A measure of how acid water or soil may be. Water or soil with a pH of less than 7.0 is considered acidic.

Acre-foot (FEMA, 2004): A unit of volumetric measure that would cover 1 acre to a depth of 1 foot. An acre-foot is equal to 43,560 ft^3.

Admixture (ASTM C 822, 2002): A material other than water, aggregates, cement, and reinforcement used as an ingredient of concrete and added to the batch immediately before or during its mixture.

Aggregate (ACI, 2000): Granular material, such as sand, gravel, crushed stone, and iron blast furnace slag, used with a cementing medium to form a hydraulic cement, concrete, or mortar.

Aging: The process of changing properties over time.

Air vent: A system used to permit air to enter the conduit to prevent collapse or to prevent the formation of low pressures within flowing water that could lead to cavitation and its possible attendant damage.

Alkali-aggregate reaction (ACI, 2000): Chemical reaction in concrete or mortar between alkalies (sodium and potassium) from Portland cement or other sources and certain constituents of some aggregates; under certain conditions, deleterious expansion of the concrete or mortar may result.

Alternate design method: See **Working stress design method**.

Anaerobic: An environment or a condition which is free of oxygen or a organism which can grow in the absence of oxygen.

Anchor: To fasten to prevent movement.

Annulus: The space between an existing conduit and a newly installed slipliner.

Antiseep collar: An impermeable diaphragm, usually of sheet metal or concrete, constructed at intervals within the zone of saturation along the conduit that passes through an embankment dam. In theory, antiseep collars were designed to increase the seepage length along the conduit and thereby prevent backward erosion piping by lowering the hydraulic gradient along the conduit.

Approach channel: The channel upstream from an entrance structure. This channel is generally unlined, excavated in rock or soil, and with or without riprap, soil cement or other types of erosion protection.

Appurtenant structure (FEMA, 2004): An ancillary feature of an embankment dam, such as an outlet, spillway, powerplant, or tunnel.

Auguring: A drilling technique that advances a hole into a soil material. The drill bit used can be one of a wide variety of helical style bits.

Autogenous growth (ASCE, 2000): Self-generating growth produced without external influence.

Auxiliary spillway: See **Spillway, auxiliary**.

Backfill: Soil or concrete used to fill excavations.

Backward erosion piping (piping): The term "piping" has often been used generically in literature to describe various erosional processes, not all of which hold to the classic definition of the term piping. Piping in the classic sense is characterized by the formation of an open tunnel that starts at a downstream seepage exit point and progresses back upstream toward the reservoir. This classic type of piping is often termed "backward erosion piping," and this term is used in this document. Blowout (also known as heave or blowup) is another term used to describe the condition where hydraulic head loosens a uniform body of cohesionless sand to the point where the permeability of the sand increases and flow concentrates in that zone that is blown out. Failures by blowout may not be exactly the same as "backward erosion piping," but for the purposes of this document, are grouped under this blanket term. Backward erosion piping involves the following essential conditions:

- Backward erosion piping is associated with intergranular seepage through saturated soil zones, not along concentrated flow paths (such as cracks).

- Backward erosion piping begins at a seepage discharge face where soil particles can escape because of the lack of a filter or an improperly designed filter at the exit face. As particles are removed, erosion progresses backward toward the source of seepage.

- The material being piped must be able to support a "pipe" or "roof," or must be adjacent to a feature such as an overlying clay layer or concrete structure that would provide a roof.

- For backward erosion piping to progress to the point where a failure occurs, soils susceptible to backward erosion piping must occur along the entire flow path.

- Backward erosion piping requires a hydraulic gradient high enough to initiate particle movement in soils that are susceptible to this phenomenon. Piping can begin with relatively low gradients for horizontal flow. For flow exiting a deposit vertically, if gradients are very high, the soil may be loosened, creating a condition sometimes termed heave.

- The term blowout is used to describe backward erosion piping that results when a sand horizon is overlain by a clay horizon with a defect in it, and an excessive hydraulic gradient causes backward erosion piping through that defect in the blanket. Defects in the blanket may consist of crayfish holes, fence post holes, animal burrows, and drying cracks. The transported sand forms a conical deposit on top of the surface clay horizon that itself is resistant to backward erosion piping.

In this document, the term "backward erosion piping" is used to describe the condition where piping occurs as defined above. The term "internal erosion" is used to describe all other erosional processes where water moves internally through the soil zones of embankment dams and foundations.

Bedding: Concrete used to provide transverse and lateral support under precast concrete conduits. Bedding generally comes up to about 25 percent of the conduit height.

Bedrock (FEMA, 2004): Any sedimentary, igneous, or metamorphic material represented as a unit in geology; being a sound and solid mass, layer, or ledge of mineral matter; and with shear wave threshold velocities greater than 2,500 ft/s.

Bentonite: A type of clay derived from weathered volcanic ash that expands when wet; commonly used as well drilling mud and annular seal.

Bond (ACI, 2000): Adhesion and grip of concrete or mortar to reinforcement or to other surfaces against which it is placed, including friction due to shrinkage and longitudinal shear in concrete engaged by the bar deformations.

Borehole: Any exploratory hole drilled into an embankment dam or its foundation to gather geophysical data.

Breach (FEMA, 2004): An opening through an embankment dam that allows the uncontrolled draining of a reservoir. A controlled breach is a constructed opening. An uncontrolled breach is an unintentional opening caused by discharge from the reservoir. A breach is generally associated with the partial or total failure of the embankment dam.

Buckling: Failure by lateral or torsional instability of a conduit, occurring with stresses below the yield strength.

Bulkhead gate: See **Gate, bulkhead**.

Bulking: The low density condition in a fine sand that occurs when negative capillary stresses develop when the sands are placed at intermediate water contents. Sands placed at a bulking water content have a much lower density than those placed very dry or saturated. Sands that may have been placed at a bulking water content may be densified by flooding and vibratory compaction.

Calibration hose (ASTM F 1743, 1996): An impermeable bladder, which is inverted within the resin-impregnated fabric tube by hydrostatic head or air pressure and may optionally be removed or remain in place as a permanent part of the installed cured-in-place pipe.

Caliper: An instrument used to measure the diameter of a conduit.

Caliper measurements: Measurement of the internal dimensions of a conduit, either by a physical device or by reflection of acoustic waves from a sled or cart. This method can be used to locate areas of conduit corrosion or excessive deformation.

Camera-crawler: A video camera attached to a self-propelled transport vehicle (crawler). Typically, the camera-crawler is used for closed circuit television inspection of inaccessible conduits.

Canal: A channel that conveys water by gravity to downstream users.

Cast-iron pipe: See **Pipe, cast-iron**.

Cathodic protection system (CPS): A system used to supplement protective coatings used for corrosion control.

Cavitation (ACI, 2000): Pitting of a material caused by implosion, i.e., the collapse of vapor bubbles in flowing water that form in areas of low pressure and collapse as they enter areas of higher pressure.

Cement (Portland) (ACI, 2000): A hydraulic cement produced by pulverizing clinker, consisting essentially of hydraulic calcium silicates, and usually containing one or more of the forms of calcium sulfate as an interground addition.

Chemical grout: Grout used for the repair of leaking joints and cracks within conduits or for the treatment of embankment materials surrounding a conduit.

Chimney drain: A drainage element located (typically) immediately downstream of a chimney filter. A chimney drain parallels the embankment dam's core and is either vertical or near vertical and placed from one abutment completely to the other abutment.

Chimney filter: See **Filter, chimney**.

Clay (ASTM D 653, 2002): Fine-grained soil or the fine-grained portion of soil that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when dry.

Closed circuit television (CCTV): A method of inspection utilizing a closed circuit television camera system and appropriate transport and lighting equipment to view the interior surface of conduits.

Closure section: The portion of a permanent embankment dam placed to fill a gap that has been left in the dam to pass diversion flows.

Coating (ACI, 2000): Material applied to a surface by brushing, dipping, mopping, spraying, troweling, etc. to preserve, protect, decorate, seal, or smooth the substrate.

Coefficient of thermal expansion (ACI, 2000): Change in linear dimension per unit length or change in volume per unit volume per degree of temperature change.

Cofferdam (FEMA, 2004): A temporary structure enclosing all or part of a construction area, so that construction can proceed in the dry. A diversion cofferdam diverts a stream into a conduit, channel, tunnel, or other watercourse.

Cohesion (ASTM D 653, 2002): The portion of the shear strength of a soil indicated by the term c, in Coulomb's equation, $s = c + p \tan \theta$.

Cohesionless soil (ASTM D 653, 2002): A soil that, when unconfined, has little or no strength when air dried and that has little or no cohesion when submerged.

Cohesive soil (ASTM D 653, 2002): A soil that, when unconfined, has considerable strength when air dried and that has significant cohesion when submerged.

Collapse: The movement or damage of a structural member that makes it unable to support loads.

Compaction (FEMA, 2004): Mechanical action that increases density by reducing the voids in a material.

> **Controlled**: A compaction process that includes requirements for maximum lift thickness and other criteria to ensure that the compacted soil has the intended properties.
>
> **Method**: A compaction process that only specifies the equipment and its operation in compacting the soil.

Compressible foundation: Foundation materials that will compress significantly when loaded.

Compressive strength (ASTM C 822, 2002): The maximum resistance of a concrete specimen to axial compressive loading; or the specified resistance used in design calculations.

Concrete (ACI, 2000): A composite material that consists of a binding medium (Portland cement and water, with or without admixtures) within which are embedded particles of fragments of fine and coarse aggregate.

> **Precast (ACI, 2000)**: Concrete that is cast somewhere other than its final location.
>
> **Reinforced cast-in-place (ACI, 2000)**: Structural concrete containing reinforcement that is placed and allowed to cure in the location where it is required to be when completed.

Condition assessment rating: A method for evaluating the condition of a conduit based on inspection.

Conduit (FEMA, 2004): A closed channel to convey water through, around, or under an embankment dam.

Consequences (FEMA, 2004): Potential loss of life or property damage downstream of a dam caused by floodwater released at the embankment dam or by water released by partial or complete failure of the dam.

Consolidation (ASCE, 2000): The process of densifying a material both naturally and mechanically.

Construction joint: See **Joint, construction**.

Contamination: The introduction of undesirable or unsuitable materials.

Contraction (ACI, 2000): Decrease in either length of volume.

Contraction joint: See **Joint, contraction**.

Control: The location within a conduit where regulation of flow occurs.

Control features: Typically gates or valves located in the entrance structure, conduit, gate chamber, or a terminal structure.

Control joint: See **Joint, control**.

Control testing: Laboratory tests performed on embankment material during construction to check if the specified material properties are being achieved.

Controlled breach (or wet breach): Excavation of a channel through an embankment dam to lower the reservoir to a safe level in the event of an emergency at the dam.

Controlled compaction: See **Compaction, controlled**.

Controlled low-strength material (CLSM): A self-compacting, cementitious material typically used as a replacement for compacted backfill around a conduit.

Core (FEMA, 2004): A zone of low permeability material in an embankment dam. The core is sometimes referred to as central core, inclined core, puddle clay core, rolled clay core, or impervious zone.

Corrosion (ACI, 2000): Disintegration or deterioration of a material by electrolysis or chemical attack.

Corrugated metal pipe (CMP): See **Pipe, corrugated metal**.

Crack: A narrow discontinuity.

Cradle: Reinforced, formed concrete that provides both longitudinal and lateral structural support to a circular conduit. A cradle extends for the full length of the conduit and encases the lower half of the conduit up to the springline.

Creep ratio: The ratio of the seepage path through an embankment dam divided by the head differential between the upstream and downstream toes of the dam. Weighted creep ratio includes proportioned vertical distances added to the horizontal seepage path length. The proportions are weighted based on the ratio of horizontal to vertical permeabilities in layered embankment and foundation soils. "Creep ratio" is no longer in common use as a design tool.

Cross section (FEMA, 2004): An elevation view of an embankment dam formed by passing a plane through the dam perpendicular to the axis.

Cured-in-place pipe (CIPP) (ASTM F 1743, 1996): A hollow cylinder consisting of a fabric tube with cured (cross-linked) thermosetting resin. Interior or exterior plastic coatings, or both, may be included. The CIPP is formed within an existing conduit and takes the shape of and fits tightly to the conduit.

Cutoff trench (FEMA, 2004): A foundation excavation later to be filled with impervious material to limit seepage beneath an embankment dam.

Dam (FEMA, 2004): An artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material, for the purpose of storage or control of water.

> **Earthfill (FEMA, 2004)**: An embankment dam in which more than 50 percent of the total volume is formed of compacted earth layers comprised of material generally smaller than 3 inches.
>
> **Embankment (FEMA, 2004)**: Any dam constructed of excavated natural materials, such as both earthfill and rockfill dams, or of industrial waste materials, such as a tailings dams.
>
> **Rockfill (FEMA, 2004)**: An embankment dam in which more than 50 percent of the total volume is comprised of compacted or dumped cobbles, boulders, rock fragments, or quarried rock generally larger than 3 inches.
>
> **Tailings (FEMA, 2004)**: An industrial waste dam in which the waste materials come from mining operations or mineral processing.

Dam failure (FEMA, 2004): A catastrophic type of failure characterized by the sudden, rapid, and uncontrolled release of impounded water or the likelihood of such an uncontrolled release. There are lesser degrees of failure, and any malfunction or abnormality outside the design assumptions and parameters that adversely affect an embankment dam's primary function of impounding water is properly considered a failure. These lesser degrees of failure can progressively lead to or heighten the risk of a catastrophic failure. They are, however, normally amenable to corrective action.

Dam safety (FEMA, 2004): Dam safety is the art and science of ensuring the integrity and viability of dams, such that they do not present unacceptable risks to the public, property, and the environment. Dam safety requires the collective application of engineering principles and experience, and a philosophy of risk management that recognizes that an embankment dam is a structure whose safe function is not explicitly determined by its original design and construction. Dam safety also includes all actions taken to identify or predict deficiencies and consequences related to failure, and to document and publicize any unacceptable risks, and reduce, eliminate, or remediate them to the extent reasonably possible.

Decant: To draw off the upper layer of liquid after the heaviest material (a solid or another liquid) has settled.

Defect: A discontinuity whose size, shape, orientation, location, or properties make it detrimental to the useful service of the structure in which it occurs.

Deflection: The decrease in the vertical diameter of a conduit due to loading.

Deformation (ACI, 2000): A change in dimension or shape due to stress.

Delamination: A separation of layers.

Desiccation: The process for evaporating water or removing water vapor from a material.

Design: An iterative decisionmaking process that produces plans by which resources are converted into products or systems that meet human needs or solve problems.

Designer: A registered engineer representing a firm, association, partnership, corporation, agency, or any combination of these who is responsible for the supervision or preparation of plans and specifications associated with an embankments dam and its appurtenances.

Destructive testing: Testing of a physical specimen or structure to determine its material properties. This may require the removal of a portion of a structure (such as a core sample of concrete removed from a conduit) for testing in a laboratory or for petrographic analysis.

Deterioration (ACI, 2000): Disintegration or chemical decomposition of a material during a test or service exposure.

Differential settlement (ASTM D 653, 2002): Settlement that varies in rate or amount, or both, from place to place across a structure.

Discharge channel: The channel downstream from a terminal structure. This channel conveys releases back to the "natural" stream or river. This channel may be excavated in rock or soil, with or without riprap, soil cement or other types of erosion protection.

Disking: Mechanical mixing (deep disking to blend materials) or scarifying (shallow disking to roughen the surface).

Dispersive clays: Dispersive clays differ from "normal" clays because of their electrochemical properties. Dispersive clays usually have a preponderance of sodium cations on the clay particles compared to a preponderance of calcium and magnesium on "normal" clays. The imbalance of electrical charges that result from this makeup causes dispersive clays to deflocculate in the presence of water. This deflocculation occurs because the interparticle forces of repulsion exceed the attractive forces. The clay particles go into suspension even in slowly moving or standing water. This means that dispersive clays are extremely erosive, and flow through cracks in dispersive clays can quickly erode the cracks and lead to rapid enlargement of the cracks. Failures caused by internal erosion in dispersive clay dams are common. Dispersive clays are also subject to severe rilling and jugging on exposed natural and constructed slopes because they are so erosive. Dispersive clays are not detectable with normal soil tests, such as mechanical analyses and Atterberg limit tests, and special tests, such as the crumb test, double hydrometer, and pinhole test, are required to detect the presence of dispersive clays.

Diver: A specially trained person who performs underwater inspection of structures or other underwater activities.

Dowel (ACI, 2000): A steel pin, commonly a plain or coated round steel bar that extends into adjoining portions of a concrete construction, as at an expansion or contraction joint; to transfer shear loads.

Downstream access: Entry through a terminal structure or exit portal of a conduit.

Downstream control: Regulation of flow within a conduit located near or at the terminal structure or exit portal.

Drain: A pipe that collects and directs water to a specified location.

Drawdown (FEMA, 2004): The difference between a water level and a lower water level in a reservoir within a particular time. Used as a verb, it is the lowering of the water surface.

Drilling: The process of penetrating earth and/or rock formations.

Dry density (ASTM D 653, 2002): The mass of solid particles per the total volume of soil or rock.

Dry spot (ASTM F 1743, 1996): An area of fabric of finished CIPP that is deficient or devoid of resin.

Dry unit weight (ASTM D 653, 2002): The weight of soil or rock solids per unit of total volume of soil or rock mass.

Durability (ACI, 2000): The ability of a material to resist weathering, chemical attack, abrasion, and other conditions of service.

Ductile iron pipe: See **Pipe, ductile iron**.

Earthfill dam: See **Dam, earthfill**.

Earthquake (FEMA, 2004): A sudden motion or trembling in the earth caused by the abrupt release of accumulated stress along a fault.

Electrical resistivity: A geophysical mapping method based on the principle that the distribution of an applied electrical potential (voltage) in the ground depends on the composition and density of surrounding soils and rocks.

Embankment dam: See **Dam, embankment**.

Emergency (FEMA, 2004): A condition that develops unexpectedly, which endangers the structural integrity of an embankment dam and/or downstream human life or property, and requires immediate action.

Emergency Action Plan (EAP) (FEMA, 2004): A plan of action to be taken to reduce the potential for property damage and loss of life in an area affected by an embankment dam failure or large flood.

Emergency classification: The act of classifying an emergency at an embankment dam, to determine the severity of the emergency condition and the proper response to prevent a dam failure, and to reduce loss of life and property damage, should the dam fail.

Emergency gate: See **Gate, emergency**.

Emergency spillway: See **Spillway, emergency**.

Engineer: A person trained and experienced in the profession of engineering; a person licensed to practice the profession by the appropriate authority.

Entrance structure: A structure located at the upstream end of a conduit. Entrance structures often include gates or valves, bulkheads, trashracks, and/or fish screens. Entrance structures are often referred to as intake structures for outlet works and inlet structures for spillways.

Epoxy: Any of various resins capable of forming tight, cross-linked polymer structures characterized by toughness, strong adhesion, and corrosion resistance. Commonly used as a two-part adhesive.

Erosion (FEMA, 2004): The wearing away of a surface (bank, streambed, embankment, or other surface) by floods, waves, wind, or any other natural process.

Evacuation: The act of removing water from a reservoir.

Event tree: A graphical representation of a series of events.

Excavation: Any manmade cut, trench, or depression in a surface, formed by earth and/or rock removal.

Expansion (ACI, 2000): Increase in either length of volume.

Expansion joint: See **Joint, expansion**.

Extensometer (ASCE, 2000): An instrument that measures the change in distance between two anchored points.

Fabric tube (ASTM F 1743, 1996): Flexible needled felt, or equivalent, woven or nonwoven material(s), or both, formed into a tubular shape, which, during installation, is saturated with resin and holds the resin in place during installation and curing process.

Failure (ICOLD, 1974): Collapse or movement of a part of an embankment dam or its foundation, so that the dam cannot retain water. In general, a failure results in the release of large quantities of water, imposing risks on the people or property downstream from the embankment dam.

Failure mode (FEMA, 2004): A physically plausible process for an embankment dam failure, resulting from an existing inadequacy or defect related to a natural foundation condition, the dam or appurtenant structure's design, the construction, the materials incorporated, the operation and maintenance, or aging process, which can lead to an uncontrolled release of the reservoir.

Filter cake: A thin layer of soil particles that accumulate at the face of a filter when water flowing through a crack carries eroding particles to the face. The filter cake forms when eroded particles embed themselves into the surface voids of the filter. The filter cake is effective in reducing further water flow to that which would occur through a layer of soil with the permeability of the eroded soil particles.

Filter collar: See **Filter, collar**.

Filter diaphragm: See **Filter, diaphragm**.

Filter: A zone of material designed and installed to provide drainage, yet prevent the movement of soil particles due to flowing water.

> **Chimney**: A chimney filter is a vertical or near vertical element in an embankment dam that is placed immediately downstream of the dam's core. In the case of a homogenous embankment dam, the chimney filter is typically placed in the central portion of the dam.
>
> **Collar**: A limited placement of filter material that completely surrounds a conduit for a specified length within the embankment dam. The filter collar is located near the conduit's downstream end. The filter collar is usually included in embankment dam rehabilitation only when a filter diaphragm cannot be constructed. A filter collar is different from a filter diaphragm, in that a filter diaphragm is usually located within the interior of the embankment dam.
>
> **Diaphragm**: A filter diaphragm is a zone of filter material constructed as a diaphragm surrounding a conduit through an embankment. The filter diaphragm protects the embankment near the conduit from internal erosion by intercepting potential cracks in the earthfill near and surrounding the conduit. A filter diaphragm is intermediate in size between a chimney filter and a filter collar. The filter diaphragm is placed on all sides of the conduit and extends a specified distance into the embankment.

First filling: Usually refers to the initial filling of a reservoir or conduit.

Flood (FEMA, 2004): A temporary rise in water surface elevation resulting in inundation of areas not normally covered by water. Hypothetical floods may be expressed in terms of average probability of exceedance per year, such as

1-percent-chance flood, or expressed as a fraction of the probable maximum flood or other reference flood.

Flood control: The regulation of flood inflows to reduce flood damage downstream.

Flotation: The act or state of floating.

Fly ash (ACI, 2000): The finely divided residue that results from the combustion of ground or powdered coal and that is transported by flue gases from the combustion zone to the particle removal system.

Footbridge: A structure that allows for pedestrian travel.

Forensics: The branch of science that employs scientific technology to assist in the determination of facts.

Foundation (FEMA, 2004): The portion of a valley floor that underlies and supports an embankment dam.

Frost heave (ASTM D 653, 2002): The raising of a structure due to the accumulation of ice in the underlying soil or rock.

Fully deteriorated conduit (ASTM D 5813, 2004): The existing conduit is not structurally sound and cannot support soil and live loads or is expected to reach this condition over the design life of the rehabilitated pipe.

Gate: A movable water barrier for the control of water.

> **Bulkhead (FEMA, 2004)**: A gate used either for temporary closure of a channel or conduit before unwatering it for inspection or maintenance, or for closure against flowing water when the head difference is small, as in a diversion tunnel.
>
> **Emergency (FEMA, 2004**): A standby or reserve gate used only when the normal means of water control is not available for use.
>
> **Guard (FEMA, 2004)**: A standby or auxiliary gate used when the normal means of water control is not available. Sometimes referred to as an emergency gate.
>
> **Regulating (regulating valve) (FEMA, 2004)**: A gate or valve that operates under full pressure flow conditions to regulate the rate of discharge.

Gate chamber: An outlet works structure containing gates or valves located between the upstream and downstream conduits.

Gauge (ASCE, 2000): A device that measures something with a graduated scale.

Geophysical techniques: Methods used to study the physical characteristics and properties of embankment dams. Geophysical techniques are based on the detection of contrasts in different physical properties of materials.

Geotextiles (FEMA, 2004): Any fabric or textile (natural or synthetic) when used as an engineering material in conjunction with soil, foundations, or rock. Geotextiles have the following uses: drainage, filtration, separation of materials, reinforcement, moisture barriers, and erosion protection.

Gradation (ASTM C 822, 2002): The distribution of particles of granular material among standard sizes, usually expressed in terms of cumulative percentages larger or smaller than each of a series of sieve openings.

Gravel (ASTM D 653, 2002): Rounded or semirounded particles of rock that will pass a 3-inch (76.2)-mm) and be retained on a No. 4 (4.75-µm) U.S. standard sieve.

Ground-penetrating radar: A geophysical method which uses high-frequency radio waves to locate voids at shallow depths, less than about 15 to 20 feet (the effective depth is very limited in clayey soils).

Grout (FEMA, 2004): A fluidized material that is injected into soil, rock, concrete, or other construction material to seal openings and to lower the permeability and/or provide additional structural strength. There are four major types of grouting materials: chemical, cement, clay, and bitumen.

Grout mix (ASTM D 653, 2002): The proportions or amounts of the various materials used in the grout, expressed by weight or volume (The words "by volume" or "by weight" should be used to specify the mix).

Grout pipe: The pipe used to transport grout to a certain location. The grout may be transported through this pipe by either gravity flow or pressure injection.

Guard gate: See **Gate, guard**.

Hazard (FEMA, 2004): A situation that creates the potential for adverse consequences, such as loss of life, or property damage.

Hazard potential classification: A system that categorizes embankment dams according to the degree of adverse incremental consequences of a failure or misoperation of a dam. The hazard potential classification does not reflect in any way on the current condition of the embankment dam (i.e., safety, structural integrity, flood routing capacity).

Heat fused joint: See **Joint, heat fused**.

Height (above ground): The maximum height from natural ground surface to the top of an embankment dam.

High density polyethylene (HDPE): A polymer prepared by the polymerization of ethylene as the sole monomer.

Holiday: A discontinuity in a coating, such as a pinhole, crack, gap, or other flaw, that allows an area of the base metal to be exposed to any corrosive environment that contacts the coating surface.

Homogeneous: Constructed of only one type of material.

Horizontal directional drilling (HDD): A trenchless construction method that uses guided drilling. The method involves three main stages: drilling of a pilot hole, pilot hole enlargement, and pullback installation of the carrier pipe.

Hydraulic fracture: A separation in a soil or rock mass that occurs if the applied water pressure exceeds the lateral effective stress on the soil element. Hydraulic fracture may occur if differential foundation movement is allowed. Soils compacted dry of optimum water content are more susceptible to hydraulic fracture.

Hydraulic gradient: The slope of the hydraulic grade line. The hydraulic gradient is the slope of the water surface in an open channel.

Hydrophilic: Having a strong affinity for water.

Hydrophobic: Having a strong aversion for water.

Hydrostatic head (ASTM D 653, 2002): The fluid pressure of water produced by the height of the water above a given point.

Hydrostatic pressure: The pressure exerted by water at rest.

Ice lens: A mass of ice formed during the construction of an embankment dam, when a moist soil is exposed to freezing temperatures. In certain types of soils (silts and silty soils) the size of the ice mass will increase as it draws unfrozen capillary water from the adjacent soil. A void in the soil may remain after the ice lens melts.

Impervious: Not permeable; not allowing liquid to pass through.

Incident (ICOLD, 1974): Either a failure or accident, requiring major repair.

Inclinometer (ASCE, 2000): An instrument for measuring the angle of deflection between a reference axis and casing axis.

Infiltration: The flow of water through a soil surface or the flow of water into a conduit through a joint or defect.

Inspection: The review and assessment of the operation, maintenance, and condition of a structure.

Inspector: The designated on-site representative responsible for inspection and acceptance, approval, or rejection of work performed as set forth in the contract specifications. The authorized person charged with the task of performing a physical examination and preparing documentation for inspection of the embankment dam and appurtenant structures.

Instrumentation (FEMA, 2004): An arrangement of devices installed into or near embankment dams that provide for measurements that can be used to evaluate the structural behavior and performance parameters of the structure.

Intake structure (FEMA, 2004): Placed at the beginning of an outlet works waterway (power conduit, water supply conduit), the intake establishes the ultimate drawdown level of the reservoir by the position and size of its opening(s) to the outlet works. The intake may be vertical or inclined towers, drop inlets, or submerged, box-shaped structures. Intake elevations are determined by the head needed for discharge capacity, storage reservation to allow for siltation, the required amount and rate of withdrawal, and the desired extreme drawdown level.

Internal erosion: A general term used to describe all of the various erosional processes where water moves internally through or adjacent to the soil zones of embankment dams and foundation, except for the specific process referred to as "backward erosion piping." The term "internal erosion" is used in this document in place of a variety of terms that have been used to describe various erosional processes, such as scour, suffosion, concentrated leak piping, and others.

Inundation map (FEMA, 2004): A map showing areas that would be affected by flooding from releases from a dam's reservoir. The flooding may be from either controlled or uncontrolled releases or as a result of a dam failure. A series of maps for a dam could show the incremental areas flooded by larger flood releases.

Inversion (ASTM F 1743, 1996): The process of turning the calibration hose inside out by the use of water pressure or air pressure.

Invert: The bottom or lowest point of the internal surface of the transverse cross section of a conduit.

Job Hazard Analysis (JHA): A procedure which helps integrate accepted safety and health principles and practices into a particular operation.

Joint (ASTM F 412, 2001): The location at which two sections of conduit or pipe are connected together.

> **Construction (ACI, 2000)**: The surface where two successive placements of concrete meet, across which it is desirable to develop and maintain bond between the two concrete placements, and through which any reinforcement that may be present is not interrupted.
>
> **Contraction (ACI, 2000)**: Formed, sawed, or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure. The concrete surface is unbonded. No reinforcement extends across the joint. Smooth dowels may be provided to maintain proper alignment of monolithic units.
>
> **Control**: Joints placed in concrete to provide for control of initial shrinkage stresses and cracks of monolithic units. The concrete surface is unbonded. Control joints are constructed as described for contraction joints, except that reinforcement is always continuous across the joint. The reinforcement prevents the longitudinal forces from opening the joints.
>
> **Expansion (ACI, 2000)**: A separation provided between adjoining parts of a structure to allow movement where expansion is likely to exceed contraction or an isolation joint intended to allow independent movement between adjoining parts.
>
> **Heat fused (fusion) (ASTM F 412, 2001)**: A joint using heat and pressure only.
>
> **Mechanical (ASTM F 412, 2001)**: A connection between piping components employing physical force to develop a seal or produce alignment.

Joint meter (ASCE, 2000): A device used to measure the movement of a joint in concrete or any other material.

Leakage (FEMA, 2004): Uncontrolled loss of water by flow through a hole or crack.

Linear coefficient of thermal expansion: The change in length with temperature for a solid material relative to its original length.

Lining: A material applied to the inside surface of a conduit to provide a protective covering.

Lubricant (ASTM F 412, 2001): A material used to reduce friction between two mating surfaces that are being joined by sliding contact.

Maintenance: All routine and extraordinary work necessary to keep the facility in good repair and reliable working order to fulfill the intended designed project purposes. This includes maintaining structures and equipment in the intended operating condition, and performing necessary equipment and minor structure repairs.

Man-entry: A conduit size large enough for personnel to access and perform the required actions.

Mastic: A permanently flexible waterproofing material used for sealing water-vulnerable joints.

Maximum water surface: The highest acceptable water surface elevation considering all factors affecting the dam.

Mechanical caliper: An instrument used for measuring the distance between two points.

Mechanical joint: See **Joint, mechanical**.

Method compaction: See **Compaction, method**.

Microtunneling: A trenchless construction method that uses a tunnel boring machine normally controlled from the surface. The method simultaneously installs pipe as the spoil is excavated and removed.

Modulus of soil reaction (E'): An empirical value used to express the stiffness of the embedment soil in predicting flexible pipe deflection.

Moisture content: The water content in a soil.

Monitoring: The process of measuring, observing, or keeping track of something for a specific period of time or at specified intervals.

Mortar (ACI, 2000): A mixture of cement paste and fine aggregate. In fresh concrete, this is the material occupying the interstices among particles of coarse aggregate.

Mud slab (ACI, 2000): A 2- to 6-inch layer of concrete below a structural concrete floor or footing over soft, wet soil; also called mud mat. Mud slabs are used to protect foundations during construction.

Multichannel Analysis of Surface Waves (MASW): An extension of Spectral Analysis of Surface Waves, MASW uses multiple geophones (usually 24 or more) to simultaneously acquire surface wave data on many points from a single seismic source.

Nondestructive testing (NDT): Geophysical methods for assessing the condition of a conduit, embankment dam, or other structure, which do not require that a physical sample be removed from the structure. These methods include seismic tomography, electromagnetic tomography, ground penetrating radar, and ultrasonic pulse-echo. When combined with modern computer processing software, the data obtained from the testing can be used to create detailed images of the structure.

Nonpressurized flow: Open channel discharge at atmospheric pressure for part or all of the conduit length. This type of flow is also referred to as "free flow."

Normal water surface (FEMA, 2004): For a reservoir with a fixed overflow sill, this is the lowest crest level of that sill. For a reservoir whose outflow is controlled wholly or partly by moveable gates, siphons, or other means, it is the maximum level to which water may rise under normal operating conditions, exclusive of any provision for flood surcharge.

Nuclear gauge: An instrument used to measure the density and water content of both natural and compacted soil, rock, and concrete masses. The gauge obtains density and water contents from measurements of gamma rays and neutrons that are emitted from the meter. Gamma rays are emitted from a probe inserted into the mass being measured. Measurement of the gamma rays transmitted through the mass, when calibrated properly, reflects the density of the mass. Neutrons are emitted from the base of the gauge. Measuring the return of reflected neutrons when the gauge is calibrated properly can be related to the water content of the mass.

Offset (ACI, 2000): An abrupt change in alignment or dimension, either horizontally or vertically.

Open cut: An excavation through rock or soil made through topographic features to facilitate the passage of a conduit.

Optimum moisture content (optimum water content) (ASTM D 653, 2002): The water content at which a soil can be compacted to a maximum dry unit weight by a given compactive effort.

Outlet works (FEMA, 2004): An embankment dam appurtenance that provides release of water (generally controlled) from a reservoir.

Overburden (ASTM D 653, 2002): The loose soil, sand, silt, or clay that overlies bedrock. All materials overlying a conduit.

Oxygen content: The amount of dissolved oxygen.

Partially deteriorated conduit (ASTM D 5813, 2004): An existing conduit that can support the soil and live loads throughout the design life of the rehabilitated conduit. The soil adjacent to the existing pipe must provide adequate side support. The pipe may have longitudinal cracks and some distortion of the diameter.

Penetrometer: A device used to determine the resistance to penetration (bearing capacity) of a soil.

Penstock (FEMA, 2004): A pressurized pipeline or shaft between the reservoir and hydraulic machinery.

Permeability: A measure of the rate at which water can percolate through soil.

Pervious: Permeable, having openings that allow water to pass through.

Pervious zone (FEMA, 2004): A part of the cross section of an embankment dam comprising material of high permeability.

Phreatic line (ASCE, 2000): Water surface boundary. Below this line, soils are assumed to be saturated. Above this line, soils contain both gas and water within the pore spaces.

Phreatic surface (ASCE, 2000): The planar surface between the zone of saturation and the zone of aeration. Also known as free-water surface, free-water elevation, groundwater surface, and groundwater table.

Piezometer (ASCE, 2000): An instrument for measuring fluid pressure (air or water) within soil, rock, or concrete.

Pig: A cylindrical device inserted into a conduit to perform cleaning or internal inspection.

Pipe jacking (ASCE, 2001): A system of directly installing pipes behind a shield machine by hydraulic jacking from a drive shaft, such that the pipes form a continuous string in the ground.

Pipe: A hollow cylinder of concrete, plastic, or metal used for the conveyance of water.

> **Cast iron**: A type of iron-based metallic alloy pipe made by casting in a mold.
>
> **Corrugated metal**: A galvanized light gauge metal pipe that is ribbed to improve its strength.
>
> **Ductile iron**: A type of iron-based metallic alloy pipe that is wrought into shape.
>
> **Plastic (ASTM F 412, 2001)**: A hollow cylinder of plastic material in which the wall thicknesses are usually small when compared to the diameter and in which the inside and outside walls are essentially concentric.
>
> **Precast concrete**: Concrete pipe that is manufactured at a plant.
>
> **Steel**: A type of iron-based metallic alloy pipe having less carbon content than cast iron, but more than ductile iron.

Piping: See **Backward erosion piping**.

Pitting: A form of localized corrosive attack characterized by holes in metal. Depending on the environment and the material, a pit may take months, or even years, to become visible.

Plastic pipe (ASTM F 412, 2001): A hollow cylinder of plastic material in which the wall thicknesses are usually small when compared to the diameter and in which the inside and outside walls are essentially concentric.

Polyester (ASTM D 883, 2000): A polymer in which the repeated structural unit in the chain is of the ester type.

Polyethylene: A polymer prepared by the polymerization of ethylene as the sole monomer.

Polyvinyl chloride (PVC): A polymer prepared by the polymerization of vinyl acetate as the sole monomer.

Pore pressure (ASCE, 2000): The interstitial pressure of a fluid (air or water) within a mass of soil, rock, or concrete.

Power conduit: A conduit used to convey water under pressure to the turbines of a hydroelectric plant.

Precast concrete pipe: See **Pipe, precast concrete**.

Pressure flow: Pressurized flow throughout the conduit length to the point of regulation or control or terminal structure.

Principal spillway: The primary outlet device through an embankment dam for flood regulation. Typically, consists of riser structure in combination with an outlet conduit.

Pumping: The release or draining of a reservoir by means of a machine or device that creates pressure and water flow.

Quality assurance: A planned system of activities that provides the owner and permitting agency assurance that the facility was constructed as specified in the design. Construction quality assurance includes inspections, verifications, audits, and evaluations of materials and workmanship necessary to determine and document the quality of the constructed facility. Quality assurance refers to measures taken by the construction quality assurance organization to assess if the installer or contractor is in compliance with the plans and specifications for a project. An example of quality assurance activity is verifications of quality control tests performed by the contractor using independent equipment and methods.

Quality control: A planned system of inspections that is used to directly monitor and control the quality of a construction project. Construction quality control is normally performed by the contractor and is necessary to achieve quality in the constructed system. Construction quality control refers to measures taken by the contractor to determine compliance with the requirements for materials and workmanship as stated in the plans and specifications for the project. An example of quality control activity is the testing performed on compacted earthfill to measure the dry density and water content. By comparing measured values to the specifications for these values based on the design, the quality of the earthfill is controlled.

Radiography: A nondestructive testing method that provides an internal examination of a metallic structure or component by exposing it to a beam of X-ray or gamma radiation. Internal defects can be seen on a screen or recorded on film.

Regulating gate: See **Gate, regulating**.

Reinforced cast-in-place concrete: See **Concrete, reinforced cast-in-place**.

Reinforcement (ASTM C 822, 2002): Steel in the form of continuous wire, welded wire fabric, or bars embedded in concrete in such a manner that the concrete and steel act together to resist stresses.

Relative density: A numerical expression that defines the relative denseness of a cohesionless soil. The expression is based on comparing the density of a soil mass at a given condition to extreme values of density determined by standard tests that describe the minimum and maximum index densities of the soil. Relative density is the ratio, expressed as a percentage, of the difference between the maximum index void ratio and any given void ratio of a cohesionless, free-draining soil; to the difference between its maximum and minimum index void ratios.

Remotely operated vehicle (ROV): An unoccupied, highly maneuverable underwater robot controlled by a remote operator usually located in a ship or on the shore. Most vehicles are equipped with a video camera and lights. Additional equipment can be added to expand the vehicle's capabilities.

Renovation: The repair or restoration of an existing structure, so it can serve its intended purpose.

Repair: The reconstruction or restoration of any part of an existing structure for the purpose of its maintenance.

Replacement: The removal of existing materials that can no longer perform their intended function and installation of a suitable substitute.

Reservoir (FEMA, 2004): A body of water impounded by an embankment dam and in which water can be stored.

Reservoir evacuation: The release or draining of a reservoir through an outlet works, spillway, or other feature at an embankment dam.

Resin (ASTM F 412, 2001): A solid or pseudosolid organic material, often with high molecular weight, which exhibits a tendency to flow when subjected to stress, usually has a softening or melting range, and usually fractures conchoidally (shell-like fracture).

Resistivity: A measure of the resistance to current flow in a material.

Resolution (ASCE, 2000): The smallest increment in measurement that can be distinguished.

Riprap (FEMA, 2004): A layer of large, uncoursed stone, precast blocks, bags of cement, or other suitable material, generally placed on the slope of an embankment or along a watercourse as protection against wave action, erosion, or scour. Riprap is usually placed by dumping or other mechanical methods and in some cases, is hand placed. It consists of pieces of relatively large size, as distinguished from a gravel blanket.

Riser pipe: A vertical pipe section at the upstream end of a spillway that allows water to drop into the conduit and be discharged downstream.

Risk (FEMA, 2004): A measure of the likelihood and severity of adverse consequences (National Research Council, 1983). Risk is estimated by the mathematical expectation of the consequences of an adverse event occurring, that is, the product of the probability of occurrence and the consequence, or alternatively, by the triplet of scenario, probability of occurrence, and the consequence.

Risk reduction analysis: An analysis that examines alternatives for their impact on the baseline risk. This type of analysis is begun once the baseline risk indicates risks are considered too high and that some steps are necessary to reduce risk.

Rockfill dam: See **Dam, rockfill**.

Rutting: The tire or equipment impressions in the surface of a compacted fill that result from repeated passes of the equipment over the compacted fill when the soil is at a moisture and density condition that allows the rutting to occur. Rutting usually occurs when soils are not well compacted and/or are at a water content too high for effective compaction.

Sand (ASTM D 653, 2002): Particles of rock that will pass the No. 4 (4.75–μm) sieve and be retained on the No. 200 (0.075-mm) U.S. standard sieve.

Sand boil: Sand or silt grains deposited by seepage discharging at the ground surface without a filter to block the soil movement. The sand boil may have the shape of a volcano cone with flat to steeper slopes, depending on the size and gradation of particles being piped. Sand boils are evidence of piping occurring in the foundation of embankments or levees from excessive hydraulic gradient at the point of discharge.

Scaling: The deposition and adherence of insoluble products on the surface of a material.

Scarification: The process of roughening the surface of a previously compacted lift of soil before placement of the next lift. Scarification is accomplished with discs, harrows, and similar equipment. The purpose of scarification is to promote bonding of lifts and reduce interlift permeability. Scarification is usually required in construction specifications written by designers concerned over stratification of earthfills.

Scour: The loss of material occurring at an erosional surface, where a concentrated flow is located, such as a crack through a dam or the dam/foundation contact. Continued flow causes the erosion to progress, creating a larger and larger eroded area.

Seepage (ASTM D 653, 2002): The infiltration or percolation of water through rock or soil or from the surface.

Seepage paths (ASCE, 2000): The general path along which seepage follows.

Segregation: The tendency of particles of the same size in a given mass of aggregate to gather together whenever the material is being loaded, transported, or otherwise disturbed. Segregation of filters can cause pockets of coarse and fine zones that may not be filter compatible with the material being protected.

Seismic activity: The result of the earth's tectonic movement.

Seismic tomography: A geophysical method that measures refraction and reflection of small, manmade seismic waves and high-level imaging software to create cross-sectional views of the internal portions of a structure, similar to computerized axial tomography (CAT) scans used in medicine.

Self potential (or streaming potential): A geophysical method that maps fields of electrical potential (voltage) generated by water flowing through a porous material to locate seepage areas in a dam or foundation.

Self-healing: The property of a sand filter that reflects its ability to deform and fill a crack that is transmitted to the filter.

Service life: Expected useful life of a project, structure, or material.

Service spillway: See **Spillway, service**

Settlement (FEMA, 2004): The vertical downward movement of a structure or its foundation.

Shear strength (ASCE, 2000): The ability of a material to resist forces tending to cause movement along an interior planer surface.

Shear stress: Stress acting parallel to the surface of the plane being considered.

Shore (ACI, 2000): A temporary support for formwork and fresh concrete.

Silt (ASTM D 653, 2002): Material passing the No. 200 (75-µm) U.S. standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air-dried.

Sinkhole: A depression, indicating subsurface settlement or particle movement, typically having clearly defined boundaries with a sharp offset.

Siphon: An inverted u-shaped pipe or conduit, filled until atmospheric pressure is sufficient to force water from a reservoir over an embankment dam and out of the other end.

Slaking: Degradation of excavated foundation caused by exposure to air and moisture.

Sliplining: The process of inserting a new, smaller-diameter lining or pipe into an existing larger-diameter conduit.

Slope (FEMA, 2004): Inclination from the horizontal. Sometimes referred to as batter when measured from vertical.

Slurry: A mixture of solids and liquids.

Soil (ASTM D 653, 2002): Sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter.

Soil resistivity: The measure of the resistance to current flow in a soil.

Soluble salt: A salt that can be dissolved in water.

Sonar: A geophysical method which measures the reflection of acoustic waves to map underwater structures. Often refers to side-scan radar.

Sonic caliper: An instrument that utilizes pulses to measure the distance between two points.

Spacer: A specially fabricated material used during the sliplining of conduits to keep a smaller diameter pipe centered within the larger diameter pipe.

Spall (ACI, 2000): A fragment, usually in the shape of a flake, detached from a larger mass by a blow, by action of weather, pressure, or expansion within the larger mass.

Specifications: The written requirements for materials, equipment, construction systems, and standards.

Spectral Analysis of Surface Waves (SASW): A nondestructive, geophysical procedure for characterizing in-situ materials based on the principle that different materials have varying surface (Rayleigh) wave velocities. Surface wave data from geophones and small seismic sources are processed with specialized computer software to evaluate material properties, such as density, stratification, and location of voids.

Spigot: The plain end of a bell and spigot pipe. The spigot is inserted into the bell end of the next pipe.

Spillway (FEMA, 2004): A structure, over or through which water is discharged from a reservoir. If the rate of flow is controlled by mechanical means, such as gates, it is considered a controlled spillway. If the geometry of the spillway is the only control, it is considered an uncontrolled spillway.

> **Auxiliary (FEMA, 2004)**: Any secondary spillway that is designed to be operated infrequently, possibly in anticipation of some degree of structural damage or erosion to the spillway that would occur during operation.
>
> **Emergency (FEMA, 2004)**: See **Spillway, auxiliary**.
>
> **Service (FEMA, 2004)**: A spillway that is designed to provide continuous or frequent regulated or unregulated releases from a reservoir, without significant damage to either the dam or its appurtenant structures. This is also referred to as principal spillway.

Spray lining: The application of cement mortar or epoxy resin against the inside walls of an existing conduit, using a revolving spray head moved through the conduit.

Stability (ASCE, 2000): The resistance to sliding, overturning, or collapsing.

Standard Proctor compaction test: A standard laboratory or field test procedure performed on soil to measure the maximum dry density and optimum water content of the soil. The test uses standard energy and methods specified in ASTM Standard Test Method D 698.

Standardized dimension ratio (SDR): Ratio of the average specified outside diameter to the minimum specified wall thickness for outside diameter controlled plastic pipe.

Standards (ASCE, 2000): Commonly used and accepted as an authority.

Steel pipe: See **Pipe, steel**.

Stoping: The sequence of soil removal at the bottom of hole followed by roof collapse. This bottom-up erosion process forms a cavern in the embankment material, typically with steep sides.

Stoplogs (FEMA, 2004): Large logs, timbers, or steel beams placed on top of each other with their ends held in guides on each side of a channel or conduit so as to provide a cheaper or more easily handled means of temporary closure than a bulkhead gate.

Storage (FEMA, 2004): The retention of water or delay of runoff either by planned operation, as in a reservoir, or by temporary filling of overflow areas, as in the progression of a flood wave through a natural stream channel.

Strain gauge (ASCE, 2000): A device that measures the change in distance between closely spaced points.

Strength design method (ACI, 2000): A design method that requires service loads to be increased by specified load factors and computed theoretical strengths to be reduced by the specified phi factors. Also, known as ultimate strength design method.

Subsidence: A depression, indicating subsurface settlement or particle movement, typically not having clearly defined boundaries.

Suffosion: Seepage flow through a material that causes part of the finer grained portions of the soil matrix to be carried through the coarser grained portion of the matrix. This type of internal erosion is specifically relegated only to gap graded soils (internally unstable soils) or to soils with an overall smooth gradation curve, but with an overabundance of the finer portions of the curve represented by a "flat tail" to the gradation curve. While a crack is not needed to initiate this type of internal erosion, a concentration of flow in a portion of the soil is needed.

Sulfate attack (ACI, 2000): Either a chemical reaction, physical reaction, or both between sulfates usually in soil or ground water and concrete or mortar; the chemical reaction is primarily with calcium aluminate hydrates in the cement-paste matrix, often causing deterioration.

Surface air voids (ACI, 2000): Small regular or irregular cavities, usually not exceeding about 0.5 inch in diameter, resulting from entrapment of air bubbles in the surface of formed concrete during placement and consolidation. Commonly referred to as bugholes.

Surface hardness: The surface hardness of concrete can be measured to provide a relative indication of the strength of in-situ concrete. Surface hardness can be measured by rebound hammer (also called Schmidt Hammer or Swiss Hammer, ASTM C 805) or by the penetration resistance test (also called Windsor Probe, ASTM C 803). Surface hardness is affected by the condition of the surface, composition of concrete, type of coarse aggregate, and degree of carbonation of the concrete surface. To improve accuracy of the inferred strength, the test methods must be calibrated with laboratory strength tests performed on samples of the concrete.

Tailings: The fine-grained waste materials from an ore-processing operation.

Tailings dam: See **Dam, tailings**.

Tailwater (ASCE, 2000): The elevation of the free water surface (if any) on the downstream side of an embankment dam.

Terminal structure: A structure located at the downstream end of the conduit. Terminal structures often include gates or valves and may include some type of structure to dissipate the energy of rapidly flowing water and to protect the riverbed from erosion

Tether: A cable that attaches two things together.

Thermocouple (ACI, 2000): Two conductors of different metals joined together at both ends, producing a loop in which an electric current will flow when there is a difference in temperature between two junctions.

Thermoplastic (ASTM F 412, 2001): A plastic that can be repeatedly softened by heating and hardened by cooling through a temperature range characteristic of the plastic, and that in the softened state can be shaped by flow into articles by molding or extrusion.

Thermoset (ASTM F 412, 2001): A plastic that, when cured by application of heat or chemical means, changes into a substantially infusible and insoluble product.

Toe of the embankment dam (FEMA, 2004): The junction of the downstream slope or face of a dam with the ground surface; also referred to as the downstream toe. The junction of the upstream slope with ground surface is called the heel or the upstream toe.

Transducer (ASCE, 2000): Any device or element that converts an input signal into an output signal of a different form.

Transverse crack: A crack that extends in an upstream and downstream direction within an embankment dam.

Trashrack (FEMA, 2004): A device located at an intake structure to prevent floating or submerged debris from entering the entrance of the structure.

Tremie concrete (ACI, 2000): Concrete which is deposited through a pipe or tube, having at its upper end a hopper for filling and a bail for moving the assemblage.

Trench: A narrow excavation (in relation to its length) made below the surface of the ground.

Trenchless technology (ASCE, 2001): Techniques for conduit renovation with minimum excavation of the embankment dam or ground surface.

Tunnel (FEMA, 2004): An long underground excavation with two or more openings to the surface, usually having a uniform cross section, used for access, conveying flows, etc.

Turbidity meter (ASCE, 2000): A device that measures the loss of a light beam as it passes through a solution with particles large enough to scatter the light.

Ultimate strength design method: See **Strength design method**.

Ultrasonic pulse-echo: A nondestructive testing method that measures the time for an ultrasonic (acoustic) wave generated by a transducer to travel through a structure and return to a sensor.

Ultrasonic pulse-velocity: A nondestructive testing method that measures the speed of an ultrasonic (acoustic) wave generated by a transducer to travel through a structure to a remotely located sensor.

Unwater: Removal of surface water; removal of water from within a conduit.

Uplift (ASCE, 2000): The pressure in the upward direction against the bottom of a structure, such as an embankment dam or conduit.

Upstream access: Entry through an entrance structure or inlet portal of a conduit.

Upstream control: Regulation of flow within a conduit located near or at the entrance structure or inlet portal.

Utility conduit: A conduit utilized for electricity, gas, telecommunications, water, or sewer service.

Valve (FEMA, 2004): A device fitted to a pipeline or orifice in which the closure member is either rotated or moved transversely or longitudinally in the waterway so as to control or stop the flow.

Void: A hole or cavity within the foundation or within the embankment materials surrounding a conduit.

Water content (ASTM D 653, 2002): The ratio of the mass of water contained in the pore spaces of soil or rock material, to the solid mass of particles in that material, expressed as a percentage.

Water quality: The condition of water as it relates to impurities.

Waterstop (ACI, 2000): A thin sheet of metal, rubber, plastic, or other material inserted across a joint to obstruct the seepage of water through the joint.

Watertight (ASTM C 822, 2002): Will restrain the passage of water to not exceed a specified limit.

Weir (ASCE, 2000): A barrier in a waterway, over which water flows, serving to regulate the water level or measure flow.

Working stress design method (ACI, 2000): A method of proportioning structures or members for prescribed working loads at stresses well below the ultimate, and assuming linear distribution of flexural stresses. Also known as the alternate design method.

Zone: An area or portion of an embankment dam constructed using similar materials and similar construction and compaction methods throughout.

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Appendix A

History of Antiseep Collars

The purpose of this appendix is to explain:

- How antiseep (cutoff) collars were historically used in an attempt to prevent failures from uncontrolled seepage of water along the outside of conduits

- How research and improved understanding of failure mechanisms caused antiseep collars to be abandoned as a standard design element in embankment dams

- How filters provide improved protection against failures caused by water flowing along the outside of conduits installed in earthen embankment dams

Designers have always been concerned with seepage along conduits extending through earthfill or earth and rockfill embankment dams. Many observed failures of embankment dams have occurred near conduits, which accentuated this concern. Until about the mid-1980s, antiseep collars and careful compaction of backfill around conduits were the traditional methods for attempting to prevent problems caused by water flowing along the outside of conduits. As additional failures occurred, research was instituted to determine the basic mechanisms causing these problems. Once failure mechanisms were understood more completely, filters replaced antiseep collars as the preferred design tool to control seepage along conduits.

A.1 Concrete gravity dam experience

Concrete gravity dams were a popular type of water control structure early in the 1900s. Several failures of these structures caused a reexamination of design procedures. One approach to preventing failures of these dams from uncontrolled seepage under them was to increase the length of the flow path under the structures by using cutoff walls at the upstream and downstream edges of the dam. Studies of concrete gravity dam failures showed that some foundation soil types were much more likely to fail from water flowing under the dams than others were. Tables were developed that showed typical values of hydraulic gradients that were considered safe for different soil types. If a preliminary design showed that a hydraulic gradient was excessive, the structure was lengthened, or the dimensions of the cutoff walls were increased to reduce the gradient at the toe of the dam.

An example of a table formerly used in designing concrete gravity dams is reproduced in table A-1 from Reclamation's *Design of Small Dams*, First Edition (1960). The table shows twelve soil types and the "critical creep ratio" for each soil type. The term creep ratio basically represents the inverse of the hydraulic gradient in a structure design. Early designs without downstream drainage considered a structure to be safe against seepage forces if the computed creep ratio was larger than the values listed for the foundation soil type in the table. If the preliminary design was inadequate by this criterion, the length of the structure was increased, or the dimensions of the cutoff walls were lengthened. Later, more reliance was placed on drainage and filters downstream of the structures.

Table A-1.—Critical creep ratios for various soil types

Soil type	Critical creep ratio*
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

* A structure is considered safe only if the computed creep ratio is larger than the listed value for the foundation soil type. Higher creep ratios result from longer seepage distances and lower head differences. To increase the creep ratio of a design, cutoff walls were lengthened.

A.2 Creep ratio tables

In the table in the previous section, the term creep ratio is defined as the weighted seepage path length divided by the difference in head between the upstream pool and the downstream discharge elevation.

Several important points are illustrated with this listing:

- Soils with the highest susceptibility to backward erosion piping are very fine sands or silts and fine sands, and soils with the lowest susceptibility are boulders with some cobbles and gravel, and clays.

- Clay soils have a high resistance to backward erosion piping, even when subjected to large hydraulic gradients.

- Although a soil may not be susceptible to backward erosion piping, internal erosion of cracks may pose serious problems for these soils. The table does not address the resistance to internal erosion of various soil types, only the resistance to piping. The resistance to internal erosion primarily depends on the plasticity of the soil fines, the dispersivity of clay in the soil, and whether the soil is very broadly graded.

A.3 Antiseep collar design in earthen and earth-rock dams

Designers of earthen and earth-rock dams adopted the philosophy of increasing the length of the seepage path used for concrete gravity dams. Concrete collars termed antiseep collars were constructed at regular intervals along conduits through the dams to increase the length of the flow path of water along the outside of the conduit. The theory was that forcing water to take a longer seepage path would dissipate hydraulic forces and reduce the likelihood of piping at the downstream embankment toe. The collars usually extended outward from the conduit by a dimension equal to the diameter of the conduit, or more.

Antiseep collars were often constructed using the same materials used for the conduits. Probably the most common material was formed concrete. Steel, corrugated metal, and plastic collars have been used for conduits made of similar materials. Collars were spaced at regular intervals along the conduit within the predicted zone of saturation of the earthfill zone. In the case of central core fills with rockfill shell zones, the collars were usually installed only within the compacted core of the embankment.

The Soil Conservation Service, now the Natural Resources Conservation Service, constructed thousands of small, earthen embankment dams, many of them between

1950-1970. Most of these sites had concrete conduits that would slowly release the temporarily impounded floodwaters. The design criterion used for most of those embankment dams arbitrarily required that the seepage path through the saturated portion of the embankment be increased by 15 percent by adding antiseep collars. This requirement did not vary with the soil type in the embankment dam. Usually, collars were spaced along the conduit every 20 to 25 feet through the earth core of zoned embankment dams or through the central portion of homogeneous dams.

Figure A-1 shows a conduit with antiseep collars with hand compacting of earthfill next to the conduit. Figure A-2 shows a failure of a conduit with antiseep collars constructed around the conduit.

A.4 Changes in philosophy

From 1960 to 1980, a number of small embankment dam failures occurred, even though antiseep collars were carefully installed in well constructed earthen embankments. Several of the failures were at structures designed by the Natural Resources Conservation Service. Sherard (1972) reported on a study of those failures. The study showed that intergranular seepage and associated backward erosion piping was not the mode of failure for these embankment dams. The failures usually occurred almost exclusively when the completed embankments were first subjected to a pool, long before a phreatic surface had time to develop through the compacted earthfill. Other studies by Sherard (1973) on larger earthen embankment dams attributed failures and near-failures to internal erosion of clay cores through hydraulic fractures in the embankment zones.

The reasons why antiseep collars were ineffective in preventing failures near conduits may be summarized as follows:

- The antiseep collars only influenced water flowing in the immediate vicinity of the conduit. The collars did not significantly affect the remainder of the surrounding earthfill. Most of the failures were found to have occurred not immediately along the conduit, but in compacted fill outside the zone of the antiseep collars.

- The antiseep collars were designed to increase head loss in intergranular seepage, but most failures occurred long before steady seepage conditions occurred. Studies showed the failures occurred from water flowing along cracks within the earthfill, and not through the soil immediately next to the conduit.

Sherard, Decker, and Ryker (1972) discuss the mechanisms by which failures can occur in the following quote from the reference. In the quote, the term piping is

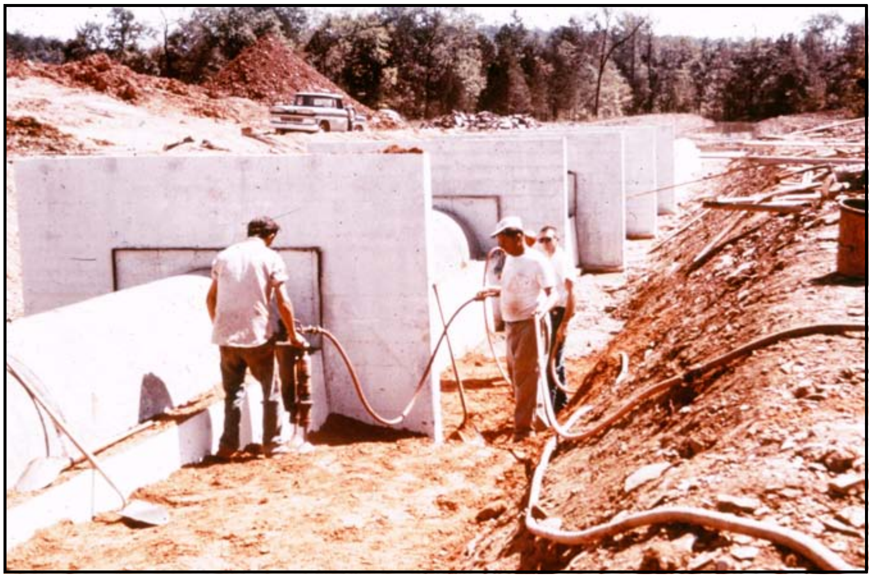

Figure A-1.—Hand tamping embankment material next to an antiseep collar.

Figure A-2.—Internal erosion failure along a conduit with antiseep collars.

generically used to describe two distinctly different mechanisms of failure, backward erosion piping and internal erosion, as defined in this document. The glossary defines the terminology used in this document to describe these mechanisms in more detail. As quoted from the reference:

> One main source of the commonly held idea that piping is most likely to damage dam structures in cohesionless soil is the experience which led to the establishment of the Bligh "creep ratio", and, later the Lane "weighted creep ratio" theory. This theory was developed from studies of failures of concrete dams on soil foundations. Under the theory, cohesionless silts and very fine sands are the materials which require the longest seepage path to avoid piping: a weighted creep ratio of 8.5 or more is needed for dams underlain by very fine sand or silt, whereas a weighted creep ratio of 2.0 is adequate for foundations of medium clay.
>
> It would appear, therefore, that the experience underlying the creep ratio theory indicates that piping of a given leak along a given seepage path is many times more likely to occur in fine sand than in clay. It is also apparent, however, that the conditions which are needed to cause piping of a leak passing through a soil foundation directly under a concrete dam are wholly different than those which are most likely to cause piping inside an earth embankment, because the concrete dam provides a roof for the leakage channel which cannot collapse. Hence, the conclusion that piping failures in homogeneous earth (embankment) dams may be statistically more likely in embankments of clay than in embankments of cohesionless soils does not conflict with the weighted creep ratio theory. Completely different mechanisms are involved in the two cases.

Because seepage was determined not to be the cause of most of the observed failures and because many of the failures occurred near conduits with properly designed and installed antiseep collars, designers reconsidered how best to prevent similar failures. Research by Sherard and others resulted in numerous seminal papers on the effectiveness of properly designed filters to collect flow through cracks in embankment dams. Other papers on causative mechanisms of cracks in embankment dams also were authored. Hydraulic fracture associated with differential movements in compacted fills was the primary mechanism identified as creating cracks through which scour could occur.

Based on this history, the major design organizations constructing embankment dams abandoned the use of antiseep collars in the mid-1980s. Seepage collars were seen to be ineffective in preventing many failures associated with conduits, and even thought possibly to have contributed to failures. By inducing additional differential settlement and impeding proper compaction around the conduits, cracking of the surrounding earthfill could more easily occur. The major design agencies adopted an alternative design measure to intercept water potentially passing through the earthfill surrounding conduits. The design includes a zone of designed granular filter surrounding the penetrating conduit. This filter zone is termed a filter diaphragm or collar.

Since this type of design has come into common usage, very few failures have occurred. The filter is thought to intercept water that can flow through cracks in embankment dams. The filter has a designed gradation that blocks eroding soil particles and prevents subsequent enlargement of the flow path by sealing the avenue for erosion. Philosophically, the filter diaphragm is more of a crack interceptor and sealer than it is a collector of seepage. If seepage in the embankment dam is a concern, more substantial zones including chimney drains are required. The design and construction of filters is discussed in more detail in the body of the document.

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Appendix B

Case Histories

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Medford Quarry Wash Water Lake Dam	Maryland	Failure of an embankment dam due to internal erosion along the conduit	67
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Project name: Anita Dam

Location: Montana

Summary: Failure of an embankment dam by internal erosion along the outside of the outlet works conduit

This case history illustrates an embankment failure likely caused by internal erosion despite the inclusion of antiseep collars. Dispersive clays also contributed to the failure.

Anita Dam is located about 22 miles north of Chinook, Montana, about 5 miles south of the Canadian border. The drainage, which normally flows only in response to snowmelt or heavy rain, is an unnamed tributary of the East Fork of Battle Creek, which flows southward into the Milk River.

Construction of Anita Dam was completed in November 1996. The embankment dam has a height of 36 feet, a crest length of about 1,012 feet, and a crest width of 14 feet. The reservoir impounds 794 acre-feet. The embankment was constructed with an overflow 36-inch diameter steel outlet conduit as the principal spillway. Two additional natural spillways were located on the reservoir rim to safely pass the probable maximum flood. The embankment was constructed as a homogeneous fill with a layer of upstream riprap. After the incident, it was determined that the fill material was a lean clay (CL) with dispersive properties.

The spillway conduit for Anita Dam utilized a series of concrete antiseep collars surrounding the conduit, but without a continuous cradle. During construction "flowable" backfill (essentially a high slump soil cement) was placed under the conduit to provide support between the antiseep collars. Rock-filled gabions were used at the downstream end of the conduit.

During the spring runoff of 1997, unusually heavy snowpack caused the reservoir to fill in the 4 days immediately prior to the failure. On the morning of March 26, a large leak beside the outlet conduit was noticed (figure B-1). Emergency response teams were dispatched to the site.

Upon arrival, the teams verified the large amount of leakage around the conduit, along with multiple vortexes in the reservoir water surface about 150 feet upstream of the embankment dam. Outflow from the outlet conduit and leakage was estimated to be about 400 ft^3/s; this greatly exceeded the capacity of the outlet conduit alone. Around-the-clock surveillance was instituted. Nineteen families downstream of the embankment dam were notified of the potential for evacuation

Figure B-1.—Outflow of water at downstream end of outlet conduit during failure. Note flow from conduit itself and from the area adjacent to the conduit.

and four families chose to evacuate. During the next day, a local National Guard Unit and Type II Incident Command Team were dispatched to the site. A helicopter was utilized to assess surrounding conditions, including flow into the reservoir.

The reservoir completely drained through the outlet conduit and caverns in the earthfill adjacent to the conduit, concluding on March 27, 36 hours after initiation of the incident. Complete embankment dam collapse did not occur. Following drainage of the reservoir, inspections indicated that the embankment material had been completely eroded from around the outside of the conduit (figure B-2). The open tunnels immediately adjacent to the outlet conduit extended from the upstream embankment toe completely through the embankment dam to the downstream toe.

Figure B-2.—A view of upstream end of outlet conduit following failure. Note formation of caverns immediately adjacent to seepage cutoff collars. These caverns extend to the downstream embankment toe.

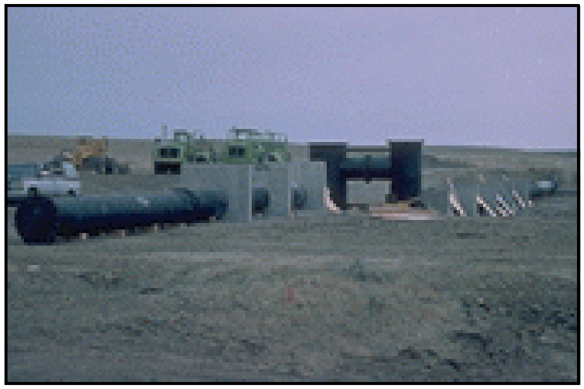

Figure B-3.—Initial construction of embankment dam and outlet conduit (upstream is to left). Note presence of antiseep collars surrounding conduit.

Figure B-4.—Initial construction. Note hand tampers being used to compact earthfill adjacent to outlet conduit.

The cause of the failure was likely the combination of the dispersive clay embankment material, hydraulic fracture, antiseep collars (figure B-3) around the conduit that required the use of hand tampers (figure B-4), "flowable" backfill for conduit support in lieu of a continuous concrete conduit support, and lack of a filter and drain around the outlet conduit in the downstream portion of the embankment dam. Cold air flowed through the conduit during the winter preceding the failure. This caused lenses of frozen material in the conduit's backfill. These lenses may have provided a path for concentrated leakage when they were thawed by the initial flow of water in the conduit during the runoff.

Lessons learned

Even though antiseep collars were utilized, a major leak occurred along the conduit, causing rapid erosion of the dispersive clays used to construct the embankment dam.

Key changes to the design that would likely have prevented embankment failure include:

- Elimination of the antiseep collars
- Utilizing a concrete encasement around the outlet conduit that allowed for better compaction of the earthfill against the conduit and to provide insulation during cold weather
- Lime treatment to stabilize the dispersive soils around the conduit
- Utilizing a filter diaphragm with drainage provisions to the downstream toe
- Provisions to stop the flow of cold air through the conduit during the winter
- Provisions for slow first filling of the reservoir

Reference

Bureau of Land Management, *Anita Reservoir (Blaine County, Montana) Dam Failure*, Report by Board of Inquiry, August 20, 1997.

Project name: Annapolis Mall Dam

Location: Maryland

Summary: Forensic investigation of a spillway conduit failure

In March 1993, a newly constructed embankment dam near Annapolis, Maryland, rapidly filled with water during a storm and failed, causing extensive environmental damage, but no loss of life or damage to downstream roadways. Figure B-5 shows the upstream section of the 54-inch diameter CMP spillway conduit, having completely collapsed when the water level reached the elevation of the weirs on the inlet structure.

The 25-foot high embankment dam, built in 1992 to manage stormwater runoff from expansion of a nearby shopping mall, collapsed less than 1 year after it was constructed. Based on the original design drawings, the spillway inlet structure (riser) was a reinforced cast-in-place concrete box about 15 feet high and 10 feet square. The spillway conduit was 54-inch diameter CMP. The overall length of the conduit was about 75 feet, and was to be constructed on a relatively steep slope of about 10 percent. The first conduit joint was to be made within 2 feet of the riser. Four corrugated metal antiseep collars were to be installed on the conduit at distances of

Figure B-5.—The upstream section of the 54-inch diameter CMP spillway conduit completely collapsed when the water level reached the elevation of the weirs on the inlet structure. Site personnel reported a "vortex" in the pool adjacent to the structure shortly before collapse.

10, 15, 20, and 25 feet from the riser. The conduit joints were to consist of 13-inch wide "hugger bands" with o-ring gaskets installed in "re-rolled" corrugations at the ends of each conduit section.

A review of a videotape of the site during construction (which was made for training purposes, not for documentation of construction, and only incidentally contained footage of the dam construction) indicated that a substantial portion of the dam embankment had been placed prior to delivery and installation of the spillway CMP. The spillway conduit was then apparently installed into a narrow trench with vertical sides, excavated through the partially completed embankment dam and into the foundation soils. The design engineer was not required to be onsite during construction, and construction inspection was at the discretion of the contractor.

Site personnel noted that just before failure, the pool level was at the upper weir elevation, and a vortex (whirlpool) was observed in the pond adjacent to the spillway. Failure occurred at about midday on March 4, 1993. After the failure, about 26 feet of the upstream section of the CMP was observed to have completely collapsed. The bottom of the collapsed portion of the conduit exhibited an inverted "V" shape. A large amount of upstream portion of the embankment had washed out through the downstream portion of the CMP, which remained partially intact. Deep, vertical troughs were visible on the downstream slope directly above the sides of the CMP. The sediment level in the channel below the dam obscured the bottom half of the CMP. Figure B-6 shows the downstream section of the CMP spillway remained partially intact, but deep troughs were visible directly above each side of the conduit.

An unauthorized grating with small openings (i.e., chain link fence), which had been bolted to the downstream end of the conduit by the owner to prevent vandalism, was observed to be nearly plugged over its entire area with debris, indicating that the CMP probably was full of water at the time of the failure. The grating was detached from the end of the CMP during the failure, and an o-ring joint gasket was observed entwined in the grating and debris. Grass growing on the dam embankment at the downstream toe near the spillway outlet was bent downstream, indicating that water had flowed along the outside of the conduit during the failure.

About 2 weeks after the failure, a team of geotechnical engineers, state and local officials, surveyors, lawyers, and the local pipe manufacturer observed excavation of the failed spillway conduit in order to determine the cause of failure and who was responsible. Engineers from at least three companies were present, representing the embankment dam owner, the contractor, and the original designer.

Two large excavators carefully removed soil from above the conduit. The sides of the excavation were sloped as required for stability. A surveyor documented the

Figure B-6.—The downstream section of the CMP spillway remained partially intact, but deep troughs were visible directly above each side of the conduit.

location of items of interest (elevation of top of conduit, conduit invert, locations of joints and antiseep collars, etc.) as directed by the engineers.

The conduit included three joints. When the hugger band at the downstream joint was removed, one of the o-ring gaskets was observed to have been displaced into the conduit, and there was debris from the pool under the band. This indicates that water from the pool may have flowed along the outside of the conduit, despite the antiseep collars, and into the joint. Figure B-7 shows the spillway conduit and portions of the soils that were carefully removed and documented during a forensic investigation about 2 weeks after the failure. Figure B-8 shows the two large excavators that removed the majority of the embankment dam. Figure B-9 shows the o-ring gasket that was found to have been displaced.

The forensic investigation confirmed that the CMP was installed in a trench with near vertical sides. In addition, it appeared that the trench may have been overexcavated and backfilled with poorly compacted fill material, which was quickly eroded away by flow along the outside of the conduit and/or into open joints. Loss of soil support would have caused additional conduit deformation, further opening the joints, resulting in an ever-increasing cycle of leakage and loss of earthfill by internal erosion. Figure B-10 shows the presence of undisturbed foundation soils

Figure B-7.—The spillway conduit and portions of the embankment soils were carefully removed and documented during a forensic investigation about 2 weeks after the failure.

Figure B-8.—Starting at the downstream end of the conduit, two large excavators removed the majority of the earthfill under the watchful eyes of State and local officials, geotechnical engineers, surveyors, lawyers and the pipe manufacture.

Figure B-9.—When the "hugger band" at the most downstream conduit joint was removed, the o-ring gasket was found to have been displaced. In addition, debris from the pool was found under the band, indicating that water may have flowed unrestricted along the outside of the conduit from the pool and into the joint.

that confirmed that the conduit was placed in a trench with nearly vertical sides, making it difficult to obtain good compaction of the fill soils under and along the CMP and around the antiseep collars.

In addition, it was determined that the antiseep collars were installed in locations substantially downstream of the designed location. Seepage along the sides of the CMP and under the "haunches" and the resulting loss of backfill soils caused the CMP and joints to deform. Figure B-11 shows the result of seepage along the sides of the CMP causing loss of soil support, leading to conduit and joint deformation.

Lessons learned

A private engineer designed the embankment dam to control stormwater runoff associated with enlargement of a local shopping mall. The original design engineer was not onsite during construction, and the contractor was to provide construction supervision.

The contractor constructed the embankment dam for a local government highway agency, who also planned to use the pond for stormwater management for a nearby road improvement project. The contractor specialized in utility construction, not embankment dam construction. The inspection firm had no control over the

Figure B-10.—The presence of undisturbed foundation soils confirmed that the conduit was placed in a trench with nearly vertical sides, making it difficult to obtain good compaction of the fill soils under and along the CMP and around the antiseep collars.

Figure B-11.—Seepage along the sides of the CMP resulted in loss of soil support, causing conduit and joint deformation.

project, and served only to document the placement of earthfill and compaction. The local government agency overseeing the project was a highway department, which had little dam construction experience.

The embankment dam was completely removed, and a new embankment dam and concrete spillway conduit were constructed 2 years later in the same location. When the foundation for the new dam was being prepared, the State inspector observed roots and other debris, under the original embankment fill. Since such material should have been removed, this reinforces the notion that supervision of embankment dam construction by qualified engineers is essential. After reconstructing the embankment dam, the owner eventually decided not to pursue legal action to determine fault. The original contractor went bankrupt just before the failure occurred, the local government may have incurred some liability for overseeing the construction, and the design was apparently completed in accordance with the approved standards in place at the time the design was started. (Although more restrictive standards requiring different conduit joints had been developed before the original embankment dam was constructed, the design approval was apparently "grandfathered" under the older standard.)

The failure resulted in new requirements that spillway conduits not be installed into near-vertical trenches excavated into the foundation or partially completed embankment dam. This trench installation technique is common procedure for highway culverts, because the sides of the trench facilitate "arching" of the backfill, reducing the load on the culvert. However, this results in areas of low soil pressure along the conduit, facilitating seepage along the conduit. Filters are now routinely constructed around the downstream portion of the conduit to intercept this type of flow and prevent internal erosion. In addition, the use of large diameter flexible conduits for embankment dam spillways has been substantially reduced in the last 10 years because of this and other failures related to large deformations, difficulty in obtaining watertight joints, and difficulty placing earthfill under the sides of the conduits (Van Aller, 1993).

References

State of Maryland, Dam Safety Division, unpublished investigation notes and file photographs (MD Dam No. 372). The reports of the forensic investigation were not submitted to the State and are not public information.

Van Aller, *Recent Failures of Large Corrugated Metal Pipe Spillways*, ASDSO 1993 Annual Conference, 1993.

Project name: Arkabutla Dam

Location: Mississippi

Summary: Construction error leads to defective joints in an outlet works conduit

Arkabutla Dam is an embankment dam located in northwest Mississippi on the Coldwater River (figure B-12).

The USACE designed the embankment dam for the purpose of flood control. The embankment dam was completed in 1943 and is 83 feet high, 10,000 feet long, and controls a drainage area of approximately 1,000 square miles. Runoff from the drainage area is stored in the lake created by the embankment dam, and the water is released at a controlled rate through a gated intake structure located in the lake. Water passing through the control structure is released to the downstream river channel through an egg-shaped reinforced concrete conduit. The conduit is 325 feet long, 18.25 feet high, and 16 feet wide, and the sides of the conduit are 42 inches thick. At spillway crest, the embankment dam has a storage capacity of 525,000 acre-feet.

Arkabutla Dam is constructed across a broad alluvial valley. The intake structure is located in the reservoir in the alluvial valley, not in the abutment. The clay top stratum was removed and the intake structure, conduit, and stilling basin are founded on alluvial sands. Since the conduit is founded on sand, it was imperative that the

Figure B-12.—Arial view of Arkabutla Dam, Mississippi.

waterstops in the joints of the conduit be designed and installed properly. Unfortunately, this did not happen. There is no written record why this error was not discovered sooner. A cross section of the outlet works conduit is shown in figure B-13. The designers intended that the lower one-third of the conduit be constructed as one continuous monolith. Above the field joint shown in figure B-13, the conduit was designed to be cast in 25-foot long monoliths.

At each monolith joint, a rubber waterstop was placed in the top two-thirds of the conduit. The waterstop was placed in the center of the conduit walls and extended from 2 feet below the field joint on one side of the conduit to 2 feet below the field joint on the other side. No waterstop was installed along the lower one-third of the conduit, but the contractor also constructed the lower one-third of the conduit monolithically. This was not the design intent. With no waterstop along the bottom of the conduit, the very fine sand in the foundation was eroded through each joint and was continually being flushed downstream during operation of the outlet works.

Problems with the joints were discovered soon after the project went into operation. Lead wool was used for several years to control the erosion of fine sand into the conduit, but problems were experienced in keeping the lead wool in the joints. The designers estimated that the maximum settlement of the conduit would eventually be 0.25 feet. However, by 1950, the conduit had settled as much as 0.75 feet. Much of this unexpected settlement was attributed to the loss of sand through the joints of the conduit. The first attempt to grout the joints was undertaken in 1950. Grout "takes" were not significant in 1950, except at joints 5-6 and 6-7, where 47 and 98 cubic feet of grout, respectively, were pumped. There are 13 monoliths, with monolith 1-2 being at the upstream transition and monolith 12-13 being at the downstream end of the conduit. Monolith 6-7 is about 60 feet downstream of the centerline of the embankment dam. Monolith 4-5 is about 10 feet downstream of centerline of dam. Additional settlements since 1950 have been less than 1 inch; however, in 1970 an attempt to grout the joints again was made due to sand being eroded into the conduit through joints. Water and trace amounts of sand were again noted coming from some joints in 1977, but attempts to grout the joints resulted in only insignificant amounts of grout "take." Since 1977, at least three attempts have been made to stop water coming from the joints using chemical grout. Grout takes were significant only at joints 3-4, 5-6, and 6-7, where 410, 33, and 68 cubic feet of grout, respectively, were placed. Joints 3-4, 5-6, and 6-7 have caused the most trouble, but essentially all joints have had to be grouted at least once with either chemical grout or neat cement grout.

The above background provides an introduction to the problem experienced in the fall of 2003. While in the conduit to replace the filler in the outer dove-tail portion of the joints, joint 6-7 broke lose and started to make sand at a significant rate (2/3 cubic yard in 1 hour). Even though it has been known since construction that there were no waterstops in the lower portion of the conduit, it was thought that the joints

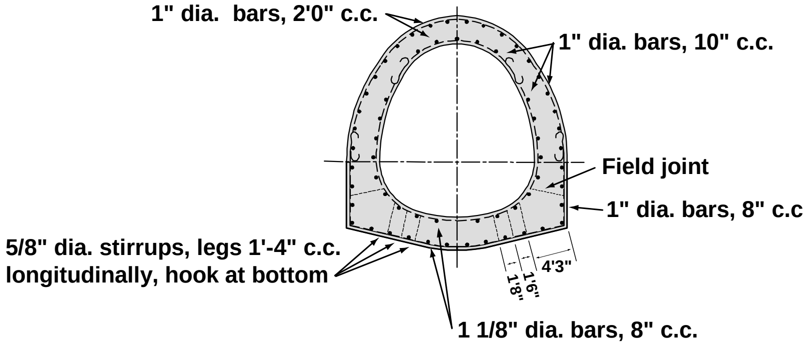

Figure B-13.—Cross section of the outlet works conduit.

had been satisfactorily sealed with grout and that periodic grouting with chemical grout would keep the conduit watertight. However, since joint 6-7 broke lose suddenly and flowed sand at a significant rate, the conduit could no longer be considered safe for static loading and definitely could not be considered safe for earthquake loadings. Therefore, a team was formed to determine what should or could be done to ensure the continued safety of the conduit. The team solicited advice from personnel throughout the USACE. The consensus felt that the obvious solution was to line the conduit with a steel liner. However, the team recognized that this would be very expensive and would require bypassing outflow. Also, going through the design process, review process, and budgeting process would take several years. Therefore, the team elected to experiment with an interim measure that may or may not be the final solution. The team felt that if a plate were bolted across the joint with gasket material beneath it to prevent sand from exiting from the joint, then this would solve the problem. However, no one had any experience in doing this, and there was no assurance that the plate would stay in place. Therefore, the USACE elected to experiment with two joints during the time that the structure was unwatered for the 5-year formal inspection. A metal plate was bolted across joints 5-6 and 6-7 during a shutdown of the outlet works (figure B-14).

The following gives a brief outline of what was done:

- *Cement mortar*.—Cement mortar was used to smooth the surface across the joint. This mortar was from 1.8 inches thick to 1 inch thick (had this much differential settlement at one joint).

- *Steel plate*.—Twenty-two feet of stainless steel plate was installed along the bottom of the joint. At a distance of about 10.5 feet from the centerline of the conduit, a 1.25-inch diameter hole was drilled in the side of the conduit to the existing waterstop (about 21 inches).

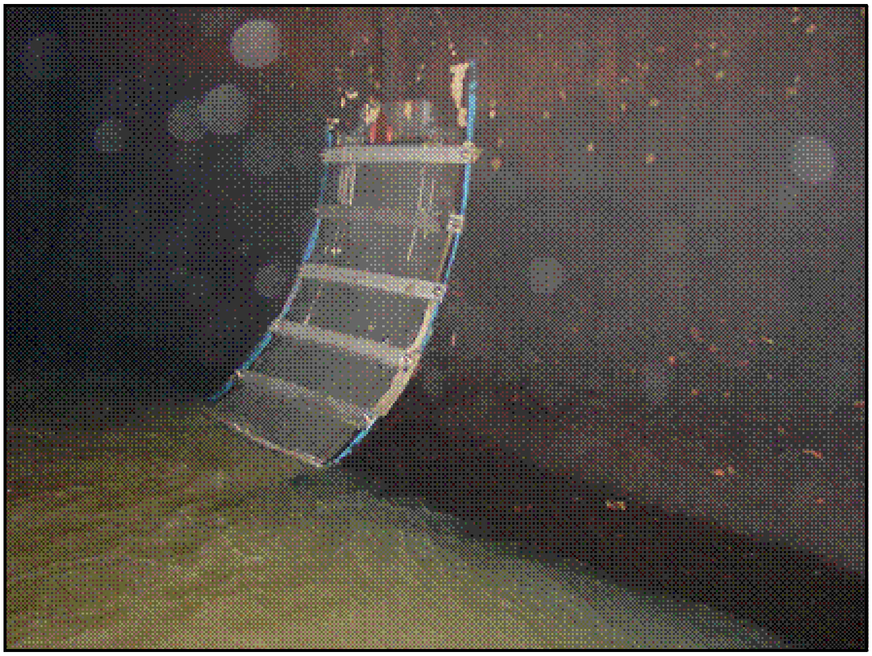

Figure B-14.—Metal plate bolted across the joint.

- *Waterstop*.—A waterstop was formed in this hole using backer rod saturated with chemical grout.

- *Chemical grout*.—After chemical grout in the backer rod had hardened, chemical grout was pumped through a tube that had been installed to the bottom of the hole. Grout was pumped under pressure to fill any voids and try to get a good contact with the rubber waterstop.

- *Anchors*.—The metal plate was designed to have metal straps hold it down. The metal straps were anchored to the conduit with stainless steel anchors installed 4 inches deep. Anchor straps were installed on 1-foot centers. Therefore, 46 holes had to be drilled at each installation. A two-part epoxy was used to hold the anchors in place.

- *Fabric*.—Two layers on engineering fabric were installed at each joint.

- *Compressible rubber*.—A 1.25-inch thick layer of compressible rubber was installed about 2 inches from the joint on each side of the joint.

- *Additional steel plates*.—A 1/8-inch stainless steel plate 36 inches wide was installed on top of everything. Several holes had been previously drilled in this plate along its centerline to let any water that seeped up along the joint pass through the filter fabric and out through the holes without building up pressure under the plate.

- *Metal straps.*—The metal straps $^{3}/_{8}$ inch thick and 3 inches wide were then bolted across the plate, compressing the material and producing a slight bow in the plate.

- *Outer edges.*—The outer edges of the plate were then sealed to temporarily hold the cement grout in place. This grout was placed between the steel plate and the engineering fabric. No grout was placed between the two rubber seals, so that any seepage along the joint could pass through the fabric and not build up pressure. This was done to give the plate solid support to help minimize vibrations.

After installation, the project released 1,500 ft^3/s for 2 weeks and then inspected the joint. The inspection team found the plate at one of the two repair joints displaced, bent, and torn (figure B-15).

As of September 2004, the USACE had yet to determine why the patch did not stay in place and can only speculate that a log may have hit it. Currently, the USACE is evaluating installation of a steel liner as the only reliable solution for the defective joints.

Figure B-15.—Damaged metal plate.

Lessons learned

Close construction oversight is required for constructing conduits through embankment dams. Repair efforts are not always successful, and complete renovation may be required.

Reference

USACE project files and periodic inspection reports.

Project name: Balman Reservoir Dam

Location: Colorado

Summary: Reservoir evacuation by pumping and controlled breaching techniques

Balman Reservoir Dam is an earthfill embankment dam located in San Isabel National Forest in south central Colorado near Cotopaxi and is owned by the U.S. Forest Service. The embankment dam has a maximum height of 31 feet with a crest length of 75 feet and impounds approximately 51 acre-feet of water. The embankment dam was constructed at approximately elevation 9,400 feet and is located in a remote area. The embankment dam has a small earthcut, 12-foot wide spillway and no gated outlet works. The embankment dam was built in 1965.

On November 4, 1996, a large sinkhole was discovered on the upstream slope and crest of the embankment dam. The sinkhole was measured to be approximately 8 to 10 feet in diameter and approximately 6 to 8 feet deep. A small whirlpool was observed in the reservoir near the sinkhole, indicating the presence of continual flow into the cavity and apparent sediment transport into the sinkhole. Cracks were developing in the embankment dam crest above the sinkhole (figure B-16) and sloughing of the embankment materials into the sinkhole was observed. Extensive water flow was occurring all along the downstream toe of the embankment dam and along the right abutment groin up to approximately mid-height of the dam. Water exiting the slope was also occurring on the downstream face above an 18-inch diameter drainpipe for the chimney drain.

After reviewing the original construction plans for the embankment dam and taking into account its past poor operational performance and the worsening condition of the sinkhole and dam, it was apparent that the dam was experiencing an internal erosion failure, which could eventually result in a sudden and catastrophic breach of the dam and a release of the reservoir. The downstream hazard consists of a church camp, a hiking trail, a campground, and a county road. Based on this, it was decided that the reservoir needed to be lowered to a safe storage level and to a level where no more seepage was exiting on the downstream face of the dam and along the downstream toe. Since the embankment dam lacked an outlet works, it was decided to try lowering the reservoir by an alternative means.

The first attempt to lower the reservoir water surface to a safe level was made by diverting the reservoir inflow away from the reservoir. The diversion structure above Rainbow Lake, located upstream of Balman Reservoir, was adjusted on November 8, 1996 to direct all flows from the drainage basin into Rainbow Lake and away from Balman Reservoir in the hope that this would help lower the water

Figure B-16.—Cracking at the embankment dam crest above the sinkhole on the upstream slope of the embankment dam. Note the simple staking of the area to monitor movement of the crack.

surface level of Balman Reservoir. After five days of diverting the water, the water surface level in Balman Reservoir was lowered only approximately 1 to 2 inches, and the reservoir level remained just below the spillway crest. At this rate, the reservoir could not be drawn down in a reasonable period of time and be maintained at a safe level. Therefore, it was decided to partially breach the embankment dam.

On November 15, 1996, a portable 3,000 gal/min pump (figure B-17) was delivered to the site. The pump was set up adjacent to the spillway and placed into operation. The pump was operated continuously for 3 days, and the water surface level of the reservoir was lowered to approximately 8 feet below the spillway crest. At this point, the excavation to breach the embankment dam was commenced. A Caterpillar 330 track-mounted excavator was used to perform the excavation. The area of the sinkhole and the upstream slope were first excavated as far into the reservoir as the excavator could reach, and then the embankment dam crest was benched down approximately 4 feet on either side of the breach. The breach was excavated on both sides at a slope of 1.5H:1V down to the water surface level. Rocks of varying sizes were placed in the bottom of the excavation and down the downstream slope of the embankment dam to help control erosion. Then, a small amount of the embankment dam was removed from the breach entrance, and water was allowed to flow through the breached section. Some minor slope erosion occurred, but no backcutting of the channel into the embankment dam was observed.

Figure B-17.—Portable pump used to initiate draining of the reservoir.

The initial flow of water through the breach (figures B-18 and B-19) was allowed to stabilize and diminish, and then the process was repeated to remove another small portion of the embankment dam. The excavation of the embankment material was kept at a minimal amount to limit the quantity of water discharging through the breach section. This process was repeated, and each time the water flowing through the breach was allowed to flow out and stabilize before removing additional embankment material. The partial breaching of the embankment dam was completed over 2 days. The final dimensions of the breach obtained consisted of an 8-foot wide bottom, a 65-foot top width, a depth of 20 feet, and side slopes of 1.5H:1V. The bottom of the breach section was provided with rock riprap erosion protection. The embankment dam was not completely breached to allow for a small reservoir with a depth of 8 feet to remain for the purpose of controlling silt deposits in the reservoir and to maintain a fish habitat. With the partial breaching of the embankment dam, the dam was considered to no longer pose a safety hazard to the general public.

Lessons learned

This event demonstrated that with a relatively small reservoir and small embankment dam height, the reservoir can be released in a controlled manner by pumping and performing a controlled breach of the embankment dam. Care should be employed when attempting to release the reservoir by means of a controlled breach of the embankment dam.

Figure B-18.—Initialization of the breach in the embankment dam near the left abutment.

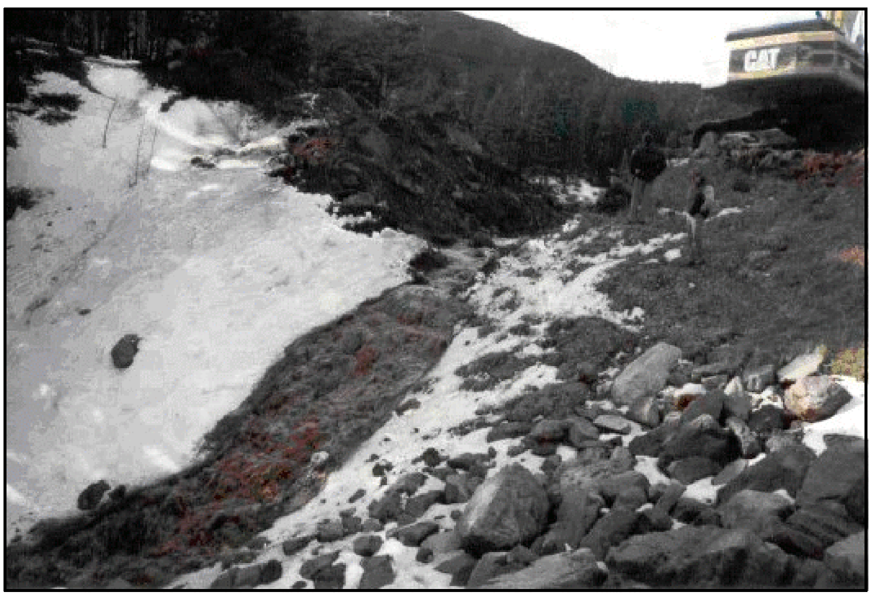

Figure B-19.—Discharge of water through partially breached section and down the downstream slope of the embankment dam.

Reference

Colorado Division of Water Resources, *Dam Safety Engineers Inspection Report Files—Incident Report for Balman Reservoir Dam*, State Engineers Office, Dam Safety Branch, 1996.

Project name: Beltzville Dam

Location: Pennsylvania

Summary: Conduit crack survey

Beltzville Dam is located in northeastern Pennsylvania. The embankment dam has a structural height of 170 feet, a crest length of 4,560 feet, and a crest width of 30 feet. Appurtenant features include a spillway and an outlet works. The outlet works is used for flood control. The outlet works consists of a gated intake structure, a 7-foot diameter concrete conduit approximately 1,165 feet in length, and a stilling basin. The USACE with assistance from the Beltzville Lake operations personnel performed a condition survey (also called a crack survey) of the outlet works conduit in July 2003. Previous formal surveys had taken place in 1971, 1988, 1992, and 1999. Although not specifically meeting all of the parameters defining a confined space, the outlet conduit was treated as such for man-entry. Personnel were trained in confined space operations and air monitoring equipment, and a hard-wire communications tool was used during the survey. Drawings showing that the results of the previous surveys were used as a baseline for performing the current mapping. Stationing within the conduit is marked periodically on the conduit walls, although some markings have lessened in intensity. Digital photographs were taken of some of the more prominent features.

Spalling had occurred at joints and other localized spots. Minor cracking, spalling, surface abrasion, and calcitic efflorescence were observed and mapped. Figures B-20 through B-23 are typical of these features. No major leakage was evident; however, minor seepage of water was observed in two locations but with no material being carried.

The 2003 survey noted changes in the sizes of some of the spalls and seven additional (or not-previously-mapped) spalls. Two new (or not-previously-mapped) occurrences of cracking and three new (or not-previously-mapped) occurrences of calcitic efflorescence were observed. Conversely, calcitic efflorescence no longer existed in six locations where it had been previously mapped. Flow through the conduit during high releases appeared to have removed these materials. In general, the condition of the conduit had changed little from the 1999 survey. A comparison of photos taken in 1999 and 2003 also indicated little change in the more prominent features. Drawings showing the features are developed after each survey is completed using different colors to denote the different surveys in order to follow changes in condition of the conduit.

Figure B-20.—Large spall at the construction joint located at station 12+13.

Figure B-21.—Exposed aggregate located at station 11+28.

Figure B-22.—Popout located at station 10+79.

Figure B-23.—Spall at the construction joint located at station 4+93.

The conduit is considered to be in a continued serviceable condition. Some new or increased instances of spalling, cracking, and efflorescence are apparent. Several instances of efflorescence were no longer apparent. No material is being carried with the existing minor seeping flows observed. The next conduit condition survey is scheduled for 2008 in conjunction with the next periodic inspection of the project.

Lessons learned

- Periodic conduit condition surveys and walkthroughs are essential for thorough dam safety monitoring.

- Use of different colors for different surveys in the drawings enhances interpretation of the condition of the conduit and allows for comparison with the results of previous inspections.

Reference

U.S. Army Corps of Engineers, Engineering Regulation 1110-2-100, *Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures*, 1995.

Project name: Bohemia Mill Dam

Location: Maryland

Summary: Undermining and failure of a new spillway conduit constructed on piles

The Bohemia Mill Dam is a very old structure, constructed in the early 1900s with a timber spillway structure supported on piles. The embankment dam is about 15 feet high with a two-lane county road on the crest. In the 1990s, deterioration of the timber bridge over the spillway led the county to enact weight restrictions and restrict traffic to a single lane until a new structure could be designed and built.

Geotechnical investigations of the embankment dam and foundation revealed that the underlying soils are very soft, and a decision was made to replace the spillway with a reinforced cast-in-place box culvert supported on 60-foot long steel piles. Seepage control along the culvert was to be provided by a bentonite slurry wall near the center of the embankment dam and by a filter drain at the downstream end.

Construction of the new spillway, slurry cutoff wall, filters, and new lake drain was completed in 2002 (figures B-24 and B-25).

In early 2004, an engineer inspecting the bridge notified the owner that he observed clear seepage from under the downstream end of the spillway conduit, but there was no indication of internal erosion or backward erosion piping of the embankment or foundation soils.

However, within a few months, a sinkhole was noted in the pavement on the embankment dam crest (figure B-26). The seepage flow had significantly increased, and soil particles were observed moving downstream. The condition rapidly worsened, and the size of the sinkhole increased (figure B-27). Particles of the bentonite slurry cutoff wall were observed to be washing downstream from an area that appeared to be boiling (figure B-28). Attempts to create a sandbag weir around the boil to reduce leakage under the spillway conduit were unsuccessful, and the lake was drained.

Currently, the structure has not been repaired. Repair options under consideration include construction of a jet grout slurry wall along the upstream side of the embankment dam and spillway or installation of sheetpile cutoff wall along the upstream side of the dam and culvert.

Figure B-24.—Because of soft foundation soils, numerous 60-foot long pipe piles were installed in winter 2001 to support a new reinforced cast-in-place concrete spillway structure.

Figure B-25.—This the downstream end of the spillway at the end of construction in 2002.

Figure B-26.—Less than 2 years later, an engineer inspecting the bridge over the spillway reported that seepage flow was visible from under the downstream end of the spillway. The seepage flow was clear, and no migration of soils was evident. A few months later, the roadway on the dam crest collapsed, and large quantities of sediment were observed in the pool below the dam.

Figure B-27.—The road was closed immediately, and lake level was lowered. However, the sinkhole rapidly enlarged.

Figure B-28.—Seepage flow at the downstream end of the spillway appeared to be boiling. Attempts to create a sandbag weir around the boil to reduce leakage under the spillway were unsuccessful.

Lessons learned

Avoid constructing conduits on piles, because the conduit may become undermined, allowing uncontrolled seepage to occur under it.

Reference

Maryland Dam Safety Division, dam file No. 158

Project name: Clair Peak Dam

Location: Maryland

Summary: Grouting from the embankment dam surface to fill voids along the outside of a spillway conduit

In April 2003, a police officer traveling on State Highway 235 near Lexington Park, Maryland reported a large "pothole" in the roadway. A highway repair crew dispatched to the site promptly filled the 12-foot long, 1.5-foot deep hole with asphalt patching material, and the road remained open to traffic (figure B-29). However, one lane of the roadway was closed a few hours later when it was observed that the patched area had again subsided and a sinkhole was located directly above the spillway conduit.

The lake at Clair Peake Dam had been in existence for many years before the highway was widened in 1983. As part of the highway modification, the original low-level concrete spillway pipe was abandoned by backfilling with concrete (The State Dam Safety Division was unable to determine why this was done). After the 26-foot high embankment dam was widened by placement of new fill on the downstream side, a new 24-inch diameter CMP spillway was installed under the

Figure B-29.—A sinkhole, which appeared in a heavily traveled roadway above a 20-year old CMP spillway, was filled with asphalt. Part of the roadway was closed when the asphalt patch subsided a few hours later.

roadway. The new pipe was installed on a steep slope completely through the new and old embankment zones, with the upstream end of the pipe set at the normal pool elevation and the downstream end at the toe of the new earthfill.

An inspection of the embankment dam in 1996 noted some "sinkholes" in the downstream slope of the embankment near the location of the pipe, but no repairs were made. Another inspection of the embankment dam and pipe exterior the following year noted substantial deposits of sediment in the stream channel just below the downstream end of the pipe. An interior evaluation of the pipe could not be made, because the sediment deposits at the downstream end obscured the pipe, and debris placed at the upstream end by a local beaver precluded inspection at the upstream end.

After the roadway collapsed in 2003, an inspection revealed extensive deterioration of the CMP and that substantial loss of embankment material had occurred. Ground penetrating radar (GPR) detected large voids in the embankment along the pipe, and a decision was made to construct a new spillway at a different location (figures B-30 and B-31). The failed CMP spillway and adjacent voids were filled with a stiff cement and flyash-based "compaction grout" (figure B-32). A specialty contractor performed the grouting and pumped the grout into the 24-inch diameter CMP using a trailer mounted, diesel powered, piston type concrete pump, specially mounted for grout injection work. The pipe fill mixture was a flowable, nonshrink, moderate strength (500-700 lb/in^2) grout with the following specifications:

- 600 pounds type I Portland cement
- 500 pounds flyash
- 500 pounds pea gravel
- 1800 pounds concrete sand
- 40 to 45 gallons of water
- 40 ounces of superplastizer

The compaction grouting to fill the voids in the embankment dam was performed from the roadway and median along the abandoned pipe alignment, with each grout pipe extending to approximately the pipe invert elevation. The grout-hole layout plan consisted of 128 compaction grout locations based on a 5-foot offset square grid throughout the anticipated treatment zone. The compaction grouting was performed utilizing a diesel powered track drill to install the grout casing through the pavement and fill soils to the invert elevation of the pipe. The grout was a blend of

Figure B-30.—Ground penetrating radar investigations were performed from the embankment dam crest.

Figure B-31.—Ground penetrating radar identified the location of voids along the CMP.

concrete sand, type I Portland cement, flyash, and water proportional to provide a pumpable mix with about 500 lb/in^2 strength in 28 days. The compaction grout mixture specifications are as follows:

- 200 pounds of Type I Portland cement

- 850 pounds of flyash

Figure B-32.—The failed CMP and voids were filled with a stiff compaction grout.

- 2300 pounds of concrete sand
- 35 gallons of water per cubic yard

During compaction grouting, vertical ground movements were monitored and recorded. A surveyor's level was being utilized to monitor the vertical ground movements. The system used was capable of detecting 1/8-inch movements. The slump of the grout was required to be maintained at 2 inches or less.

A filter diaphragm surrounding both the CMP spillway and the abandoned concrete pipe was constructed near the downstream toe of the embankment dam to intercept and control any seepage along the outside of the conduits.

Lessons learned

Compaction grouting may be satisfactorily used to fill voids existing along a spillway conduit, when conventional excavation cannot remove the conduit.

Reference

Maryland Dam Safety Division, dam file No. 275.

Project name: Como Dam

Location: Montana

Summary: Sliplining of an existing outlet works conduit using a steel pipe slipliner

Como Dam was constructed by semihydraulic fill method from 1908 to 1910. Como Dam has a crest length of 2,550 feet and a crest width of 25 feet. The structural height is 70 feet, and the base width is 400 feet. The spillway was constructed in 1923 and is located on the left abutment. The outlet works conduit through the embankment dam was a 6-foot inside diameter, 526-foot long concrete-encased redwood stave pipe. The wood staves in the pipe are 3 inches thick. Approximately half of the downstream conduit was relined in 1936 with 3/8-inch thick steel plate liner. Figure B-33 shows an aerial view of Como Dam.

The reservoir was maintained at a reduced level, because increased seepage through the embankment dam during the summer of 1992 caused concern over the safety of the dam. An emergency situation was declared, which required modification to the embankment dam and appurtenances to be completed in the fall of 1992 and winter of 1993 to allow operation of the reservoir in the spring. An original plan included replacing 130 feet of the downstream conduit, since the embankment above the pipe had to be removed and replaced. However, this was not feasible because of time restrictions. Potential seepage through the concrete in some areas of the pipe and the deteriorated condition of the redwood liner plates led to the plan to line the

Figure B-33.—Aerial view of Como Dam, Montana.

entire conduit with a steel pipe slipliner. This steel pipe slipliner would provide additional structural stability and a good flow surface, prevent seepage through the concrete, and could be placed in an expeditious manner to allow reservoir operation in the spring of 1993.

The existing redwood liner was removed from the inside of the concrete conduit, upstream and downstream of the gate chamber. A 66-inch diameter steel pipe slipliner (figure B-34) was placed inside the existing concrete conduit, upstream and downstream of the gate chamber. The steel pipe slipliner was placed in 20-foot sections. These sections were pulled into place, and the ends were butt strapped and welded. The voids between the steel pipe slipliner and the concrete conduit were then grouted through grout plugs in the steel pipe slipliner. This work (about 300 feet of pipe) was completed in about 1 month. A new transition structure was placed between the conduit and the terminal structure. Also, the exit channel was modified by placing grouted riprap for about 150 feet downstream of the terminal structure.

Lessons learned

Rapid installation of a steel pipe slipliner can be done to facilitate reservoir operations.

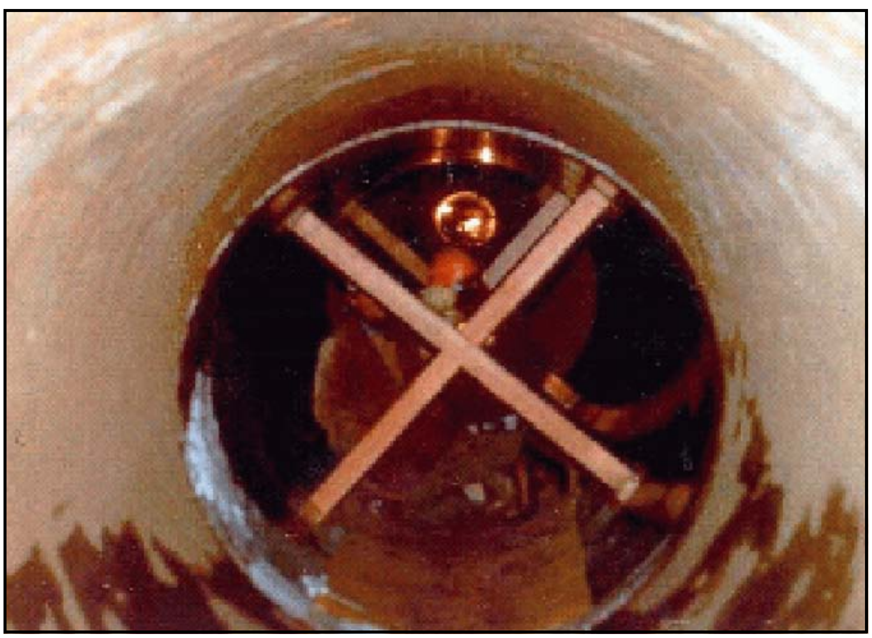

Figure B-34.—Installing the steel pipe slipliner within the existing outlet works conduit.

Reference

Bureau of Reclamation project files.

Project name: Crossgate Dam

Location: North Carolina

Summary: Problems encountered during the construction of a new siphon spillway

Crossgate Dam was constructed in 1955 in a rural area outside of Raleigh, North Carolina. The site was eventually incorporated into the Raleigh city limits, and development in the area downstream of the 25-foot high embankment dam resulted in a high hazard classification. Very little maintenance was performed on the embankment dam after it was constructed, and 40 years of neglect took its toll on the dam and the CMP spillway conduits.

In 1996, a developer agreed to upgrade the embankment dam in order to build new homes around the reservoir. A new 12-inch diameter siphon spillway was designed to be installed in a trench excavated through the embankment dam crest (see figure B-35). Leumas (1998, p. 710) discussed the design of the siphon:

> The design for the siphon was somewhat unique in that it served not only as a normal pool regulating device which discharged water from the surface of the reservoir, but also as a "bottom drain" structure which could drain the reservoir by discharging flow from near the bottom of the reservoir. Also, the siphon was designed to be self-priming, so that it would not need to be filled initially to start the siphon in order to drain the reservoir.

Unfortunately, a hurricane arrived during construction of the siphon and proved disastrous. The CMPs, which served as the only spillway, had already been abandoned with grout, and the only outflow was by means of a small, temporary siphon installed by the contractor. The flood overtopped the embankment dam, and

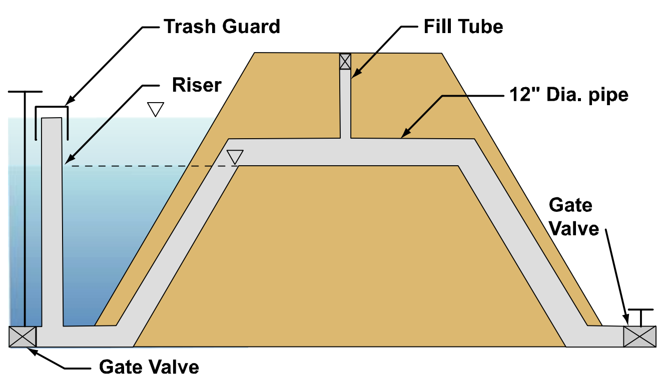

Figure B-35.—A well designed siphon was to be installed through the dam crest (from Leumas, 1998).

erosion severely damaged the partially constructed siphon. Temporary repairs were under way a few days later, but not yet completed, when another storm caused the embankment dam to again be overtopped. Erosion of the embankment dam, at the location of the siphon, by the second storm event caused the embankment dam to breach.

A decision was made to abandon the siphon spillway and instead construct a concrete riser and barrel structure. However, the siphon was later repaired and was successfully used to drain the reservoir (although several large pumps were also required).

Lessons learned (after Leumas, 1998)

- A siphon may be an attractive option for providing an existing embankment dam with a permanent means to drain a reservoir in lieu of excavation of the embankment to install a traditional bottom drain. However, many design elements should be considered in deciding whether to install a siphon, a low level outlet works, or sliplining the existing deteriorated conduit. Each method has its own advantages and disadvantages. The long term performance of the final selection and public safety considerations, rather than cost, should be the basis for the selected design.

- Diversion during construction is a vital element to be considered in the design process. An acceptable level of risk for diversion requirements, which balances economics for the project and an owner's liability, must not compromise the safety of the downstream public.

- Bad weather can occur during any project. Contingency planning should be made during the design process, which addresses what to do in the event that the capacity of diversion measures is exceeded. Such planning should have a readily available notification list of State dam safety program staff, emergency management officials, and other State and local representatives, who can assist in the event of an emergency.

Reference

Leumas, James, *To Siphon or Not To Siphon: That is the Question (Among Others)—A Repair History of the Crossgate Dam*, 1998 ASDSO Annual Conference, Las Vegas, 1998.

Project name: Dalewood Shores Dam

Location: Mississippi

Summary: Man-entry inspection of a deteriorated corrugated metal outlet works conduit

Dalewood Shores Dam was constructed in 1960 near Lauderdale, Mississippi without benefit of a qualified professional engineer. The 34-foot high embankment dam has a crest length of 3,800 feet and impounds a surface area of roughly 1,000 acres at normal pool elevation. Normal outflow occurs by way of a concrete chute spillway. Due to downstream development, the embankment dam is now classified as a high hazard structure.

During construction, a 60-inch CMP conduit with an upstream flap gate was installed through the embankment dam as a means to lower the reservoir level. The flap gate failed the first time it was operated, and the owner abandoned the outlet works by covering the intake and gate with soil and rock to prevent loss of the reservoir.

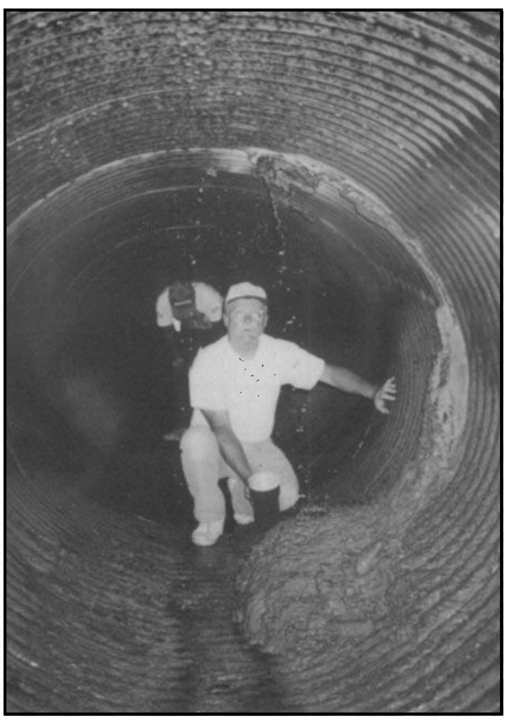

Figure B-36.—A man-entry inspection of this 60-inch CMP noted seepage and extensive loss of embankment soil at two locations.

Subsequent sloughing of the upstream slope damaged the upstream end of the CMP. An man-entry inspection of the pipe in 1993 noted that the pipe had ruptured, and seepage into the conduit was observed at two locations. At that time, the seepage flow was clear, and no soil loss was evident. However, by 1995 the seepage flow had increased, and soil deposits in the pipe indicated that internal erosion or backward erosion piping of embankment material was occurring, and failure of the embankment dam was a distinct possibility. Figure B-36 shows the man-entry inspection being performed. Due to the potential for loss of life if the embankment dam were to fail, the State dam safety agency directed the owner to hire a professional engineer to develop remedial plans to immediately repair the dam.

Although all previous man-entry inspections had noted that the seepage into the CMP was clear, the accumulation of soil in the conduit indicated that material was being transported into the conduit. Until remedial repairs could be made, the engineer made frequent man-entry inspections of the conduit in order to take emergency action, if dam failure was imminent. A decision was made to slipline the existing CMP with a 48-inch outside-diameter HDPE pipe and then grout the annulus between the pipes. The HDPE pipe joints were fusion welded in the field. A filter diaphragm was constructed downstream around the downstream end of the existing conduit to control seepage along the outside of the conduit. The work was completed in 1996 at a total cost of about $140,000.

Lessons learned

- Internal erosion or backward erosion piping of embankment material may occur in an embankment dam, even if seepage appears clear during infrequent inspections.

- CMP is a poor choice for use in outlet works conduits through an embankment dam.

- Sliplining a 60-inch CMP with 48-inch outside-diameter HDPE was cost effective.

References

Newhouse, Scott, and Dan McGill, *This Old Dam: Must It Have Outlet Works?*, 1997 ASDSO Annual Conference Proceedings, Pittsburgh, Pennsylvania, 1997.

Clevenger, Charles, *Dalewood Shores Dam Conduit Repair*, FEMA/ICODS Dam Safety Seminar No. 6, Piping Associated with Conduits through Embankment Dams, 1999.

Project name: Empire Dam

Location: Colorado

Summary: Reservoir evacuation using controlled breaching techniques

Empire Dam is an earthfill embankment dam located in San Isabel National Forest in central Colorado near Leadville and is privately owned. The embankment dam has a maximum height of approximately 10 feet with a crest length of 100 feet and impounds approximately 80 acre-feet of water above the natural ground surface. The embankment dam was constructed at approximately elevation 11,000 feet and is located in a remote area with limited access consisting of a very rough 4-wheel drive access road. The embankment dam was provided with a small earthcut spillway and a gated outlet works. The outlet works, however, was found to be inoperable. Records showed the embankment dam to be fairly old and was constructed to enlarge the storage of a natural high altitude lake.

On June 27, 1997, a backpacker reported that the embankment dam was being overtopped, and a considerable amount of erosion damage was occurring on the downstream slope of the dam. The backpacker was concerned that the dam may fail and indicated that a 2-hour hike is required to reach the site.

An investigation of the damsite revealed that the embankment dam had been overtopped, but was not being overtopped at the time of the inspection. On the day of the inspection, the reservoir was at a level of approximately 2.5 feet below the dam crest. A large scarp and eroded area was noted on the downstream slope and dam crest, where the embankment dam had been overtopped (figure B-37). The scarp was eroded approximately 3 feet into the dam crest. Approximately 0.5 ft^3/s of seepage was exiting from the bottom of the scarp area, and the discharge was clear. The flow into the reservoir was estimated to be approximately equal to the seepage through the erosional scarp. The embankment dam did not appear to be in imminent danger of failure, but the erosion of materials through the scarp and the reduced section of the embankment dam would eventually lead to a failure of the dam.

The downstream hazard consisted of one residence located approximately 2 miles downstream of the embankment dam at an elevation approximately 1,500 feet lower than the dam. A failure of the embankment dam would release approximately 80 acre-feet of water into a very narrow and steep natural channel for approximately 0.5 mile downstream of the dam, at which point the channel grade lessens and the

Figure B-37.—Erosion scarp on downstream slope of the embankment dam at the location of the overtopping of the dam crest.

channel widens significantly. The dam break flood would attenuate rapidly at this point; however, the flood would still pose a threat to the one home and anyone fishing and hiking in or along the stream channel. With the long Independence Day weekend approaching, it was anticipated that heavy recreational use in the area could be expected, and the embankment dam in its present condition posed a real threat to the safety of the general public.

Several futile attempts were made to operate the outlet works. Since the outlet works was inoperable, it was decided to perform a controlled breach of the embankment dam down to a safe level. After some discussion concerning the relative difficulty in getting heavy equipment to the damsite, and the significant environmental damage that would ensue, it was suggested that it would not take all that much effort to perform a controlled breach using manual labor. Prison inmates had been used previously to perform routine maintenance on embankment dams, such as tree and brush removal. A phone call late in the afternoon to the Buena Vista Correctional Facility found them willing to provide eight young men, equipped with picks and shovels the following morning.

After the inmates and a guard were transported to near the damsite via four-wheel drive vehicles, a plan was devised and explained to the inmates. Breach excavation would start at the downstream toe, and slope gently upward to within 1 foot of the upstream slope. The excavated breach would be lined with rock, which had to be hauled in by hand. The final cut would be made through the embankment dam, allowing the reservoir to be drained in a controlled manner. The bottom and sides of the breach were excavated and lined with a graded rock riprap (figure B-38).

The excavation was accomplished by a few well aimed pick swings, and the embankment dam was breached, and the water began flowing (figures B-39 and

Figure B-38.—Excavation of the controlled breach in the embankment dam.

Figure B-39.—Completion of the initialization of the breach in the embankment dam.

Figure B-40.—Deepening of the initial breach channel.

B-40). However, the control section, where the flow transitions from subcritical to supercritical flow, was located in the breach channel, where backcutting could occur and cause a large uncontrolled release. The reservoir basin upstream of the breach would have to be excavated in order to move the control section back into the reservoir. The inmates donned hip-waders and began the arduous task of excavating underwater, and after an hour's effort, the control section was safely located several feet back into the reservoir. Empire Dam was completely breached by July 2, and the reservoir drained in a very efficient and cost-effective manner.

Lessons learned

This event demonstrated that with a relatively small reservoir and small embankment dam height, the reservoir can be released in a controlled manner by performing a controlled breach of the embankment dam. The event also demonstrated that heavy equipment is not always needed to perform such a task. Care should be employed when attempting to release water from the reservoir by means of a controlled breach of the embankment dam.

Reference

Colorado Division of Water Resources, *Dam Safety Engineers Inspection Report Files—Incident Report for Empire Dam*, State Engineers Office, Dam Safety Branch, 1997.

Project name: Hernandez Dam

Location: California

Summary: Conduit constructed over a cutoff trench

Hernandez Dam is a 124-foot high embankment dam located in San Benito County, 50 miles southeast of the town of Hollister, on the San Benito River. The embankment dam was built in 1961. Hernandez Dam is a zoned earthfill with ballast berms on both slopes. Slopes are about 3.5H:1V on the upstream side and 3H:1V on the downstream side. The embankment dam has a large central clay core with outside slopes of 3/4 H:1V. The upstream shell is composed of a pit run sand and gravel zone. The downstream shell zone is clay to sandy clay from the spillway excavation. An 18-inch thick layer of river run sand and gravel filter was installed to two-thirds of the embankment dam height in the abutments, and a blanket drain was used in the center part of the fill. There is no chimney drain. An idealized cross section is shown in figure B-41.

The outlet conduit of Hernandez Dam is a 48-inch diameter steel pipe encased in reinforced concrete. The pipe is 3/16 inch thick, and the concrete is 12 inches thick. There are six concrete antiseep collars along the outlet conduit in the clay core section. The discharge is controlled on the upstream end of the outlet conduit by two hydraulically operated 30-inch butterfly valves. There is no downstream valve. The maximum discharge capacity is 400 ft^3/s with the reservoir water level at the spillway crest. The conduit crosses the cutoff trench in the core section of the embankment dam as shown on figure B-41. The antiseep collars are separated from the concrete encasement by 3/4-inch asphaltic expansion filler material. Steel reinforcement in the reinforced concrete encasement varies with the loading conditions. The longitudinal rebar is continuous across the construction joints. The joints have 1 5/8-inch shear keys and 6-inch dumbbell-type rubber waterstops.

The outlet conduit was installed on variable thicknesses of compressible material. The first 50 feet of the outlet conduit at the upstream toe of the embankment dam is chert bedrock. From this point to the upstream edge of the core trench, a layer of gravel backfill above chert bedrock serves as foundation for the outlet conduit. The gravel thickness varies from 1 foot at its upstream end to 3 feet at the edge of the core trench. In the core section, the outlet conduit bridges above 20 feet of compacted clay backfill, which is in the underlying cutoff trench. Downstream from the core, the bedrock dips steeply, and the outlet conduit is founded on streambed gravels in the upper zones, grading to gravelly clays in the lower levels above bedrock. The depth of the alluvium to bedrock was 24 feet prior to the embankment dam foundation excavation. The outlet conduit was constructed in the

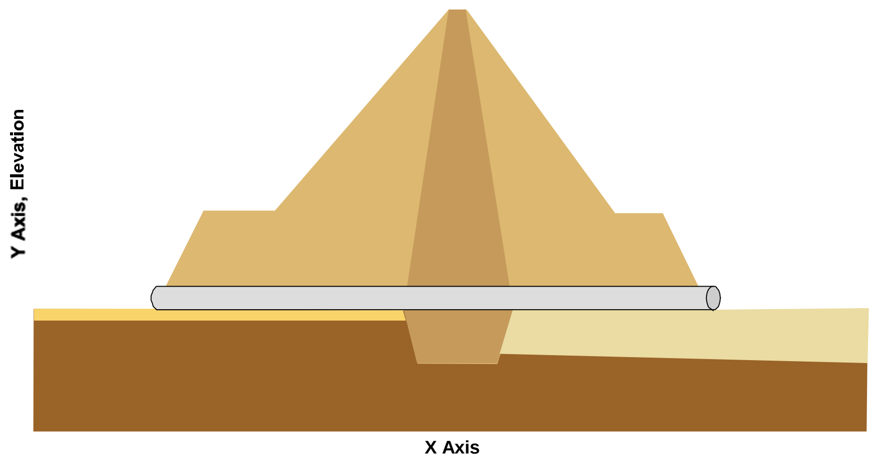

Figure B-41.—Idealized cross section of Hernandez Dam.

trench condition under the upstream shell and the first half of the downstream shell; it was constructed in the positive projecting condition in the sections under the core and second half of the downstream shell. The impervious core backfill in the key trench was brought up to ½ foot above grade; the earthfill was then excavated down to grade prior to the installation of the outlet conduit.

In February 1997, San Benito County Water District (District) staff attempted to operate the valves to reduce the water release and found that the right valve was inoperable in the open position. Later inspection revealed that the flexible ¾-inch hydraulic hose connector to the hydraulic cylinder was severed by the operation of the left butterfly valve. The reservoir emptied through the outlet conduit because the open gate was the only control for the conduit. Once the reservoir was emptied, staff from the California State Division of Safety of Dams, the District, and a consultant hired by the District inspected the valves and the outlet conduit. Observations of water flowing through the conduit, showed it to have a sag in its profile. This was confirmed in a subsequent survey of the outlet conduit.

The outlet conduit was found to have settled more at locations over the cutoff trench than it had at locations upstream and downstream. The sharp differential in the thickness of compressible materials at that point is depicted in figure B-41. The difference in settlement was attributed to the condition where the center portion of the outlet conduit was underlain by up to 24 feet of compacted fill in the cutoff trench while the adjacent sections under the embankment shells had rock or gravel foundations that were much less compressible. In addition to the problems with the sag in the outlet conduit, some welded joints had cracked open and others were in various stages of corrosion. The cracked joints had corroded through the ruptured coal tar coating and were rimmed with calcium deposits. A small amount of water was dripping at these joints.

The 1961 design of the outlet conduit recognized the need to camber the pipe, and the initial design included a camber of up to 1.3 feet, waterstops across construction joints, and planned access to the conduit. The steel pipe is accessible along its entire length for visual inspection and in case repair is needed. For unknown reasons, a decision was made during construction to reduce the camber from the planned 1.3 feet to what the as-built drawings show as a finally constructed camber of half, about 0.61 foot. This was the camber prior to any fill placement. By the end of construction in early 1962, 0.31 foot of settlement had been measured in the foundation under the pipe at the cutoff trench location, leaving a remaining camber of about 0.3 foot. Measurements have shown continued settlement of the outlet conduit after completion of construction. In June 1997, 35 years later, the pipe invert at station 6+00 was 0.86 foot below the as-built elevation. This reflects about a 0.3-foot sag in the conduit at the worst section.

Settlement along the outlet conduit is not uniform, and the largest settlement is concentrated in the section that spans the cutoff trench. Figure B-42 shows the measured settlement along the conduit. Settlement is least under the upstream sections of the outlet conduit that has a rock foundation, and highest in the portion of the conduit overlying the cutoff trench backfill, as would be expected. Because the outlet conduit rested on a gravelly clay foundation in the part of the embankment dam downstream of the cutoff trench, that part of the conduit had also settled to some extent, although less than that at the cutoff trench location.

The sharp differential settlement of the outlet conduit at the upstream end of the trench is attributed as the cause of the cracking of the joints in the steel pipe liner in this area. The concrete encasement has cracked as well. Exposed longitudinal steel across the cracks are subject to corrosion. At the downstream end of the trench, the settlement decreased gradually because the transition in compressible materials under the outlet conduit was more gradual. No cracks in the pipe joints were found in this area.

In October 1997, the District overhauled the valves, hydraulic operators, hydraulic piping, air vent piping, and hydraulic control equipment at the control house on the embankment dam crest. The valves are now operating satisfactorily. In the same time frame, the steel pipe was repaired by welding $^3/_{16}$-inch thick steel plate butt straps along the 5 most badly cracked joints. The repairs used $^3/_{16}$-inch fillet field welds in accordance with AWWA Standard C206-82. Included in the five repaired joints were the two joints cracked by the settlement of the line. Heavy corrosion of the original welds damaged the other three joints. All repaired joints were coated with two coats of coal tar emulsion. The repairs to the joints are expected to be lasting, excluding other factors, because the rate of settlement has decreased. The

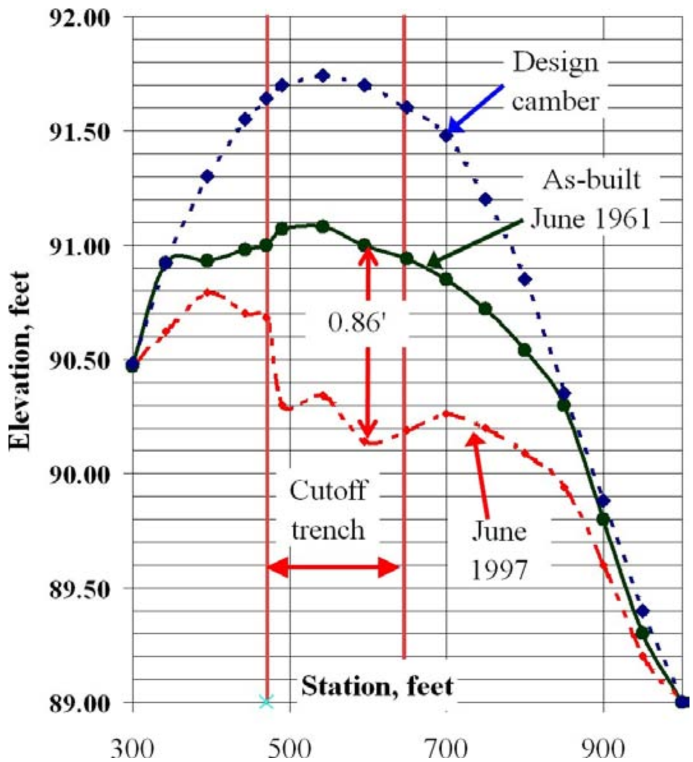

Figure B-42.—Settlement of the outlet conduit at Hernandez Dam.

outlet conduit remains accessible for inspections and maintenance work. Grouting of the cracked joints in the concrete encasement outside the steel pipe will be considered, if problems persist.

If additional problems develop, a recommendation to install a downstream valve on a new and smaller pipe placed inside the conduit, grouting the annulus, and preserving access to the inside of the new pipe have been proposed.

Lessons learned

This case history illustrated the need to install a outlet conduit on a uniform foundation. Sharp differences in thicknesses of compressible material beneath sections of a conduit that are near one another can lead to differential settlement that can damage rigid conduits. Cutoff trenches that are spanned by a outlet conduit should be designed to be compacted to a degree necessary to achieve similar strain characteristics in the cutoff trench backfill compared to the foundation materials on either side of the cutoff trench. Another approach to improve this situation is to flatten the side slopes of the cutoff trench to reduce differential strain. This case history also illustrates how important it is to have operating control on gates that are easily operated and maintained. Complicated mechanisms may be prone to malfunction and excessive maintenance.

References

Enzler, Y-Nhi and Anna Kolakowski, *Experiences with Outlets Constructed Across Cut-Off Trenches in Earthfill Dams*, USCOLD Nineteenth Annual Lecture Series, 1999.

Project name: Lake Darling Dam

Location: North Dakota

Summary: Grouting voids existing outside an outlet works conduit

Lake Darling Dam is a zoned embankment dam built in 1935 on the Upper Souris River for the purpose of providing water supply for fish and wildlife habitat and production. The embankment dam has a structural height of about 40 feet, a hydraulic height of about 32 feet. The embankment dam holds 112,000 acre-feet of water at the maximum normal water surface elevation. The catchment basin above the embankment dam is over 9,000 square miles. The original outlet works consisted of twin (side-by-side) 10-foot by 14-foot cast-in-place, reinforced concrete conduits placed on alluvial foundation soils in the central embankment area. Two antiseep collars were constructed around the exterior of the conduits at locations that roughly align with the upstream and downstream edges of the embankment dam crest. Figure B-43 shows the outlet works discharge at the downstream embankment toe. The discharge from the outlet filled a tailwater pool used for fish and wildlife purposes. A small control structure on the downstream end of the tailwater pool provided limited water surface elevation control capabilities. A cross section with a profile of the outlet works and design cross section of the embankment dam is shown on figure B-44. Roller gates controlling releases from the reservoir are located immediately downstream of the intake structure near the upstream toe of the

Figure B-43.— Lake Darling Dam outlet works discharge, circa 1990.

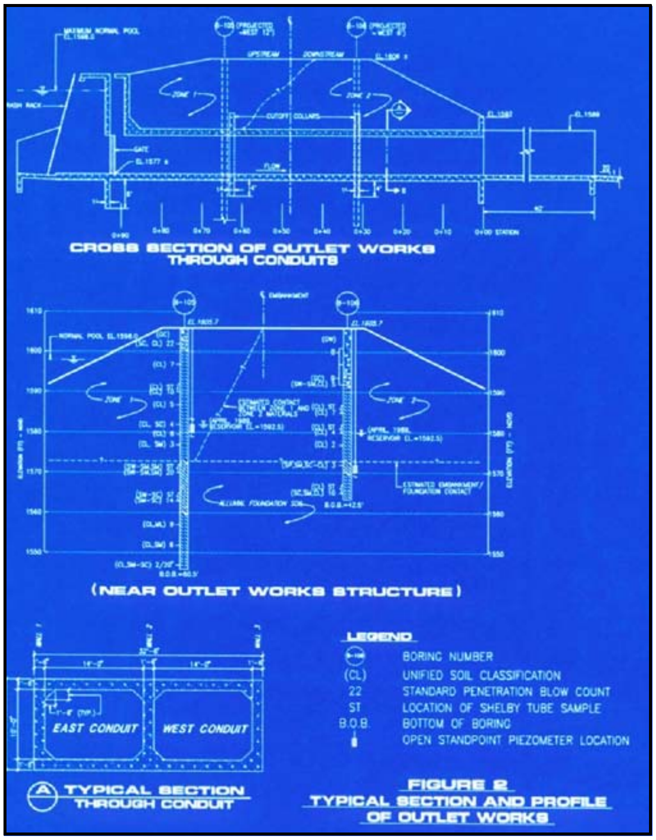

Figure B-44.—A cross section with a profile of the outlet works and design cross section of Lake Darling Dam.

embankment dam. The original design and construction did not include any control joints with waterstops in the conduits. The alluvial foundation materials beneath the conduits consist of up to 80 feet of sands, silts, and clays, some of which have moderate compressibility.

The U.S. Fish and Wildlife Service undertook a comprehensive safety assessment of Lake Darling Dam beginning in about 1988. The comprehensive safety assessment included borehole investigations and instrumentation of the embankment and foundation at representative locations, including the area around the existing outlet works conduit. During the investigations in the vicinity of the outlet conduit, a large amount of grout being used to backfill an instrumented boring at the downstream edge of the embankment dam crest was lost into an internal erosion or backward erosion feature. The grout loss occurred at the estimated contact between the embankment and foundation materials (see figure B-44) at the top of a bentonite seal installed at the top of the foundation piezometer influence zone.

Because of water demands and minimum flow needs downstream of the embankment dam, and other concerns, the original investigation program did not include dewatering and inspection of the outlet works conduits. Based on the results of the observed grout loss, however, it was determined that the outlet works conduits must be dewatered and inspected to further evaluate seepage and safety concerns, including the possible cause of the grout loss. Following dewatering, the condition of the conduits was thoroughly assessed in a two-phase program. During the first phase, a nonintrusive investigation was performed using geophysical impulse response to evaluate support conditions, impact echo to evaluate the condition of the concrete, and structural condition surveys (mapping of cracks and locations of seepage discharging from the cracks, inspecting the general condition of the concrete, and surveying the conduit invert along four upstream-to-downstream profiles). During the second phase, information from the nonintrusive investigation was used to design a program of intrusive investigations, including (1) drilling through the concrete to inspect the condition of the subgrade and backfill around the conduits including the presence of voids around the conduits, and (2) installation of vibrating wire piezometers to measure water pressures and estimate seepage gradients along the bottom and side walls of the conduits at different locations along the conduit profile.

Small voids were found beneath the conduit at a number of locations near observed cracks with seepage discharge. The lost grout was not found. Based on the additional information, final design and construction of corrective actions were initiated. The USACE was going to remove and replace the outlet works within 5 to 10 years as part of a comprehensive flood control project. The design of corrective actions took into consideration the limited planned life expectancy of the outlet. Corrective actions included in the rehabilitation design included both primary and secondary "lines of defense." Primary lines of defense included (1) installation of six relief wells into the foundation soils to collect and safely discharge foundation seepage—these wells were installed about 15 feet downstream of the roller gates and discharged up through the floor of the conduit through a flap valve; and (2) sealing of all cracks in the conduit floors and walls with an elastic filler that would adhere to the concrete and expand and contract during seasonal changes in the width of the crack. The secondary lines of defense included (3) grouting around the base and exterior of the outlet conduits to seal existing voids and increase the effectiveness of the relief well system (see figure B-45); and (4) construction of a new floor with a filtered underdrain system.

While the investigation program found voids around the conduits and unfiltered seepage water discharging through cracks in the conduit floor, instrumentation also showed locations within the embankment and foundation with relatively high water pressures and seepage gradients, particularly near the discharge end. Corrective actions would not be performed on the basin downstream of the discharge. To address concerns related to potential high discharge gradients in the downstream toe area, a key part of the remediation program would be continued monitoring of instruments installed during the investigation/assessment phase of the project. Target "safe" gradients were identified. Based on estimated gradients from instrument measurements, the water in the tailwater pool would be increased if necessary to reduce the exit gradients to "safe" levels.

Figure B-45.—Grouting operations within the conduit.

Lessons learned

- The location and extent of internal erosion or backward erosion piping features developing within a embankment dam or dam foundation is difficult to find even when direct evidence is available on the approximate location of such features. Conservative judgment is required in the assessment of piping and erosion problems and development of appropriate corrective actions. In many instances, "multiple lines of defense" will be required to adequately resolve the deficiency and provide appropriate risk reduction.
- Periodic inspection of an outlet works conduit is required to make a complete assessment of its current condition and safety. While inspections can sometimes be difficult and expensive to perform, they are the only way to observe certain conditions. Likewise, explorations and instrumentation around the exterior of a conduit may also be the only way to detect developing conditions that warrant further investigations or corrective actions and to monitor the effect of corrective actions, once completed, on the overall safety of the embankment dam and its foundation.

References

Ferguson, K.A., "Rehabilitation of the Lake Darling Dam Outlet Works, North Dakota," *Proceedings of the Fourteenth Annual USCOLD Lecture Series*, Phoenix, Arizona, June 1994.

Ferguson, K.A., J.R. Talbot, S.J. Poulos, and A. Arzua, *Advanced Seepage Short Course*, prepared for the Association of State Dam Safety Officials and presented at MIT (July 2003), University of Colorado at Boulder (October 2003), University of California at Davis (July 2004), and Georgia Tech (November 2004).

Project name: Lawn Lake Dam

Location: Colorado

Summary: Failure of an embankment dam by a combination of internal erosion and backward erosion piping caused by pressurized leakage from the outlet works conduit

Lawn Lake Dam was located on the Roaring Fork River in Rocky Mountain National Park approximately 10 miles upstream from Estes Park, Colorado. The embankment dam was constructed in 1903 by the Farmers Irrigation Ditch and Reservoir Company to impound additional water in an existing natural mountain lake for the purpose of irrigation storage. The original Lawn Lake was created by a natural glacial moraine and had a reservoir surface area of approximately 16.4 acres. An earthen dam was constructed with a maximum height of approximately 26 feet, which created a reservoir with a surface area of approximately 47.1 acres and a storage volume of 817 acre-feet. The reservoir was at an elevation of approximately 11,000 feet. The outlet works for the embankment dam consisted of a 36-inch diameter steel pipe with a gate valve to control releases from the reservoir situated at approximately the center of the embankment dam.

At approximately 5:30 in the morning of July 15, 1982, Lawn Lake Dam failed. Estimates indicate that the breach of the embankment dam released 674 acre-feet of water, and the resulting dambreak flood had an estimated peak discharge of 18,000 ft^3/s (USGS, 1982). The final breach dimensions through the embankment dam were surveyed to have a maximum depth of 28 feet, a top width of 97 feet, and a bottom width of 55 feet. The embankment dam materials exposed at the face of the breach showed the dam to have been constructed mainly of a silty and poorly graded sand with varying amounts of fine gravels and considerable amounts of organic materials. Figures B-46, B-47, and B-48 show the breached embankment dam. The embankment dam was not observed as it was failing.

The embankment dam failure occurred when a large leak developed in a pressurized outlet conduit. Flow from the leak quickly eroded the surrounding low plasticity embankment soils. The probable cause of the defect in the outlet conduit system that led to the embankment dam failure was deterioration of the lead caulking at the joint between the steel outlet conduit and the gate valve used to control releases through the conduit. The opening at this juncture in the conduit allowed pressurized water to escape the conduit, because the gate valve was closed, and the reservoir was full.

Figure B-46.—Aerial view of the breached Lawn Lake Dam and downstream floodplain.

Figure B-47.—Right side of the breached Lawn Lake Dam.

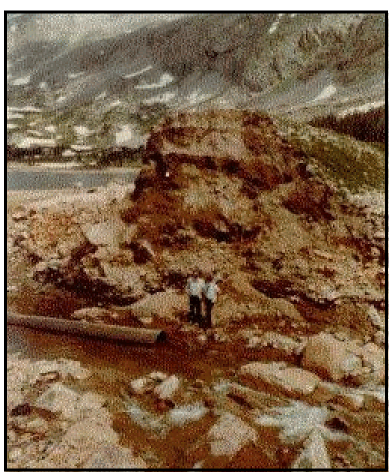

Figure B-48.—Left side of the breached Lawn Lake Dam.

The embankment dam was constructed of low plasticity soils susceptible to backward erosion piping, and the failure could have occurred from this mechanism. The high seepage pressures caused by the conduit leak would have been sufficient to initiate backward erosion piping, and the exit face for the seepage was not protected by an adequate filter. Another possibility is that the pressurized leakage hydraulically fractured the earthfill surrounding the conduit and the failure occurred from the mechanism of internal erosion. Probably, a combination of these mechanisms was responsible for the earthfill erosion. Regardless of the mechanism, the failure demonstrated the dangers of a pressurized conduit that develops a defect allowing full reservoir head to be imposed on soil that is not protected by filters.

The embankment dam failure resulted in the loss of three lives and approximately $35 million in property damage in the town of Estes Park. The resulting dam break flood also overtopped and failed the Cascade Lake Dam located 6.7 miles downstream of Lawn Lake Dam. Cascade Lake Dam was a 12-foot high concrete gravity dam with a reservoir with a storage volume of 12.1 acre-feet. The concrete dam was overtopped by an estimated 4.2 feet. Extensive erosional damage also occurred along the Roaring and Fall Rivers downstream of Lawn Lake Dam. The river channels were widened by several tens of feet in some locations, and scour depths varied from 5 to 50 feet. A large alluvial fan was also created at the confluence of the Roaring and Fall Rivers. The alluvial fan covered an area of approximately 42 acres and had an estimated maximum thickness of 44 feet. The fan also dammed the Fall River, creating a reservoir with a surface area of 17 acres. The largest boulder believed to have been moved by the floodwaters was estimated to have a weight of 450 tons.

Lesson learned

If possible, construction of fully pressurized conduits beneath embankment dams should be avoided. When pressurized conduits are constructed within embankment dams, the joints in the conduit should be properly designed to assure watertightness under all loading conditions.

Reference

U.S. Geological Survey, *Hydrology, Geomorphology, and Dam-Break Modeling of the July 15, 1982, Lawn Lake Dam and Cascade Lake Dam Failures, Larimer County, Colorado*, Open-File Report 84-612, 1982.

Project name: Little Chippewa Creek Dam

Location: Ohio

Summary: Separation of spillway conduit joints due to foundation movement

The joints of the 48-inch reinforced concrete spillway pipe separated when foundation movement occurred during final stages of embankment construction. The failed spillway was removed, a new structure was constructed in a different location, and stabilizing berms were added to the embankment design.

Little Chippewa Creek Dam, known officially as "Chippewa Conservancy District Structure VIIc" is a high-hazard dam located about 3 miles northwest of the city of Orrville, Ohio. The 27-foot high embankment dam is a single-purpose, dry flood control structure designed in 1971 by the Soil Conservation Service under the authority of Public Law PL-566.

The embankment dam was designed with an upstream slope of 3H:1V and a downstream slope of 2.5H:1V. The site lies on the glaciated, moderately rolling Allegheny Plateau. The site was glaciated during a series of advances and retreats during the Wisconsin Stage of the Pleistocene Epoch. The foundation soils consist of glacial outwash deposits of layered sand, silt and clay.

Construction of the embankment dam started in July 1972. During spillway construction, the inspector noted the presence of soft, gray silt at the bottom of the excavation for the pipe. The foundation was overexcavated by 1 foot and backfilled with AASHTO No. 46 coarse aggregate. The pipe was installed on top of the aggregate and bedded in concrete. After the pipe joints were covered with 12-inch wide sheet metal shields, embankment material was backfilled around the pipe. Construction was suspended in late 1972 due to winter weather.

Construction of the embankment dam resumed in July 1973. Earthfill placement proceeded rapidly without incident until mid-August 1973. As the embankment dam was nearing completion, the downstream portion failed suddenly, severely damaging the spillway conduit. The downstream end of the conduit moved about 2.4 feet in the downstream direction. A 1.5-foot high bulge was observed in the stream bottom, and some cracks were observed in the slope below the dam. However, no settlement, cracking, or other distress of the embankment dam itself were observed.

The bottom of the pipe under the maximum earthfill height settled by about 1.5 feet, and the joints of one section of the pipe separated by more than 1 foot (figure B-49). The engineers investigating of the failure reported that the foundation soils under the

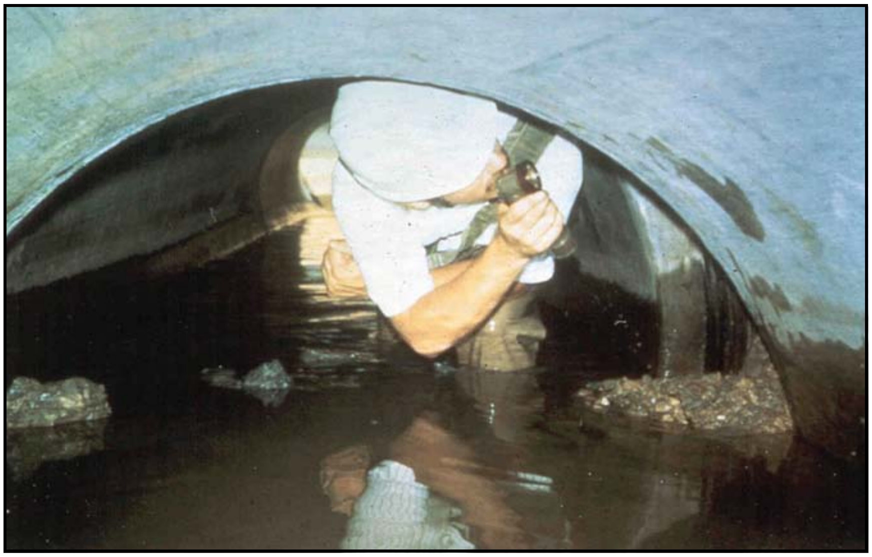

Figure B-49.—The pipe joints separated when foundation movement occurred during construction of the embankment dam.

pipe, visible through the open joints, were soft, wet silts that resembled "stiff grease." The engineer reported that no sands or gravels were encountered, and that the soil could be probed easily with a hand ruler to depths of 6 feet below the pipe. The report also indicated that the sheet metal shields on the outside of the joints appeared to support the embankment soils and minimized migration of soil into the open joints.

Because the nearly completed structure was capable of impounding water, but was unsafe, temporary modifications were required to prevent its failure during a storm. A 30-foot wide bypass channel was immediately excavated through the emergency spillway. Also, a 30-inch diameter CMP was temporarily installed inside of the failed concrete pipe to carry stream flow and to prevent flow of water over the open joints.

The permanent repairs included removal and relocation of the failed principal spillway conduit and appurtenances; reconstruction of the embankment dam in the vicinity of the failed spillway; installation of a new spillway at another location; addition of 70-foot-wide stabilizing berms to the upstream and downstream sides of the dam; addition of chimney drain across the reconstructed area of the embankment dam and around the new spillway pipe; and addition of a cut-off trench across the emergency spillway control section to seal the emergency breach channel.

Reconstruction of the embankment dam was successfully completed in late 1974.

Lessons learned

Embankment dams constructed on soft clay foundations may experience excessive settlement and spreading, and conduits associated with them may be damaged. Design of embankment dams on soft foundations must consider the undrained strength likely to be operative during construction and incorporate design measures, such as wide berms and special conduit joint details to address these problems.

Reference

Ohio DNR and *Phase 1 Inspection Report*, U.S. Army Corps of Engineers, 1981, which includes the 1973 "Report of Investigation of Structural Deficiency" as an appendix.

Project name: Loveton Farms Dam

Location: Maryland

Summary: Failure of an embankment dam by internal erosion along the spillway conduit

Loveton Farms Dam is a 23-foot high earth embankment dam in Baltimore County, Maryland. The dam was constructed in 1985 as a stormwater management structure to attenuate increased runoff due to commercial and residential development of the watershed. The embankment dam is a "dry structure" which does not normally impound any water. The spillway consists of large diameter (78-inch diameter) CMP constructed through the embankment dam. A vertical section of CMP about 16 feet high (riser) was constructed at the upstream end of the spillway pipe. Low flows pass through a small (1-foot) opening at the base of the riser. Flows in excess of the 100-year storm bypass the embankment dam via an emergency spillway channel excavated in the left abutment.

The embankment dam is essentially a homogeneous embankment constructed of local residual soils. These soils are micaceous silty fine sands and sandy silts weathered from the parent rock (Piedmont Geologic Province). They are classified as SM and ML under the Unified Soil Classification System. Typical liquid limits are about 30 percent, with a plasticity index of about 7.

The embankment dam failed less than a year after it was completed, when a relatively small storm filled the pool to the top of the riser. Failure was attributed to internal erosion of embankment fill along the outside of the pipe (figures B-50 and 51). The original spillway pipe was likely placed in a vertically sided trench excavated through the nearly completed embankment dam. This construction technique is not recommended, as it makes compaction of the soil under the sides of pipe very difficult.

Poorly compacted fill in this area results in poor support of the pipe, which causes excessive deformation of the pipe and may cause the joints to separate. In addition, the sides of the trench may tend to support the fill, allowing it to "bridge" across the excavation, preventing the fill from consolidating under its own weight. This can create areas of low soil density under the pipe where seepage can occur. In addition, differential settlement and hydraulic fracture can result.

The embankment dam was redesigned to include seepage controls. The structure was rebuilt in 1990 using essentially the same embankment dam and spillway

Figure B-50.—Loveton Farms Dam failure as viewed from upstream. Note that the walls of the failure area are nearly vertical. The construction records indicate that a large portion of the embankment dam was placed prior to installation of the CMP.

Figure B-51.—Loveton Farms Dam after failure as viewed from the downstream end of the 78-in diameter CMP spillway. Note that one of the antiseep collars, which were about 14 feet square, is visible in the breach.

configuration as the original (although a second riser was added at the upstream end of the spillway). However, a chimney filter was installed which ran axially along the embankment dam. A sand filter diaphragm was also constructed around the downstream portion of the pipe to control seepage and prevent internal erosion of embankment material along the sides of the conduit. The side slopes of the excavation through the remaining embankment dam were designed and constructed as 3H:1V to minimize problems with bridging of the fill. Compaction of the embankment material was carefully monitored and tested during the repairs. Powdered bentonite was added to the backfill under the pipe, because the bentonite would presumably swell to eliminate any voids.

In the years that followed the repair of the embankment dam, Maryland began to experience problems with other dams constructed with large diameter CMP spillways. In particular, it was noted that many of the joints between sections of the pipe were not watertight. This deficiency is primarily the result of deflection of the pipe, (the design standard allowed 5 percent of the pipe diameter), but poor construction techniques and manufacturing tolerances also contributed to the problem (figure B-52). Embankment dam owners were advised to carefully monitor their spillway conduits.

Accordingly, the owner of Loveton Farms Dam (a local government agency) scheduled inspections of the structure twice per year to document the condition of

Figure B-52.—The failed embankment dam was repaired with a new 78-in diameter CMP spillway. However, soon after reconstruction, this failed joint was discovered at the upstream end of the pipe near the riser. A large void was also observed in the adjacent embankment fill. Note the o-ring gasket has been displaced from the joint.

pipe. During an inspection in 1994, large voids (3 feet diameter and 20 feet long) were noted in the embankment around the upstream end of the pipe, and the first joint downstream of the riser had suddenly separated by about 0.1 m.

The embankment dam was determined to be unsafe, and the riser portion of the spillway was removed to minimize impounding of water until a more detailed inspection could be conducted. A more thorough investigation of the embankment dam utilizing seismic tomography revealed that nearly all of the embankment fill around the conduit was of low density. Since the soils are frost susceptible (silty sands and sandy silts of low plasticity), it is quite possible that freezing damaged the soils adjacent to the conduit. The melting of ice lenses that may have formed in the backfill would leave voids through which internal erosion could occur. Also, the formation of ice lenses can create forces large enough to deform the thin steel pipe, causing the joints between pipe sections to open.

The embankment dam was deemed to be unsafe and is scheduled to be removed.

Lessons learned

Use of large diameter CMP conduits in embankment dams should be avoided. Vertical trenches transverse to the embankment dam are never permissible, unless they are in rock and backfilled with concrete. Sloping the sides of excavations to no steeper than 2H:1V is always recommended.

Reference

Schaub, W., *Investigation of the Loveton Farms StormWater Management Pond*, prepared for Baltimore County Bureau of Engineering and Construction, June 1996.

Project name: McDonald Dam

Location: Montana

Summary: Steel lining of an existing outlet works conduit

McDonald Dam is located near Polson, Montana on the Flathead Indian Reservation. A section of the 6-foot diameter elliptical conduit (figure B-53) was removed and replaced. Installation of both 52- and 16-inch diameter bypass pipes was completed in the conduit replacement section, and a hydrostatic test of the 16-inch diameter bypass pipe was performed. The hydrostatic testing was performed in increments and eventually tested the entire lengths of the 52- and 16-inch pipes, as well as existing pipe installed in an earlier contract. Due to the existing 2-inch diameter air vent at the intake structure (tower), a portion of the pipe was pressure tested at 20 lb/in^2 instead of the 30 lb/in^2 required by the specifications.

An independent testing company performed dye-penetrant tests on the welds of all installed 52-inch diameter pipe sections (from sta. 5+79 to sta. 6+60.12).

The annular space around the pipes was grouted. Prior to beginning grouting, the 52-inch diameter pipe was anchored to the existing conduit to prevent the pipe from floating during the grouting operation. Anchorage was accomplished with ¾- by 7-inch diameter mechanical anchors placed through the steel liner and secured to the

Figure B-53.—Existing 6-foot diameter elliptical conduit.

invert of the existing conduit. A total of 11 anchors were used from stations 6+60 to 8+10. The annular space was grouted in two stages. The first stage was to a level just below the lower grout connections. The second stage—the remaining annular space—was grouted 24 hours later. Grouting pressure was limited to 5 lb/in^2. Air vents had been previously installed through the steel bulkheads at stations 6+02, 6+60, and 8+10. The vents at stations 6+02 and 6+60 were extended approximately 5 feet above the top of the conduit to allow placement of embankment to proceed prior to completion of the grouting.

Second stage grouting of the conduit from stations 5+79 to 6+02 began at the upstream grout connection, working downstream until grout had risen to the top of the air vent at station 6+02. The next day, it was observed that the grout had receded completely from the air vent standpipe. An additional 5 ft^3 of grout was pumped into the air vent. For several days prior to beginning the grouting, it was observed that water was flowing from the bottom edge of the bulkhead at station 8+10. The water was assumed to be entering the existing conduit through a joint in the concrete at approximately station 7+00. Grouting of the first stage proceeded from stations 6+60 to 8+10, in an effort to push the water ahead of the grout and out the bulkhead. The flow of water from the bulkhead stopped after the initial grout set, but then resumed several hours later, with just a trickle flowing from the bottom edge of the bulkhead. Problems were encountered with grouting of the second stage due to leakage of grout from the contact between the bulkhead and the pipe. The contractor attempted to use various fillers, but resorted to placement of a filet weld between the pipe and the bulkhead. Grouting of the second stage proceeded from stations 8+10 to 6+60, in an effort to minimize entrapment of air and dilution of grout in the downstream portion of the conduit. Grouting continued until grout was observed in the air vents at stations 6+60 and 8+10. The following day, the grout had receded in the vent at station 8+10. Additional grout was added at the vent, requiring less than 2 gallons to fill the vent. Seepage of water from the bulkhead did not occur after grouting was completed.

The plugs installed in the grout connections were ground down flush with the interior pipe surface and seal welded, per specification requirements. Several days later, it was noticed that water was seeping from two of the connection points. These connections were in the vicinity of the concrete joint in the existing conduit approximately at station 7+00. Presumably, a small void had developed between the pipe and the surrounding grout, allowing a path for the water to seep into the existing conduit at the joint. The leaking grout connections were welded again, which effectively stopped the seepage.

The 2-inch diameter air vents extending through the bulkheads at stations 6+03 and 6+60 (used for grouting of the annular space) were removed. The 2-inch diameter pipe was removed at the threaded couplers (10 inches below the top of the conduit) and the void repaired using sand and cement dry-pack.

The uncoated interior surfaces of the newly installed 52-inch diameter pipe and damaged paint coating on the existing pipe were sand blasted and painted. These areas consisted of the weld joints, grout connections, and miscellaneous scrapes and gouges throughout the length of the conduit. Paint coatings were applied with two applications of DeVoe High Build Epoxy. Mil thickness readings taken indicated the first coat was 7 mils and the second coat, applied the next day, checked out at only 14 mils. The paint subcontractor subsequently returned and applied a third coat, to build the coating thickness to the 16 mils required by the specifications. Final applications of DeVoe High Build Epoxy were made on weld areas and damaged paint surfaces of the 54-inch diameter pipe, which completed all work associated with the pipe.

Lessons learned

- Grouting operations must be well planned and closely monitored.
- Grouting operations may require field adjustment to accommodate any seepage encountered.

Reference

Bureau of Reclamation, *Technical Report of Construction for McDonald Dam Modification (Draft)*, Flathead Construction Office, Ronan, Montana, October 2000.

Project name: Medford Quarry Wash Water Lake Dam

Location: Maryland

Summary: Failure of an embankment dam due to internal erosion along the conduit

Medford Quarry Wash Water Lake Dam is a 26-foot high, significant hazard embankment dam. Downstream hazards include roadways, railroad tracks, and a residence. The embankment dam is essentially an offstream basin, and nearly all inflow is pumped into the basin from a nearby wash plant.

The embankment dam was constructed in 1988 as a "temporary sediment basin." Although an engineer prepared plans for the structure, no engineering supervision was provided during construction. Local materials (decomposed shale and erodible silts) were used to construct the embankment dam. A CMP spillway with conventional antiseep collars was constructed in a trench excavated into the foundation and backfilled with low plasticity silts and decomposed rock fragments from the excavation (figure B-54).

The structure was placed into use the following year, and it failed upon first filling. When the pool level was only about 2 to 3 feet deep, flow along the outside of the pipe resulted in loss of the adjacent soils by internal erosion (figure B-55).

Figure B-54.—The embankment dam was constructed with antiseep collars.

Figure B-55.—When the pool level of this embankment dam was only about 2 to 3 feet deep, flow along the outside of the CMP resulted in loss of the adjacent soils.

The spillway structure was removed and replaced and has performed satisfactorily.

Lessons learned

This significant hazard embankment dam was improperly designed and constructed as a "temporary sediment basin" (which has less rigid construction requirements) and did not have proper inspection during construction.

Reference

Maryland Dam Safety Division, Dam file No. 318.

Project name: Olufson Dam

Location: Washington

Summary: Outlet works conduit failure

Olufson Dam was a privately owned embankment dam located in Pierce County, near Gig Harbor, Washington that experienced an outlet works conduit failure. The embankment dam was 18 feet high, with a storage capacity of 15 acre-feet and 21 acre-feet at the top of the dam. The principal spillway consisted of a 2-foot square, concrete, drop inlet conduit. An open channel in the abutment served as an emergency spillway. The embankment dam was constructed in the 1960s without the benefit of engineering plans. The owner did all the work himself, including placing earthfill and mixing his own concrete onsite. Conditions exposed by the failure suggest that the elements of the construction that required skill were substandard. In particular, the concrete work suffered from inadequate cement content, poor overall mix gradation, and improper reinforcing. Thick steel cable was substituted, in part, for conventional reinforcing steel. Likewise, these cables were improperly positioned in the conduit section thus minimizing its enhancement of the tensile load capacity of the conduit. To limit concrete volumes, it appeared the owner had embedded bricks, rocks and concrete rubble into the walls as a filler during concrete pours. This practice, termed cyclopean concrete construction, has been successfully used in large gravity structures, but was inappropriate for thin walled, concrete box conduits.

On December 11, 1996, a sinkhole 20 feet in diameter and 17 feet deep opened up in the crest of the embankment dam (figure B-56). At the time the sinkhole developed, the property on which the embankment dam sat was uninhabited due to the recent death of the property owner. The sinkhole was discovered by neighbors walking the streambed to investigate the cause of muddy streamflows. This was fortuitous in that the sinkhole was discovered before it lead to an embankment failure. The sinkhole appeared to have resulted from a collapse in the top section of the cast-in-place box culvert that served as the principal reservoir outlet. The failed segment of the conduit allowed overlying masses of embankment soil, over time, to repeatedly drop into the conduit, where flows then flushed the soil downstream. This sequence of events was supported by the record of stream flows in a downstream gauging station. The gauge record shows normal flows interrupted by a series of near zero creek flows immediately followed by short, abnormally high channel discharges. The zero flows are interpreted as incidences of soil masses falling into and plugging the conduit. The following anomalous high flows represent a blowing out of the plug and a release of backwater in the conduit and inlet tower upstream of the plug.

Figure B-56.—Sinkhole in the dam crest the night of December 11, 1996.

As an immediate response to the threat of an embankment dam breach, county maintenance staff filled the sinkhole with some 200 yards of angular cobbles and boulders. The State dam safety staff saw no viable alternative to the county's scheme to address the immediate crisis. Finer grained soils would likely have been sluiced through the top of the collapsed box conduit. This could have worsened the situation by plugging what limited outlet capacity remained after sediment had largely blocked the conduit. Nonetheless, it was obvious that the rockfill was but an interim measure, and immediate follow-up action was necessary to lower the reservoir and permanently resolve the public safety threat. Three days of pumping were necessary to lower the reservoir to allow excavating a trapezoidally shaped breach of the embankment dam (figure 57). The floor of the breach was armored with a geotextile fabric and capped with much of the rock originally dumped into the void the night of the failure. To improve fish passage, an attempt was made to include a number of pools along the breach channel at the direction of the Washington State Department of Fish and Wildlife. The Washington State Water Quality staff assisted in blanketing the disturbed sections of the embankment dam with hay to minimize further sediments entering the water course.

Damage downstream was limited to the streambed. Primarily, it occurred in the form of stream habitat degradation from sediment deposition. Many of the salmon eggs in this fish-producing stream were smothered under sediments for several hundreds of yards downstream of the embankment dam. As bad as it was, the emergency action prevented a likely failure of the embankment dam. Thus, the

Figure B-57.—View of upstream dam crest nearing completion of breach.

possible threat of loss of life was averted along with extensive damage to property abutting the streambed.

Lessons learned

An examination of the failed conduit through the embankment dam revealed it to be of poor quality with minimal reinforcing. What reinforcing was provided, consisted of misplaced, steel cable rather than conventional deformed bars. Given the construction of the conduit, it is remarkable that it functioned for over 30 years.

This failure reinforces the concern that conduits have a definite service life, measured in decades. At the end of that service life, they require retrofitting for their continued satisfactory functioning. A failure to do so, risks a failure of the embankment dam. Proper care taken in the design and construction can materially increase the conduit service life. Conversely, poor workmanship may reduce it.

Periodic inspection of conduits is required to confirm that they are structurally sound, and to provide timely notice of a developing problem with age.

Reference

Washington State Department of Ecology, Project file.

Project name: Pablo Dam

Location: Montana

Summary: Removal and replacement of an existing outlet works

Pablo Dam is located near Polson, Montana on the Flathead Indian Reservation. The embankment dam is an earthfill structure consisting of a main dam and dikes, which flank both sides of the dam, south and north. The crest elevation of the main dam is at 3220, and the dikes are at 3217. The main dam has a structural height of 43 feet, a crest length of 10,550 feet, a crest width of 20 feet, a 3:1 upstream slope, and a 2:1 downstream slope. The north dike has a crest length of 5850 feet, and the south dike has a crest length of 10,250 feet. The crest width of both dikes is 12 feet.

Pablo Dam was constructed in three phases over 24 years. In 1911, the embankment was constructed to elevation 3202. The second construction in 1918 raised the embankment dam to elevation 3209, and the final construction in 1934 raised the dam to the present elevation 3220. Pablo Dam is an offstream structure that is fed by the Pablo Feeder Canal. The purpose of the embankment dam is to impound water for irrigation. The reservoir has a capacity of 28,400 acre-feet at elevation 3211.0.

The original outlet works was situated at the maximum section of the dam and consisted of a 42-foot high concrete intake structure with two 3- by 5-foot slide gates. The original outlet works consisted of three box shaped conduits. The middle and south conduits were 172 feet long and 4.5 feet wide by 5 feet high. The north conduit was about 136 feet long and 3.0 feet square. This north conduit was abandoned prior to the third phase of original construction.

Differential settlement between the intake tower and the outlet works conduits caused some offset in "sliding joints." This settlement was expected, as "sliding joints" (no reinforcement crossing the joint) were included in the original design. However, continued settlement of the intake structure and the first 13 feet of the conduits required grouting of the foundation shortly after construction. No further settlement has been detected in the last 50 plus years. The first sliding joint is displaced vertically about 2 inches and sprays water at high reservoir head. Mortar filling in all sliding joints was disbonded, cracked, and deteriorating. Tensile cracks were also discovered along the length of the conduit. Water was commonly leaking from both the cracks and the sliding joints, and there are signs of possible internal erosion of embankment material occurring in a few areas. Spalling concrete had been discovered in the walls of the conduits. The concrete in the center wall at the downstream end of the conduits was deteriorated, resulting in exposed aggregate and rebar.

Dam safety modifications were begun in 1993, consisting of injection of polyurethane grout into cracks and conduit joints. A two-man crew from McCabe Brothers Drilling of Idaho Falls, Idaho, mobilized to the job site. They installed ventilation ductwork into the two outlet works conduits and began drilling injection holes in the south conduit. Existing cracks (mostly at construction joints) upstream of station 1+27 were injected with polyurethane resin grout to stop leakage through the cracks. This was done prior to repair of spalled concrete in the conduits. The subcontractor used a ratio of polyurethane to water of 1.3:1, which effectively stopped 90 percent of the seepage. However, after completing injection of cracks in the south conduit, seepage began to migrate downstream and appear in cracks that were previously dry.

During drilling of the injection holes, two voids were discovered, one in the crown of each conduit at station 0+13. The voids were approximately 12 inches deep and 24 inches wide and seemed to be connected to each other. Old construction drawings showed this as the location where concrete counterfort walls, which support the intake tower, meet the conduits. No voids were found behind any of the other cracks. The voids at station 0+13 were injected with polyurethane. As injection of the south conduit was completed, some migration of polyurethane was noted through the crown and divider wall of the middle conduit.

In mid-November, McCabe Brothers Drilling completed injecting polyurethane resin into cracks in the outlet works conduits. They injected a total of 305 gallons into the two conduits (the specified quantity was 50 gal). As the injection operation progressed from upstream to downstream, cracks that had been previously dry near the canal outlet began to seep water. Therefore, these cracks were injected also. Because the seepage appeared to be following the exterior of the conduits and exiting farther downstream, the seepage continued to be unfiltered and may increase the internal pressures in the embankment. A decision was made to install weep drains in the conduit and to construct a filter collar about the exterior of the walls. A modification to the contract was issued to provide for this additional work.

After the polyurethane injection was completed, the conduits were unwatered and inspected. Repair areas were marked, and the contractor began chipping out and preparing the surfaces of the repair areas for epoxy-bonded concrete. Approximately 30 small repairs and one large repair at the conduit outlet (splitter wall) were done to complete the conduit repairs option of the work. Smaller and shallow areas were repaired using an approved two-part epoxy material. Larger areas were repaired with epoxy-bonded concrete.

During an inspection of the interior of the conduits in April 2001, it was discovered that material had been deposited inside the middle conduit near an opening in a construction joint. This was occurring through a hole in the floor of the middle conduit at a construction joint near station 1+30. Approximately 1 ft^3 of silt and fine sand were deposited on the floor. However, this deposit was observed during the winter when no irrigation releases are made. More deposition may have occurred during irrigation season that was washed downstream and not observed. Consequently, the total volume of material could have been much greater than the 1 ft^3 observed in 2001. Reclamation theorized that plugging this opening could result in redirecting the erosion through a different hole or crack in the conduit. Also, redirecting the erosion might cause a more dangerous path to develop along the foundation contact of the conduits, and a piping exit might develop downstream of the embankment dam. If the exit point were located within the outlet channel, early detection would be very difficult.

Another area of concern was the condition of the north conduit that was reportedly plugged at each end prior to the final raise of Pablo Dam in 1932, but was never confirmed. Therefore, it could be possible that a nearly full reservoir head could exist at the end of the north conduit, which was less than 100 feet from the downstream toe of the dam. After much discussion between all involved parties, it was decided to completely remove and replace the original outlet works.

As an interim measure, a temporary patch was installed over the opening to prevent additional material from being eroded into the conduit while allowing for relief of water pressures. The patch consisted of filter fabric under a metal screen. During March 2002, the geotextile portion of the patch ruptured and approximately 0.5 ft^3 of silt and fine sand were deposited into the conduit. The patch was repaired soon after the rupture was discovered. Reservoir level restrictions were implemented in April 2003 and were to be kept in place until the removal and replacement modifications could be completed.

The construction of a new outlet works began in November 2004 and was completed in the by the spring of 2005. The major aspects of the work included:

- Construction of a cofferdam to maintain an area free of water during construction.

- Clearing, grubbing, and stripping prior to excavation.

- Removing existing embankment dam slope protection.

- Excavating embankment materials to accommodate construction of the new outlet works (Slopes transverse to the dam centerline were excavated at 4H:1V).

Figure B-58.—Pablo Dam nearing completion.

- Removal of the existing reinforced concrete intake structure, conduits, retaining walls, and apron.

- Constructing a lean concrete mudslab, on which to found the new outlet works.

- Constructing reinforced cast-in-place intake structure, conduit, retaining walls, and apron. The new conduit was double barreled with each barrel having a 6-foot 3-inch inside diameter. The exterior surface of the conduit was sloped at 1H:10V below springline and was curved above springline to provide a good surface to compact earthfill against. Each conduit joint was a treated as control joint with longitudinal reinforcement extending across the joint and 6-inch PVC waterstop.

- Installing two emergency guard gates and two regulating gates within the upstream intake structure.

- Constructing a chimney filter and drain system. The filter extended downstream and encased the outlet works conduit. Filter materials encasing the conduit consisted of sand processed to a specified gradation from an approved offsite source.

- Placing and compacting zoned earthfill in the embankment dam closure section.

- Replacing the embankment dam slope protection.

Figure 58 shows Pablo Dam as it was nearing completion.

Lessons learned

Sometimes repairs alone are not fully robust enough to address all the unknown erosional mechanisms existing within an embankment dam. Due to continued dam safety concerns, more extensive measures may be warranted.

Reference

Bureau of Reclamation, *Technical Construction Report-Pablo Dam Modification Contract No. CSKT/SOD 06*, August 1996.

Project name: Pasture Canyon Dam

Location: Arizona

Summary: Closed circuit television inspection of an outlet works conduit

Pasture Canyon Dam is located on the Hopi Indian Reservation in Arizona. Pasture Canyon Dam is a homogenous embankment dam with a height of 17 feet. The embankment dam crest is at elevation 4890.0 feet, 20 feet wide, and 632 feet in length. The embankment dam was apparently founded on pervious, sandy alluvium. No information was available as to its construction. The embankment dam was completed in 1920s or 1930s and modified in 1975. The 1975 modification included a 3-foot crest raise.

Appurtenant structures at the site include an uncontrolled earthen spillway and an outlet works. The outlet works is located within the embankment dam approximately 200 feet from the left abutment. The outlet works consists of a concrete intake structure, approximately 55 feet of 12-inch by 12-inch masonry conduit connected to 35 feet of 14-inch diameter CMP connected to 20 feet of 14-inch diameter concrete pipe. The intake structure contains a hand-operated slide gate. The discharge capacity of the outlet works has been estimated to be 9 ft^3/s.

The Bureau of Reclamation's Technical Service Center performed a CCTV inspection of the outlet works conduit at Pasture Canyon Dam in April 2004. The conduit was accessed for inspection, via an existing manhole located at the downstream end of the outlet works. The camera-crawler was inserted into the 14-inch diameter concrete pipe and was advanced upstream approximately 18 feet, where the concrete pipe ended and CMP began. Water clarity was somewhat poor and limited viewing throughout the conduit. The camera-crawler was advanced upstream within the CMP for approximately 35 feet, where the CMP ended and a masonry conduit began. The camera-crawler was advanced upstream within the masonry conduit for approximately 30 more feet, where numerous piles of sand were observed near the sidewalls of the masonry conduit. Figure B-59 shows a typical pile of sand near the sidewall of the conduit. In addition, just a few feet into the masonry conduit existed an open defect (crack) in the crown of the conduit. Figure B-60 shows the defect at the crown of the conduit. This defect allowed sand materials to enter the conduit. Based on observations made during the CCTV inspection, it was concluded that Pasture Canyon Dam was in the process of failing by internal erosion or backward erosion piping, and immediate action was required. Visual monitoring of the embankment dam and monitoring of the area around the outlet works was performed every 4 hours. The Bureau of Indian Affairs (BIA) imposed reservoir and gate operating restrictions at Pasture Canyon Dam.

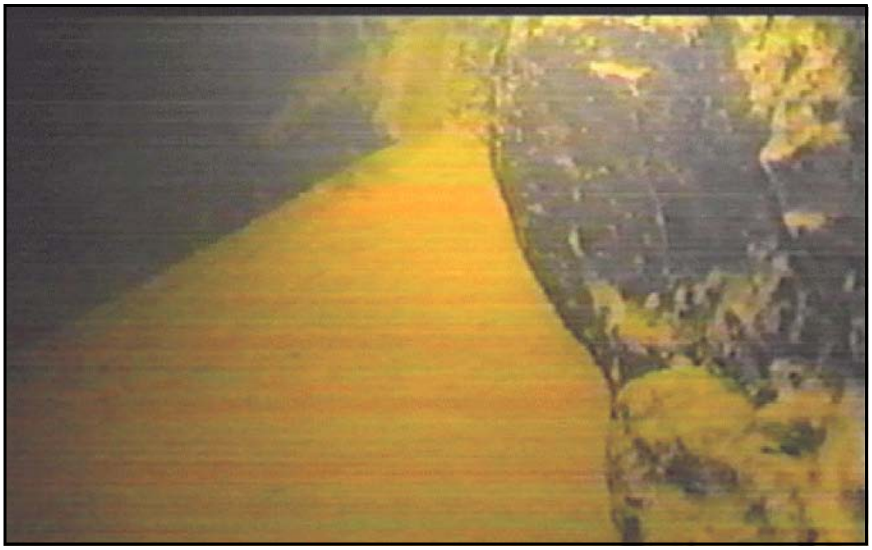

Figure B-59.—The camera-crawler encountered fine sandy materials that appeared to be entering the conduit through a defect in the sidewall. This view is looking upstream at the right side wall. The conduit had not been operated since the fall of 2003. These materials had collected within the conduit over the last 7 months.

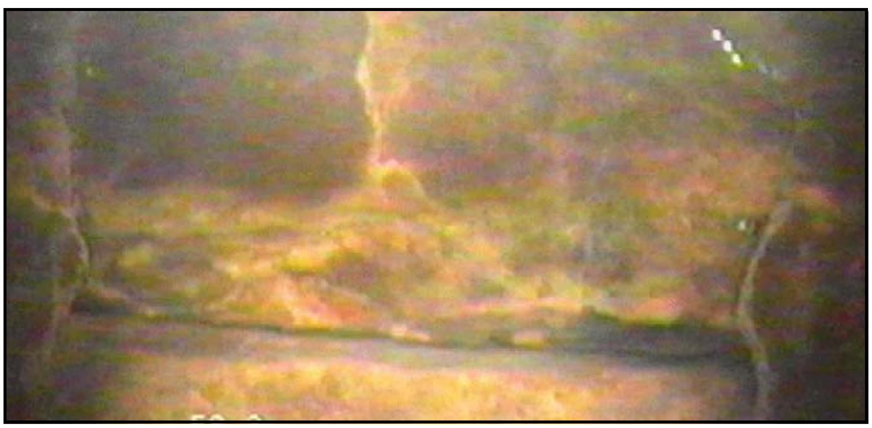

Figure B-60.—Just into the masonry conduit an open defect (crack), shown in the lower part of the figure, exists in the crown of the conduit. This defect allowed sandy materials to enter the conduit.

A contractor was mobilized to lower the reservoir level to elevation 4880.0 using high capacity, low head pumps (12-inch diameter). The reservoir drawdown was limited to 1 foot per day. After the reservoir was lowered to the desired elevation, a siphon was installed to provide downstream irrigation releases. Figure B-61 shows the siphon discharge irrigation releases. Based on the deteriorated condition of the conduit, the BIA decided to abandon the outlet works by grouting it closed. The

Figure B-61.—Siphon constructed over to crest of the embankment dam for discharging irrigation releases.

Figure B-62.—Grout mix being conveyed directly into the grout mixer from the transit mixer truck.

Bureau of Reclamation's Farmington Construction office accomplished the grouting using three 1½-inch diameter PVC schedule 40 pipes. The upstream end was plugged by an inflatable bladder. The downstream end was plugged using a burlap pig sealed with redi-mix dry-pack. The burlap pig acted as a filter to prevent grout leakage, and the dry-pack held the pig in place. Two grout plants were brought onsite. The grout mixers had a volume of 45 gallons each. The plants were powered by gasoline engine over a hydraulic system and were found to be very adequate for the grouting operations. Figure B-62 shows the grout plant used. Type II cement was utilized in the grout mix. The initial mix was 0.8:1 (water/cement ratio by volume). When it was determined that no problems were encountered with the grouting operations, the mix was reduced to 0.7:1. A grout fluidfier was used in the grout mix. Upon completion of the grouting operations, an ASTM C33 sand filter was installed at the downstream end of the conduit.

Lessons learned

- CCTV inspection equipment can be used to identify deteriorated areas within inaccessible conduits.

- Expedited dam safety actions require good communication between all interested parties and agencies involved.

- A siphon can be constructed quickly and inexpensively in order to provide downstream irrigation requirements.

Reference

Bureau of Reclamation, *Pasture Canyon Dam—Outlet Works Abandonment*, November 2004.

Project name: Piketberg Dam

Location: South Africa

Summary: Failure of an embankment dam by internal erosion resulting from hydraulic fracture of earthfill adjacent to the outlet conduit

In 1986, Piketburg Dam, a 40-foot high embankment dam was built across a minor tributary of the Verlore Vlei River, near the town of Piketberg in the Western Cape, South Africa. The new embankment dam was constructed over an existing dam at the site to increase storage. Figure B-63 shows a cross section of the embankment dam.

After 5 weeks following construction, during which water was pumped into the reservoir, and when it was almost full, major leakage suddenly appeared at the downstream toe of the embankment dam near the outlet. Within less than a day, the entire contents of the reservoir had been lost through a cavern adjacent to the outlet conduit.

Inspection after the event revealed a major tunnel through the entire width of the embankment dam along the outlet conduit. At the time of the inspection, the roof of the tunnel had collapsed over the entry and exit. The center portion of the tunnel beneath the dam crest, however, remained intact arching almost 33 feet across the tunnel. Large sinkholes were present in the upstream slope of the dam.

The embankment material was broadly graded from coarse gravel sizes to clay sizes, typically with a liquid limit of 28 percent and plasticity index of 9. Both residual soil (decomposed phyllite and greywacke) and transported soil were utilized. The latter

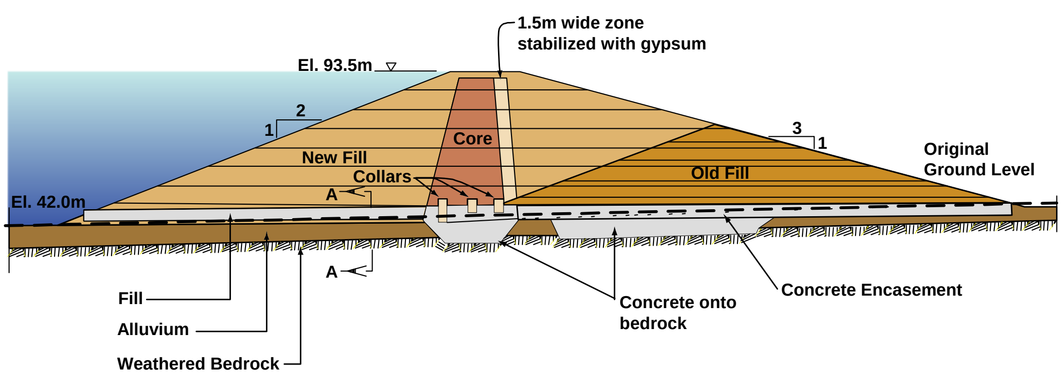

Figure B-63.—Cross section of the dam as designed.

included gravelly clay gully wash deposited in the form of an alluvial fan, as well as colluvium. Fine grained material was intended to be reserved for the designated core zone. The embankment dam design did not include provisions for either filtering or drainage of the core.

Tests showed that the earthfill material was dispersive. During construction, gypsum was added to portions of the core as a treatment for dispersivity. The new outlet conduit was laid roughly along the original ground surface, under the highest section of the new dam. A pipe was placed in the bottom of a wide slot cut through the old embankment dam. The pipe was laid in a trench dug into the lowest layers of compacted fill that had already been placed, and then encased in reinforced concrete. Across the new core's foundation cutoff, plus along one other section, the outlet trench was deepened to weathered bedrock prior to filling with concrete, to improve bearing upon soft material present under those sections. Concrete antiseep collars were found to have been cast over only the top and upper sides of the outlet encasement. The collars did not extend below the pipe encasement.

Breaching took place soon after filling began and before the reservoir was completely filled. This suggests that one or more concentrated leaks must have existed, to enable flow to reach the downstream toe so soon, long before the saturation front could have advanced very far into the earthfill. Failure started at the downstream toe in the vicinity of the outlet conduit, and the erosion tunnel terminated immediately adjacent to the upstream end of the outlet encasement.

The initial concentrated leaks alongside the outlet conduit are postulated to have occurred due to hydraulic fracture of the earthfill by the rapid rise of the reservoir. The conduit appears to have allowed for a low stress zone to occur in the earthfill next to the wall of the encasement. In essence, the wall "shielded" the adjacent fill from the full weight of the overlying embankment. The stresses in this area were likely lower than the reservoir head. A somewhat compressible foundation material beneath the conduit could have assisted in the formation of the crack completely along the embankment dam's cross section. Once concentrated flow started, the dispersive nature of the embankment fill would have allowed for rapid erosion from the downstream exit of the crack, progressing upstream. Lack of any defensive designs for embankment cracking, such as a filter and drain, contributed to the failure.

Also contributing to the failure was the poor compaction of the earthfill material adjacent to the encasement wall that was found during the forensic investigations. Also thought to contribute to the failure were the potential for differential settlement of the new and old earthfill.

Figure B-64 shows a cross section through the conduit area after failure.

Lessons learned

The dam failure was likely caused by poor compaction of the soil adjacent to the outlet conduit. Factors contributing to the poor compaction include inclusion of antiseep collars and a poorly constructed concrete encasement. A compressible foundation may have assisted in the formation of a crack next to the conduit.

Key changes to the design and construction that would have likely prevented failure of the embankment dam include:

- Utilizing a conduit design that accommodated the likely settlements caused by the foundation.
- If the concrete casement around the conduit had used battered side slopes rather than vertical ones, compacting soils against the conduit would have created more positive pressures and lessened the potential for hydraulic fracture.
- Inclusion of a filter diaphragm.
- Elimination of the cutoff collars.

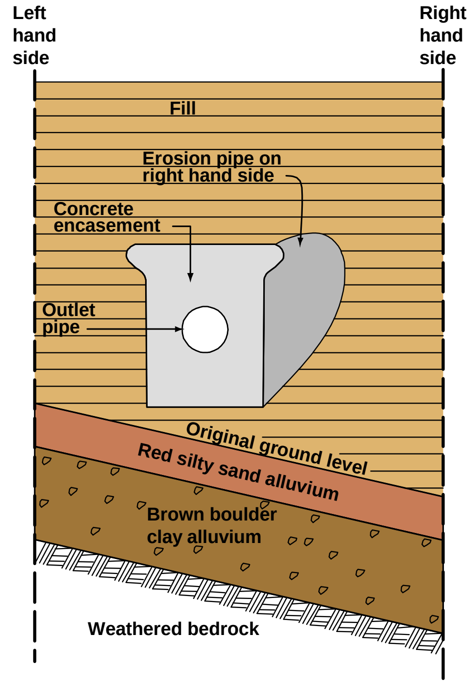

Figure B-64.—Cross section through the outlet conduit showing pipe, encasement and erosion tunnel.

Reference

Wilson, Clive and Louis Melis, "Breaching of An Earth Dam in the Western Cape by Piping," *Geotechnics in the African Environment*, Blight et al. (eds), Balkema, Rotterdam, 1991.

Project name: Ridgway Dam

Location: Colorado

Summary: Grouting of cracks in an existing outlet works conduit

Ridgway Dam is a zoned earthfill embankment across the Uncompahgre River in Ouray County near Montrose, Colorado. The embankment dam has a maximum height of 335 feet above the streambed, and a crest length of approximately 2,460 feet. The river outlet works is located near the right abutment of the embankment dam and crosses the dam axis at station 11+69.07. Most of the outlet works was constructed under a Stage I contract during 1980 and 1981. The completed outlet works consists of a 50-foot long, 9-foot diameter diversion conduit, a drop inlet intake structure with a concrete plug, a 500-foot long, 9-foot diameter upstream conduit, a gate chamber with a 5- by 6-foot high pressure guard gate, 64-inch square regulating gates, a control house and equipment, and hydraulic jump stilling basin. The spillway consists of a gloryhole intake exiting into a 6.5-foot diameter conduit located on a shelf on the left abutment of the dam. The conduit exits into an open chute and then into a Type II hydraulic jump stilling basin. Figure B-65 shows an aerial view of Ridgway Dam.

Figure B-65.—Aerial view of Ridgway Dam, Colorado.

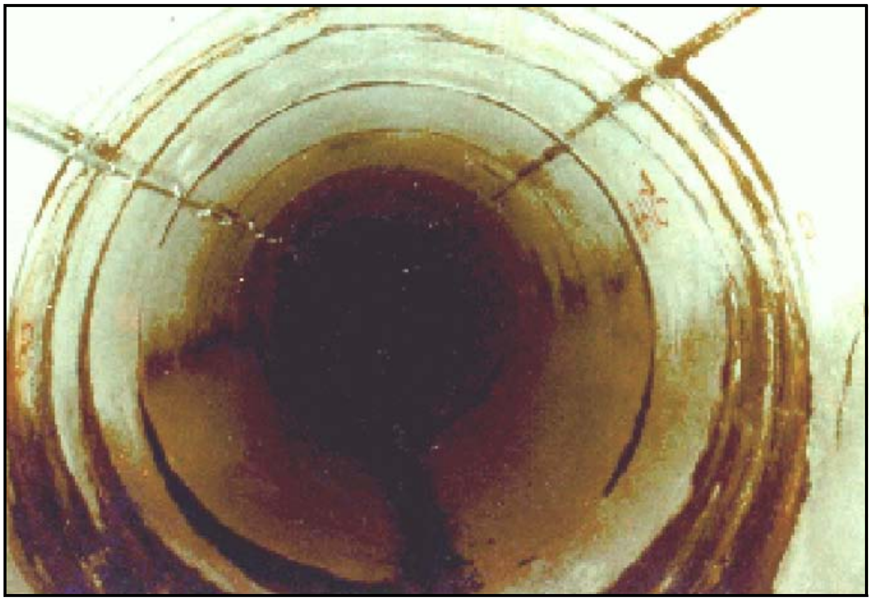

Figure B-66.—View of the interior of the outlet works after grouting.

Settlement and crack surveys were taken, in the upstream and downstream conduits, in January of 1986 and October of 1986. The maximum settlement recorded in January of 1986 was about 0.74 feet at station 11+24 in the downstream conduit. For reference, the station at the centerline of the gate chamber is located just downstream of the centerline of the embankment dam, station 7+50. In October of 1986, an additional settlement of about 0.22 feet was recorded at station 11+24. Settlements of other points in the conduit varied, seemingly based on the stiffness of the foundation. Maximum transverse cracking occurred in the upstream conduit at station 7+80 and station 8+80. The downstream conduit had very little transverse cracking, but a maximum joint opening of about ½ inch occurred at station 10+73. Longitudinal cracking occurred in the crown and invert between stations 6+50 and 7+65 and between stations 11+74 and 14+35 in the upstream and downstream conduits, respectively. The settlement and cracking occurred during or after constructing the dam embankment to elevation 6830 in the 1985 construction season and topping out of the dam in 1986 at elevation 6886.

The settlement and cracking of concrete in the upstream conduit was addressed by required injecting grout in and around the cracks to reduce water leakage and potential backward erosion piping or internal erosion of surrounding soils in the embankment dam, along with protecting the conduit reinforcement from corrosion. Polyurethane resin was used to grout the transverse cracks. Also, the polyurethane resin grout was considered (due to its more flexible characteristics over more rigid epoxy material) for grouting the longitudinal cracks. However, longitudinal cracks were grouted with epoxy material to provide a degree of structural stability. Approximately 500 feet of cracks were designated for grouting. The contractor grouted the cracks in February and March 1987 (figure B-66). Instrumentation was installed across selected cracks in the upstream conduit, which could be read in the gate chamber to track opening of the cracks and additional settlement. This type of tracking was selected because it is difficult and expensive to unwater and inspect the upstream conduit. The downstream conduit is accessible, and cracking is routinely inspected.

Lessons learned

- Settlement can occur even with best efforts to locate the conduit on competent foundation. Settlement along the alignment is not uniform and can result in cracking of the conduit.

- Grouting is an effective method for seating cracks and making the conduit watertight.

Reference

Bureau of Reclamation, Unpublished notes and file photographs.

Project name: Rolling Green Community Lake Dam

Location: Maryland

Summary: Sliplining of an existing spillway conduit using Snap-Tite® HDPE

Rolling Green Community Lake Dam failed in February 1999. Constructed in 1965, the 22-foot high, low hazard embankment dam contained a 24-inch diameter CMP spillway. The spillway riser had been gradually deteriorating, and the owner had attempted repairs at the top of the riser by use of a larger CMP sleeve and concrete grout. However, no repairs to the lower portion of the riser were attempted, and the base of the riser collapsed on February 6, 1999. A large portion of the embankment dam was washed away, leaving a void about 30 feet in diameter and 10 feet deep around the original riser location (figure B-67).

A CCTV inspection of the barrel portion of the CMP revealed that the remaining sections of pipe were in good condition. The engineer elected to slipline the existing 24-inch diameter barrel with 20-inch (outside diameter) Snap-Tite® pipe (SDR 32.5). The space between the two pipes was filled with a grout composed of fly ash and cement (compressive strength 2,500 lb/in^2), and a new aluminum riser was constructed within the upstream portion of the embankment dam.

Figure B-67.—When the 35-year old corrugated metal pipe riser collapsed, a large portion of the low hazard embankment dam was washed away. This left a void about 30 feet in diameter and 10 feet deep around the original riser.

A filter diaphragm was constructed around the downstream end of the pipe to control seepage along the outside of the conduit.

Lessons learned

The use of Snap-Tite® HDPE allowed for rapid installation of a slipliner at a low hazard facility. No specialized contractors were needed to heat fuse the joints of the slipliner.

Reference

Deegan, J., *Rolling Green Dam Completion Report*, 2001.

Project name: Round Rock Dam

Location: Arizona

Summary: Sliplining of an existing outlet works conduit using HDPE

Constructed in 1937 and enlarged in 1953, Round Rock Dam is a 35-foot high embankment dam, located about 3 miles from the town of Round Rock, Arizona on the Navajo Reservation. The original outlet works at Round Rock Dam was constructed by cut and cover methods. Both the upstream and downstream outlet works conduits were constructed of 24-inch diameter CMP. Reservoir releases are controlled by a 24-inch diameter slide gate in a concrete wet well located about 15 feet upstream of the dam crest. In 1991, the Bureau of Reclamation's Deficiency Verification Analysis identified the structural integrity of the outlet works conduits as a dam safety deficiency. CCTV inspection had detected corroded portions and joint separations in the CMP.

Reclamation designed a modification, to address this dam safety deficiency, which consisted of an 18-inch O.D. HDPE pipe that was sliplined (figures B-68 and B-69) and then grouted into both the upstream and downstream CMP conduits. The HDPE pipe was designed to withstand external loads, disregarding any additional

Figure B-68.—Heat fusion of an HDPE pipe joint.

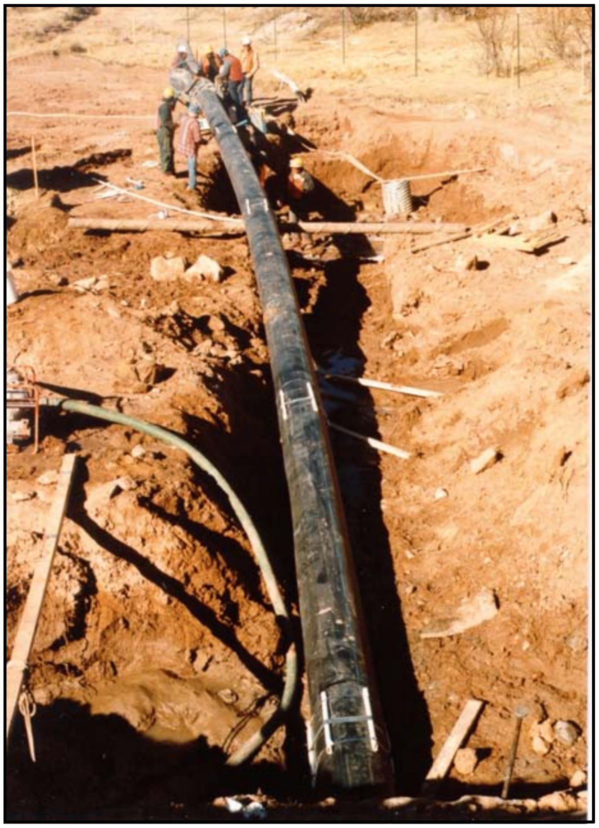

Figure B-69.—Inserting the HDPE slipliner into the existing CMP conduit.

support from the existing CMP. The design called for the HDPE pipe to withstand embankment fill loads of 38.2 feet, hydrostatic loads to 37 feet and a construction surcharge H-20 live load with a minimum of 5 feet cover. The HDPE pipe was also designed to withstand the loads associated with grouting. Installation of the liner was completed in the summer of 1994. The outlet works has operated without any liner-related incidents since that time. CCTV inspection of the slipliner was performed in May 2001, and the liner was found to be in good condition.

Lessons learned

Sliplining provides a low cost and less disruptive alternative to the conventional removal and replacement renovation method.

Reference

Bureau of Reclamation, *Outlet Works Video Inspection at Round Rock Dam—Bureau of Indian Affairs (BIA) Safety of Dams Program—Navajo Indian Reservation, Arizona*, July 26, 2001.

Project name: St. Louis Recreation Lake Dam (actual name withheld by request of owner)

Location: Missouri

Summary: Conduit abandonment by grout injection

In 2000, a 118-foot high embankment dam was constructed in the St. Louis area to create a 325-acre recreation lake. The lake is an integral part in an upscale land development. When the construction permit application was submitted to the State for approval, the designer included a 16-inch diameter PVC diversion pipe in the base of the embankment dam to prevent impoundment of water while the dam was being built. The construction permit was ultimately approved with the condition that the pipe would be filled with grout when the embankment dam was completed.

During construction, some foundation problems were discovered that required grouting of the foundation. The contractor was allowed to construct the lower portion of the embankment dam, and the grouting contractor was allowed to drill through it to grout the foundation. During this process, the contractor apparently observed a small amount of grout flowing from the outlet end of the temporary PVC diversion pipe (However, this was not reported to the State until after a problem with the pipe was later discovered).

As the work on the embankment dam neared completion and it came time to abandon the temporary diversion pipe with grout, the owner (through his designer) requested permission to alter the plans. Instead of filling the pipe with concrete, the owner proposed to retain the pipe and place a valve on the downstream end of the pipe. Their argument was that this would allow them to use the PVC pipe to lower the lake level in the future. The state Dam Safety Program balked at this, and it quickly became a contentious issue between the State and the dam owner. In an effort to assess the condition of the PVC pipe prior to making a final decision the State used a remotely operated video camera to examine the interior of the pipe.

With the owner and his engineer present, a video camera was inserted into the downstream end of the pipe. At approximately 450 feet upstream of the pipe outlet, a separated joint was observed (figure B-70). Just upstream of that joint, the pipe had collapsed, leaving a space only a few inches high at the bottom (figure B-71) The pipe was immediately abandoned by completely filling it with grout.

The owner later admitted that if the State had not been able to demonstrate why it did not want a valve on the downstream end of the pipe, he would have tried to coerce the State to allow the pipe to remain by contacting his legislators or by going

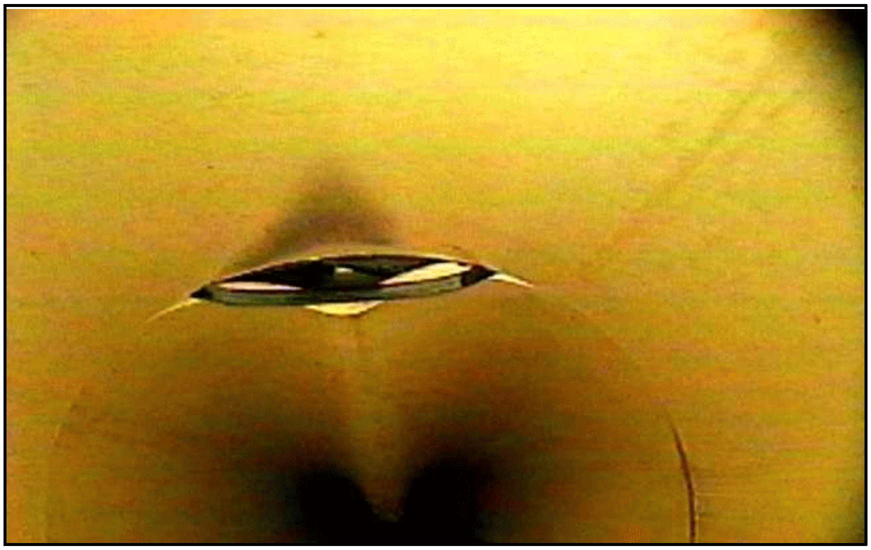

Figure B-70.—An open joint was discovered in a 16-inch PVC temporary diversion pipe under the 118-foot high embankment dam.

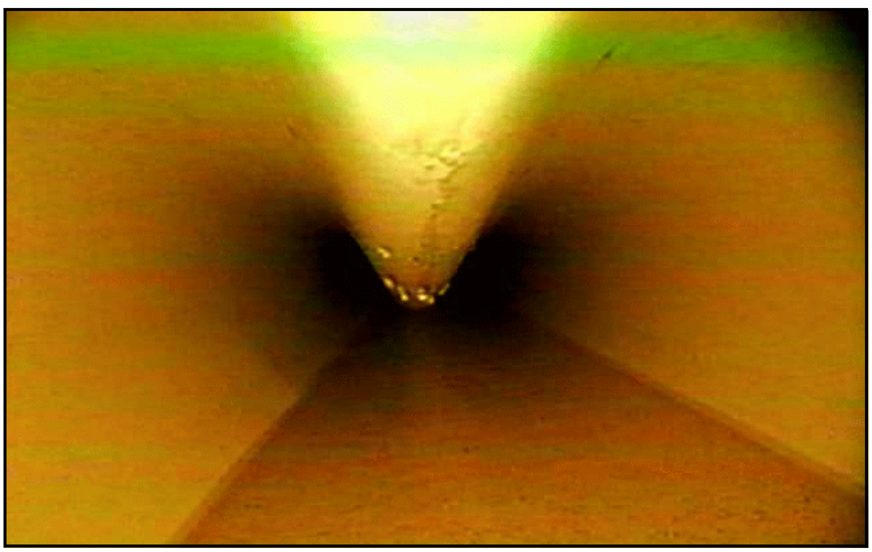

Figure B-71.—A portion of the temporary PVC diversion pipe was found to be severely deformed.

to court. If they had proceeded with the modification, this would have resulted in an unsafe pipe, subjected to full reservoir of more than 100 feet of head pressure running through the base of the embankment dam, with a valve at the downstream end. The presence of the separated pipe joint could ultimately have resulted in a disaster.

The cause of failure of the PVC diversion pipe was not determined. The cause may have been the result of the grouting work, poor construction practices, faulty pipe, or a combination of these factors.

Lessons learned

Internal inspection of all conduits within an embankment dam should be conducted at the end of construction to ensure that the conduits are not excessively deformed and that they will perform as intended. PVC pipe is not recommended for use in high or significant hazard embankment dams, unless it is fully encased in reinforced cast-in-place concrete. A better design would allow the permanent spillway to also function as a temporary diversion and would avoid the use of temporary conduits, which are intended to be abandoned in place.

Reference

Personal communications with Mr. Jim Alexander, P.E., Program Director, Chief Engineer, Water Resources Program, P.O. Box 250, Rolla, Missouri 65402.

Project name: Salmon Lake Dam

Location: Washington

Summary: Man-entry and underwater inspections of an outlet works conduit

Salmon Lake Dam is an offstream embankment dam located above the town of Conconully in Okanogan County, Washington. The Salmon Lake Dam reservoir (Conconully Lake) was a natural lake prior to construction of the embankment dam in 1921. The embankment dam is an earthfill structure with a structural height of 54 feet, a crest length of 1,260 feet at elevation 2325.1 (2330.25 original datum), and a crest width of 14 feet. The reservoir has an active storage capacity of 10,540 acre-feet and a surface area of 310 acres at spillway crest and normal reservoir elevation 2318.68 (2324.25 original datum).

The uncontrolled automatic siphon spillway and gate tower are located in the left abutment of the embankment dam. A trashrack prevents debris from entering the spillway. The siphon spillway consists of a trashrack intake structure, a vertical shaft, and a "goose-neck" transition section that leads to the downstream outlet conduit. The spillway discharge capacity at reservoir water surface elevation 2318.68 (2324.25 original datum) is 400 ft^3/s.

Outlet releases are controlled by two 3-foot by 4-foot 6-inch cast-iron slide gates with two hand-operated gate lifts. The gates have a combined discharge capacity of about 500 ft^3/s at reservoir elevation 2318.68 (2324.25 original datum) when operated separately from the spillway. The outlet works consists of a trashrack intake structure, a 4-foot 6-inch diameter upstream conduit, a control tower containing the slide gates, a 4-foot 6-inch diameter downstream conduit, and transition into an open cut channel. The concrete conduit has a minimum concrete thickness of 9 inches. Both the outlet works and spillway share a common downstream conduit with a transition immediately downstream from the gate.

Man-entry inspection of the downstream conduit was preformed in November 2000 and revealed poor quality concrete on eroded lift lines where, in at least one location, a ruler could be inserted beyond the thickness of the concrete. The most severe damage to the concrete was located approximately 110 feet upstream from the outlet portal.

A decision was made to perform concrete repairs on the poorest areas the downstream conduit. In preparation of the repair work in March 2001, a tap was inserted into a crack in the conduit approximately 190 feet upstream of the outlet portal in order to control seepage that was entering the conduit. Figure B-72 shows

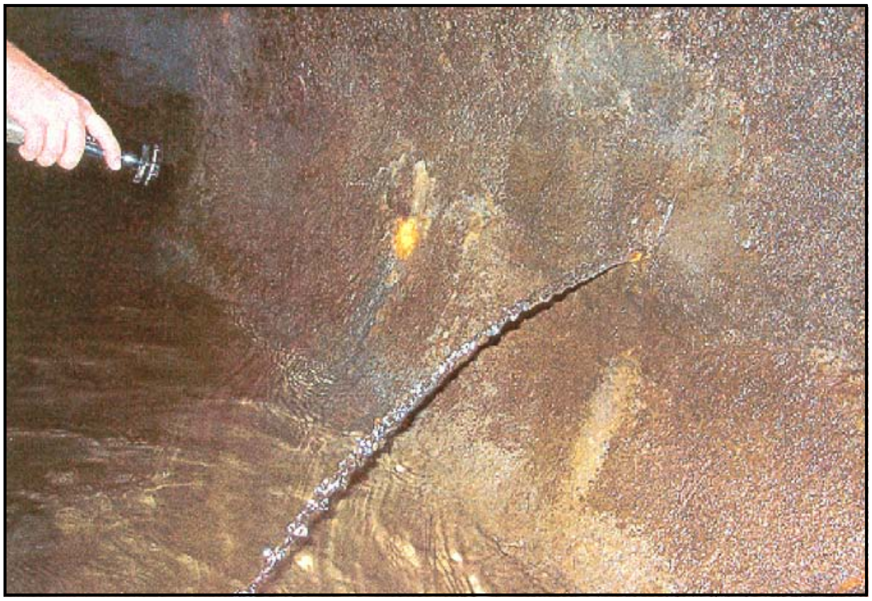

Figure B-72.—Seepage entering the downstream conduit.

the seepage into the conduit prior to installation of the tap. At this location, it was discovered that a void approximately 12 inches in depth existed behind the conduit wall. The concrete in this section of the conduit, which had previously been described as good, was determined to have 3 to 4 inches of somewhat sound concrete, backed by approximately 6 inches of loosely bonded aggregate or rubble. In subsequent explorations, it was determined that the void extended a minimum of 3 feet upstream and 4 feet downstream from the tap and was approximately 3 feet high.

The repairs amounted to jackhammering the eroded areas and soft spots and patching holes in the concrete. Repairs were made at locations identified during man-entry inspections and at some other locations where poor quality concrete was encountered. A largely unsuccessful attempt was made to grout the void behind the conduit wall located about 190 feet upstream of the outlet portal. This was attempted using a hand grout pump, which proved to be inadequate (partially due to the existing seepage gradients). One observation during concrete repairs was that only hoop steel was encountered in the concrete conduit.

Additional man-entry inspection was performed in May 2001 following the completion of the concrete repairs with the intent of evaluating possible modification alternatives. The concrete deterioration and erosion in the interior of the downstream conduit was considered unusual and possibly an indication of concrete with low strength and poor durability. The current condition of the concrete indicates that deterioration and leakage into and out of the conduit will worsen with time. The condition of the concrete may also be indicating a potential for loss of structural strength over time. Water tests were performed to determine if the water in the reservoir and the conduit were acidic or contained anything that would be detrimental to the concrete. The water tests results were negative, and it was concluded that the condition of the concrete could be a result of poor consolidation and possibly other poor construction practices.

The upstream conduit had never been inspected, since the reservoir would need to be drawn down to the level of the intake structure. In lieu of reservoir drawdown, an underwater inspection was performed in September 2001 to assess the condition of the concrete. The divers found the concrete in the intake structure to be in very good condition with no signs of deterioration. The concrete in the upstream conduit did not appear to be in as good of condition as the intake structure. The divers inspected and videotaped approximately 80 linear feet of the conduit. A knife was used to probe cracks and deteriorated areas in the concrete. Figure B-73 shows the diver using a knife to probe a crack in the conduit. In general, the crown of the upstream conduit was in the best condition, with most of the concrete being smooth and free of voids. The floor of the conduit is mostly smooth to 1/8-inch relief. The sides of the conduit were in the poorest condition with concrete relief being 1/8 to 1/4 inch thick with localized areas to 3/4 inch thick. Some areas of unconsolidated concrete were observed on the sides of the upstream conduit, but did not appear to be as severe as what was observed on the downstream conduit. The divers also inspected the upstream sides of the two 3-foot by 4-foot 6-inch cast-iron slide gates. During the inspection, the divers took care not to stir up particles on the invert, to avoid reducing the water visibility to near zero.

Due to the poor condition of the concrete within the downstream conduit and voids on the outside of the conduit, a decision was made to perform outlet works modifications to mitigate the existing dam safety deficiencies. In 2003, a 48-inch inside diameter steel liner was installed, and the annulus between the steel liner and existing concrete conduit was backfill grouted. As part of the outlet works modifications, a filter collar was installed around the downstream end of the conduit. The upstream conduit was determined to be of adequate condition to continue in service without repair, but underwater inspection should be made at regular 6-year intervals.

Lessons learned

- Man-entry inspection should be used to evaluate the condition of the conduit for both temporary repairs and permanent renovations.

- Where feasible, divers should be used to perform underwater inspections of conduits that cannot be dewatered.

Figure B-73.—The diver used a knife to probe cracks in the concrete. Divers prefer knives with blunt ends in underwater inspection, since it is less likely that a hole could accidently be poked into their dry suits.

Reference

Bureau of Reclamation, *Report of Findings—Spillway and Outlet Works Conduit Modifications—Corrective Action Alternatives*, February 7, 2002.

Bureau of Reclamation, *Design Summary—Salmon Lake Dam Modifications, Okanogan Project, Washington*, October 2003.

Project name: Sardis Dam

Location: Mississippi

Summary: A sinkhole developed over an outlet works conduit due to material being eroded through a joint

Sardis Dam is a hydraulic fill embankment dam constructed by the Corps of Engineers and was placed in service in 1940. Sardis Dam is 15,300 feet in length with an average height of 97 feet. The outlet works is located in the left abutment. The outlet works consists of an intake tower with four gated passages, and these passages transition in a 64-foot long monolith to a single "egg" shaped reinforced concrete conduit. The 18.25- by 16-foot conduit is founded on fine Tertiary sand and was cast in place. The walls of the conduit are 3.25 feet thick. The conduit consists of 17 monoliths, each 30 feet in length. Copper waterstops were placed at each monolith joint. The conduit discharges into a concrete stilling basin, which has baffle blocks for energy dissipation.

In December 1974, a sinkhole occurred above the monolith joint at the junction of the intake tower and the upstream end of the transition monolith. Figure B-74 shows the location of the sinkhole. Sinkhole investigation revealed that the intake tower is founded on piles, but the transition is not founded on piles. This allowed the transition monolith to settle about 1 inch more than the intake tower. This differential settlement was enough to rupture the copper waterstop. With water in the conduit being free flowing (nonpressurized conduit), and water pressure outside the conduit being near lake stage, a large pressure differential existed across this joint. This large pressure differential caused flow through the joint after the waterstop ruptured. The water flowing through the joint carried enough material to eventually cause the sinkhole to occur.

The solution to this problem was to fill the sinkhole with impervious material and to drill grout holes in this monolith joint all the way through the concrete into the surrounding soil along the entire perimeter of the joint. The holes were drilled from inside the transition. Neat cement grout was then pumped through these holes to fill any voids outside the transition and to seal the waterstop as well as possible. Prior to grouting, the gates were closed and sealed with saw dust, air compressors and a grouting machine were set up in the backfill area of the stilling basin, and supply lines were run up the conduit to the transition monolith to be grouted. The lake stage was at its normal level for that time of the year, and the elevation of the lake was about 20 feet above the invert of the conduit. Twelve grout holes were drilled in the monolith joint, four in the invert, four in the crown, and two in each side wall. Each hole was installed as follows: (1) A hole about 2 inches in diameter was drilled

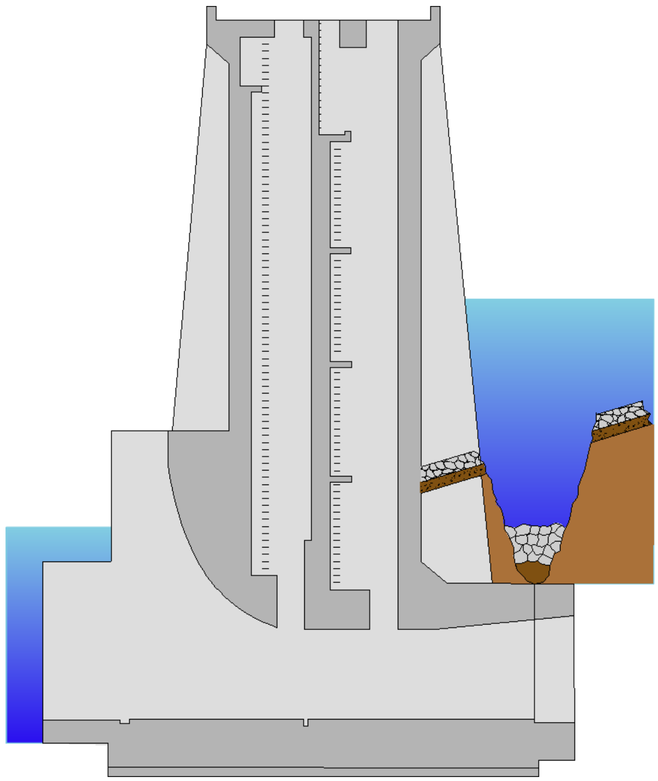

Figure B-74.—A sinkhole occurred above the monolith joint at the junction of the intake tower and the upstream end of the transition section.

with a jack hammer to a depth of about 2 feet, and (2) a 1½-inch pipe with a ball valve on the upper end was then grouted into the hole. After the grout had set up, a jack hammer with bit small enough to go inside the 1½-inch pipe was used to drill the rest of the way through the conduit to the foundation or backfill material. When foundation or backfill material was encountered, the holes would start to flow. The jack hammer with bit was then removed, and the ball valve was closed to prevent the hole from flowing. A water jet pipe was used to clean out the grout pipe just prior to grouting. At the end of the grouting operation, the ball valves were removed, and caps were placed on the grout pipes.

The first hole grouted took 12 cubic feet, and the take per hole decreased for each succeeding hole. A total of 40.5 cubic feet of neat cement grout was pumped around the joint.

The cement grout was anticipated to be brittle, and there was some concern that any additional settlement of the foundation could cause this neat cement to crack. However, this repair was completed more than 30 years ago and no other sink holes have developed.

Lessons learned

The problem was caused by a broken waterstop, which was the result of differential settlement. To prevent this on future designs, the monolith joints should be designed so that there can be essentially no differential settlement at the monolith joint.

Reference

Sardis Lake Project, Little Tallahatchie River, Mississippi; Dam, Outlet Works, and Spillway; Periodic Inspection Report No.2, Supplement D, September 1971.

Project name: Sugar Mill Dam

Location: Georgia

Summary: Siphon spillway failure

Sugar Mill is a residential subdivision that was developed in the early 1990s in north Fulton County, Georgia (Atlanta metropolitan area). A central amenity of the development was an existing reservoir impounded by an old earthen embankment dam with inadequate spillway capacity.

In addition to widening the earthen emergency spillway, five PVC siphon pipes were installed in trenches excavated through the crest of the embankment. The design called for the pipes to be bedded in concrete. Control valves were installed in the siphons at the top of the embankment dam inside of manhole structures.

In 2002, the owner noted the presence of water flowing out of a hole in the embankment adjacent to the siphons, approximately 15 feet downstream of the valve manhole. The owner contacted the design engineer, who performed exploratory investigations in an attempt to locate the source of the seepage. The engineer recommended installation of a drainage system to control the seepage. However, this did not work, and the seepage situation continued to get worse. In 2003, during a storm, the owner attempted to operate the siphon spillways, and found the manholes full of water and that the seepage had substantially increased.

An internal CCTV inspection of the siphons found no problems with the PVC pipes. The engineer suspected that flow was occurring under the pipes and advised the owner to replace the siphons. Upon excavation and removal of the pipes, it was found that the original contractor had not achieved adequate placement of the concrete cradle, resulting in voids under the center of each siphon. Constant flow through these voids under the pipes resulted in internal erosion of the underlying embankment soils.

Lessons learned (adapted from Wilson and Monroe)

- Siphon spillways do not always work as expected, especially when constructed by an inexperienced contractor. Thorough construction oversight is required.

- A filter should be used in conjunction with conduit penetrations through embankment dams.

References

Wilson, Charles, and Joseph Monroe, *Dam Surgery—Repairs to Sugar Mill Dam, Fulton County, Georgia*, presented at the ASDSO Southeast Regional Conference, Norfolk, Virginia (entire reference is not available), 2004.

Sugar Mill Community Association, Minutes of Board of Directors' meetings: April 18, 2002; May 7, 2002; and January 14, 2003

Project name: Turtle (Twin) Lake Dam

Location: Montana

Summary: Sliplining of an existing outlet works conduit using HDPE

Constructed in 1932 by the U.S. Indian Irrigation Service (now Bureau of Indian Affairs), Twin or Turtle Lake Dam is a 20-foot-high embankment dam, located about 4 miles southeast of Polson, Montana on the Flathead Indian Reservation. The original outlet works at Turtle Lake Dam was constructed by cut and cover methods. The outlet works conduits were constructed of concrete pressure pipe in approximately 4-foot long sections. The upstream conduit and the first 40 feet of the downstream conduit were 21-inch diameter pipe. The vertical wet well shaft at the upstream edge of the dam crest separates the upstream and downstream conduits and houses a 24-inch diameter slide gate, which regulates discharges. The remainder of the downstream conduit (340 feet, much of which extends downstream of the toe of the embankment dam) is 18-inch diameter.

During the 1996-1997 winter, a sinkhole was discovered above the lower reaches of the downstream conduit. Investigations showed that a root ball had partially plugged the outlet, and portions of the pipe had partially collapsed. During the late spring of 1997, the Bureau of Reclamation designed a temporary repair for the deteriorating outlet works. Because of the small size of the conduit and the concern for seepage coming into the conduit, it was decided that a watertight liner should be used. The conduit downstream of the embankment dam toe was excavated, but it was not desirable to excavate in the embankment dam itself. For this reason and because remotely CCTV inspection seemed to indicate potential for offsets and changes in the conduit alignment, an HDPE sliplining was proposed.

A 16-inch O.D. HDPE pipe was sliplined (figures B-75 and B-76) into the existing downstream conduit beneath the dam up to the regulating gate, and the annulus between the existing and new pipes was grouted with a cement grout with superplasticizer. Downstream of the dam toe, the pipe transitions to a 22-inch O.D. pipe, which extends downstream to the original portal location.

The HDPE-lined outlet works at Turtle Lake Dam has been in operation since modification without further incident. A CCTV inspection was conducted in April 2001. The inspection indicated that the HDPE slipliner was performing well.

Figure B-75.—HDPE slipliner being installed at Turtle (Twin) Lake Dam. Note the HDPE grout lines welded onto top of new liner. The end flange was used to attach the two different sized liners together.

Figure B-76.—Upstream end of the HDPE slipliner modified to act as a pulling head.

Lessons learned

Sliplining provides a low cost and less disruptive alternative to the conventional remove and replacement renovation method.

Reference

Cooper, Chuck, Ernest Hall, and Walt Heyder, *Case Histories Using High Density Polyethylene Pipe for Slip-Lining Existing Outlet Works and Spillways*, October 2001.

Project name: Upper Red Rock Site 20 Dam

Location: Oklahoma

Summary: Failure of an embankment dam by internal erosion resulting from hydraulic fracture of earthfill adjacent to the flood control conduit

Upper Red Rock Site 20 Dam was a low hazard earthfill embankment structure constructed by the NRCS for flood control in 1973. The embankment height was about 31 feet and it contained about 61,000 cubic yards of earthfill. The principal spillway conduit is a 36-inch diameter reinforced concrete pipe. The embankment dam was constructed in an area of Oklahoma now known to have a high concentration of dispersive clays. The embankment soils classify as CL in the Unified System with an LL of about 35 and a PI of about 15. The dispersive clays are produced from weathering of Permian or Pennsylvanian age shales of marine origin. The sodium rich parent material produce dispersive clays that are highly erodible.

The embankment dam failed in 1986 by internal erosion when the reservoir filled suddenly to a reservoir elevation higher than the reservoir had previously ever impounded water. At the time of the failure, the reservoir was about 1.6 feet below the top of the crest of the embankment dam. The embankment dam had impounded water continuously at a lower reservoir elevation for most of its history, until the rainfall event that filled the reservoir to this unprecedented higher elevation.

Most often, failures similar to this one occur when the reservoir fills suddenly soon after completion of the embankment dam. This failure occurred however when a "second first filling" type of event occurred. The site had previously filled to a pool level corresponding to about one-third of the embankment dam height (the dam is a flood control single purpose reservoir), and maintained that pool for most of the 13 years of its life. A large rainfall event caused water to rapidly fill the reservoir and water flowed through the auxiliary spillway, but not over the crest of the embankment dam in 1986. Cracks in the earthfill in the upper part of the embankment dam allowed water to find a pathway through unsaturated dispersive clay fill used to construct the embankment dam. The crack(s) in the embankment dam quickly eroded and a breaching type failure of the dam resulted. The cracks in the embankment dam were thought to be a result of a combination of hydraulic fracture and desiccation. The failure tunnel was located about 40 feet to the side of the conduit that penetrated the embankment dam, in the area of the old stream channel.

The differential settlement that helped to create stress conditions favorable to hydraulic fracture was probably associated with the presence of a channel through the embankment dam that had relatively steep side slopes and the presence of the principal spillway conduit in the vicinity. Internal erosion failures in similar embankment dams constructed by the NRCS in Oklahoma were often near principal spillway conduits. Differential settlement is a primary contributor to conditions favorable to hydraulic fracture.

The failure of the embankment dam was observed as it occurred from an aerial survey of the site. Figure B-77 shows the failure of the embankment dam as it occurred. Water in the reservoir had risen to about 1.6 feet below the crest of the embankment dam following about 19 inches of rainfall which had occurred over several days. Water had flowed over the crest of the auxiliary spillway, but had not overtopped the embankment dam. Water was observed to be flowing through a tunnel developing in the embankment dam at about 40 feet to the side of the conduit location. Water entered the upstream slope of the embankment dam at about the maximum reservoir level in several locations and exited the downstream slope of the embankment dam through an erosion tunnel. Water exited the downstream slope about one-third of the way up from the toe of the embankment dam. The tunnel that developed in the embankment dam eroded quickly and drained the pool to about one-half of the embankment height.

Figure B-77.—Aerial view showing the failure of Upper Red Rock Site 20 Dam.

Lessons learned

The incident demonstrated that although failures by hydraulic fracture may be most common in first filling incidents, a potential for failures still exists years after the first filling. The soils were known to be highly dispersive, but designers thought that by placing the dispersive soils above the elevation of the permanent pool, the threat to the embankment dam could be reduced. Embankments constructed of dispersive clays are extremely susceptible to internal erosion failures unless protected by chimney filter zones.

Reference

Oklahoma NRCS State Office files.

Project name: Waterbury Dam

Location: Vermont

Summary: Design and construction of a filter diaphragm around an existing outlet works conduit

Waterbury Dam is 150 feet high across much of a valley except over the original river gorge, where it approaches 190 feet high. The embankment dam consists of a wide central impervious core (CL and ML) flanked by sand and gravel (SM and GM) shells upstream and downstream. A large rock fill zone extends across the toe of the main embankment section. The dam is founded on a thick glacial silt deposit beneath the western two-thirds of the embankment dam, and directly on the schist bedrock beneath the eastern third of the dam.

The internal erosion of embankment dam and/or foundation silts into and through the rockfill zone attracted attention in the late 1970s. A section of the rockfill toe along the western portion of the embankment dam was reconstructed and treated by a filter injection process in the mid-1980s. At that time, a separate internal erosion condition was revealed in the portion of the embankment dam over the original river gorge. Internal voids were treated using the filter injection process.

From 1985 through the late 1990s, seepage conditions within the gorge area of the dam failed to completely stabilize. In 1999, investigations concluded that additional remedial action was needed. The remediation included placing a filter drain to intercept any seepage that might occur along the interface between the conduit and the embankment materials around the downstream end of the existing outlet conduit.

The outlet conduit consists of a horseshoe-shaped reinforced concrete conduit placed within a bedrock excavation. Within the impervious core zone, the conduit is mostly within the confined bedrock excavation, and impervious material was compacted using hand tampers. The east wall of the excavation consists of a near-vertical excavated rock wall with some localized overhanging rock ledges. These conditions led to the concern for potential internal erosion along the conduit or along the interface with the steep rock surface.

Exposure of the conduit required a large excavation into the downstream sand and gravel shell (20,000 yd^3) and rock fill (10,000 yd^3) zones of the embankment dam. Hand-placed riprap from the dam surface was stripped, processed to remove fine materials, and eventually replaced on the embankment surface. Within the shell, excavation slopes were 1.5H:1V, and within the rock fill slopes were 1H:1V. In spite of several very hard rains, there were no stability problems. About 160 feet of the conduit, entirely within the rockfill zone, was exposed.

The drain, consisting of a coarse inner zone and a fine outer filter zone, was wrapped around the outer surface of the exposed conduit. At the upstream end of the excavation, an expanded filter diaphragm was placed from the eastern bedrock surface to the western rock fill excavation surface. The diaphragm extended about 5 feet above the crown of the conduit.

The drain consisted of a coarse drainage fill (AASHTO No. 7 stone) to act as a carrying medium for collected seepage, and a finer sand zone (Vermont concrete sand) to act as a filter to prevent the infiltration of fines from the surrounding rockfill materials. The 18-inch thick course zone was placed around the drainpipe and filled the irregular space between the overhanging eastern bedrock wall and the conduit surface. The fine filter zone was also a minimum of 18 inches thick, but was broadened to fill the excavation wedge along the western side of the conduit. See figure B-78 for cross section.

The filter and inner coarse drainage fill zone were designed to meet filter criteria with respect to each other, and the filter was designed to handle the silts within the fine matrix material within the adjacent rock fill zone. The low-plasticity glacial silts, whether foundation silts, core material, or rock fill matrix fines, have always been the primary concern with respect to internal erosion. A relatively fine filter material is required to meet filter criteria.

The exact limits of the rockfill zone were not known, although some of the original drawing information suggested that the proposed excavation would completely penetrate the rock fill to the contact with the shell zone. As the excavation progressed, it became apparent that the rock fill zone would not be completely penetrated as hoped. Therefore, it was decided to install a horizontal drain through the remaining wedge of rockfill to intercept seepage before it dispersed into the rockfill zone. The drain was drilled within the confined space between the west side of the conduit and the bedrock surface. Drilling was very difficult due to the presence of an unknown concrete plug adjacent to the conduit. However, the drain was completed, and seepage flows were intercepted near the upstream limits of the rockfill zone. The discharge pipe from the horizontal drain was extended to the downstream toe of the dam, along with the drainage pipe placed at the bottom of the conduit drainage zone along both the east and west sides of the conduit.

Because of the confined spaces and irregular bedrock surfaces, the drainage and filter zones were placed in thin lifts and compacted with hand-operated vibratory compaction equipment (figure B-79). Although the work was time consuming, the uniformly graded materials were very easy to place and compact. To accommodate

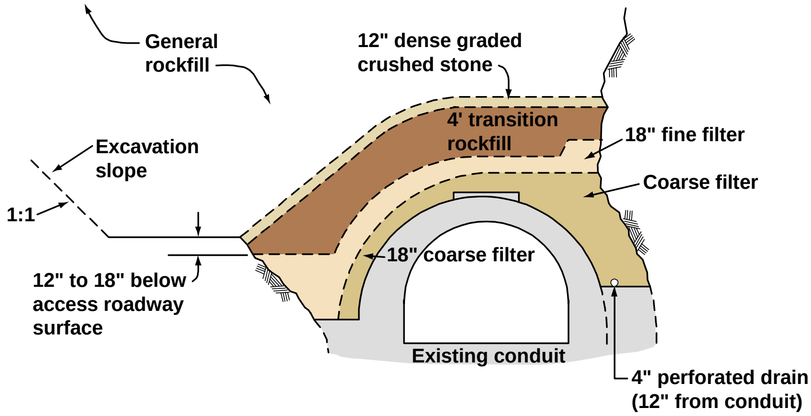

Figure B-78.—Cross section showing filter zone and conduit.

Figure B-79.—Filter material being placed in thin lifts and compacted with hand-operated vibratory compaction equipment.

construction, the zones were expanded on both the east and west sides of the conduit to reach to the exposed bedrock surfaces paralleling the conduit before switching back to rockfill materials. A transition zone of smaller rockfill was placed immediately adjacent to the filter before going to general rock fill backfill materials.

Lessons learned

- The designers must be actively involved onsite during the construction phase as the actual subsurface conditions are revealed. Designers should not rely on construction personnel to make critical judgments about the need for field changes—especially where filter and drain features are involved.

- The highest risk to the embankment dam occurred when the excavation was at the maximum extent, and before the new drains were installed. The contractor was required to prepare a contingency plan for mobilization of a horizontal drilling specialist to minimize the time the excavation had to be open to the maximum depth. When it was determined that a horizontal drain would be required, the drilling contractor was mobilized. The drilling contractor was onsite by the time the excavation was completed, resulting in minimum impact on the duration of the critical phase of the excavation.

- This type of construction, due to subsurface conditions, is difficult to investigate. Since as-built records from that construction era were lacking, a high degree of uncertainty was involved. Anticipating field changes is required, as well as carrying a higher contingency than might be necessary for above-ground construction.

- Any evidence of slime bacteria deposits should be addressed as a potentially serious problem for filter and drain features.

Reference

U.S. Army Corps of Engineers, Summary Report CENAB-EN-GF, *Waterbury Dam—Seepage Control Modifications*, January 30, 2003.

Project name: Willow Creek Dam

Location: Montana

Summary: Lining of an existing outlet works conduit using CIPP

Willow Creek Dam is located in western Montana. The 84-foot high embankment dam impounds a reservoir of 32,300 acre-feet, used primarily for irrigation. The outlet works consists of a 54-inch diameter concrete-lined tunnel through the right abutment, with guard and regulating gates provided within a gate shaft upstream of the dam axis. The embankment dam and outlet works were originally constructed between 1907 and 1911, and were modified in 1917 and 1941. The Bureau of Reclamation owns the embankment dam, and the Greenfields Irrigation District operates it.

A large sinkhole was discovered on the crest of the embankment dam in June 1996. The sinkhole was located directly above the outlet works tunnel, about 50 feet downstream from the gate shaft and near the dam axis. Earth materials were found to be eroding periodically from a 1-inch weep hole in the tunnel sidewall. Four weep holes in the tunnel lining were sealed, and the sinkhole was temporarily stabilized by backfilling with sand and gravel materials. The reservoir was gradually lowered using the outlet works, for a total drawdown of 27 feet.

Excavation of the embankment dam revealed the sinkhole extended through 40 feet of bedrock to a large cavity surrounding the concrete tunnel lining. Tremie grout was used to fill the voids around the tunnel, followed by the placement of backfill concrete to the excavated bedrock surface at elevation. The embankment dam was restored by the placement of a filter blanket on the excavated foundation, followed by the placement of compacted glacial till materials to dam crest elevation, and replacement of the slope protection on the downstream face.

The outlet works tunnel was originally excavated in 1907 by hand-drilling and blasting, with considerable water and soft materials encountered. Heavy timber beams and posts with timber lagging were used to support the tunnel excavation throughout its length, resulting in a square, excavated opening for the circular tunnel lining. Although it is unclear how the concrete tunnel lining was actually constructed, the specifications called for a uniform concrete thickness of 8 inches, without reinforcing bars, and the placement of "puddled fill" outside the tunnel lining to the excavated surface. Such construction could have resulted in a significant quantity of fine grained backfill material along the outlet works tunnel.

The downstream tunnel lining was severely damaged in 1958, when maximum outlet releases of 550 ft^3/s were reported. Approximately 70 feet of unreinforced concrete in the tunnel invert was removed by the flow, beginning 10 feet downstream from the regulating gate, and the foundation rock was eroded to a depth of 3 feet. A 10- to 15-foot long section of the tunnel crown was also removed, revealing a large void surrounding the tunnel. The structural damage is believed to have been initiated by negative pressures resulting from an insufficient air supply to the downstream tunnel during maximum releases, due to an undersized, 6-inch diameter air vent pipe. Repairs included replacement of the missing concrete invert and crown, and placement of rubble fill outside the tunnel lining. Weep holes were later drilled in the tunnel lining for pressure relief.

The development of the large tunnel cavity was probably a combination of overexcavation during construction, gradual erosion of the puddled fill and soft bedrock materials, and collapse of the harder bedrock materials into the tunnel following the lining failure. The sinkhole may have developed gradually by the internal erosion of glacial till embankment materials through open joints and fractures in the bedrock, progressive collapse or "stoping" of the bedrock into the void below, and erosion of earth materials through open cracks and weep holes in the tunnel lining.

Continued concern for the long term stability and structural integrity of the downstream tunnel lining, and the potential for renewed erosion of earth materials through open cracks and joints (despite grouting efforts) resulted in the consideration of tunnel lining options. The downstream tunnel extends 429 feet from the regulating gate to the downstream portal, where outlet releases enter a diffusion-type stilling basin. A structural lining was required for the first 100 feet of tunnel, which seemed to be the most susceptible to future problems, since it included the portion damaged in 1958, the sinkhole location, a significant longitudinal crack along the crown, and continuing seepage from various other open cracks and joints, and was located directly below the wide embankment dam crest.

The configuration of the stilling basin at the downstream portal, and grade changes within the tunnel (including one of over 3 degrees), made the proposed installation of a rigid structural lining more difficult. So the search for alternatives to rigid linings focused on CIPP systems, originally introduced in the United States by Insituform Technologies in 1977. A CIPP lining consists of a flexible, resin-impregnated, needled polyester felt tube, which is expanded under hydrostatic head, and cured by the circulation of heated water. Construction access through the outlet works gate shaft, for installation of a CIPP lining from the upstream end of the tunnel, would have been severely affected by the gate house and existing mechanical equipment, including the gate operator and stem, air vent pipe, ladders, and landings. Installation of a CIPP lining by the inversion method from the downstream portal would have resulted in the exposure of the entire unreinforced concrete lining to high water temperatures, and the requirement of an additional 329 feet of waste tube material. A finite element analysis of potential thermal stresses within the 8-inch concrete lining, using an ABAQUS computer program, predicted large tensile stresses sufficient to produce extensive cracking, which was unacceptable. Use of an alternative low temperature resin, with a curing temperature of only 80 °F, would avoid thermal stresses and produce acceptable results, but would require special handling and a longer curing period.

Reclamation prepared design specifications for a partial tunnel lining using CIPP and issued them in May 1997. A construction contract was awarded to the low bidder, Western Slope Utilities, Inc. (WSU) of Breckenridge, Colorado, in July 1997. An InLinerUSA licensee, WSU was experienced in the pulled-in-place installation method for linings up to 36 inches in diameter, and obtained the services of an InLinerUSA representative with the required experience for large diameter linings.

For design purposes, the existing tunnel was assumed to be in a "fully deteriorated" condition (due to the longitudinal crack in the crown) and subject to internal pressure under maximum discharge conditions. Design loads included a 10-foot external fill height on the tunnel crown, a 10-foot external hydrostatic head on the tunnel invert, and a maximum internal pressure of 20 lb/in^2. The CIPP was designed to carry the external loads with no contributing support from the circular tunnel lining with a factor of safety of 2.0. An ovality reduction factor, based on the average minimum and maximum diameters of the tunnel lining, was included to properly estimate the stiffness of the elliptically deflected pipe. For internal loads, the CIPP was designed as a thin-walled cylinder with uniform pipe wall stresses, using a hoop stress equation for plastic pipe.

An epoxy vinyl ester resin was selected over a polyester resin for greater strength and longevity. Design properties for the resin included an initial flexural modulus of 300,000 lb/in^2 and an initial flexural strength of 5,000 lb/in^2 for external loads, and an initial tensile strength of 3,000 lb/in^2 for internal loads. To characterize the long term performance of the CIPP over the minimum 50-year design life, a 33-percent creep reduction was assumed for the flexural modulus and flexural strength, and a 50-percent hydrostatic stress regression was assumed for the tensile strength. The final design thickness for the CIPP was 1.06 inches, including an additional 5 percent thickness to provide sufficient resin to fill the interior felt of the calibration hose, which was to remain in place.

The contractor began mobilizing equipment at the embankment dam on August 8, 1997. A steel platform was installed 12 feet above the bottom of the regulating gate shaft, and a steel elbow section was centered within the upstream end of the tunnel, to support a short flexible hose for a water column. One end of a 110-foot long calibration hose, consisting of a single layer of felt fabric with a watertight polyurethane coating, was carefully lowered down the shaft and through the flexible hose and elbow, where it was turned inside out and securely fastened to the outside of the elbow. A winch and roller were set up at the gate house doorway, and a second roller was positioned at the bottom of the shaft. The tunnel surfaces were swept clean, and utility lines (for lighting, ventilation, and water circulation) were established within the shaft.

The resin-filled tube was delivered to the site on August 11 in a refrigerated truck (figure B-80). The nonwoven fabric tube was manufactured in Houston, Texas at InLinerUSA headquarters, and the resin was added in Alma, Colorado at a "wet-out" plant used by WSU. Total weight of the liner was 10,000 pounds. The liner was removed from the refrigerated truck using a truck-mounted winch, and was carefully fed into the tunnel at the downstream portal and slowly pulled upstream. The liner was pulled into final position in the tunnel within about 1.5 hours and was securely fastened to the steel elbow outside the calibration hose. Reservoir water from the upstream gate shaft was pumped into the water column to begin inversion of the calibration hose under a 1-foot head. Within 20 minutes, the calibration hose had been turned inside out and extended the full length of the liner, pressing the liner tightly against the tunnel surface. Two perforated water supply hoses inside the calibration hose were used to circulate heated water from a heat exchanger truck under the full 12-foot head.

Return water temperatures at the truck reached 135 °F in 2 hours and were held constant for 4 hours, and then were raised to 175 °F within 1 hour and were held constant for 6 hours for curing the resin. After curing was completed, the circulating water was gradually cooled to 100 °F in 4 hours, finishing by noon on August 12. Epoxy vinyl ester resin contains styrene, a possible carcinogen, which is released during the curing process. Styrene vapors are heavier than air, and potentially flammable and explosive. Installers and inspectors must follow OSHA regulations pertaining to workers in hazardous and confined spaces. Fresh air had to be introduced into the tunnel before the contractor could cut a small hole in the end of the hardened liner to release the water.

The waste water was fully contained within the downstream stilling basin to permit final cooling to 70 °F, removal of resin residue from the water surface, and dissipation of dissolved styrene.

Both ends of the liner were trimmed using chain saws and circular saws, and a 0.5-inch deep groove was provided around the periphery to accommodate installation of end seals. Amex 10/WEKO seals were used, each consisting of a 14.5-inch wide rubber seal with three stainless steel bands spread by a hydraulic expanding device to ensure a tight fit. The work was completed on August 15, one week after site mobilization, for a total cost of about $145,000. Subsequent laboratory tests on field samples confirmed the design parameters for tensile and flexural properties.

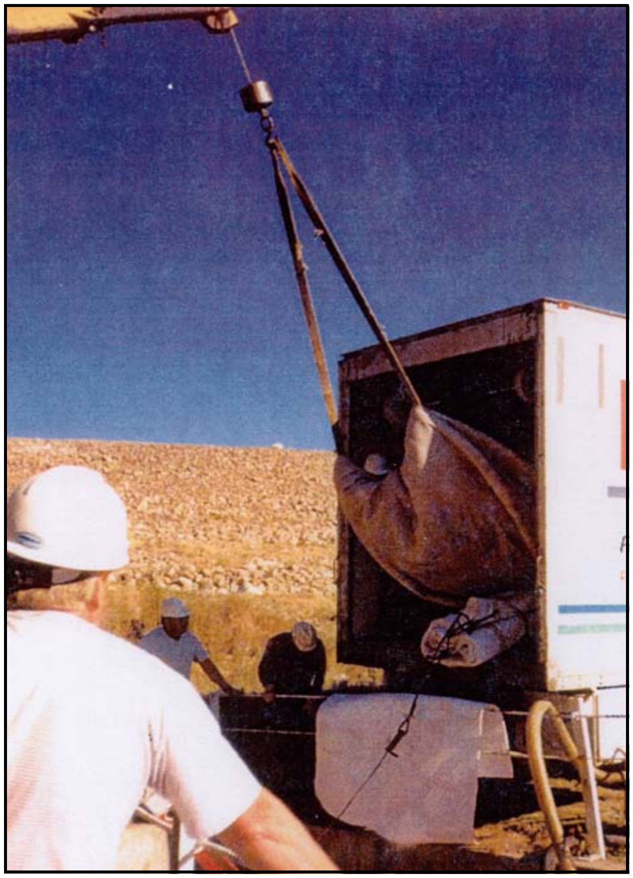

Figure B-80.—Lowering of the CIPP liner into the stilling basin, so it can be winched up the tunnel.

The CIPP installation has been performing satisfactorily since completion of construction.

Lessons learned

- CIPP can be used for conduit renovation.
- Use of CIPP can be applicable to conduits with changes in invert slope. CIPP provides a conduit lining with minor loss of flow cross-sectional area.

Reference

Hepler, Tom, Ron Oaks, and Roger Torres, *Sinkhole Development and Repairs at Willow Creek Dam, Montana*, ASDSO Conference, 1997.

Project name: Wister Dam

Location: Oklahoma

Summary: Near failure of an embankment dam due to internal erosion

The descriptions of this case history are extracted from several articles written by Sherard, including his 1986 article, and an article by Rutledge and Gould (1973). Arthur Casagrande (1950) also discussed this case history.

Lake Wister is located in the San Bois Mountains on the Poteau River in far eastern Oklahoma. The Tulsa District Corps of Engineers designed and built the project. Construction began in April 1946, and the project was placed in full flood control operation in December 1949. The embankment dam is a rolled, impervious earthfill with rock-protected slopes. The embankment dam was constructed as a homogeneous clay fill without a chimney filter. At the time the embankment dam was constructed, chimney filters were not a standard design element in major dams as they are now. The embankment dam is 5,700 feet long and rises to a maximum height of 99 feet above the streambed. Later tests on soils from the embankment dam conclusively demonstrated that the clays were highly dispersive. The bedrock in the area is Pennsylvanian age shale known to commonly produce residual soils with dispersive properties.

Heavy rains caused the reservoir to fill quickly beginning in January 1949. When water had risen to a height of about 60 feet, muddy water was seen discharging on the downstream slope of the embankment dam. The quantity of flow was initially estimated at 2,200 gal/min, and it increased to about 8,000-9,000 gal/min in the next several days. The spillway radial gates were opened, and within 3 days, the reservoir had dropped about 13 feet. This exposed tunnels on the upstream slope, through which the water was entering the embankment dam. The tunnels were about 2 feet in diameter and extended along the upstream face of the embankment dam for a distance of about 300 feet, at about the same elevation on the slope. Dye was injected into a vortex on the upstream slope, and the test showed the water was flowing along a nearly horizontal seam in the embankment dam for a distance of about 740 feet, with a head on the tunnel at the time flow began of only about 13 feet (a gradient of a little over 50:1). The dye tests showed the time for flow to travel this distance was less than 13 minutes, a velocity of about 1 foot per second. Figure B-81 below shows an idealized sketch of the embankment cross section with the flow path causing the erosion identified.

After the reservoir level had dropped farther, the erosion tunnels exposed were excavated and plugged, and several remedial measures were implemented, including

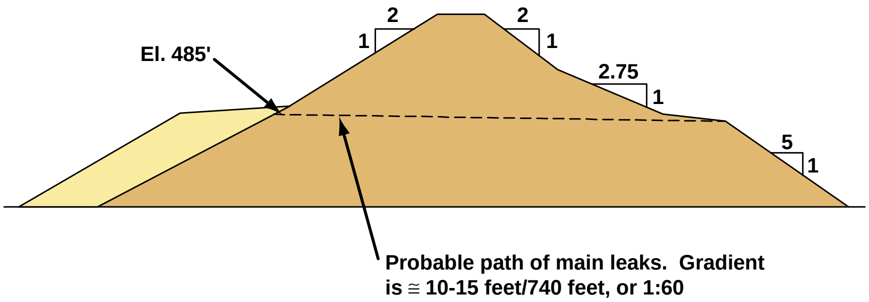

Figure B-81.—Cross section of Wister Dam showing probable path for internal erosion through embankment. The length of the flow path was about 740 feet, and the head on the entrance tunnels was less than 15 feet.

extensive grouting, a steel sheet pile wall, and additional upstream and downstream berms and drains. After completion of the remedial work, the embankment dam has been in operation continuously with little trouble. A major renovation program was finished in 1990. The renovation included a slurry panel wall installed for the full height of the embankment.

Sherard concluded that the cause of the leakage path in the embankment dam could only be attributed to hydraulic fracture in the embankment. The flow path developed immediately above the closure section on the embankment dam, also just above the old stream channel. Aggravating this condition was the fact that the right bank of the stream channel (viewed downstream) consisted of a bedrock shelf that contributed to differential settlement. A plan view of the area of the leak is shown in figure B-82. A longitudinal profile of the area where the leak developed is shown as figure B-83. Note that this view is looking upstream.

The embankment dam was compacted to a relatively high dry density corresponding to about 97 percent of the maximum Standard Proctor dry density, and it was compacted at about optimum water content. The soils were silty, relatively low in plasticity, and highly dispersive. Compaction at these conditions probably resulted in a somewhat brittle fill likely to crack when subjected to differential settlement.

Lessons learned

The incident demonstrated that even well constructed embankment dams built by what was then state-of-the-art technology are susceptible to hydraulic fracture and internal erosion if they are not protected by internal chimney filters. Modern embankment dam design concepts include protective filters for all significant and high hazard embankment dams, and even low hazard dams if constructed of

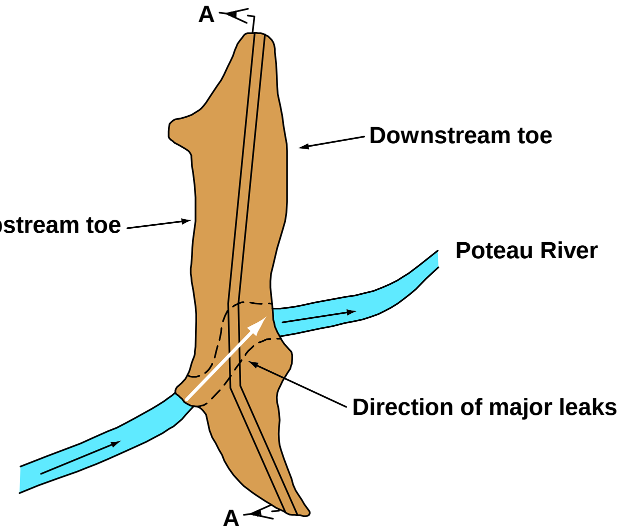

Figure B-82.—Plan View of Wister Dam showing flow path for internal erosion tunnels in fill.

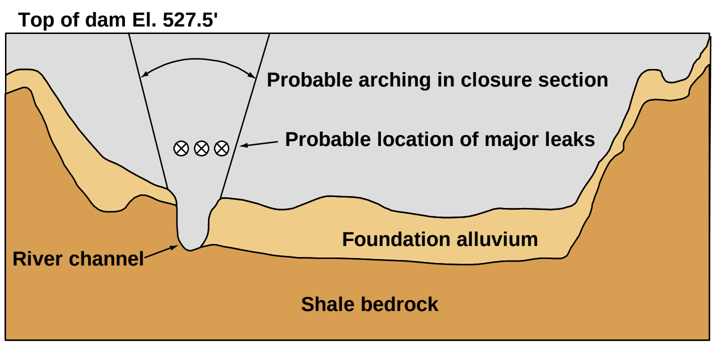

Figure B-83.—Cross section A-A from figure B-82. Profile along centerline of embankment viewed upstream.

problematic soils, such as dispersive clays. This case history also illustrates the increased potential for arching and hydraulic fracture associated with closure sections in embankment dams.

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Rutledge, Philip, and James P. Gould, *Embankment Dam Cracking*, Casagrande Volume, John Wiley and Sons, New York, 1973, pp. 272-353.

Sherard, James L., "Hydraulic Fracturing in Embankment Dams," *Journal of Geotechnical and Geoenvironmental Engineering*, October, 1986.