Technical Manual: Plastic Pipe Used in Embankment Dams

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

November 2007

FEMA

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Federal Emergency Management Agency

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Preface

Plastic pipe has been used for many decades in water and sewer applications. More recently, plastic pipe has been used in new embankment dam construction and in the renovation of existing conduits. However, most of the available design information is targeted toward water distribution and sewer pipes and does not address the unique factors involved in using plastic pipe in embankment dams. In general, information on plastic pipe is too dispersed for the best use of lessons learned from past performance, and compilation of information into a more readily available source was needed. Due to the absence of any single recognized standard for plastic pipe used in embankment dams, there is significant inconsistency in the design and construction rationale. 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. The goal of this document is to provide a single, nationally recognized standard to promote greater consistency between similar project designs, facilitate more effective and consistent review of proposed designs, and result in increased potential for safer, more reliable facilities.

This document is intended to supplement the plastic pipe information in the Federal Emergency Management Agency's (FEMA) *Technical Manual: Conduits through Embankment Dams* (2005). This document provides in-depth analyses of loading conditions, structural design, and hydraulic design of plastic pipe.

This document attempts to condense and summarize the body of existing information, provide a clear and concise synopsis of this information, and present a recommended design approach. The authors reviewed most of the available information on plastic pipe as it relates to use within embankment dams in preparing this document. Where detailed documentation exists, they cited it to avoid duplicating available materials. The authors have strived not to reproduce information that is readily accessible in the public domain. Where applicable, the reader is directed to selected portions of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) and other consensus-accepted references for additional guidance. 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 preparing this document, the authors frequently found conflicting procedures and standards in the many documents 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 differ from some of the participating agencies' own policies.

Since this is a supplemental document, the authors adopted the same approach toward hazard potential classification as used in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). The reader is directed to that document for a complete discussion of hazard potential classification. The hazard potential classification does not reflect in any way on the current condition of the dam (i.e., safety, structural integrity, or flood routing capacity). The three hazard potential classification levels used in this document are low, significant, and high as defined in FEMA 333, *Federal Guidelines for Dam Safety: Hazard Potential Classification Systems for Dams* (1998):

- *Low hazard potential*.—Embankment dams assigned the low hazard potential 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 owners' property.

- *Significant hazard potential*.—Embankment dams assigned the significant hazard potential classification are those 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 potential 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)

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

FEMA, as the lead agency for the National Dam Safety Program, sponsored development of this document in conjunction with the Association of State Dam Safety Officials, Bureau of Reclamation, Mine Safety and Health Administration, Natural Resources Conservation Service, and U.S. Army Corps of Engineers. The primary authors of this document are Wade Anderson, P.E. (Natural Resources Conservation Service), Chuck Cooper, P.E. (Bureau of Reclamation), John Fredland, P.E. (Mine Safety and Health Administration), Michele Lemieux, P.E. (Montana Department of Natural Resources and Conservation), Mark Pabst, P.E. (Bureau of Reclamation), David Pezza, P.E. (U.S. Army Corps of Engineers), and Hal Van Aller, P.E. (Maryland Department of the Environment). The technical editor for this document was Lelon A. Lewis (Bureau of Reclamation). Illustrators for this document were Bonnie Gehringer (Bureau of Reclamation), John Markley (Bureau of Reclamation), and Wendy Pierce (Natural Resources Conservation Service). Additional technical assistance was provided by Cynthia Fields (Bureau of Reclamation), Cindy Gray (Bureau of Reclamation), and Gia Price (Bureau of Reclamation).

Peer review of this document was provided by Darren Blank P.E. (Mine Safety and Health Administration), Kurt Hafferman P.E. (Montana Department of Natural Resources and Conservation), Bruce Harrington P.E. (Maryland Department of the Environment), Greg Hughes P.E. (U.S. Army Corps of Engineers), John LaBoon P.E. (Bureau of Reclamation), Danny McCook P.E. (Natural Resources Conservation Service), Chuck Redlinger, P.E. (Bureau of Reclamation), Greg Reichert P.E. (URS Corporation), Sal Todaro P.E. (URS Corporation), and Ken Worster P.E. (Natural Resources Conservation Service).

The National Dam Safety Review Board (NDSRB) reviewed this document prior to issuance. The NDSRB plays an important role in guiding the direction of the National Dam Safety Program. 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 State implementation of the assistance program. The NDSRB consists of five representatives appointed from federal agencies, five State dam safety officials, and one representative from the U.S. Society on Dams.

A number of additional engineers and technicians provided input in preparation of this document, and the authors greatly appreciate their efforts and contributions. The authors also extend their appreciation to the following agencies and individuals for graciously providing additional reviews, information, and permission to use their materials in this publication:

Advanced Drainage Systems, Inc., Jim Goddard and Robert Slicker
American Concrete Institute
American Society of Civil Engineers
American Water Works Association
Association of State Dam Safety Officials
ASTM International
Bureau of Reclamation, Mark Baker, Richard D. Benik, Richard Fuerst, Mark Gemperline, Ernest Hall, Walter Heyder, Steven Robertson, and Jay Swihart
Bureau of Indian Affairs
Federal Emergency Management Agency
Florida Department of Environmental Protection, Steve Partney
Geo/Environmental Associates, Inc., Barry Thacker
Amster Howard
Inuktun Services, Ltd.
ISCO Industries, Dudley Burwell
Knight Piesold, Allen H. Gipson Jr.
Maryland Department of the Environment
Mine Safety and Health Administration, Carol L. Tasillo
Montana Department of Natural Resources and Conservation
Montana Tech. of the University of Montana, Rich McNearny
Natural Resources Conservation Service, Bill Irwin
Jim Norfleet
Robert Peccia and Associates, Robert Peccia
Performance Pipe, Larry Petroff
Plastic Pipe Institute
Simpson Gumpertz & Heger Inc., Timothy J. McGrath
Tetra Tech. Inc., Larry Cawlfield and Mike Hatten
Uni-Bell PVC Pipe Association, Michael Luckenbill
URS Corporation, Scott Jones
U.S. Army Corps of Engineers, Ed Ketchum, Terry J. Matuska, and Kevin L Pavlik
Utah State University, Steve Folkman, A.P. Moser, Blake Tullis, and Reynold K. Watkins
Virginia Tech University, Michael J. Duncan

Designers must continue to explore the advantages and limitations of plastic pipe. 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 when plastic pipe is used within an embankment dam, the designer must be experienced with all aspects of the design and construction of these structures.

The authors caution the users of this document 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.

The authors utilized many sources of information in the development of this document, including:

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

This document is available in two formats: print copy and digital versatile disc (DVD). The DVD format includes built-in Adobe Acrobat Reader software, hyperlinks, and search capabilities. A hyperlink is a highlighted word or image within the document which, when clicked, takes the user to another place within the document or to another location altogether. Hyperlinks are especially useful when the user wants to see the full reprint of a cited reference or the exact location in a reference from which the material was cited. The available document formats and a description of their contents are as follows:

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- DVD.—The DVD contains this document, portable document format (PDF) copies of the cited references that were available in the public domain or where permission for reprint was granted, plus "additional reading" references in PDF format. The "additional reading" references are references that have not been specifically cited in this document, but may be of additional interest to the user. As DVD drives become more common, the DVD format will eventually become the preferred format for all users, since it allows the user to utilize all the available features.

Suggestions for changes, corrections, or updates to this document should be directed to:

Bureau of Reclamation
Denver Federal Center, Bldg. 67
6th Avenue and Kipling
Denver CO 80225-0007
Attention: Chuck Cooper (86-68130)

Please reference specific pages, paragraphs, or figures within the document, together with proposed new material in any convenient format. Sources of proposed new material should be completely cited. Submission of material signifies permission for use in a future revised edition of this document, but credit for such new material will be given where appropriate.

Contents

Tables

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Appendix A

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Common Abbreviations

AASHTO, American Association of State Highway and Transportation Officials
ABS, acrylonitrile-butadiene-styrene
ACI, American Concrete Institute
ADS, Advanced Drainage Systems, Inc.
AGI, American Geological Institute
ASCE, American Society of Civil Engineers
ASDSO, Association of State Dam Safety Officials
ASTM, ASTM International
AWWA, American Water Works Association
CANDE, Culvert Analysis and Design
CCFRPM, centrifugally cast fiber reinforced polymer mortar
CCTV, closed circuit television
CIPP, cured in place pipe
CLSM, controlled low strength material
CLSM-CDF, controlled low strength material—controlled density fill
CMP, corrugated metal pipe
DOS, disk operating system
DR, dimension ratio
DVD, digital versatile disc
EM, embedment/encasement material
ESC, environmental stress cracking
F, Fahrenheit
FEMA, Federal Emergency Management Agency
FFP, fold and formed pipe
FHWA, Federal Highway Administration
FS, factor of safety
HDB, hydrostatic design basis
HDPE, high density polyethylene
HDS, hydrostatic design stress
LL, liquid limit
MSA, maximum size aggregate
NAWIC, National Association of Women in Construction
NCHRP, National Cooperative Highway Research Program
NCLS, notched constant ligament stress
NDSP, National Dam Safety Program
NDSRB, National Dam Safety Review Board
NRCS, Natural Resources Conservation Service

PB, polybutylene
PC, pressure class
PDF, portable document format
PE, polyethylene
P.E., Professional Engineer
PI, plasticity index
PM, pipe material
PP, polypropylene
PPI, Plastic Pipe Institute
PR, pressure rating
PS, pipe stiffness
PUR, polyurethane
PVC, polyvinyl chloride
ROV, remotely operated vehicle
SCC, self consolidating concrete
SCR, stress crack resistance
SCS, Soil Conservation Service
SDR, standard dimension ratio
SI, International System of Units
SSHB, Standard Specifications for Highway Bridges
SIDR, standard inside dimension ratio
UP, unsaturated polyester
USACE, U.S. Army Corps of Engineers
USCS, Unified Soil Classification System
USSD, United States Society on Dams
USU, Utah State University
UV, ultraviolet

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	1233.489	cubic meters
cubic feet	0.028317	cubic meters
cubic feet per second	0.028317	cubic meters per second
cubic inches	16.38706	cubic centimeters
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
miles	2.589988	kilometers
mils	0.000025	meters
mils	0.025400	millimeters
pounds	0.453592	kilograms
pounds per cubic foot	16.01846	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	6894.757	pascals
square feet	0.092903	square meters
square inches	6.451600	square centimeters

Symbols

$\%\Delta Y/D$, percent deflection
α, coefficient of thermal expansion, in/in/°F
γ, total unit weight of soil, lb/ft^3
γ_b, buoyant unit weight of soil, lb/ft^3
γ_m, moist unit weight of soil, lb/ft^3
γ_s, saturated unit weight of soil, lb/ft^3
γ_w, unit weight of water, 62.4 lb/ft^3
ΔH, increase in dam height, ft
ΔH, surge pressure, feet of water
ΔP_s, increase in soil loading due to dam raise, lb/ft^2
ΔP, surge pressure, lb/in^2
ΔT, change in temperature, °F
ΔV, change in velocity of water, ft/s
$\Delta Y/D_M = \%\Delta X/D$, percent deflection expressed as a decimal
ε, maximum combined strain in pipe wall, in/in of pipe wall circumference
ε_{all}, allowable strain for the pipe material, in/in
ε_f, maximum strain in the pipe wall due to ring deflection, in/in
ε_h, maximum strain in the pipe wall due to hoop stress, in/in
η, porosity, percent of void volume, %
μ, coefficient of friction, tan ϕ
ϕ, effective friction angle of backfill
ρ, density of water, slugs/ft^3
σ, allowable long-term compressive stress, lb/in^2
a, percentage of soil passing the No. 200 sieve, fines content
a, velocity of the pressure wave, ft/s
A, filter or foundation area through which flow passes, ft^2
A_{pw}, area of the pipe wall, in^2/in of pipe length
A_R, required area of the end restraint, ft^2
B', empirical coefficient of elastic support
c, distance from the inside surface to the neutral axis, in
C, constant ranging from 0.2 to 0.6, averaging 0.35
C, reduction factor for buckling pressure
C_c, positive projection load coefficient
c_u, coefficient of uniformity, D_{60} / D_{10}
D_{10}, particle size diameter in millimeters of the 10th percentile passing grain size

D_{15}, particle size diameter in millimeters of the 15th percentile passing grain size
D_{50}, particle size diameter in millimeters of the 50th percentile passing grain size
D_{60}, particle size diameter in millimeters of the 60th percentile passing grain size
D_{85}, particle size diameter in millimeters of the 85th percentile passing grain size
D_{15}B, particle size diameter in millimeters of the 15th percentile passing grain size of the base soil
D_{85}B, particle size diameter in millimeters of the 85th percentile passing grain size of the base soil
D_{15}E , particle size diameter in millimeters of the 15th percentile passing grain size of the envelope
D_{85}E, particle size diameter in millimeters of the 85th percentile passing grain size of the envelope
D_{10}F, particle size diameter in millimeters of the 10th percentile passing grain size of the filter
D_{15}F, particle size diameter in millimeters of the 15th percentile passing grain size of the filter
D_{85}F, particle size diameter in millimeters of the 85th percentile passing grain size of the filter
D_i, inside diameter of the pipe, in
D_L, deflection lag factor
D_M, mean pipe diameter, in
D_O, outside diameter of the pipe, ft
e, base of natural logarithms, 2.7183
E, modulus of elasticity of pipe material, lb/in^2
E, short-term modulus of elasticity of pipe material, lb/in^2
E', modulus of soil reaction, lb/in^2
F, force due to expansion/contraction of the pipe, lb
FS, factor of safety
g, acceleration due to gravity, 32.2 ft/s^2
h, height of fill above the top of pipe, in
h_w, height of water above the top of the pipe, ft
H, height of soil above the top of the pipe, ft
H_e, height of plane of equal settlement above the top of the pipe, ft
H_i, initial height of existing dam, ft
HDB, hydrostatic design basis of the pipe, lb/in^2
HDS, hydrostatic design stress, lb/in^2
i, hydraulic gradient, head loss outside the pipe divided by the distance over which that head loss occurs, ft/ft
I_{pw}, pipe wall moment of inertia, in^4/in of pipe length
K, bedding constant (typically 0.1 for soil embedment)
K_L, bulk modulus of water, lb/in^2
K, Rankine's active lateral earth pressure coefficient, $\tan^2(45-\phi/2)$
k, coefficient of permeability of the surrounding filter or foundation, whichever is greater, ft/yr

L, distance within the pipe that a pressure wave moves before it is reflected back by a boundary condition, ft
LL, liquid limit, %
P, design pressure, lb/in^2
p, projection ratio
PC, pressure class, lb/in^2
P_{CR}, unconstrained collapse pressure, lb/in^2
P_G, external hydrostatic pressure, lb/ft^2
PI, plasticity index, %
PR, pressure rating, lb/in^2
P_s, pressure due to weight of soil on top of pipe, lb/ft^2
PS, pipe stiffness, lb/in^2
P_V, internal vacuum pressure, lb/in^2
P_W, pressure on the pipe from a wheel load, lb/ft^2
q_a, allowable buckling pressure, lb/in^2
q_{All}, allowable soil bearing capacity, lb/ft^2
q_{ar}, reduced allowable buckling pressure, lb/ft^2 or lb/in^2
Q, rate of flow of water into a drainpipe, ft^3/yr
r, mean pipe radius, in
r_{sd}, settlement ratio
R_w, water buoyancy factor
SDR, standard dimension ratio of pipe, D_0/t
S_{EC}, stress due to temperature change, lb/in^2
SIDR, standard inside dimension ratio
t, wall thickness of the pipe, in
t_w, top width of existing dam crest, ft
T_{CR}, critical time, s
T_{pw}, thrust in pipe wall, lb/in
v, Poisson's ratio
w/c, water-cement ratio by volume
W_c, soil load, lb/linear foot of pipe
W_L, wheel load, lb
W_v, vacuum load per linear foot of pipe, lb/ft

AASHTO Standards

AASHTO Standard	Title
M252	Corrugated Polyethylene Drainage Pipe.
M294	Corrugated Polyethylene Pipe, 300- to 1200-mm Diameter
SSHB	Standard Specifications for Highway Bridges
T99	Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop

ASTM Standards

ASTM Standard	Title
C 33	Standard Specification for Concrete Aggregates
C 117	Standard Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing
C 136	Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates
C 150	Standard Specification for Portland Cement
C 618	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
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 kNm/m^3])
D 1504	Standard Specification for Folded Poly(Vinyl Chloride) (PVC) Pipe for Existing Sewer and Conduit Rehabilitation
D 1556	Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method
D 1785	Standard Specification for Poly(Vinyl Chloride) (PVC) Plastic Pipe, Schedules 40, 80, and 120
D 2241	Standard Specification for Poly (Vinyl Chloride) (PVC) Pressure-Rated Pipe (SDR Series)
D 2321	Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications

D 2412	Standard Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading
D 2434	Standard Test Method for Permeability of Granular Soils (Constant Head)
D 2487	Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)
D 2657	Standard Practice for Heat Fusion Joining of Polyolefin Pipe and Fittings
D 2837	Standard Test Method for Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials or Pressure Design Basis for Thermoplastic Pipe Products
D 2922	Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)
D 3034	Standard Specification for Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings
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 4318	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D 4439	Standard Terminology for Geosynthetics

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
F 412	Standard Terminology Relating to Plastic Piping Systems
F 477	Standard Specification for Elastomeric Seals (Gaskets) for Joining Plastic Pipe
F 679	Standard Specification for Poly(Vinyl Chloride) (PVC) Large-Diameter Plastic Gravity Sewer Pipe and Fittings
F 714	Standard Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Outside Diameter
F 794	Standard Specification for Poly(Vinyl Chloride) (PVC) Profile Gravity Sewer Pipe and Fittings Based on Controlled Inside Diameter
F 894	Standard Specification for Polyethylene (PE) Large Diameter Profile Wall Sewer and Drainpipe
F 949	Standard Specification for Polyvinyl Chloride (PVC) Corrugated Sewer Pipe With a Smooth Interior and Fittings
F 1216	Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube
F 1668	Standard Guide for Construction Procedures for Buried Plastic Pipe
F 1743	Standard Practice for Rehabilitation of Existing Pipelines and Conduits by Pulled-in-Place Installation of Cured-in-Place Thermosetting Resin Pipe (CIPP)
F 1803	Standard Specification for Poly (Vinyl Chloride)(PVC) Closed Profile Gravity Pipe and Fittings Based on Controlled Inside Diameter

F 2164	Standard Practice for Field Leak Testing of Polyethylene (PE) Pressure Piping Systems Using Hydrostatic Pressure
F 2306	Standard Specification for 12 to 60 in. [300 to 1500 mm] Annular Corrugated Profile-Wall Polyethylene (PE) Pipe and Fittings for Gravity-Flow Storm Sewer and Subsurface Drainage Applications

AWWA Standards

AWWA Standard	Title
C900	Polyvinyl Chloride (PVC) Pressure Pipe, and Fabricated Fittings, 4 - 12 in. (100-300 mm), for Water Dist.
C901	Polyethylene (PE) Pressure Pipe and Tubing, ½ in. (13 mm) Through 3 in. (76 mm), for Water Service
C905	Polyvinyl Chloride (PVC) Pressure Pipe and Fabricated Fittings, 14 - 48 in. (350-1,200 mm)
C906	Polyethylene (PE) Pressure Pipe and Fittings, 4 in. (100 mm) Through 63 in. (1,575 mm), for Water Dist. and Trans.

Websites

The following websites can provide additional information and publications related to plastic pipe, embankment conduits, drainpipes, and embankment dams:

American Society of Civil Engineers: http://www.asce.org

American Society of Civil Engineers Publications: http://www.pubs.asce.org

Association of State Dam Safety Officials: http://www.damsafety.org

Bureau of Reclamation: http://www.usbr.gov

Bureau of Reclamation Publications:
http://www.usbr.gov/pmts/hydraulics_lab/pubs/index.cfm

Canadian Dam Association: http://www.cda.ca

Federal Emergency Management Agency:
http://www.fema.gov/plan/prevent/damfailure

Federal Emergency Management Agency Publications:
http://www.fema.gov/plan/prevent/damfailure/publications.shtm

Federal Energy Regulatory Commission:
http://www.ferc.gov/industries/hydropower.asp

International Commission on Large Dams: http://www.icold-cigb.org

Mine Safety and Health Administration: http://www.msha.gov

National Performance of Dams Program: http://npdp.stanford.edu

Natural Resources Conservation Service: http://www.nrcs.usda.gov/technical/eng

Natural Resources Conservation Service Publications:
http://www.info.usda.gov/ced

Plastic Pipe Institute: http://www.plasticpipe.org

U.S. Army Corps of Engineers: http://www.usace.army.mil

U.S. Army Corps of Engineers Publications:
http://www.usace.army.mil/publications

United States Society on Dams: http://www.ussdams.org

Uni-Bell PVC Pipe Association: http://www.uni-bell.org

Introduction

Plastic pipe used in embankment dams serves different purposes than pipe used in water and sewer applications. Failure of plastic pipe in water and sewer applications rarely results in loss of life. However, failure of plastic pipe in dams can have catastrophic consequences. Removal and replacement can be difficult, time consuming, and costly. Plastic pipe used in dams must be conservatively designed to provide for a long service life, strength to accommodate all loading conditions and foundation movements, and have adequate access for cleaning and inspection. For a discussion of the importance of good design and construction and the ramifications that can result if these are lacking, see the *Introduction* of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

Recommendations in this document are based on well founded engineering principles fundamental to the safety dams. However, in some cases, a distinction is made between plastic pipe used in significant/high hazard potential dams and low hazard potential dams. Significant and high hazard potential dams require stringent and conservative design measures, because failure or misoperation could result in loss of human life or economic damages. Generally, this is not the case for low hazard potential dams. While low hazard potential dams could certainly benefit from the design measures discussed in this document, some measures may be considered overly conservative for this type of structure. The designer of low hazard potential dams needs to carefully consider the requirements of their particular application.

Plastic pipe has been used in the construction and renovation of conduits and drainpipes within embankment dams (i.e., earthfill and rockfill) since about the 1980's. The term "conduit" as used in this document refers to conduits used for outlet works, spillways, and siphons in embankment dams. These types of conduits regulate or release water impounded by the dam and are grouped together as "embankment conduits." The term "drainpipe" is used to refer to toe drains that act as a downstream extension of the dam's internal drainage system to collect and transport seepage passing through the dam or foundation to a desired outfall location. Plastic pipe has also been used for decants and drainpipes in tailings disposal and slurry impoundment facilities since about 1980.

Plastic pipe is lightweight, abrasion resistant, and inert to most forms of chemical attack. This facilitates installation and benefits durability and service life. Plastic pipe is often used in toe drain systems for collecting and measuring seepage and safely discharging it into a channel located downstream from the dam. Plastic pipe is commonly used for toe drain construction, since it is relatively inexpensive, readily available in many diameters, can be manufactured with slots or perforations, and can be rapidly installed (figure 1). Another frequent use of plastic pipe is for the sliplining of deteriorating outlet works conduits. Plastic pipe is preferred for sliplining due to its ease of installation, ability to re-establish the watertightness of the conduit, and improved hydraulic performance.

Dam designers and dam safety officials often rely upon precedent and recognized guidelines to design critical features of dams; therefore, many dam designers and dam safety officials have been reluctant to use plastic pipe. Currently, the primary source of design information for plastic pipe is from manufacturers. However, most of this information is targeted to sewer and water pipe installations and does not address the unique factors involved in using plastic pipe within embankment dams. Most dam designers have never had training on the behavior of plastics and must weigh decisions on the use of plastic pipe by considering the initial costs, operating requirements, maintenance costs, dependability, and long-term performance. Some State dam safety officials have attempted to address the use of plastic pipe in their policies and regulations since the early 1990's. Their efforts have resulted in imposing various design requirements, including reinforced concrete encasement, restrictions on the use of plastic pipe, and use restrictions based upon dam hazard classification. However, because of the many potential benefits, more projects are being designed and constructed using plastic pipe. The manufacture of plastic pipe will continue to evolve, based on the requirements of the engineering community. Continued improvements in manufacturing processes will provide products with enhanced strength, durability, and efficiency. This document is intended to serve as a guide for dam designers and dam safety officials to address the unique design requirements of plastic pipe used in dams for embankment conduits and drainpipes. This document provides the reader with detailed procedures for design, inspection, maintenance, renovation, and repair for plastic pipe applications used in embankment dams.

This document specifically addresses plastic pipe applications involving embankment conduits and drainpipes in traditional water-retention embankment dams. The information in this document also applies to the design and use of plastic pipe for conduits and drainpipes in tailings or mine waste-disposal impoundments. However, chapter 7 discusses how the unique characteristics of these impoundments can affect the design of plastic pipe when used for this application.

This document does not address other uses of plastic pipe often associated with embankment dams, such as instrumentation (e.g., piezometer riser pipes), relief wells (relief wells are considered part of the foundation drainage system), and structure underdrains (i.e., drains located under spillway floor slabs). Also, this document does not address plastic pipe used to deliver tailings or slurry to a mine-waste-disposal impoundment. However, some portions of this document may have limited applicability to these uses of plastic pipe.

Figure 1.—Plastic pipe is lightweight, which facilitates installation.

Flat drains (edge drains) may have very limited application for drainpipes within embankment dams where overburden depths are small and future access is not a problem. However, concerns exist that the geotextile fabric wrapped around the drain has the potential for clogging, rendering the drain ineffective. Due to concerns with the potential for clogging, numerous inspection difficulties associated with flat drains, unknown performance under large fill loads, and the lack of precedent for use, they will not be addressed further in this document. Another recent innovation involving plastic pipe that will not be discussed in this document includes prefabricated riser intake structures. These prefabricated units are typically used to replace deteriorated corrugated metal pipe (CMP) risers.

New and improved plastic pipe products are continuously being developed. Some may have potential applicability for use in embankment dams and others may not. For any new plastic pipe product without a proven record of successful use in embankment dams, the designer must exercise a cautious approach and closely evaluate all the characteristics and properties of the particular pipe. A number of research needs are presented in chapter 8 to better understand the performance of plastic pipe and embedment/encasement materials used in embankment dam applications.

Chapter 1

General

Many types of plastic pipe are available from manufacturers and suppliers. However, certain types of plastic pipe are preferred for use within embankment dams due to their ability to accommodate a variety of internal and external loading conditions that may be experienced during the service life of the project. This document is intended to address parameters unique to plastic pipe and its applications within dams. The designer should understand that design criteria for plastic pipe used in dams differ from criteria used in design of plastic pipe in other types of applications, such as municipal water distribution and sewers. The most significant differences are the limited accessibility should something go wrong and the resulting potential impacts to downstream populations. Plastic pipe used in dams is often buried deeply where access is nearly impossible due to the amount of overburden existing above it and the existence of a reservoir pool. For these reasons, dam designers considering the use of plastic pipe must be cautious and select pipe that meets or exceeds conservative design criteria affecting watertightness, durability, structural performance, and design life.

This chapter discusses the history, common types of plastic pipe, and their advantages and disadvantages for use in the construction of embankment conduits and drainpipes within embankment dams. Chapters 2 and 3 provide guidance on loading conditions and structural/hydraulic design.

1.1 Historical Perspective

Plastic pipe has been commonly used for embankment dam drainage systems (e.g., drainpipes) since the early 1970's. Drainage applications in dams are typically nonpressurized. The use of plastic pipe for embankment conduit applications (e.g., outlet works and spillways) within traditional earthen dams is less common. Plastic pipe has been used in the construction and modification of embankment conduits since the early 1990's. These types of applications can either be pressurized or nonpressurized. Typically, the designs for embankment conduits have been prepared without the use of a nationally recognized guideline and by default, have largely been based upon manufacturers' information developed for differing and less critical applications.

The mining industry has used plastic pipe in dams since the mid-1980's for decant pipes, internal-drain collector pipes, and delivery pipes for slurry or tailings disposal. As with embankment conduits, no nationally recognized design guideline is available for this type of application.

While no standardized guidelines exist for the design of plastic pipe used for embankment conduits and drainpipes, numerous codes, standards, and recommended practices do exist that regulate and influence the plastic pipe industry. These publications cover a wide range of product performance requirements, materials, manufacture, and test methods related to plastic pipe. ASTM International (ASTM) publishes standard specifications, practices, and test methods. Standard specifications define specific performance and product requirements, standard practices define how a particular activity is to be performed, and standard test methods define how a particular test is to be performed. The American Water Works Association (AWWA) also publishes standards. ASTM and AWWA are consensus standards and are voluntary. They only become mandatory when specified by some user or entity such as a government agency. For example, if an agency specifies that pipe must meet AWWA C900, the finished product specifications found in AWWA C900 must be met. At the same time, the ASTM requirements called out in AWWA C900 also become mandatory. As changes in plastic pipe are made and newer products, applications, or test methods are developed, the standards are revised accordingly. The use of up-to-date publications is strongly advised.

Additional information concerning plastic pipe is available in a number of publications, such as AWWA's *PE Pipe—Design and Installation* (2006) and *PVC Pipe—Design and Installation* (2002), the Plastic Pipe Institute's (PPI) *Handbook of Polyethylene Pipe* (2006), and Uni-Bell PVC Pipe Association's *Handbook of PVC Pipe—Design and Construction* (2001).

1.2 Common Types of Plastic Pipe Used in Embankment Dams

Many types of plastic pipes are available, but not all types should be used in dams. The formulations used for the production of plastic pipe can vary slightly from manufacturer to manufacturer. The designer must specify the type, grade, and class required for each plastic pipe application. Due to the numerous options available, selection of the proper plastic pipe can become a bewildering experience for the designer. Fortunately, many standards, such as those from ASTM and AWWA, have been developed to ensure plastic pipe products have uniform characteristics, regardless of the manufacturer. This section will discuss some of the types of plastic pipe that have been "commonly" used in dams. Section 1.3 discusses how these types of plastic pipes are used in embankment conduit and drainpipe applications. The types of plastic pipes discussed in these sections have been successfully used in the past for applications in dams. If the designer wants to consider other types of plastic pipe not discussed in this document, all design implications must be carefully evaluated. Also, as the industry introduces newer plastic pipe products, the designer will need to carefully determine their applicability for the intended project. The information contained in this document should be used to assist in this determination.

Plastic pipe used in dams primarily consists of two types: thermoplastic and thermoset plastic. The differences between thermoplastic and thermoset plastic pipes are discussed in the following sections. Some of the information in these sections has been adapted from FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). Selected information has been updated where applicable.

1.2.1 Thermoplastic

Thermoplastics are plastics that can be repeatedly softened by heating and hardened by cooling without deterioration of their properties. In thermoplastics, the polymer molecules are not crosslinked (not chemically bonded to other polymer molecules). The molecules not being connected by crosslinks allows the molecules to spread farther apart when the plastic is heated. With the application of heat, thermoplastics may be shaped, formed, molded, or extruded. This is the basic characteristic of a thermoplastic.

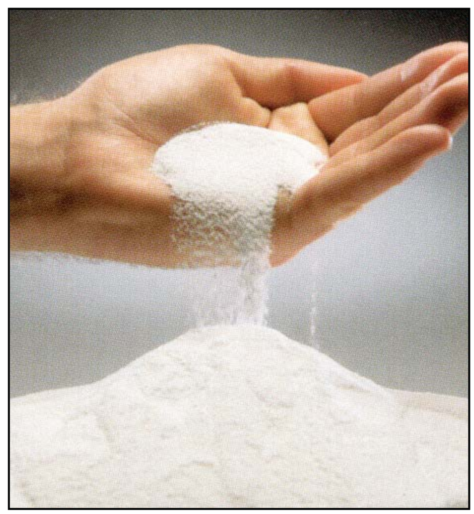


Figure 2.—Resin. Photo courtesy of Uni-Bell PVC Pipe Association Members and Associate Members.

Plastics used for the manufacture of thermoplastic pipe are compounds consisting of resins (figure 2) mixed with additives. Each additive serves a specific purpose, such as (Willoughby, 2002, p. 2.3):

- *Antioxidants.*—Extends the temperature range and service life.
- *Colorants.*—Provides color to the plastic material.
- *Coupling agents.*—Improves the properties of the plastic material.
- *Fibrous reinforcements.*—Improves the strength to weight ratio.
- *Fillers and extenders.*—Improves the properties of the resin.

- *Heat and ultraviolet stabilizers*.—Helps prevent degradation from heat and sunlight.

- *Preservatives*.—Helps prevent bacterial attack on the plastic material.

The formulations, proportions, and actual ingredients used provide the specific properties dictated by the particular application.

The thermoplastics class of materials commonly includes polyethylene (PE), polyvinyl chloride (PVC), acrylonitrile-butadiene-styrene (ABS), polybutylene (PB), and polypropylene (PP). However, the thermoplastics most commonly used in the construction of embankment dams are PE and PVC:

- *Polyethylene*.—Polyethylene pipe is classified into several different categories based mostly on its density and branching. These categories include, among others, low, medium, and high density PE. The mechanical properties of PE depend significantly on variables such as the extent and type of branching, the crystal structure, and the molecular weight. ASTM D 3350 is used to classify polyethylene materials used for piping. High density polyethylene (HDPE) is the most common type of PE used in dam construction.

- *Polyvinyl chloride*.—Polyvinyl chloride pipe is classified into several categories: pressure class (AWWA C900), pressure rating (ASTM D 2241 and AWWA C905), schedule 40, 80, and 120 (ASTM D 1785), and nonpressure (ASTM D 3034). The pressure class and rating products offer a pressure capacity independent of pipe size, whereas the schedule product pressure ratings vary between different pipe diameters.

The general properties, advantages, and disadvantages of HDPE and PVC pipe in dam construction are discussed in section 1.3.

Thermoplastic pipe is produced by the extrusion process, as illustrated in figure 3. The extrusion process produces an inherently strong finished product. 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. The speed at which the molten material is drawn away from the extruder determines the wall thickness. After a number of additional processes, such as cooling of the extruded pipe, the final product can be handled without distortion and can be cut into the specified pipe lengths. The process described here is typically used for solid wall pipe. Additional steps are required in the manufacturing process for adding corrugations or belling the ends of the pipe. For example, to add the bell end to a PVC pipe, one end of the PVC pipe is reheated and placed into a belling machine to enlarge the pipe diameter. The bell is formed by means of a belling mandrel which is slipped through the heated end of the pipe to enlarge it and shape it into the bell. In

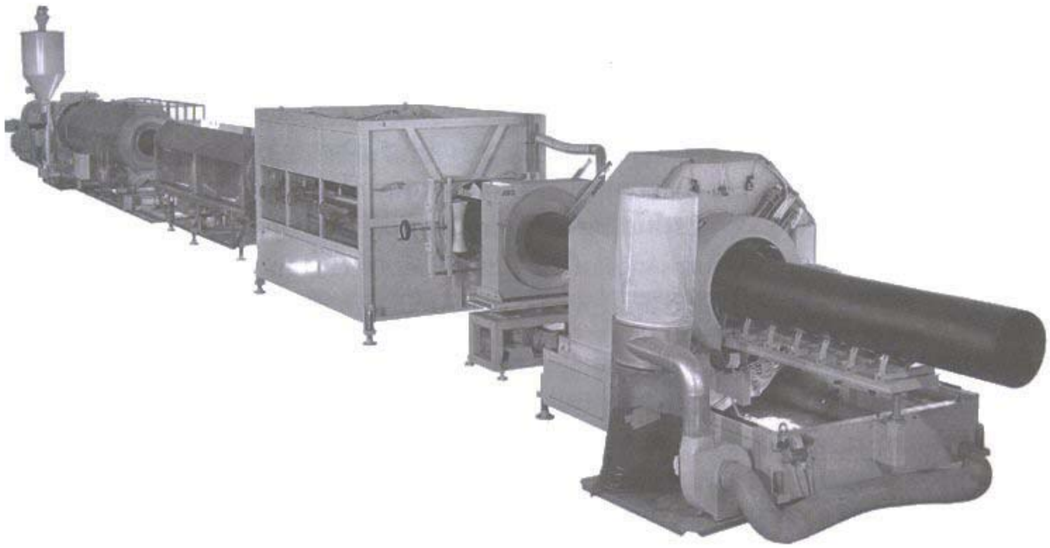


Figure 3.—Conventional extrusion line. Photo courtesy of the Plastic Pipe Institute.

this machine, the bell is formed along with a groove for installation of a rubber gasket.

Thermoplastic pipe fittings are required for changes in alignment, size, or connections (e.g., bends, wyes, tees, and reducers). Pipe fittings can be manually fabricated or made by the injection mold process (for nominal diameters of 12 inches or less). Manually fabricated fittings are normally constructed by joining sections of pipe or machined from blocks. Pressure rated fittings are joined by heat fusion. To ensure that manually fabricated fittings have the same exact dimensions and properties as the pipe to which they will be connected, straight lengths of the same type of pipe are used to fabricate the fitting. The straight lengths of pipe are precision cut and joined together using heat fusion to form the fitting.

1.2.1.1 HDPE

Two general classes of HDPE materials are commonly used to make pipe for dam applications. One material (ASTM F 714) is classified by ASTM D 3350 as having a hydrostatic design basis and is suitable for pressure applications. The other material (ASTM D 3035) is classified by ASTM D 3350, but is not pressure rated. This material is used to make corrugated pipe manufactured to American Association of State Highway and Transportation Officials (AASHTO) standards M252 and M294 respectively. In some special cases, corrugated pipe can be made from "pressure rated" material. The four HDPE pipe types, described in the following paragraphs, have been used in dam construction (figure 4 shows cross-sectional illustrations of each type). Other plastic pipe wall configurations exist, but have had very infrequent

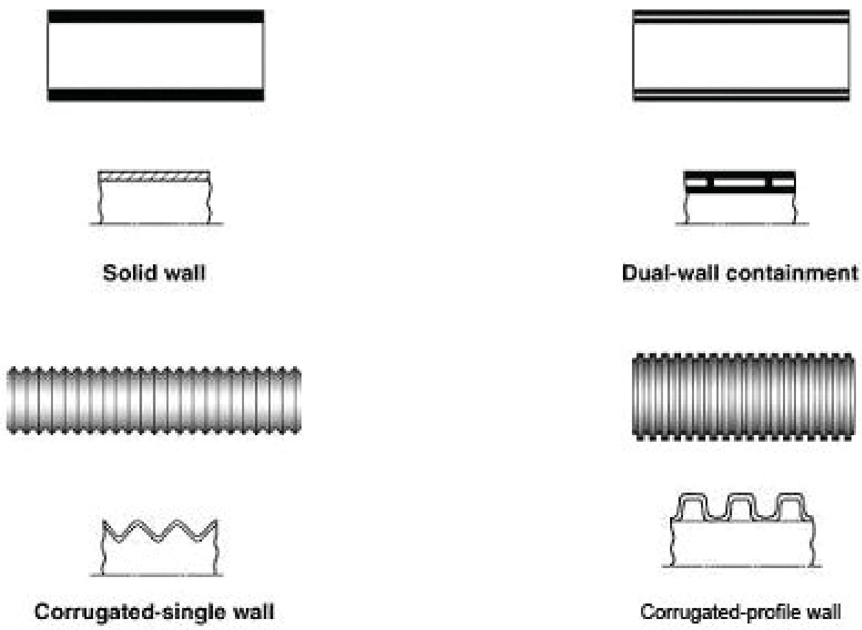


Figure 4.—Types of HDPE pipe walls.

use or have not been used in embankment dam applications. Those pipes will not be discussed in this document.

HDPE plastic pipe used in dam construction includes:

- *Solid wall.*—Solid wall pipe is made of a continuous wall of HDPE with uniform thickness. Solid wall pipe has smooth interior and exterior surfaces. Although solid wall pipe is pressure rated to meet the requirements as specified in ASTM F 714, dual-wall containment pipe should be used for pressurized embankment conduit applications in dams due to its added factor of safety. Solid wall pipe is available in diameters up to about 63 inches in typical lengths of 40 to 50 feet. Figure 5 shows an example of solid wall HDPE pipe.

Figure 5.—Solid wall HDPE pipe to be used for sliplining of an existing outlet works conduit.

- *Dual-wall containment*.—Dual-wall containment pipe is made from two solid wall pipes. Dual-wall containment pipe consists of an inside pipe (carrier pipe) which is centered within an outer pipe (containment pipe). Dual-wall containment pipe should be used use in pressurized embankment conduits, since it affords the added protection of a second pipe. The annular space between the carrier and containment pipes allows for quick detection of leaks in the carrier pipe. The manufacturer can preassemble this type of pipe at the factory, or the pipe can be assembled at the job site using two solid wall pipes. End spacers (centralizers) located at each end of a section of pipe center the carrier pipe within the containment pipe. The end spacers are made to form a tight fit and are extrusion welded in place. Intermediate spacers (known as spiders) are placed at intermediate points between the end spacers to provide additional support. Figure 6 shows an example of a dual-wall containment pipe. The containment pipe and carrier pipe should be pressure rated to meet the requirements as specified in ASTM F 714. Dual-wall containment pipe is available in diameters up to about 54 inches for the carrier pipe and 63 inches for the containment pipe. The Wheatfields Dam Case History in appendix B discusses the use of dual-wall containment pipe for an outlet works conduit renovation.

Figure 6.—Dual-wall containment HDPE pipe. A 14-inch diameter carrier pipe is being inserted into a 20-inch diameter containment pipe. Intermediate spacers are attached to the carrier pipe. Grout lines for grouting of the annulus between the existing conduit and containment pipe can be seen.

- *Corrugated (single wall)*.—Single wall corrugated pipe has corrugated interior and exterior surfaces. This pipe is manufactured using a corrugated cross section for increased strength to allow the pipe wall to support soil loads. Single wall corrugated pipe is distributed in coils (figure 7). Single wall corrugated pipe is available in both perforated and nonperforated products. Perforations can be slots or circular holes. Single wall corrugated pipe is available in diameters up to about 24 inches.

Figure 7.—Single wall corrugated HDPE pipe has corrugations on both interior and exterior surfaces. Photo courtesy of Advanced Drainage Systems, Inc.

Figure 8.—Profile wall corrugated HDPE pipe has smooth interior and corrugated exterior surfaces.

- *Corrugated (profile wall)*.—Profile wall corrugated pipe has a smooth interior surface and a corrugated exterior surface (figure 8). The corrugations add ring stiffness to the pipe to assist in maintaining cross-sectional shape. The smooth interior surface reduces friction and resistance to flow. Profile wall corrugated pipe economizes on the amount of material needed for fabrication; by altering the wall the same stiffness may be achieved with less material. However, not all types of wall corrugation are equal. Parametric studies were conducted by Burgon, Folkman, and Moser (2006) to examine the influence of profile height, shape, and thickness. Results of this research show that the corrugation shape had a dramatic effect on profile stability. Profile wall corrugated pipe is available in both perforated and nonperforated products. Perforations can be slots or holes. This pipe is supplied in standard 20-foot lengths. Profile wall corrugated pipe is available in diameters up to about 60 inches.

HDPE pipe is typically black due to the addition of carbon black during the manufacturing process. The addition of carbon black prevents degradation of the pipe when exposed to ultraviolet (UV) radiation. HDPE pipe is also available in shades of gray to reduce glare and improve conduit inspection using closed circuit television (CCTV) equipment.

The most common method used to join solid wall pipe and dual wall containment pipe is by heat fusion (ASTM D 2657; and PPI, 2005). Although a number of different fusion techniques exist, the butt fusion technique is the most widely used and industry-accepted method for joining sections of HDPE pipe. Butt fusion is typically used to join pipes that have the same nominal outside diameter and wall thickness. Butt fusion is accomplished by heating two surfaces to a designated temperature, and fusing them together by application of sufficient force. The application of force causes the melted materials to flow and mix together. As the joint cools, the molecules return to their crystalline form, the original joint interfaces are gone, and the two pipes have become one homogenous pipe. If performed according to recommended procedures, the fused joint is watertight and as strong or stronger than the HDPE pipe in both tensile and compressive properties. Butt fusion is performed at the site, by an operator who has been trained by an experienced pipe distributor, fusion equipment manufacturer, or pipe manufacturer using a portable fusion machine. Improper operation of the equipment can produce a poor fusion. Six steps are involved in making a properly performed butt fusion

Figure 9.—HDPE pipe joint being butt fusion welded.

joint using a fusion machine (figure 9) (PPI, 2005, p. 14; Performance Pipe, 2006, p. 9):

1. Securely fasten the pipe components into the clamping jaws of the fusion machine, so that they will not move.

2. Face (trim and square off) the ends of the pipe components to establish clean, parallel mating surfaces. Most fusion machines have a rotating planer to perform this task. Poor preparation and any contaminants remaining on the pipe surfaces will produce a poor joint.

3. Align the pipe ends to minimize mismatch of pipe walls.

4. Heat both ends of the pipes (usually to about 400 to 450 °F). The heating tools are integrated into the fusion machine. A melt pattern that penetrates into the pipe must be formed around both pipe ends.

5. Join the ends of the pipe by bringing them together with sufficient pressure to properly mix the molten pipe materials on the ends of the pipe components. A small melt bead will form at the joint on the interior and exterior surfaces of the pipe as the ends are joined. A properly performed fusion will form a double melt bead that is rolled over to the surface on both ends of the pipe. The pipe manufacturer will specify proper pressure required for the thickness and diameter of the pipe.

6. Hold the molten joint together under pressure until it has cooled adequately to develop proper strength. The amount of time required for cooling depends upon the material, pipe diameter, and wall thickness. The manufacturer will specify proper cooling times for their product.

Fusion machines are available for pipe sizes up to 63 inches in diameter. Modern butt fusion machines are hydraulically assisted and semiautomatic requiring only one operator. Hydraulic power is used to operate all fusion functions including the clamping jaws, heater, and facer. Some machines have the capability to record important data, such as heater surface temperature, and heating, fusion, and cooling times. A printed record for each joint can be created to ensure consistency. Trial fusions should be considered at the beginning of the day, so the fusion procedure and equipment settings can be verified for the actual job site conditions. During cold weather, additional time is required to warm up the fusion machine and to heat the ends of the HDPE pipe. A temporary shelter may need to be constructed for joining the sections of HDPE pipe in case of inclement weather to avoid precipitation, wind, and heat loss. For additional cold weather procedures, see ASTM D 2657. Dual-wall containment pipe is typically butt fused together simultaneously or by staggering the welds of the carrier and containment pipes. Manufacturers' recommended procedures should always be observed for butt fusion. HDPE pipe cannot be joined by field threading or solvent bonding.

The need for melt bead removal is uncommon, and has negligible impact on the hydraulic performance of the pipe. If melt bead removal is required, it can be accomplished using special tools after the joint has thoroughly cooled to ambient temperature. Personnel using the debeading tool should be properly trained, so the pipe is not needlessly gouged.

Figure 10.—A butt fused HDPE pipe joint being checked for gaps and voids.

The beads should be thoroughly inspected for uniformity and proper size around the entire joint. Visual inspection criteria should be obtained from the pipe manufacturer. Nondestructive evaluation methods have been performed using ultrasonic equipment to detect voids or other discontinuities. Radiographic methods are considered unreliable because x-rays are a poor indicator of fusion quality. For destructive testing, a bent strap test (ASTM D 2657) can be performed in the field to confirm joint integrity, operator procedure, and fusion machine setup (PPI, 2006, p. 8). Figure 10 shows a joint being tested. The test is easy to perform on thin wall pipes, but can be difficult on thick wall pipes (greater than about 1½ inches). For thicker walled pipes, nondestructive evaluation methods should be considered. Field fusion should not proceed until joint quality on a test sample has been properly evaluated. Use of fusion machine operators who are skilled, knowledgeable, and certified will produce a good joint. Improperly butt fused joints cannot be repaired and must be cut out, and the ends must be properly joined (ASTM D 2657). Upon completion of the repair, the HDPE pipe should be retested for leaks. For guidance on leak testing, see section 3.8.2

Unlike plastic pipe joined by couplers—as in corrugated HDPE, bells and spigots in PVC, or flanged joints—butt fusion creates a continuous joint-free pipe of nearly constant outside diameter. In sliplining applications for embankment conduits, the butt fusion joint does not take up any additional space, so a larger inside diameter slipliner can be used. This is an advantage over bell and spigot pipe or pipe with flanged joints.

Other joining methods for solid wall pipe include:

- *Joints made by extrusion welding.*—Many prefabricated fittings (i.e., elbows, bends, and tees) can be joined to HDPE pipe with heat fusion (ASTM D 3261) in the field using an extrusion gun. Extrusion welding is a manual process utilizing a hand held extruder (figure 11). The process involves continuously extruding molten HDPE onto the plastic components to be joined. The welding gun has the appearance of an electric drill with a small extrusion barrel attached to the front. The extrusion barrel is heated either by cartridge heaters or hot air.

Figure 11.—Hand held extrusion gun.

HDPE rod or granule feedstock is fed into the rear of the extrusion barrel and the material is heated as it is drawn through the barrel. The molten HDPE is continuously ejected through a specially designed shoe attached to the front of the extrusion barrel. At the leading edge of the shoe, hot gas is used to preheat the surfaces where the molten HDPE is to be applied, so a proper weld can be formed. Generally, no further work is required to complete the joint. Typical welding speeds are 1 to 3 feet per minute. Extrusion-welded joints are significantly weaker than butt fusion joints. Weld quality depends upon the skill of the operator. Proper training and certification are required to maintain high standards of fabrication. Extrusion welding has also been successfully used for connecting HDPE grout and air vent pipes to plastic pipe slipliners. Extrusion welding cannot be used to repair damaged HDPE pipe.

Figure 12.—HDPE flange adapter connection.

- *Mechanical joints*.—Mechanical joints are used to join HDPE pipe and fittings to themselves or to other types of pipe materials. The most common mechanical joint is the flange adapter (figure 12). 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. A backup ring should be used with flanged connections. The backup ring is placed behind the HDPE flange. When the flange bolts are tightened, the backup ring compresses against the HDPE flange to the steel pipe flange, providing a seal. Flanged connections allow for easy assembly and disassembly of the joint. Flange joints tend to require more annular space than butt fusion joints. Depending on the application, the use of a gasket may be required with the flange adaptor connection. Other mechanical joining methods, such as couplings, are available from various manufacturers, but have not had much applicability for use in embankment dam construction. Although couplings are meant to allow HDPE connections to other pipe materials, there are special concerns. These include the low coefficient of friction of HDPE making gripping of the outside of the pipe more difficult than for other materials and the need for internal stiffeners.

- *Snap joints*.—This type of patented joint is used in ISCO's Snap-Tite pipe joining system and consists of solid wall HDPE pipe specially machined to form two grooves around the circumference on both ends of the pipe section. The grooves on the male end are on the exterior surface, and the grooves on the female end are on the interior surface. Each new piece of pipe is snapped onto the proceeding pipe. A lubricant and gasket is normally used with this type of joint. Snap joints allow sections to be easily joined using chains wrapped around the pipe, come-alongs, and a backhoe. This type of pipe joint has been used in sliplining of nonpressurized embankment conduits in low hazard potential dams, but should not be used in significant or high hazard potential applications. Figure 13 shows an example of the male end of a snap joint.

Figure 13.—Male end of snap joint.

Corrugated pipe is most often used in embankment dams for drainpipe applications, requiring nonrated and nonpressure joints. Manufacturers typically offer a variety of joints to meet specific project requirements (i.e., prevent the infiltration of soil, exfiltration of water, etc.). Corrugated pipe products are joined using the following methods: (1) single wall pipe using an external split or snap coupler and (2) profile wall pipe using an external split coupler, snap coupler, bell/bell gasketed coupler, or integral bell and spigot gasketed joint. Figure 14 shows an example of an external split coupler.

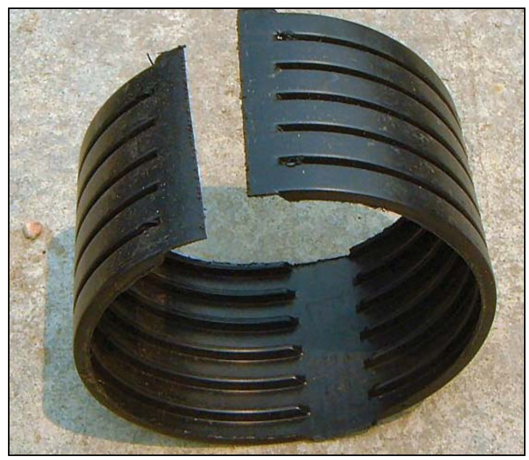


Figure 14.—External split coupler.

1.2.1.2 PVC

Pressure- and nonpressure PVC pipe is available in solid wall, which has smooth interior and exterior surfaces (figure 15). Solid wall PVC pipe is commonly available in 4- to 48-inch diameters in standard 20-foot lengths for pressure pipe. ASTM D 3034 nonpressure pipe is available in 14- or 20-foot lengths and ASTM F 679 nonpressure pipe is available in 14-foot lengths. Note that AWWA C900 and C905 are the only standards that specify a length. All others may vary from manufacturer to manufacturer. Open profile (single and double wall) (4- to 48-inch) ASTM F 794 and F 949, and closed profile (double wall) (18- to 60-inch diameter) ASTM F 1803 are also available, but have not been used in dam applications.

The common joining system for PVC pipe is a bell and spigot flexible gasketed joint (figure 16). The gasketed joint is designed so that when it is assembled, the elastomeric gasket(s) is compressed radially between the pipe spigot and bell to form a positive seal (Uni-Bell, 1995, p.1). Gasket materials should comply with the physical requirements as specified in ASTM F 477. Assembly of gasketed joints is facilitated by use of a lubricant as recommended and applied in accordance with the pipe manufacturer's instructions. Best practice for bell and spigot connections

Figure 15.—Solid wall PVC pipe has occasionally been used in embankment conduit applications within low hazard potential embankment dams. However, the bell and spigot joint connection used for this type of pipe can experience separation and seepage when improperly installed.

Figure 16.—PVC pipe joint (bell and spigot).

requires the bell ends pointing in the direction of the work progress, since it is easier to insert the spigot into the bell rather than push the bell over the spigot. Care must be taken to avoid over- or underinsertion of the spigot end into the bell end. Since gasketed joints permit some flexibility, they are preferred for drainpipe installations, especially where settlement is expected. However, since embankment conduits in significant and high hazard potential dams must be designed with a high degree of conservatism, bell and spigot joints should not be used. Bell and spigot joints are susceptible to separation as the embankment dam settles.

Other joining systems are available for PVC pipe. These proprietary joining systems include spline (figure 17), heat fusion (figure 18), and mechanical (figures 19 and 20) joints. These types of joints are being used on water distribution and sewer installations, but have not been used in applications for dams. The designer needs to carefully evaluate the watertightness and long-term suitability of these joints before they are considered for use in dam applications; see research need PM-6 in chapter 8.

Figure 17.—The splined joint has a machined groove in the PVC pipe and in the coupling to allow insertion of a flexible thermoplastic spline that provides a 360-degree restrained joint. Photo courtesy of Uni-Bell PVC Pipe Association Members and Associate Members.

Figure 18.—The heat fusion process is used to join PVC pipe, resulting in a continuous length of pipe. Photo courtesy of Uni-Bell PVC Pipe Association Members and Associate Members.

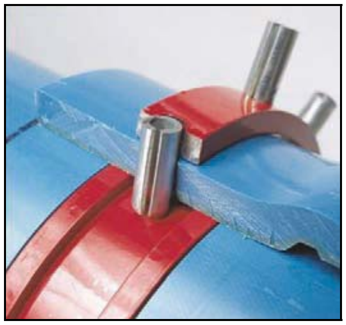


Figure 19.—This type of mechanically restrained joint is used to prevent overinsertion of bell and spigot gasket PVC pipe. Photo courtesy of Uni-Bell PVC Pipe Association Members and Associate Members.

Figure 20.—PVC pressure pipe with external mechanically restrained joints. Photo courtesy of Uni-Bell PVC Pipe Association Members and Associate Members.

1.2.1.3 Other types of thermoplastic pipe

Another type of thermoplastic is called fold-and-formed plastic (FFP). This system has not been used for renovation of embankment conduits, but may have applicability at some low hazard potential dams. The FFP system utilizes thermoplastic materials that have been folded from a circular shape to produce a smaller net cross-section and can be inserted into an existing pipe (USACE, 1995, pp. 2-8). These pipe products are usually extruded PVC or HDPE pipe that is flattened and folded longitudinally. The plastic pipe is fed from a spool into an existing pipe, and hot water or steam is applied until the liner reaches a uniform temperature throughout the material elevated enough for rounding. For one system, a special rounding device is inserted in the upstream end of the FFP and propelled by steam pressure to the downstream termination point. As the rounding device progresses, it expands the FFP tightly against the walls of the host pipe. Other systems use only heat and pressure to round the FFP. Any liquids in the host pipe are pushed out ahead of the expanding liner. The flexible FFP molds to the shape of the host pipe and normally forms distinct dimples at service connections. Pressure is maintained in the rounded FFP until it cools to a rigid state. The completed FFP liner has no joints and a very small annular space. No bonding occurs between the FFP and host pipe. The diameter range is limited to the manufacturing limits of this system (4 to 18 inches). Lengths up to 700 feet are possible. Due to its limited potential for use in embankment dams, FFP will not be discussed further in this document. For additional guidance on FFP, see ASTM D 1504.

1.2.2 Thermoset plastic

Thermosetting plastics (thermosets) refer to a variety of polymer materials that cure, through the addition of energy, to a stronger form. The energy may be in the form of heat or through a chemical reaction (e.g., two-part epoxy). The curing process transforms the resin into a plastic by cross-linking. Thermoset plastic polymer molecules are cross-linked (chemically bonded) with another set of molecules to form a "net like" or "ladder-like" structure. Once cross-linking has occurred, a thermoset plastic does not soften, melt, or flow and will disintegrate when sufficent heat is added. However, if the crosslinking occurs within a mold, the shape of the mold will be formed. A thermoset material cannot be melted and remolded after it is cured. The thermoset class of materials includes unsaturated polyester (UP), epoxy, and polyurethane (PUR). Thermoset materials are generally better suited to high-temperature applications than thermoplastic materials. However, they do not lend themselves to recycling like thermoplastics, which can be melted and remolded.

The most commonly used thermoset plastic in dam applications has been cured-in-place pipe (CIPP) (figure 21). CIPP is also referred to as an "elastic sock." CIPP liners have been used mainly for sliplining of embankment conduits, as an alternative renovation method. CIPP liners are constructed to be slightly smaller than the inner diameter of the existing pipe that is being renovated. CIPP consists of a flexible polyester needle-felt or glass fiber/felt tube preimpregnated with resin. The preimpregnation process is usually done at the factory for quality control purposes. Unsaturated polyester, vinyl ester, and epoxy resins are available, with unsaturated polyester being the most widely used. These resins have a wide range of capability allowing CIPP to be designed for specific applications, unlike other types of plastic pipe, which have fixed properties. The fabric tube carries and supports the resin until it is in the final position and cured. The fabric tube must withstand stresses from installation and stretch to expand against irregularities within the existing pipe. 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.

A variety of installation methods are available, including using water or air pressure to invert the tube through the existing pipe or a winch to pull the tube through the existing pipe (figure 22). When pressure is applied for rounding out the tube, the saturated fabric stretches to conform to the inner surface of the existing pipe. Although inversion is the preferred method of installation, winching may be pursued in situations where sufficient water pressure is unavailable or scaffold towers required for inversion are not practicable (USACE, 2001, p. 11). Combinations or variations of these methods are sometimes used. Hot water or steam is used to heat the resin and allow it to harden and cure after the liner has been formed within the

Figure 21.—CIPP liner exiting from an existing outlet works conduit, via the hydrostatic inversion method.

existing pipe. Other curing methods are possible (i.e., UV and ambient), but typically have not been used with embankment conduits. When completed, the CIPP process forms at continuous tight-fitting, pipe-within-a-pipe containing no joints.

Many CIPP systems are available today. The primary differences between these systems are in the composition and structure of the tube, method of resin impregnation, installation procedure, and curing process (USACE, 1995, pp. 2-6). Commonly used standards for specification and installation of CIPP are ASTM D 5813 and F 1216. CIPP is applicable for lining existing conduits with diameters ranging from 4 to 132 inches. Maximum lengths of CIPP liners can exceed 1,000 feet. At the larger diameters, the weight and cost of the materials become significant and the economics of the process may be adversely affected. Some mechanical bonding of the resin to the inner pipe surface can occur in practice. Whether it is effective in enhancing the structural performance of the CIPP liner depends to a great extent upon the condition of the existing pipe (USACE, 1994, pp. 14-15). Grouting of the annulus is typically not possible due to the small size of the gap between the existing pipe and a properly installed CIPP liner.

Fiberglass pipe is another type of thermoset plastic, but has had very infrequent use in dam applications. Fiberglass pipe generally consist of two types, filament wound and centrifugally cast. In the filament-wound process, glass fiber is drawn, and a gelatinous or glutin like substance is applied. This substance helps protect the fiber as it is wound onto a bobbin. The particular substance applied relates to the end use

Figure 22.—Contractor installing a resin-soaked CIPP liner into an existing outlet works. Installation begins by hauling the liner up to the top of the platform. On the platform, water is run into the liner causing it to pressurize and expand downward. As the liner reaches the outlet works pipe opening, laborers on the ground maneuver the water filled liner into the outlet works. Water pressure continues to cause the liner to advance upstream in the outlet works pipe and un-invert itself. Photo courtesy of Tetra Tech Inc.

of the pipe. The winding process takes place at a very high speed. In the centrifugal casting process, materials are placed in multiple layers, building from the outside to the inside using mold rotation. Centrifugally cast fiber reinforced polymer mortar (CCFRPM) pipe is manufactured in this fashion.

The main advantage fiberglass pipe has over other types of plastic pipe is the availability in larger diameters. Fiberglass pipe typically has standard designs up to 110 inches and nonstandard designs for larger sizes. Fiberglass pipe uses bell and spigot joints and should only be used on low hazard potential dam applications. For further guidance on the design of fiberglass pipe, see AWWA's, *Fiberglass Pipe Design Manual* (2005).

1.3 Common Uses for Plastic Pipe

Not all plastic pipe can be used in the same way within dams. This section discusses some of the common applications of plastic pipe used in dam construction.

1.3.1 HDPE

1.3.1.1 Solid wall and dual-wall containment pipe

Solid wall pipe is mainly used in nonpressurized sliplining applications for renovation of existing outlet works conduits, construction of siphons, and the construction of decants in tailings and slurry impoundments. Sliplining is a renovation method where a new plastic pipe is pulled or pushed through the interior of an existing embankment conduit (i.e., outlet works), forming a watertight barrier. HDPE pipe has been used in sliplining of existing conduits since the early 1990's. The annulus between the new and existing pipes is typically filled with grout. 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 embankment conduits that are not easily renovated and cannot be easily inspected. The Worster Dam case history in appendix B illustrates how a HDPE slipliner can be used to renovate an outlet works. Dual-wall containment pipe is mainly used in pressurized sliplining applications. Use of dual-wall containment pipe in dam applications began after 2000. Figure 23 shows an example of HDPE dual-wall containment pipe arriving at a job site.

The advantages of using solid wall or dual-wall containment HDPE pipe for new construction and for renovation include:

- High strength and stiffness resists internal pressures and external loads, when properly designed.

- Lightweight material facilitates installation requiring less equipment and fewer personnel. However, dual-wall containment pipe is roughly twice as heavy as solid wall pipe.

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

- Smooth interior surface reduces friction and resistance to flow.

- Smooth interior surface minimizes adherence of soluble encrustants (e.g., calcium carbonate).

- Minimizes biological growth and attack by microorganisms, such as bacteria and fungi.

- Pipe joints can be butt fused, which provides a strong, watertight joint.

Figure 23.—HDPE dual-wall containment pipe arriving at a job site.

- Good resistance to abrasion.
- Remains flexible at subfreezing temperatures.

The disadvantages of using solid wall or dual-wall containment HDPE pipe for new construction and for renovation include:

- Has a higher coefficient of thermal expansion relative to other types of plastic pipe, which can cause movement of the pipe, requiring the use of end restraints.
- Pipe can be damaged or deformed by construction and compaction equipment.
- Pipe can be displaced during compaction of earthfill against the pipe due to its light weight.
- Heat fusion of pipe joints requires special equipment and a trained operator.
- Compaction of earthfill under the haunches of the pipe is difficult and labor intensive.
- Due to concerns with internal erosion, a properly shaped, reinforced cast-in-place concrete encasement is required for significant and high hazard potential embankment dams to accommodate compaction of earthfill against the embankment conduit.
- Combustible and can melt in fire situations.

For guidance on the design and construction of embankment conduits for new installations and renovations, see chapters 2 and 3 in this document and FEMA's

Technical Manual: Conduits through Embankment Dams (2005). For guidance on solid wall pipe used in drainpipe applications, see chapters 4 and 5 of this document. While solid wall HDPE pipe has occasionally been used in drainpipe applications, dual-wall containment pipe has not been used for drainpipes.

1.3.1.2 Corrugated pipe

Corrugated HDPE pipe (single wall and profile wall) is most often used in embankment dams for drainpipe applications, such as toe drains (figure 24). Single wall corrugated pipe was first used for drainpipes in the early 1980's. The use of profile wall corrugated pipe began in the 1990's. Plastic pipe for drainpipes has largely replaced other pipe materials including clay tile, corrugated metal, and cast iron. Most designers prefer profile wall corrugated pipe over single wall pipe due to its higher wall strength and smoother interior. Also, CCTV inspection has shown the existence of structural integrity issues with single wall pipe (see section 6.2). Underground installation of corrugated pipe should follow the guidance in ASTM D 2321 and manufacturers' instructions.

The advantages of using HDPE corrugated pipe for a drainpipe include:

- Lightweight material facilitates installation requiring less equipment and fewer personnel.
- Resists corrosion and is not affected by naturally occurring soil and water conditions.
- Smooth interior surface of profile wall pipe reduces friction and resistance to flow.
- Minimizes biological growth and attack by microorganisms, such as bacteria and fungi.
- Remains flexible at subfreezing temperatures.

The disadvantages of using HDPE corrugated pipe for a drainpipe include:

- Corrugated interior surface of single wall pipe will result in lower discharge capacity (use of profile wall pipe avoids this problem).
- Interior surface corrugations can trap sediments and allow biofouling to develop (use of profile wall pipe avoids this problem).
- Interior surface corrugations are more difficult to clean (use of profile wall pipe avoids this problem).

Figure 24.—A toe drain being constructed using profile wall corrugated HDPE pipe.

- Pipe can be damaged or deformed by construction and compaction equipment.
- Pipe can be displaced during compaction of earthfill against the pipe, due to its light weight.
- Compaction of earthfill under the haunches of the pipe is difficult and labor intensive.
- Combustible and can melt in fire situations.

For guidance on the design and construction of drainpipes, see chapters 4 and 5.

1.3.2 PVC

PVC pipe was first introduced to North America in the early 1950's. However, use of PVC pipe did not appear in dam applications until about the early 1970's.

Nonpressure PVC pipe is often used in dam applications for drainpipes. Some pressure rated pipe has been used for embankment conduits in low hazard potential dams and for siphons (figure 25). However, PVC pipe should not be used in significant and high hazard potential dams for embankment conduits. Primary

Figure 25.—A temporary siphon constructed using PVC pipe. This siphon is being used for short term operation.

concerns involve the potential for leakage of the bell and spigot joints due to foundation movement.

The advantages of using PVC pipe for embankment conduits in low hazard potential dams and drainpipes include:

- High strength and stiffness resists internal pressures and external loads when properly designed.
- Lightweight material facilitates installation, requiring less equipment and fewer personnel.
- Resists corrosion and is not affected by naturally occurring soil and water conditions. May be preferable in certain applications where aggressive water or soil chemistry would limit the life of concrete or metal pipe.
- Smooth interior reduces friction and resistance to flow.
- Smooth interior surface minimizes adherence of soluble encrustants (e.g., calcium carbonate).

- Minimizes biological growth and attack by microorganisms, such as bacteria and fungi.

- Higher beam strength than HDPE pipe helps alignment and grade control during installation.

- Greater modulus of elasticity than for HDPE pipe. This allows for thinner sections of pipe to be used for the same conditions when properly designed.

The disadvantages of using PVC pipe for embankment conduits in low hazard potential dams and drainpipes include:

- More potential leak points at bell and spigot joints since joints are located every 10 to 20 feet.

- Susceptible to impact during cold weather and requires reasonable care.

- Susceptible to extended UV exposure resulting in reduced resistance to impact and gradual decline in pipe strength. However, providing an opaque surface between the sun and pipe prevents UV degradation. Burial provides complete protection.

- Pipe can be damaged or deformed by construction and compaction equipment.

- Pipe can be displaced during compaction of earthfill against the pipe, due to its light weight.

- Compaction of earthfill under the haunches of the pipe is difficult and labor intensive.

- Limited resistance to cyclic loading under very high stress amplitudes.

For guidance using PVC pipe in the design and construction of drainpipes, see chapters 4 and 5. For guidance on design and constructions of conduits, see chapters 2 and 3 in this document and FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

1.3.3 CIPP

CIPP has been successfully used in renovating of deteriorated embankment conduits since about the mid-1990's. However, the use of CIPP has been relatively small compared to other applications using thermoplastic pipe.

The advantages of using CIPP lining for embankment conduits include:

- 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.
- Smooth interior surface reduces friction and resistance to flow.
- Smooth surface minimizes, adherence of soluble encrustants (e.g., calcium carbonate).
- Minimizes biological growth and attack by microorganisms, such as bacteria and fungi.

The disadvantages of using CIPP lining for embankment conduits include:

- High material and installation costs require a trained crew with special equipment.
- Not suited for conduits with significant bends or changes in diameter.

For guidance on the use of CIPP in embankment conduit renovation applications, see chapter 12 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). CIPP has not typically been used for drainpipe applications.

1.4 Design Life

Plastic pipe has many desirable characteristics. Unlike metal and concrete pipe, which can deteriorate over time from galvanic or chemical corrosion, plastic pipe does not rust, rot, or corrode. Aggressive soils do not affect plastic pipe, and it tolerates subzero temperatures well. Plastic pipe was introduced to the United States in about the 1950's, but its use in embankment dam applications did not begin until about the mid 1970's. The long-term performance of plastic pipe, like any pipe product, depends primarily on the quality of the installation. Excessive deflection of flexible pipe caused by inadequate compaction of the backfill material in the haunch area and at the sides of the pipe affects the long-term performance of the pipe. The plastic pipe industry has addressed other factors affecting design life by updating and improving materials. Current ASTM standards for plastic pressure pipe require use of high quality plastic materials that are designed for long-term performance under field conditions. Pipe manufacturers are continuously testing and evaluating their products in accordance with ASTM procedures to ensure the long-term strength and performance.

The design life for HDPE pipe in pressure service is based on the hydrostatic design basis testing for thermoplastic pipe (ASTM D 2837) and provides for a factor of safety of 2.0. Solid wall HDPE pressure pipe and corrugated HDPE pipe have significantly different properties and are not generally used in the same applications nor designed in the same way. The base resins used to manufacture these pipes are normally different. Polyethylene pipe resin is identified by an ASTM Material Designation Code or grade. Pressure pipe base resin material has a Cell Class of 345464C or higher as designated in ASTM D 3350. ASTM D 3350 resin cell classification provides the means for identification, close characterization, and specification of material properties for polyethylene. This is a modern improved material and provides the longest life available for pipe in pressure flow applications. Current ASTM standards allow manufacturing of corrugated pipes with base resin materials having a cell classification of 323410C or 333410C. AASHTO requirements for corrugated pipes generally require better resistance to long-term stress than specified by ASTM. In some cases, corrugated pipe can be manufactured with materials similar to those used for pressure pipes.

Manufacturers have used accelerated testing and statistical prediction methods to determine the expected life expectancy of plastic pipe. The basis recommendation for the design life of plastic pipe is 50 years. The Plastic Pipe Institute cites a recent report (PPI, 2003) that there is justification for assuming a greater design service life for corrugated polyethylene pipe when properly installed and used for gravity flow end-use applications. The PPI report pertains only to corrugated HDPE pipe that is gravity flow and operates primarily in compression. However, there is no uniformly accepted agreement concerning design life exceeding 50 years. The Florida Department of Transportation has initiated a program to verify the design life of corrugated polyethylene pipe (Hsuan and McGrath, 2005, and Hsuan, Zhang, and Wong, 2006). The study pertains only to corrugated HDPE pipe used in gravity flow applications that operate primarily in compression stress (i.e., low demand because slow crack growth is a tension failure mode). Pressure pipe operates primarily under tension and therefore requires polyethylene resins with a hydrostatic design basis (HDB) rating.

HDPE pipe resins have differing amounts of stress crack resistance (SCR). A number of early drainpipe failures have occurred in single wall corrugated pipes. These failures are often attributed to the effects of environmental stress cracking (ESC) (also called slow crack growth). This phenomenon can occur during the handling and installation of HDPE pipe or under long-term service loads. The HDPE pipe could be gouged, scratched, kinked, or stressed resulting in a weak spot on the pipe wall and subsequent cracking. Failures from ESC tend to be due to the development of cracks in areas of tensile stress that slowly grow and propagate over time. 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 virtually negate the possibility of this type of failure. This ensures a virgin, high grade resin that has been found highly resistant to environmental stress cracking.

The cell classification designated in the applicable product specification identifies the stress crack requirement. The product design and end-use applications determine the required stress crack requirement. The stress crack or notched constant ligament stress (NCLS) requirement designated in the applicable product specification have been established to insure that the HDPE resin provides the highest level of stress crack resistance for the intended end-use application for the pipe. Other grades of resin often contain some percentage of low grade recycled resins. The designer should be aware that ASTM cell classifications have changed over time and ASTM D 3350 should always be consulted for current classification designations. For additional information concerning resistance to slow growth cracking, see AWWA's *PE Pipe – Design and Installation* (2006).

The Bureau of Reclamation recommends that corrugated polyethylene pipe used in embankment dams comply with the requirements specified in AASHTO M252 (3- to 10-inch diameter), AASHTO M294 (12- to 60-inch diameter), or ASTM F 2306 (12- to 60-inch diameter). AASHTO M252 specifies an ESC resistance requirement and AASHTO M294 and ASTM F 2306 specify a notched constant ligament stress requirement for the resins used to manufacture the larger diameter products. In addition, an ESC resistance requirement is specified for the finished product in both the AASHTO and ASTM product specifications. Additional research may be necessary to determine if a higher stress-crack-resistant resin should also be required in smaller diameter pipe (see chapter 8, research need *PM-1*).

Research has shown that the actual performance of plastic pipe has exceeded the performance predicted by the long-term pressure tests more than 60 years ago. (Hulsmann and Nowack, 2004, p.8) reported that the extrapolation of 10,000-hour pressure testing is conservative and the actual service life of PVC pipe is likely to be greater than 50-years. Utah State University conducted an extensive survey of utilities in 1994 to evaluate performance of PVC in both gravity and pressure applications (Moser, 2001). The study showed that 50 percent of all problems occurred within the first year. Material-related long-term problems are few and are decreasing with time, which indicates that the problems are not a result of aging.

Although much has been written regarding the projected design life for plastic pipe, there is general agreement that 50 years is a conservative estimate. As discussed in section 1.1, the performance history of plastic pipe used in embankment dam applications has been limited. Therefore, a number of research items are proposed in section 8.1.1 (*PM-1* through *PM-6*) to further evaluate the use of plastic pipe in dams. The designer should consider all aspects of the project, installation conditions, end-use application, product specifications compliance, and established codes of practice when designing for a design life of more than 50 years. If high quality materials are used in the manufacture of plastic pipe and installation is performed in compliance with established codes of practice, a design life exceeding of 50 years may be possible.

Chapter 2

Loading Conditions

Embankment conduits and drainpipes are subjected to stresses and strains from external and internal loadings. External loads can include the soil above the pipe, vehicular loads, external hydrostatic pressure, and vacuum pressure. Internal loads can include fluid pressure and water hammer. This chapter discusses the determination of the various loadings on plastic pipe. Chapter 3 discusses the structural design principles necessary to accommodate these loadings.

2.1 Soil Loading

Many classic references have used the terms 'buried conduit' or 'conduit' when discussing loading conditions. In embankment dam applications, the term 'buried conduit' or 'conduit' often is interpreted to mean either embankment conduits (i.e., outlet works, siphon) or drainpipes. Generally, for significant and high hazard potential dams, embankment conduits constructed of plastic pipe are encased in a properly shaped reinforced cast-in-place concrete section to facilitate compaction of earthfill against the conduit. However, in some low hazard potential dams, the pipe may or may not be encased in concrete. Discussions in chapter 2 primarily focus on the application of load on plastic pipe assuming no concrete encasement. Figures 27 through 34 are intended only to illustrate the principles involved with soil loads on buried pipe. However, these figures do not present all the required details for the proper design of conduits within embankment dams. The discussions presented in this chapter are best suited for applications involving conduits within low hazard potential dams or renovation (i.e., sliplining) where no support from the existing pipe or conduit is typically assumed. See section 3.5.2.2, for discussion of the reinforced concrete encasement as it relates to plastic pipe. For further design and construction guidance for conduits within significant or high hazard dams, see FEMA's *Technical Manual: Conduits through Embankment* Dams (2005). To conform with commonly used terminology whenever possible in this chapter, the terms 'buried conduit' and 'conduit' will be used interchangeably with 'buried pipe' and 'pipe.'

Loads applied to buried conduits consist of dead and live loads. Dead loads are generally permanent, consisting of the soil above the conduit. Live loads, such as construction loadings (section 2.3), may or may not be permanent. Estimated soil loads on buried conduits have historically been computed using the Marston load theory. Soil loads may also be computed by the soil prism theory. The differences in these loading theories are as follows:

- *Marston load theory*.—This theory considers the transfer of load to or from the soil directly above the buried conduit due to the relative settlement between the soil directly above the conduit and the adjacent soil. The vertical load is made up of two parts: (1) the weight of the soil element directly over the buried conduit and (2) frictional forces acting either upward or downward on the sides of the soil column. If the soil on the sides of the column settles due to compressible soils or foundation, poor compaction, or other causes, downward friction forces will develop on the soil column. When this occurs the pressures on the buried conduit is greater than just the weight of the soil column above it. If the soil in the column above the buried conduit settles more than the surrounding soil or if a compressible foundation allows the conduit to move downward or if the conduit deflects vertically, upward friction forces will reduce pressures on the conduit. Illustrations of the relative load transfer are shown in this section (figures 28, 29, and 30).

- *Soil prism theory*.—The soil prism theory is considered the simplest method for determining vertical earth soil loading above a buried conduit. This method assumes no load is transferred to or from the prismatic soil column directly above the buried conduit and includes only the load from the entire soil column directly above the conduit (figure 34).

The designer should be aware that full load transfer onto the buried conduit may require months or even years and might not be realized until after construction is completed. However, both theories of soil loading presented in this document are only estimates of the soil loadings on the buried conduit. The designer should always consider the range of possible soil loadings based on the potential range of each of the parameters included in the soil load computations. Additional discussion of the Marston load and soil prism theories is included in this and the following sections. Recommendations for the soil load method to use are provided in section 2.1.1 and 2.1.2 with further recommendations provided in table 9 in section 3.5.6.

Classifying buried conduits is required to compute soil loads using the Marston theory. Figure 26 shows the classification of buried pipe with additional details provided in figures 27 through 33. Additional guidance on buried conduit classification is available in Spangler and Handy's *Soil Engineering* (1982).

Figures 27 through 35 show circular pipes in a discussion of loading conditions for plastic pipes, which are ordinarily circular in cross section. Other sections of this document caution against using circular pipes in significant and high hazard potential embankment projects because attaining intimate contact with the surrounding embankment soils is difficult with circular pipes. Circular pipes used in embankment dams require special considerations. A filter diaphragm as discussed in chapter 6 of

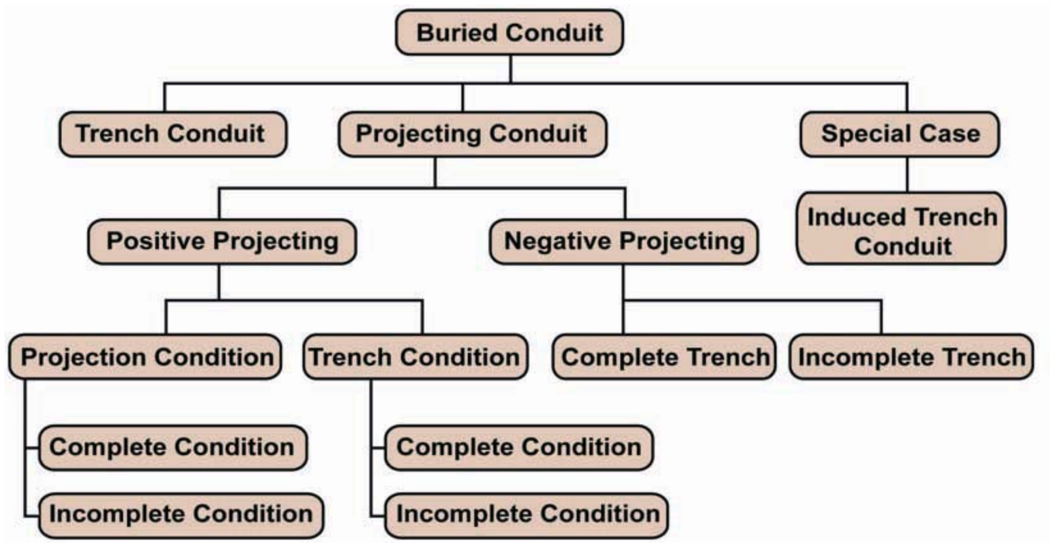


Figure 26.—Classification of buried conduits for the Marston theory (Spangler and Handy's *Soil Engineering,* 1982). Note: Use of incomplete trench and special case should not be used for embankment dam applications.

FEMA's *Technical Manual: Conduits through Embankment Dams (2005)* should always be used. Encasing plastic conduits in a cross section of concrete with a battered shape avoids the problems of compacting soil under the haunches of a circular conduit. An example of a good cross section for encasements is shown in figure 46, chapter 4 of the 2005 FEMA Technical Manual. Precautions discussed in section 3.8.8 are important for plastic conduit encasements.

Trench conduits are installed in a relatively narrow trench in passive or undisturbed soil and backfilled to the ground surface, as shown in figure 27. The consolidation and settlement of the backfill along with the settlement of the conduit cause the backfill soil to move downward relative to the soil at the side of the trench. Some load is transferred from the backfill soil to the trench sidewalls due to friction. A drainpipe buried beneath the natural ground/foundation surface is often considered a trench conduit depending on the width and side slopes of the trench excavation. Embankment conduits should not be installed as trench conduits because of the potential for seepage in the zone where reduced stresses and hydraulic fracture can occur.

Projecting conduits consist of those covered by fill material, such as embankment conduits. According to the Marston load theory, projecting conduits may be positive projecting or negative projecting. Positive projecting conduits are installed with the pipe projecting above the ground surface or compacted fill, with fill placed around and above the pipe. Positive projecting conduits are most typical for conduits through embankment dams and include conduits placed in wide or sloped-back

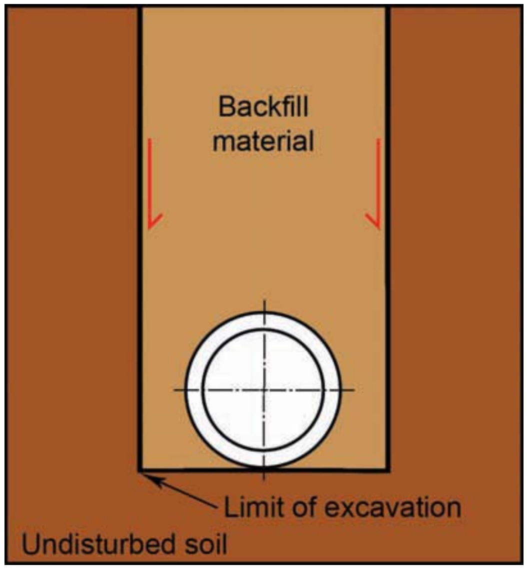


Figure 27.—Trench conduit. Backfill soil moves downward relative to the soil at the side of the trench. A drainpipe buried beneath natural ground is an example of a trench conduit. (Note: A trench conduit should not be used for an embankment conduit.)

trenches. An outlet works conduit, spillway conduit, penstock, buried siphon, or drainpipe installed within an embankment drain or filter are considered projecting conduits.

A positive projecting conduit may be in a projection condition or trench condition. If the exterior prisms settle more than the interior prism, as shown in figure 28, load is transferred from the exterior prisms to the interior prism, and a projection condition exists. The soil load on the conduit in a projection condition is greater than the weight of the fill above the conduit (soil prism load). This is caused in part by the loads from the exterior prisms as they deform being transmitted by soil shearing stresses to the interior soil prism. This increases the downward force applied to the pipe. This is sometimes referred to as negative or reverse arching. This often happens with rigid pipe because it undergoes only small deformations when loaded. If the interior prism settles more than the exterior prisms, as shown in figure 29, due to yielding foundation conditions or deflection of the pipe, a trench condition exists. The soil load on the conduit in the trench condition is typically less than the weight of the fill above the conduit (soil prism load). A flexible conduit that is installed as projecting conduit is typically considered a projecting conduit in the trench condition since the deflection of the conduit causes the interior prism to settle more than the exterior prisms.

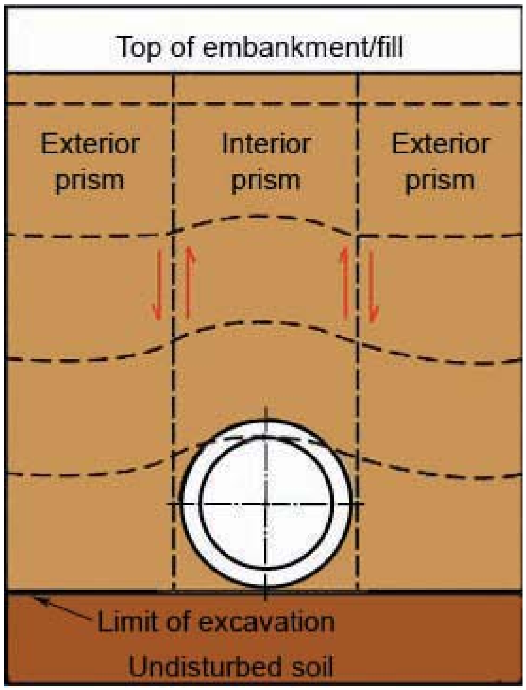


Figure 28.—Positive projecting conduit in a projection condition. The pipe is installed above the ground surface or compacted fill, with fill placed around and above the conduit. The exterior prisms settle more than the interior prism, causing load to be transferred to the interior prism. An embankment conduit is an example of a positive projecting conduit in projection condition. (Note: This figure is not intended to show all the design details required.)

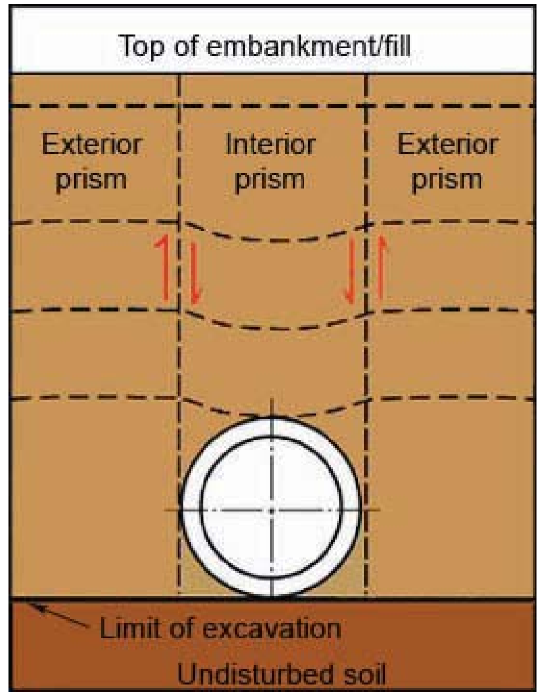


Figure 29.—Positive projecting conduit in a trench condition. If the foundation is yielding or the conduit deflects, the interior prism settles more than the exterior prisms. The soil load on the conduit is less than the weight of the soil above it. A drainpipe in an embankment dam is an example of a positive projecting conduit in a trench condition. (Note: This figure is not intended to show all the design details required.)

Negative projecting conduits are installed in shallow trenches, such that the top of the pipe is below the natural ground or compacted fill and backfilled and covered with fill material, as shown in figure 30. The soil load on a negative projecting conduit is less than that on a positive projecting conduit and typically less than the weight of the fill above the conduit (soil prism load). A negative projecting conduit may apply to a conduit through an embankment dam that is set in a narrow valley or foundation excavation, or to a drainpipe buried in the foundation beneath the embankment dam. However, good design practice requires embankment conduits and drainpipes not to be constructed as negative projecting conduits because soil arching above the conduit (figure 31) can cause potential seepage paths through the embankment. Arching can occur in all soils that have an internal angle of friction greater than zero. This includes all granular soils and most fine grained soils in the drained state. Arching is the result of grain-to-grain contact of the soil particles and

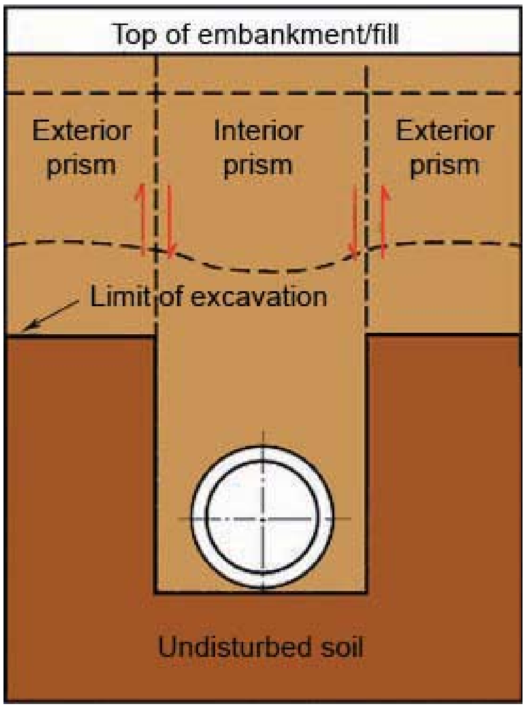


Figure 30.—Negative projecting conduit. The pipe is installed in a shallow trench such that the top of the pipe is below natural ground or compacted fill, and then covered with fill material. Negative projecting conduits should not be used for embankment conduits or drainpipes (Note: This figure is not intended to show all the design details required.)

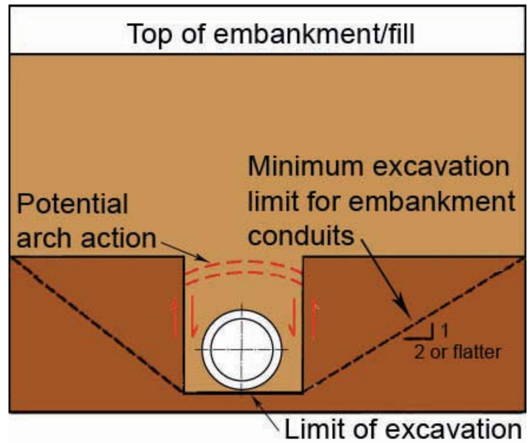


Figure 31.—Arching action of a negative projecting conduit. Negative projecting conduits should not be used for embankment conduits. For an embankment conduit, an excavation with 2:1 side slopes or flatter should be used. This causes the conduit to behave as a positive projecting conduit. (Note: This figure is not intended to show all the design details required.)

is a form of shear resistance. Arching is as stable and permanent as other forms of shear resistance (Petroff, 1990, p. 286).

Arching of the soil above the conduit can result in reduced lateral effective stress. If water pressure exceeds this stress, hydraulic fracture can occur, allowing internal erosion to develop. See chapter 5 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) for further discussion on hydraulic fracture. To avoid installing a negative projecting embankment conduit, use a trench with at least 2 to 1 (horizontal to vertical) or flatter side slopes (figure 31). The conduit will behave as a positive projecting conduit.

Positive projecting and negative projecting conduits are further divided into complete and incomplete conditions (figure 26). The transfer of load from exterior prisms to the interior prism and vice versa causes different strains in the interior and exterior prisms. At some point above the conduit the accumulated strain and settlement in the exterior prisms will equal that of the interior prism. This is defined as the plane of equal settlement. Above the plane, the interior and exterior prisms settle equally, and no shear or friction forces are transferred between the prisms. A complete condition exists when the embankment height is less than or equal to the height of the plane of equal settlement, as shown in figure 32. An incomplete condition exists when the embankment height is greater than the height of the plane of equal settlement, as shown in figure 33. Most concrete-encased embankment conduits are in the incomplete condition. At some height of fill above the pipe, but before the top of the embankment, the interior and exterior prisms are settling the same.

A summary of the classifications of buried conduits is shown in table 1. A thorough understanding of table 1 is crucial for any buried conduit design. The difference in "projecting conduits" and "trench conduits" and "projection condition" and "trench condition" must be understood. The terms "projecting conduits" or "trench conduits" refer to a classification based on construction methods while "projection condition" or "trench condition" refers to a subclassification based on relative settlements of a positive projecting conduit.

The range of potential soil loading on the buried conduit should be determined using the potential range of total unit weight of the soil. The soil prism load is not recommended for a projecting conduit in the projection condition, since settlement of the exterior prisms cause additional load on the interior prism that would be ignored using the prism theory. Thus, projecting conduits in a projection condition are typically designed using the Marston load theory. Conduits through embankment dams should be designed as shown in chapter 3, table 9. The effects of arching are typically ignored in computing the loading using the prism theory as a conservative measure. The soil prism theory typically estimates a greater (more conservative) soil load than that estimated from the Marston load theory for trench conduits, projecting conduits in a trench condition, or negative projecting conduits and should be used for these pipe classifications. Trench conduits should not be used for embankment conduits. The soil prism load theory is discussed in section 2.1.1. The Marston load theory is discussed in section 2.1.2 and by Spangler and Handy (1982).

2.1.1 Soil prism load for trench conduits, positive projecting conduits in the trench condition, and negative projecting conduits

The soil load on trench conduits, positive projecting conduits in the trench condition, and negative projecting conduits is determined by the soil prism theory as shown in figure 34 and the following equation:

$$P_s = \gamma H \tag{2-1}$$

Table 1.—Classification of buried conduits (required to compute soil loads using the Marston Load Theory)

<table>
<tr><td rowspan="2">Classification by construction method</td><td rowspan="2">Trench conduit (figure 27)—Installed in a narrow trench in a passive or undisturbed soil, then backfilled to ground surface. Consolidation of backfill and settlement of conduit cause backfill to move downward relative to soil at side. Some load transferred from backfill to trench due to friction.</td><td colspan="6">Projecting conduits—Covered by a fill material</td><td rowspan="4">Induced trench conduit—An induced trench conduit is originally constructed as a positive projecting conduit. Upon filling at least one conduit diameter above the top of the conduit, a trench is excavated to the top of the conduit and backfilled with compressible material. Induced trench conduits should not be used for embankment conduits due to the potential for preferential seepage through the compressible material placed above the conduit.</td></tr>
<tr><td colspan="4">Positive projecting conduit—Installed with conduit projecting above the grade or compacted fill with fill placed around and above the conduit.</td><td colspan="2">Negative projecting conduit (figure 30)—Installed in a shallow trench such that top of conduit is below the natural ground or compacted fill and then backfilled and covered with fill. Negative projecting conduits should not be used for embankment conduits.</td></tr>
<tr><td colspan="2" rowspan="2">Subclassification by relative settlement</td><td colspan="2">Projection condition (figure 28)—Exists when exterior prisms settle more than interior, load is transferred from exterior prisms to interior prism. Soil load on conduit is greater than weight of fill above the conduit. Typical for conduits through embankment dams; includes conduits placed in wide trenches.</td><td colspan="2">Trench condition (figure 29)—Exists when interior prism settles more than exterior prism due to yielding foundation condition or deflection of conduit. Soil load on conduit is less than weight of the fill above the conduit.</td><td rowspan="2">Complete condition</td><td rowspan="2">Incomplete condition</td></tr>
<tr><td>Complete condition (figure 32)—Exists when the embankment is higher than the plane of equal settlement.*</td><td>Incomplete condition (figure 33)—Exists when the embankment is lower than the plane of equal settlement.*</td><td>Complete condition</td><td>Incomplete condition</td></tr>
</table>

* Plane of equal settlement is some point above the conduit where the accumulated strain and settlement in the exterior prisms equals interior prism. Above the plane, exterior and interior prisms settle equally, and no shear or friction forces are transferred between the prisms.

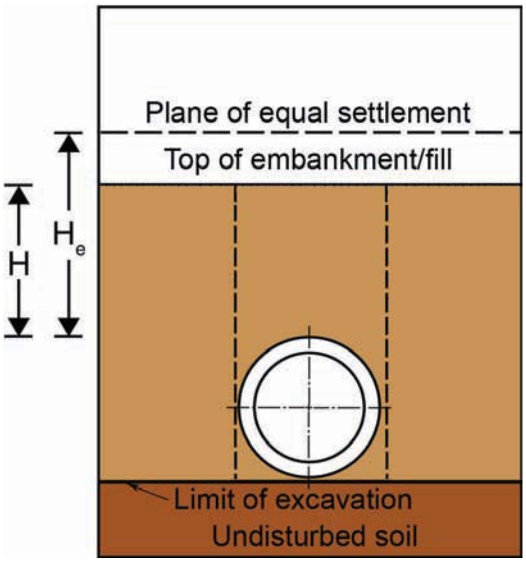


Figure 32.—Complete condition. The complete condition exists when the fill height (H) is less than or equal to the height to the plane of equal settlement (H_e). (Note: This figure is not intended to show all the design details required.)

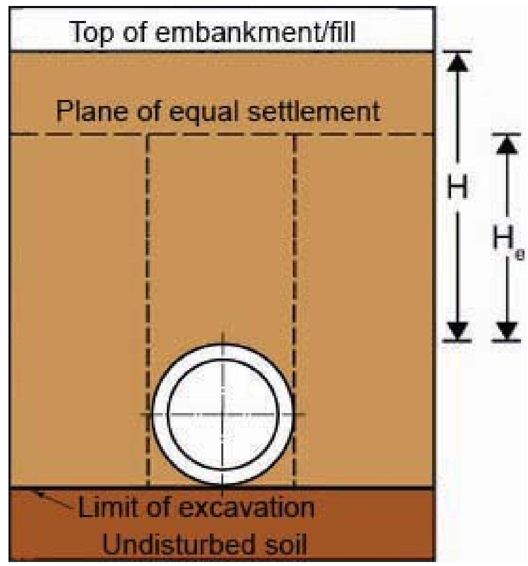


Figure 33.—Incomplete condition. The incomplete condition exists when the fill height (H) is greater than the height to the plane of equal settlement (H_e). (Note: This figure is not intended to show all the design details required.)

where:

P_s = pressure due to weight of soil on top of pipe, lb/ft^2
γ = total unit weight of soil, lb/ft^3
H = height of soil above the top of the pipe, ft

2.1.2 Marston load for positive projecting conduits

The soil load on a positive projecting conduit may be computed by:

$$W_c = C_c \gamma D_O^{\ 2} \tag{2-2}$$

where:

W_c = soil load, lb/linear foot of pipe
C_c = positive projection load coefficient
γ = total unit weight of soil, lb/ft^3
D_O = outside diameter of the pipe, ft

C_c depends on whether the buried conduit is in the projection or trench condition and in the complete or incomplete condition.

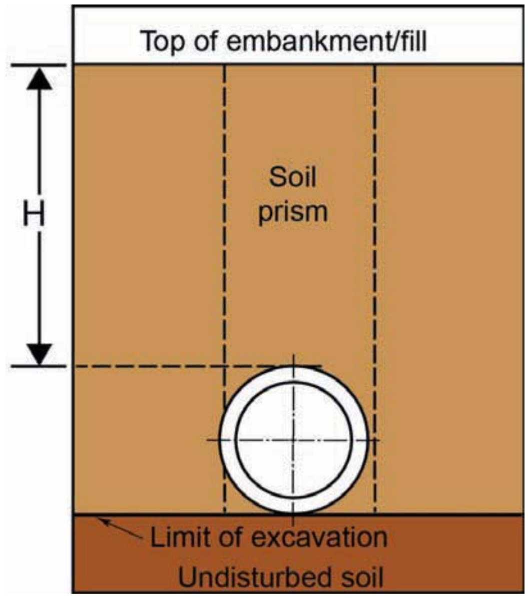


Figure 34.—The soil prism load is the weight of the soil directly above the conduit. (Note: This figure is not intended to show all the design details required.)

The positive projection load coefficient, C_c, is defined as:

$$C_c = \frac{e^{2K\mu\left(\frac{H}{D_0}\right)} - 1}{2K\mu} \text{ when } H \le H_e \text{ (complete condition)} \qquad (2\text{-}3)$$

or

$$C_c = \frac{e^{2K\mu\left(\frac{H_e}{D_0}\right)} - 1}{2K\mu} + \left(\frac{H}{D_0} - \frac{H_e}{D_0}\right) e^{2K\mu\left(\frac{H_e}{D_0}\right)} \text{ when } H > H_e \text{ (incomplete condition)} \qquad (2\text{-}4)$$

where:

C_c = positive projection load coefficient
e = base of natural logarithms, 2.7183
K = Rankine's active lateral earth pressure coefficient, $(\tan^2(45^\circ - \phi/2))$
μ = coefficient of friction (between backfill and sides of trench), $\tan \phi$
ϕ = effective friction angle of backfill
H = height of soil above the top of the pipe, ft
H_e = height of plane of equal settlement above the top of the pipe, ft
D_O = outside diameter of the pipe, ft

The height to the plane of equal settlement, H_e, may be determined by the following equation developed by Spangler (Spangler and Handy, 1982) (The solution for H_e requires an iterative procedure):

$$\left[\frac{1}{2K\mu}+\left(\frac{H}{D_0}-\frac{H_e}{D_0}\right)+\frac{r_{sd}\,p}{3}\right]\frac{e^{2K\mu\left(\frac{H_e}{D_0}\right)}-1}{2K\mu}+\frac{1}{2}\left(\frac{H_e}{D_0}\right)^2 + \frac{r_{sd}\,p}{3}\left(\frac{H}{D_0}-\frac{H_e}{D_0}\right)e^{2K\mu\left(\frac{H_e}{D_0}\right)}-\frac{1}{2K\mu}\frac{H_e}{D_0}-\frac{H}{D_0}\frac{H_e}{D_0}=r_{sd}\,p\frac{H}{D_0} \tag{2-5}$$

where:

K = Rankine's active lateral earth pressure coefficient, $(\tan^2(45-\phi/2))$
μ = coefficient of friction of fill material, $\tan\phi$
ϕ = effective friction angle of backfill
H = height of soil above the top of the pipe, ft
D_O = outside diameter of the pipe, ft
H_e = height of plane of equal settlement above the top of the pipe, ft
r_{sd} = settlement ratio (table 2)
p = projection ratio, as defined by figure 35, p is computed based on the dimensions of the installation
e = base of natural logarithms, 2.7183

Note: The value of $K\mu$ is limited to 0.19 for a projection condition.

Recommended design values for the settlement ratio (r_{sd}) are provided in table 2. These values are used to determine the Marston load on positive projecting conduits. The settlement ratio is a function of the type of installation and foundation condition.

Table 2.—Design values for the settlement ratio, r_{sd}

Installation and foundation condition		Settlement ratio, r_{sd}	
		Range	Design value
Positive projecting conduit in projection condition	Rock or unyielding soil	1.0	1.0
	Dense/well compacted soil*	0.5-0.8	0.7
	Loose/poorly compacted soil	0.0-0.5	0.3

* The value of the settlement ratio is a function of the degree of compaction of the fill material adjacent to the pipe.

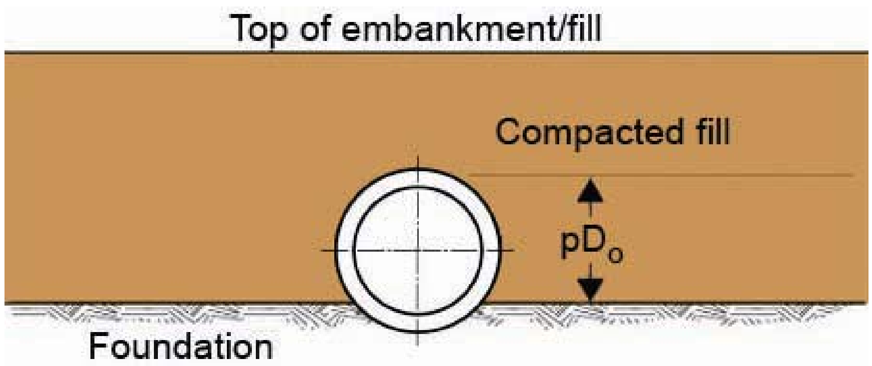


Figure 35.—Projection ratio, *p* = depth of the foundation material below the top of the conduit divided by the outside diameter of the pipe (D_O). (Note: This figure is not intended to show all the design details required.)

Figure 36 provides values for the positive projection load coefficient, C_c, for various values of the product of the settlement ratio, r_{sd}, and the projection ratio, p. Since the effect of μ is minimal, $K\mu$ is assumed to be 0.19 for the complete projection condition and 0.13 for the complete trench condition in figure 36.

The pressure on the top of the pipe may be determined by:

$$P_s = \frac{W_c}{D_O} \qquad \text{(2-6)}$$

where:

P_s = pressure due to the weight of soil on top of the pipe, lb/ft^2
W_c = soil load, lb/linear foot of pipe (see equation 2-2)
D_O = outside diameter of the pipe, ft

The range of potential soil loading on the conduit should be determined using the potential range of parameters, such as the total unit weight of the soil, settlement ratio, and projection ratio. Example A-2 in appendix A compares soil loading using both the Marston and prism theories.

2.1.3 Increase in soil loading due to a dam raise

The height of an embankment dam may be increased to provide additional flood protection or to enlarge the reservoir. The soil loading resulting from a dam raise may not be the same soil load as a dam originally constructed to the new height. The existing embankment and foundation will have experienced some, if not all, of the consolidation from the original embankment construction. The increase in soil load may be determined by finite element programs, (see section 3.1), or estimated using the following equation based on the stress distribution of an infinite footing:

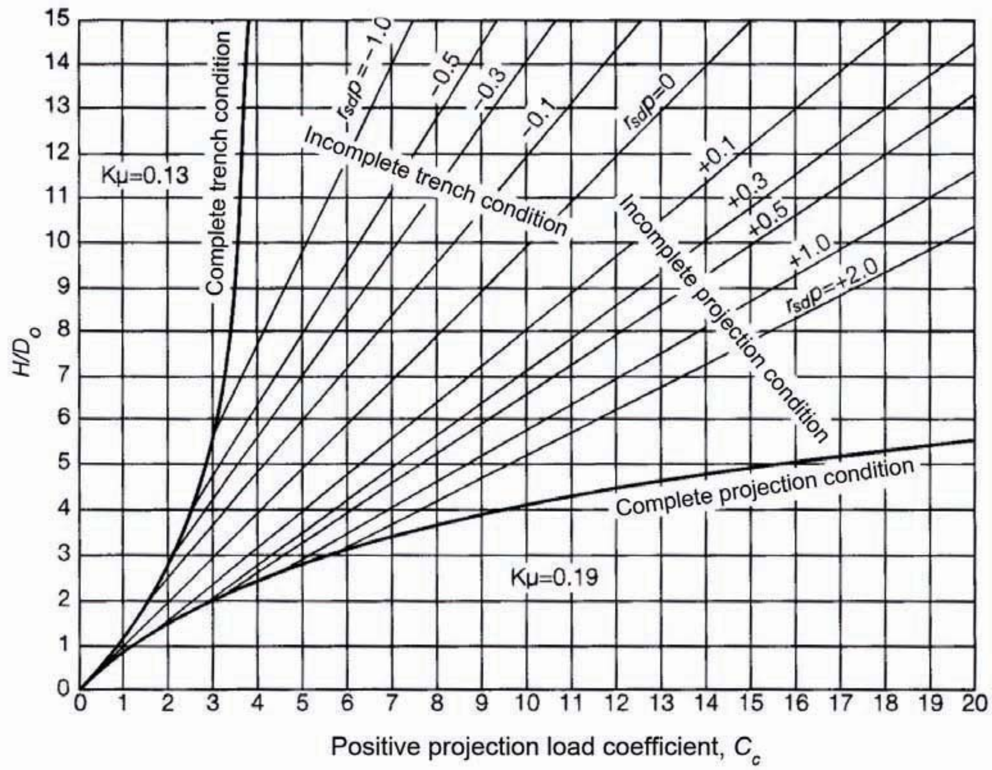


Figure 36.—Values for the positive projection load coefficient (C_c).

$$\Delta P_s = \frac{\Delta H \gamma t_w}{t_w + H_i} \tag{2-7}$$

where:

ΔP_s = increase in soil loading due to a dam raise, lb/ft^2
ΔH = increase in dam height, ft
γ = total unit weight of the soil, lb/ft^3
t_w = top width of existing dam crest, ft
H_i = initial height of existing dam, ft

2.2 Hydraulic Loading

Embankment conduits may experience hydraulic loading from internal hydrostatic pressure, surge pressure, internal vacuum pressure, or external hydrostatic pressure. Drainpipes are assumed to operate in a nonpressurized condition and typically do not experience this type of loading.

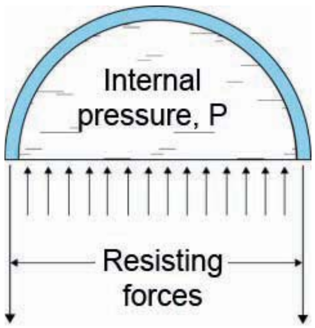


Figure 37.—Internal hydrostatic pressure.

2.2.1 Internal hydrostatic pressure

Internal hydrostatic pressure must be resisted by tensile stress (hoop stress) in the pipe walls, as shown in figure 37. The internal hydrostatic pressure is typically no more than the pressure due to the full reservoir head.

As with internal hydrostatic pressure, surge pressure must also be resisted by tensile stress (hoop stress) in the pipe walls. Surge pressure (water hammer) occurs when the flow velocity in the pipe is suddenly stopped or changed. When flow is suddenly stopped, the mass inertia of the flowing water is converted into a pressure wave or high static head on the pressure side of the pipe. Some of the most common causes of surge pressure in an embankment conduit occurs during the opening and closing of gates or valves, starting and stopping pumps, or entrapped air.

Surges may generally be divided into two categories: transient surges and cyclic surges. Transients are described as the intermediate conditions that exist in a system as it moves from one steady state condition to another. Cyclic surging is a condition that recurs regularly with time. Plastic pipe may eventually fatigue if exposed to continuous cyclic surging at sufficiently high frequency and stress.

Recurring surge pressures occur frequently and are inherent to the design and operation of the system (such as normal pump startup or shutdown and normal gate or valve opening and closure). Occasional surge pressures are caused by emergency operations. Occasional surge pressures are usually the result of a malfunction, such as power failure or system component failure, which includes pump seize-up, gate or valve-stem failure, and pressure-relief-valve failure.

The pressure wave caused by the surge travels back and forth in the pipe, getting progressively lower with each transition from end to end. The magnitude of the pressure change caused by the surge pressure wave depends on the elastic properties of the pipe and water as well as the magnitude and speed of the velocity change. The maximum surge pressure is equal to:

$$\Delta H = \frac{a\Delta V}{g} \tag{2-8}$$

or

$$\Delta P = \frac{a\Delta V}{g}\frac{\gamma_w}{144} \tag{2-9}$$

where:

ΔH = surge pressure, feet of water
a = velocity of the pressure wave, ft/s
ΔV = change in velocity of water, ft/s
g = acceleration due to gravity
= 32.2 ft/s^2
ΔP = surge pressure, lb/in^2
γ_w = unit weight of water, lb/ft^3
= 62.4 lb/ft^3

The maximum surge pressure results when the time required to stop or change the flow velocity is equal to or less than ($2L/a$) such that:

$$T_{CR} \le \left(\frac{2L}{a}\right) \tag{2-10}$$

where:

T_{CR} = critical time, s
L = distance within the pipe that the pressure wave moves before it is reflected back by a boundary condition, ft
a = velocity of the pressure wave, ft/s

The velocity of the pressure wave, a, may be estimated by:

$$a = \frac{12\sqrt{\frac{K_L}{\rho}}}{\sqrt{1+\left(\frac{K_L}{E}\right)\left(\frac{D_i}{t}\right)}} \tag{2-11}$$

or

$$a = \frac{12}{\sqrt{\frac{\gamma_w}{g}\left(\frac{1}{K_L}+\frac{D_i}{Et}\right)}} \tag{2-12}$$

where:

K_L = bulk modulus of water, lb/in^2
= 300,000 lb/in^2
ρ = density of water, slugs/ft^3
= 1.93 slugs/ft^3

E = modulus of elasticity of pipe material, lb/in^2 (140,000 lb/in^2 for HDPE, and 400,000 lb/in^2 for PVC. Note: The modulus of elasticity for surge/water hammer analysis is conservatively assumed to be higher than the value used for buried pipe analysis.)
D_i = inside diameter of the pipe, in
t = wall thickness of the pipe, in
γ_w = unit weight of water, lb/ft^3
= 62.4 lb/ft^3
g = acceleration due to gravity, ft/s^2
= 32.2 ft/s^2

For solid wall plastic pipe, the velocity of the pressure wave, *a*, may be expressed as:

$$a = \frac{12\sqrt{\dfrac{K_L}{\rho}}}{\sqrt{1 + \dfrac{K_L(SDR-2)}{E}}} \tag{2-13}$$

or

$$a = \frac{12}{\sqrt{\dfrac{\gamma_w}{g}\left(\dfrac{1}{K_L} + \dfrac{SDR-2}{E}\right)}} \tag{2-14}$$

where:
K_L = bulk modulus of water, lb/in^2
= 300,000 lb/in^2
ρ = density of fluid, slugs/ft^3
= 1.93 slugs/ft^3
SDR = Standard Dimension Ratio
= D_O/t
D_O = outside diameter of the pipe, in
t = minimum wall thickness of the pipe, in
E = modulus of elasticity of pipe material, lb/in^2 (140,000 lb/in^2 for HDPE, and 400,000 lb/in^2 for PVC. Note: The modulus of elasticity for surge/water hammer analysis is conservatively assumed to be higher than the value used for buried pipe analysis.)
γ_w = unit weight of water, lb/ft^3
= 62.4 lb/ft^3
g = acceleration due to gravity, ft/s^2
= 32.2 ft/s^2

The term "standard dimension ratio (SDR)" is widely used in the plastic pipe industry. SDR is sometimes used interchangeably with the term "dimension ratio (DR)." Both terms refer to the same ratio, which is a dimensionless term that is obtained by dividing the average outside diameter of the pipe by the minimum pipe wall thickness. These ratios were developed out of convenience rather than out of necessity. They have been established to simplify standardization in the specification of plastic pipe internationally. Since these define a constant ratio between outer diameter and wall thickness, they provide a simple means of specifying product dimensions to maintain constant mechanical properties regardless of pipe size. In other words, for a given SDR or DR, pressure capacity and pipe stiffness remain constant regardless of pipe size.

2.2.2 Internal vacuum pressure

Embankment conduits may be subject to an effective external pressure because of an internal vacuum pressure, P_V. Sudden valve closures, shutoff of a pump, or drainage from high points within the system often creates a vacuum. Embankment conduits (e.g., outlet works and siphons) are subject to internal vacuum pressures (figure 38) if they are not adequately vented. Internal vacuum pressure can lead to buckling (collapse) of the conduit. Internal vacuum pressure may be intermittent (short term), for long durations, or continuous (long term). The internal vacuum pressure determined by:

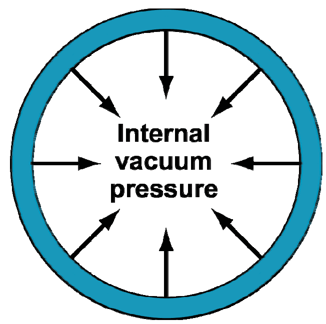


Figure 38.—Internal vacuum pressure.

$$P_V = \frac{12 \times W_V}{D_i} \tag{2-15}$$

where:

P_V = internal vacuum pressure, lb/ft^2
W_V = vacuum load per linear foot of pipe, lb/ft
D_i = inside diameter of the pipe, in

Example A-3 in appendix A demonstrates the principles involved with accommodating internal vacuum pressure in a siphon design.

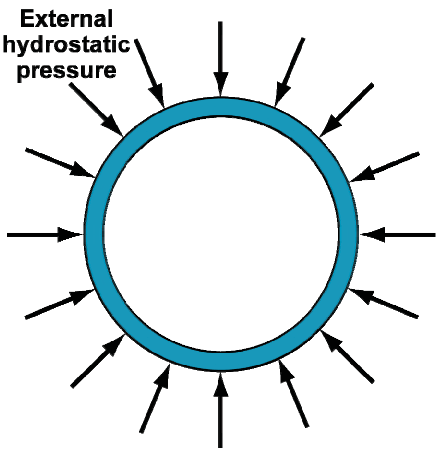


Figure 39.—External hydrostatic pressure.

2.2.3 External hydrostatic pressure

Embankment conduits and drainpipes beneath the water table or phreatic line within the embankment are subject to external hydrostatic pressure. Even pipes encased in concrete or grout are subject to external hydrostatic pressure as a result of water reaching the outside surface of the pipe by entering through cracks in the encasement material or simply seeping through the porous encasement material. External hydrostatic pressure, as shown in figure 39, may lead to buckling or collapse of the pipe. The external hydrostatic pressure may be determined by:

$$P_G = \gamma_w h_w \qquad (2\text{-}16)$$

where:

P_G = external hydrostatic pressure, lb/ft^2
γ_w = unit weight of water, lb/ft^3
= 64 lb/ft^3
h_w = height of water above the top of the pipe, ft

External hydrostatic pressure is often the controlling loading condition for plastic pipe used for conduits in embankment dams. This is due to the critical buckling pressure being directly proportional to the modulus of elasticity of the pipe. The long term modulus of plastic pipe can be as low as 1/100th as that for concrete pipe and 1/1000th of steel pipe. See sections 3.1.2 and 3.3.2 for guidance on accommodating external hydrostatic pressures.

2.3 Construction Loading

Buried conduits may be subjected to wheel loads during construction or throughout the life of the project. Pressures on the pipe depend on many factors, such as the vehicle's weight, speed, tires, surface smoothness, and depth of the pipe. Wheel loadings diminish as the depth of fill over the pipe increases. Loads from light duty vehicles tend to have little impact on buried pipe, but heavy construction equipment can seriously damage pipe with inadequate cover. For example, during construction, rough surfaces over pipes can cause scrapers to accelerate and decelerate vertically (i.e., bounce). Research has measured stresses representing impact factors with large magnitudes caused by bouncing scrapers (Bureau of Reclamation, 1984, p. 1). The higher the speed and greater the roughness, the higher the impact factor. Controls should be placed on construction practices for buried pipe. Limits on the speed of construction equipment over the pipe should be implemented. Although this is surface-roughness dependent, as a general rule, the speed of equipment crossing over pipe should be limited to 5 mi/hr until there is 2 to 4 feet of cover. Figure 40 shows an example of an HDPE pipe that has experienced damage due to insufficient cover. The effect of wheel loads lessens with the depth of fill. When the depth of fill is 2 feet or more, wheel loads may be considered as uniformly distributed over a wider area above the pipe (trapezoid shape with sides equal to 1¾ times the depth of fill) (NRCS, 2005, p. 52-8). Wheel loads may also be computed by AWWA's *PE Pipe—Design and Installation* (2006).

The pressure may be estimated by:

$$P_w = \frac{W_L}{(1.75H)^2} \tag{2-17}$$

where:

P_w = pressure on the pipe from a wheel load, lb/ft^2
W_L = wheel load, lb
H = height of soil above the top of the pipe, ft

Figure 40.—The crown of this single wall corrugated HDPE pipe has been damaged due to construction traffic crossing over it. Insufficient cover over the pipe was the likely cause.

Soil and encasement materials requiring compaction within 2 feet of the pipe should be compacted with manually operated compaction equipment. Heavier compaction equipment may be used once the depth of soil over the pipe has reached 2 to 4 feet. A more detailed analysis procedure for wheel loading may be found in NRCS's *Structural Design of Flexible Conduits* (2005, p. 52-7) or in chapter 2 of *Buried Pipe Design* (Moser, 2001). An example of the impact construction loads may have on a buried plastic pipe is included in NRCS's *Structural Design of Flexible Conduits* (2005, p. 52B-29).

Chapter 3

Structural and Hydraulic Design

The design of embankment conduits and drainpipes generally is divided into two categories: rigid and flexible. Rigid design assumes the pipe maintains its shape under loading by transferring the load to the foundation through the pipe wall. Rigid pipe is considered stiffer than the surrounding soil and does not require support from the surrounding fill. Rigid pipe will only allow minimal deflection without structural distress. Reinforced cast-in-place and precast concrete, clay, and cast iron pipe are examples of rigid pipe.

Flexible design assumes that the pipe is less stiff or only slightly less stiff than the surrounding soil and deforms without experiencing structural damage. Steel, ductile iron, CMP, aluminum, fiberglass, HDPE, and PVC are examples of flexible pipe. A flexible pipe derives its load-carrying capacity from its ability to transfer load to the surrounding soil. Under external load, the pipe deflects, developing soil support along the sides of the pipe. The deflection of the cylindrical pipe relieves the pipe of some of the load by transferring load to the soil surrounding the pipe. A flexible pipe is defined as one that deflects at least 2 percent out-of-round in cross-section without structural distress. The load that ultimately reaches the buried pipe from the dead weight of the soil and any surcharge depends upon the shear strength of the soil, its stiffness, and the buried pipe classification (see chapter 2). The transfer of load from the pipe to the surrounding soil results in lower bending and compressive stresses than would be experienced by rigid pipes. However, even small earth loadings can result in pipe deflection, if the surrounding soil provides insufficient support. For guidance on the loading conditions applied to buried pipe see chapter 2. Example A-1 in appendix A demonstrates the principles used in flexible pipe design.

As discussed in chapter 1, thermoplastic pipe such as HDPE and PVC has commonly been used in embankment dams. However, thermoset pipe (i.e., CIPP) has been used only in limited application. Therefore, this chapter will not address the structural and hydraulic design of thermoset plastic pipe. The reader is directed to ASTM F 1216 for guidance on the design considerations for CIPP and to AWWA's *Fiberglass Design Manual* (2005). Although structural and hydraulic design of thermoset pipe will not be discussed, the reader may find some of the guidance presented in chapter 3 beneficial in understanding the basic principals of plastic pipe which are of critical importance to applications for dams.

3.1 Flexible Pipe

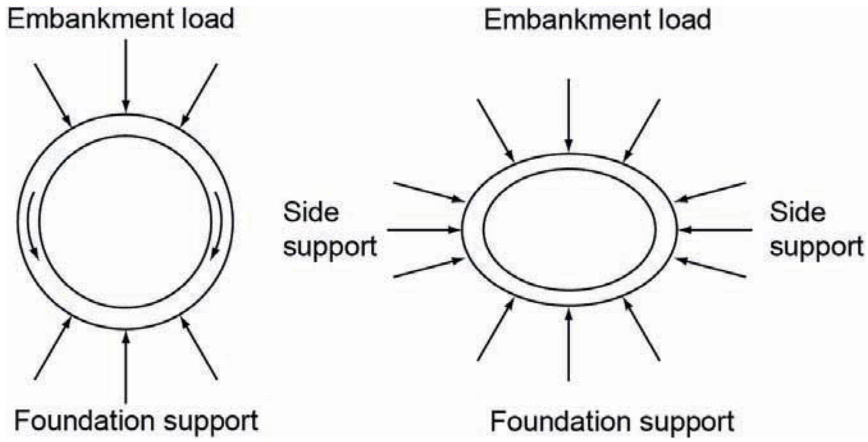
Flexible pipe design requires the load on pipe to be transferred to the soil surrounding the pipe. As the loading increases, the vertical diameter decreases and the horizontal diameter increases. Figure 41 illustrates the differences in load transfer for rigid and plastic pipe.

Since plastic pipe deflects under load, the modulus of elasticity of the plastic is an important material parameter used in the structural design of plastic pipe. The modulus of elasticity of plastic is a material property that describes the stress/strain behavior of a material in the linearly elastic region. However, for viscoelastic materials like HDPE and PVC, generally, the stress/strain curve is not linear, and the modulus of elasticity is often called "apparent modulus of elasticity," meaning it changes depending on the load amplitude and duration. As stress relaxation occurs under constant load, the modulus of elasticity decreases from a short-term modulus to a long term modulus. The ratio of the short-term to the long-term modulus of elasticity is approximately 3 for PVC and 5 for HDPE. Typical modulus of elasticity values are given in table 3:

Table 3.—Typical modulus of elasticity values for HDPE and PVC pipe

Material	Modulus of elasticity, lb/in^2	
	Short-term	Long-term
HDPE	110,000 - 140,000	22,000 - 30,000
PVC	360,000 - 400,000	100,000 - 140,000

The short-term modulus of elasticity is recommended for conditions that change through time, such as deflection or strain. Research shows that the short-term modulus of elasticity does not decrease after long-term loading. The calculated short-term modulus of elasticity actually increased when an incremental load was applied and increased the deflection (Moser, 2001, p. 415). The concept for this recommendation allows that soil settlement around a buried pipe occurs in dynamic, discrete, multiple events as the soil consolidates or soil grains are reoriented. Once movement occurs, soil arching redistributes the load, and no further deflection occurs for that particular event (AWWA, 2006, p. 58). However, as the next event occurs, these load increments are felt like impulse loads and the pipe resists them with its short-term elastic properties. In analysis for buckling, the modulus of elasticity and Poisson's ratio (for HDPE) should represent the expected duration of the expected load (i.e., the short-term properties should be used for live loads and the long-term properties for static loads, such as soil loading). The long-term modulus of elasticity should be used for all analysis on solid wall pipe with SDR values less than 13.5 because these pipes typically carry a substantial portion of the



(a) Load transfer in rigid pipe

(b) Load transfer in flexible pipe

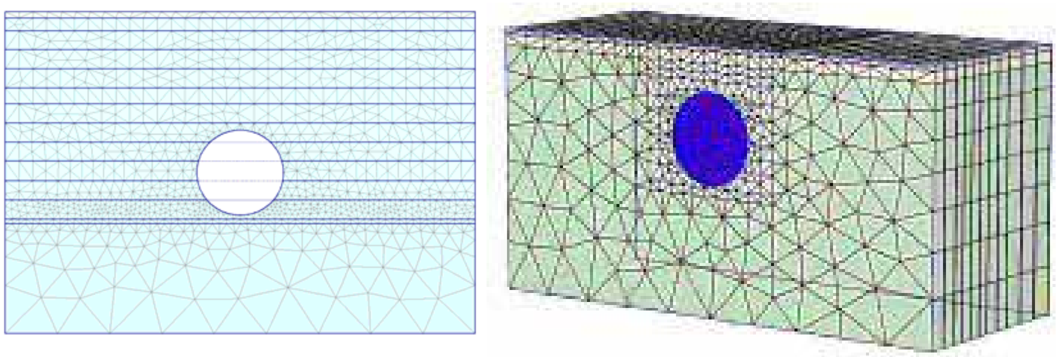
Figure 41.—Load is transferred differently for rigid and flexible pipe (Howard, 1996).

load and the long-term modulus is more conservative. For additional discussion of short- and long-tem modulus, see PPI's *Handbook of Polyethylene Pipe* (2006).

The designer can use finite element programs for structural design and analysis and evaluation of buried pipes to solve complex pipe-soil interaction problems. The soil surrounding the pipe is set up as a mesh of soil elements, and each element can be assigned different properties. Finite element programs not only allow for more realistic soil models than design equations, but also can model the effects of construction and special loadings.

Often, buried pipes in low hazard potential dams are installed with little or no field monitoring or other quality controls. In these types of situations, pipes should be designed with simplified procedures that are known to be conservative and provide ample protections against poor construction procedures. Finite element analyses are best used to investigate behavior and design buried pipes for projects with unusual installation conditions, such as deep fills or proximity to structures, on large projects where a significant investment warrants extra effort in design, or for projects where the consequences of failure are significant. Pipes buried in dams can meet all of these criteria.

Finite element analysis allows for modeling of pipe and soil using discrete elements that can each be assigned separate properties, accurate modeling of backfill soils, natural soil strata and inclusions, and pipe material behaviors. Computers now have the power to complete complex analyses in both two and three dimensions (figure 42).



2-D analysis 3-D analysis

Figure 42.—Examples of two- and three-dimensional meshes.

Any general purpose finite element program can be used to investigate buried pipe behavior, but only two have soil models specifically developed for modeling soil behavior and are widely used, Culvert ANalysis and DEsign (CANDE) (Musser, 1989) and Plaxis (Brinkgreve and Broere, 2004). CANDE is an older program developed by the U.S. Federal Highway Administration specifically to investigate the performance of pipes and culverts. This program exists only as a disk operating system (DOS) program that is relatively laborious in conducting analyses, although Soil Structure Interaction Specialists (Webb, 2005) have developed one interface using AutoCAD. The National Cooperative Highway Research Program instituted a project in 2005 to update CANDE and develop a Windows interface (NCHRP Project 15-28). Plaxis is a much newer program with an excellent user interface. Plaxis was developed to investigate soil-structure interaction for a wide variety of buried structure problems, including buried pipes. Both programs have soil models that incorporate nonlinear, stress-dependent soil stiffness and strength, which has been shown to be critical for accurate modeling of buried pipe problems. Both of these soil models require numerous input parameters, but CANDE includes standard properties for a variety of soils that allow users to select only a soil type and compaction level. CANDE and Plaxis have been used for analysis of a range of buried pipe applications both rigid and flexible, including round and arch shapes with large spans. CANDE has been and continues to be the most common program for long-span, flexible pipes. Application to long-span, flexible pipes was one of the key types of applications it was developed for. Plaxis has not been as widely used, but it is also quite suitable for analysis.

CANDE allows the user to select from several soil models. The model that has been most widely accepted incorporates the hyperbolic Young's modulus developed by Duncan, et al. (1980) and the hyperbolic bulk modulus developed by Selig (1988, pp. 99-116). The properties for this model developed by Selig have been used to develop the design procedures currently adopted by AASHTO's *Bridge Design Specifications* (2005) for concrete and thermoplastic pipe. For backfill soils, parameters are available for three types of soils: (1) coarse-grained soils with little or no fines, (2) sandy or gravelly silts or silty or clayey coarse grained soils, with low plasticity, and (3) clay soils. These three sets of parameters have been shown to be suitable for most design problems. Selig (1988, pp. 99-116) describes the procedures necessary to develop parameters for specific soils and provides parameters for a range of in situ soils. A significant drawback in the use of Plaxis is that it does not include suggested parameters for typical soils. This is in part because Plaxis was developed primarily to model in situ soils, which are much more difficult to characterize than compacted soils.

Key elements to consider in finite element analysis of buried pipe problems include:

- The power of finite element analysis lies in accurate modeling of the pipe-soil system. This power can only be truly realized with accurate input and careful interpretation of results.

- Models should include incremental construction where the backfill soils are placed incrementally as in actual construction. This has been found to be important to accurately model displacements and stresses.

- Live load analysis, though often completed with two-dimensional analysis, is actually a three-dimensional problem. This complicates interpretation somewhat, but experience has shown that by reducing the applied load at the surface of a finite element mesh to account for live load attenuation in the third dimension between the ground surface and the top of the pipe, reasonable accuracy can be achieved.

- Selecting the backfill soil model can present the same problems as selecting the appropriate modulus of soil reaction (E') value in the Iowa formula (Spangler, 1941). If the fill height is high enough to raise concerns, monitoring of actual deflections might be considered, so that deflection predictions (and soil model) can be checked based on back-calculated calibrations of the soil model under the design fill. Until enough case studies have been performed to demonstrate and document actual field performance versus CANDE's predicted performance, the designer may want to compare the results of CANDE to those obtained using more traditionally accepted methods.

- The design method selected by the designer should be no more sophisticated than the construction procedure. For pipe that is buried without much control, simplified, conservative design procedures are appropriate. For special conditions (large pipe, deep fills, long pipes), significant economies can be achieved with more sophisticated design, and the cost of increased quality control in the field is justified.

Typical failure modes of flexible pipes are shown in figure 43. Flexible pipe design of buried plastic pipe includes analyses of the wall crushing, buckling resistance, allowable long-term deflection, and allowable strain. Deflection and buckling most often control the design of flexible pipe. Table 9 in section 3.5.6 provides the appropriate method of determining the soil load based on soil type and type of conduit.

3.1.1 Wall crushing

Wall crushing in plastic pipe is characterized by localized yielding when the in-wall stress reaches the yield stress of the pipe material (Moser, 2001, p. 499). Wall crushing typically occurs at the 3 and 9 o'clock positions as illustrated in figure 43a. Figure 44 shows an example of wall crushing. This localized yielding can occur in improperly designed stiff flexible pipes installed in deep, highly compacted fill. Less stiff flexible pipe more frequently fails from wall buckling, as discussed in section 3.1.2.

Resistance to wall crushing of plastic pipe is evaluated by:

$$T_{pw} = \frac{PD_O}{2} \tag{3-1}$$

where:

T_{pw} = thrust in pipe wall, lb/in
D_O = outside diameter of the pipe, in
P = design pressure (P_S+P_V+P_W), lb/in^2 (see equations 2-6, 2-15, and 2-17)

The required wall cross-sectional area is determined by:

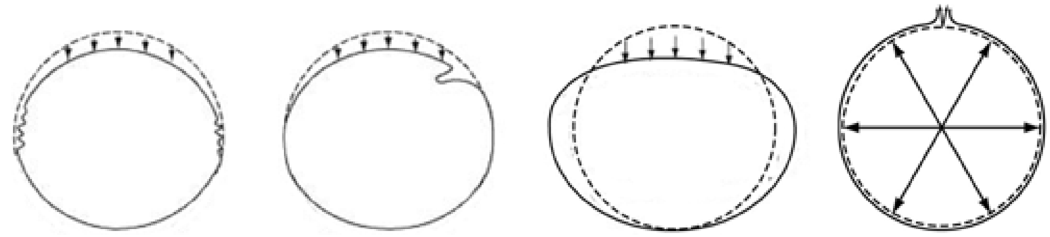
$$A_{pw} = \frac{T_{pw}}{\sigma} \tag{3-2}$$

where:

A_{pw} = area of the pipe wall, in^2/in of pipe length
T_{pw} = thrust in pipe wall, lb/in
σ = allowable long-term compressive stress, lb/in^2
= $HDB/2$
HDB = hydrostatic design basis of the pipe, lb/in^2

The actual area for a solid wall pipe wall may be computed as:

$$A_{pw} = \frac{(D_O - D_i)}{2} \text{ or } t \tag{3-3}$$



(a) Wall crushing (b) Wall buckling (c) Excessive deflection (d) Excessive internal hydrostatic pressure

Figure 43.—Typical failure modes for flexible pipes.

Figure 44.—Single wall corrugated HDPE pipe experiencing wall crushing.

where:

A_{pw} = area of the pipe wall, in^2/in of pipe length
D_O = outside diameter of the pipe, in
D_i = inside diameter of the pipe, in
t = wall thickness of the pipe, in

The actual area of the pipe wall for corrugated (single and profile wall) may be obtained from the manufacturer or ASTM standard.

3.1.2 Wall buckling

External loadings from soil pressures, external hydrostatic pressure, or internal vacuum can cause in inward deformation known as wall buckling (collapse). Wall buckling is characterized by localized yielding, as illustrated in figure 43b. Figure 45 shows an example of single wall corrugated HDPE drainpipe which has failed due to buckling. Wall buckling can occur due to insufficient pipe stiffness. The more flexible the plastic pipe, the more unstable the wall structure will be in resisting wall buckling (Moser, 2001, p. 110). Plastic pipe encased in soil may buckle due to excessive loads and deformations. The total load must be less than the allowable buckling pressure. If good backfill is used with sufficient stiffness, wall buckling is often not a concern and deflection will normally govern the design. This is true in most cases, with the exception for dam applications with shallow cover and internal vacuum pressures or fine grained backfill around embankment conduits used in low hazard potential dams.

The allowable buckling pressure may be computed with various equations (Moser, 2001; Chevron Phillips, 2002, p. 105; AWWA, 2006, p. 61-63; or Uni-Bell, 2001, p. 252). Equation 3-4 is from Moser (2001, p. 112).

$$q_a = \frac{1}{FS}\left(32R_w B' E' \frac{EI_{pw}}{D_O^3}\right)^{1/2} \tag{3-4}$$

where:

q_a = allowable buckling pressure, lb/in^2

FS = factor of safety

= 2.5 for $(h/D_O) \geq 2$

= 3.0 for $(h/D_O) < 2$

where h = height of fill above the top of pipe, in

D_O = outside diameter of the pipe, in

R_w = water buoyancy factor

$$= 1 - 0.33(h_w/h),\ 0 < h_w < h \tag{3-5}$$

where h_w = height of water above top of the pipe, in

B' = empirical coefficient of elastic support

$$= \frac{4(h^2 + D_O h)}{1.5(2h + D_O)^2} \tag{3-6}$$

E' = modulus of soil reaction, lb/in^2

E = modulus of elasticity[1] of pipe material, lb/in^2

[1] A long-term modulus of elasticity is recommended if the pipe is subject to external soil or internal vacuum pressure in normal operations. If the pipe is subject to the pressure for short time periods and infrequently, use of the short-term modulus of elasticity is recommended.

Figure 45.—Single wall corrugated HDPE drainpipe experiencing failure due to buckling.

I_{pw} = pipe wall moment of inertia, in^4/in of pipe length

$$= \frac{t^3}{12} \text{ (for solid wall pipe)} \qquad (3\text{-}7)$$

where t = wall thickness of the pipe, in

(Note: To determine I_{pw} for corrugated single and profile wall pipe, contact the pipe manufacturer)

The allowable buckling pressure depends on the surrounding soil pressure. The allowable buckling pressure increases/decreases as the effective soil pressure surrounding the pipe increases/decreases. The effective soil pressure decreases as the height of water above the pipe increases. The water buoyancy factor, R_w, accounts for the reduction in effective soil pressure for water levels in the soil above the top of the pipe.

For a siphon extending over the crest of a dam that does not have the support of surrounding soil or controlled low strength material, the pipe should be designed to withstand unconstrained wall buckling as described in section 3.3.2 and illustrated in Example A-3 in appendix A.

If plastic pipe is encased in a rigid material, such as grout, the potential for the pipe to buckle as a result of external hydrostatic pressure needs to be considered in accordance with the guidance provided in section 3.3.2.

3.1.3 Deflection

Deflection of plastic pipe in cross section is the decrease in vertical diameter and the simultaneous increase in the horizontal diameter resulting from the loadings encountered. The amount of deflection along the length of pipe can vary significantly due to the inherent differences in soil compaction, type, and loading. Deflection of a flexible pipe is a performance limit to prevent cracking of the pipe, avoid reversal of curvature, limit bending stress and strain, avoid pipe flattening, and reduce the potential for leaking joints. Deflection is illustrated in figure 43c. Figure 46 shows an example of a single wall corrugated HDPE pipe experiencing excessive deflection leading to buckling.

Excessive deflection can eventually lead to the collapse of the pipe. The normal sequence involved in pipe collapse is summarized as follows (Spangler, 1941):

1. The embankment is built high enough to cause enough loading so the pipe deflects. The vertical diameter becomes smaller and the horizontal diameter becomes greater.

2. The outward movement of the sides of the pipe against the enveloping earth brings into play the passive pressure of the earth, which acts horizontally against the pipe and reduces the rate at which the deflection occurs.

3. As the embankment is constructed higher, the deflection continues until the top of the pipe becomes approximately flat.

4. Additional load causes the curvature at the crown to reverse direction, becoming concave upward.

5. The sides of the pipe pull inward, which eliminates most of the side support of the pipe since it is a passive force and cannot follow the inward moment.

6. The pipe rapidly collapses.

The Iowa Deflection Formula was developed by Spangler (Spangler, 1941) based on research of corrugated metal pipe (CMP) under earthen embankments. Spangler realized that deflection of CMP was not a function of pipe strength alone, but rather the soil-pipe system. The formula was later modified by Watkins (Watkins and Spangler, 1958) as the modified Iowa Equation to predict deflection of a buried flexible pipe. The deflection of buried, nonpressurized, flexible pipe increases with time as the supporting soil around the conduit consolidates and the soil-pipe system approaches equilibrium. The rate of deflection and ultimate deflection vary with the surrounding soil properties, particularly material type and density. Deflection continues to increase as long as the soil around the pipe continues to consolidate. To account for this, the Modified Iowa Equation includes a deflection lag factor, D_L.

Figure 46.—Single wall corrugated HDPE drainpipe experiencing excessive deflection leading to buckling.

A D_L value of 1.0 to 1.5 is often recommended. A D_L of 1.0 has been used when the soil load is determined by the soil prism theory (Uni-Bell, 2001, p. 230). Plastic pipes designed with a D_L value of 1.5 have historically performed well in embankment dams.

At a depth of about 50 feet, these equations for deflection become conservative since they neglect the load reduction due to arching and increases in E' due to lateral earth pressure (stiffening of the soil surrounding the pipe due to over burden pressure). Other than in mine tailing impoundments, flexible plastic pipe for embankment conduits and drainpipes are rarely used in fill heights greater than 50 feet in depth, so this should not be a concern. Deeply buried pipes are outside the scope of this document and the reader should consult other methods of analysis, such as those discussed in PPI's *Handbook of Polyethylene Pipe* (2006) or by finite element analysis.

Wheel loads should be considered when estimating deflection and vehicles may cross the pipe alignment. See section 2.3 for guidance on computing wheel loadings on top of pipes.

Internal vacuum loads should be considered for siphons or when the internal hydraulic behavior of the system may allow an internal vacuum to develop. See section 2.2.2 for a discussion of internal vacuum pressure.

Deflection of nonencased plastic pipe could potentially allow pathways to open within the soil and result in the development of internal erosion along the pipe. Deflection can also reduce flow capacity and cause joint leakage. For these reasons, nonencased plastic pipe should not be allowed within significant and high hazard potential dams (see section 3.5 for guidance on encasement). Although there is not a uniformly accepted deflection limit for nonencased plastic pipe within low hazard potential embankment dams, it is often limited to 5 percent (NRCS, 2005, pp. 52-11). In mine tailings dams that are periodically monitored, a 7.5-percent deflection limit is often used (see chapter 7). Deflection limits are set to avoid the development of "reversal of curvature," limit bending stress and strain, and avoid pipe flattening. The NRCS has installed plastic pipe in hundreds to thousands of small (less than 25 feet in height), low hazard potential dams. The design deflection limit of 5 percent has resulted in satisfactory performance.

The use of a filter zone surrounding the pipe is a valuable defensive design measure, even for low hazard potential dams with favorable conditions. Some designs for low hazard potential dams may not employ a filter zone around the pipe, but eliminating this valuable feature should be carefully considered and justified. Filter diaphragms should only be eliminated when extremely favorable soil conditions, good conduit construction materials and methods, reliable construction practices, and favorable foundation conditions exist. See chapter 6 of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) for guidance on the design and construction of filters. In addition, designers of mine waste-disposal dams should consider the discussion in chapter 7 when determining the installation requirements for decant conduits in these types of dams.

For drainpipes, it is recommended that the allowable deflection be limited to 7.5 percent, as is often recommended by manufacturers for other plastic pipe applications. However, the designer will need to carefully consider where the drainpipe is being installed within the dam and may need to require more stringent deflection limitations, if deflection of the drainpipe could potentially result in internal erosion concerns. Deflection of the pipe may be decreased by the use of higher quality or more compact backfill or a thicker or stiffer pipe wall. Higher quality or more compact backfill has a greater impact on the deflection of the pipe than the stiffness of the pipe.

The Modified Iowa Equation may be modified as follows to compute the percent deflection of each type of pipe.

Solid wall pipe:

$$\frac{\%\Delta Y}{D} = \frac{(D_L P_s + P_W + P_V)K(100)}{\left[\left(\dfrac{2E}{3\left(SDR-1\right)^3}\right) + 0.061E'\right]} \tag{3-8}$$

Corrugated single and profile wall pipe:

$$\frac{\%\Delta Y}{D} = \frac{\left(D_L P_S + P_W + P_V\right)K(100)}{\left[0.149PS + 0.061E'\right]} \tag{3-9}$$

where:

$\%\Delta Y/D$ = percent deflection
$D = D_O$ = outside diameter of the pipe, in
D_L = deflection lag factor
P_s = pressure due to the weight of soil on top of the pipe, lb/in^2 (see equation 2-6)
P_W = pressure on the pipe from a wheel load, lb/in^2 (see equation 2-17)
P_V = internal vacuum pressure, lb/in^2
K = bedding constant (typically 0.1 for soil embedment)
E = short-term modulus of elasticity of pipe material, lb/in^2 (see section 3.1)
SDR = standard dimension ratio of pipe, D_O/t
t = wall thickness of the pipe, in
E' = modulus of soil reaction, lb/in^2
PS = pipe stiffness, lb/in^2

SDR is the ratio of the outside diameter of the pipe (D_O) to its wall thickness (t). A low SDR means a very strong pipe and a high SDR means a thinner wall, more flexibility, and less strength. Pipes with different outside diameters with the same SDR will tend to have similar flexibility. An SDR equal to or lower than 26 should be used for solid wall pipe since higher SDR values are extremely dependent upon the support provided by the surrounding soil.

The Modified Iowa Equation is only a guide and can be an imprecise prediction of deflection in certain situations. The accuracy of the predicted deflections is normally adequate for designs within the range of pipe and soil stiffness relationships covered by research. However, for very stiff and very flexible pipes, the Modified Iowa Equation excessively overstates the deflections on one end (very stiff) of the scale and understates them on the other end. The equation demonstrates the importance of the soil and the relatively small contribution of ring stiffness to ring deflection. The equation should never be used alone to design the wall thickness of a flexible pipe and should only be used to determine pipe deflection. The required pipe wall thickness for flexible pipe should also be determined as discussed in sections 3.1.1, 3.1.2, and 3.1.4. Limitations on the use and potential misuse of the Modified Iowa Equation are discussed in Schluter and Capossela (1998), Smith and Watkins (2004), Jeyapalan and Watkins (2004), and Howard (2006). The Modified Iowa Equation was originally developed to predict horizontal deflection, but has traditionally been used as an estimate for vertical deflection. Plastic pipe tends to deflect into a nearly elliptical shape, and the horizontal and vertical deflections may be considered to be equal for small deflections (Uni-Bell, 2001, p. 230).

The modulus of soil reaction, E', is an empirical soil stiffness used for many years to model the soil contribution to control deflection and in-ground buckling. The soil modulus cannot be measured in the laboratory or by an in-situ test and has usually been determined by measuring pipe deflection under a known load and calculating E'. Amster Howard (1977) developed E' values based on the soil prism load theory as shown in table 4 (Howard, 1996). The Howard parameters are the most commonly used E' values in general design practice. These values were back-calculated from measured vertical deflections at a number of flexible pipe installations. They provide a constant value for soil stiffness regardless of depth of fill and subsequent confinement. These values can be used to a cover depth up to 50 feet. The values for E' vary with soil type and compacted density. A conservative value for E' is recommended. The E' values presented in table 4 are average values for the type of material and percent compaction or relative densities shown. Many designers often reduce the values provided in table 4 by 25 percent to account for values below the average and variability along the length of pipe (PPI, 2006, p. 210). While Howard's E' values are most widely used in the Modified Iowa Equation in standards, in guidelines, and by designers, it should be noted that these values were derived using the prism load, a time factor to estimate long-term deflection, and vertical deflections. Howard recommends that soil loads be calculated using prism load theory, a deflection lag factor of 1.0, and a time factor that varies with backfill material and compaction. Further discussion on the use Howard's E' values and time factors can be found in Howard (2006).

Note that the Modified Iowa Equation was originally developed for a Marston load with a deflection lag factor of 1.5. However, the Marston load for a flexible embankment conduit would be similar to the soil prism load (it is sometimes less depending on the parameters assumed for the Marston load). In addition, for many years, the NRCS has predicted vertical deflection using the Modified Iowa Equation with the soil prism load and a deflection lag factor of 1.5, In summary, it is recommended that deflection be determined for a cover depth up to 50 feet by using the Modified Iowa Equation, soil prism loads, and a deflection factor of 1.5.

Whether or not the modulus of soil reaction varies with depth has been the subject of much research and conflicting opinion. Howard reported no correlation between E' and depth of fill. However, others (Hartley and Duncan, 1987) have demonstrated empirically and analytically that the value of E' increases with increasing depth of cover over the pipe. This is because of the increased confinement of the soil embedment by the surrounding soil. The increased confinement stiffens the soil embedment and raises its E' (AWWA, 2006, p. 59). Table 5 gives E' values for cover depths up to 20 feet as determined by Hartley and Duncan. The designer should compare E' values in both tables 4 and 5 and use the most conservative. The designer should base their selection of E' on project conditions, project requirements, judgment, and experience. If experience is lacking, an expert should be consulted on how to establish these values. For depths of cover exceeding 50 feet, the designer should consider alternate deflection calculations, as discussed in PPI (2006).

Table 4.—Average values of the modulus of soil reaction, *E′*, for the Modified Iowa Equation

Soil type—Pipe bedding material (Unified Soil Classification System—ASTM D 2487)	*E′* for degree of compaction of bedding, lb/in^2			
	Dumped	Slight, <85% Standard Proctor, <40% relative density	Moderate, 85%-95% Standard Proctor, 40-70% relative density	High, >95% Standard Proctor, >70% relative density
Fine-grained soil (LL ≥ 50) Soils with medium to high plasticity CH, MH, CH-MH. No data available	No data available; consult a competent soils engineer; otherwise, use *E′* = 0			
Fine-grained soil (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 30% coarse-grained particles	50	200	400	1,500
Fine-grained soil (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 30% coarse-grained particles. Coarse-grained soils with fines GM, GC, SM, SC contains more than 12% fines	150	400	1,000	2,500
Coarse-grained soils with little or no fines GW, GP, SW, SP contains less than 12% fines	200	700	2,000	3,000
Crushed rock. Not more that 25% passing ⅜-in sieve and not more that 12% fines; maximum size not to exceed 3 in.	1,000	1000	3,000	3,000

Notes:

- See table 6 for a description of soil classifications
- LL = liquid limit, %
- For use in predicting initial deflections only; appropriate deflection lag factor must be applied for long-term deflections
- If bedding falls on the borderline between two compaction categories, select lower *E′* value or average the two values
- Fines are soil particles that pass a No. 200 (75-μm) sieve
- Percent Proctor based on ASTM D 698 or AASHTO T99
- Percent Relative Density based on ASTM D 4253 and D 4254
- Values applicable only for cover of about 50 feet or less
- *E′* values are in lb/in^2
- Dumped – No compactive effort
- Slight – Some compactive effort,
  In-place density <85% standard Proctor, or <40% Relative Density
- Moderate – Intermediate level of compactive effort,
  In-place density ≥85% and <95% standard Proctor, or ≥40% and <70% Relative Density
- High – Considerable compactive effort,
  In-place density ≥95% standard Proctor, or ≥70% Relative Density

Table 5.—Hartley-Duncan's (1987) values of *E′*, modulus of soil reaction

Type of soil	Depth of cover, ft	*E′* for standard AASHTO relative compaction, lb/in^2			
		85%	90%	95%	100%
Fine-grained soils with less than 25% sand content	0-5	500	700	1,000	1,500
	5-10	600	1,000	1,400	2,000
	10-15	700	1,200	1,600	2,300
	15-20	800	1,300	1,800	2,600
Coarse-grained soils with fines (SM, SC)	0-5	600	1,000	1,200	1,900
	5-10	900	1,400	1,800	2,700
	10-15	1,000	1,500	2,100	3,200
	15-20	1,100	1,600	2,400	3,700
Coarse-grained soils with little or no fines (SP, SW, GP, GW)	0-5	700	1,000	1,600	2,500
	5-10	1,000	1,500	2,200	3,300
	10-15	1,050	1,600	2,400	3,600
	15-20	1,100	1,700	2,500	3,800

Issues concerning the use and misuse of the Modified Iowa Equation and the conflicting opinions of E' varying with depth only apply to flexible pipes. If a flexible plastic pipe is to be used as an embankment conduit in significant and high hazard potential dams, it should be encased in properly shaped reinforced cast-in-place concrete to facilitate compaction of earthfill against the conduit or grouted in place when used as a slipliner. Calculation of deflection is not necessary for encased plastic pipes. The principles of encased plastic pipe design are discussed in section 3.3.

Standard Proctor density or AASHTO relative compaction and relative density are not the same. Standard Proctor density is typically used for fine grained soils while relative density is typically used for coarse grained soils with few or no fines such as SW, SP, GP, and GW. Table 5 does not include the values for relative density. In order to use Table 5 for coarse grained soils compacted to a relative density, some experience in determining which AASHTO relative compaction column is appropriate for the percent relative density of the coarse grained soil is required.

For pipes installed in trenches, the support stiffness developed depends on the combined stiffness of the embedment material immediately adjacent to the pipe, plus the native soil in the trench. For this situation, the designer will need to determine the composite modulus of soil reaction. For guidance on determining this value, see AWWA's *PE Pipe-Design and Installation* (2006) and *PVC Pipe-Design and Installation* (2002).

Internal vacuum pressure (P_v) and pressure on the pipe from a wheel load (P_w) seldom occur at the same time. However, this could occur for siphon applications and should be considered in the computation of percent deflection.

3.1.4 Internal hydrostatic pressure

The internal hydrostatic pressure capacity of plastic pipe is given as a pressure rating for pipe manufactured in accordance with ASTM standards and as a pressure class for pipe meeting AWWA standards. Figure 43d illustrates internal hydrostatic pressure. The pressure class of PVC pipe manufactured in accordance with AWWA C900 is reduced by the surge pressure from an instantaneous velocity change of 2 ft/s. (AWWA C905 does not include an allowance for surge pressure). If surge pressure is not anticipated, the allowable internal pressure of AWWA C900 pipe may be increased accordingly. The pressure class or rating of PE pipe manufactured in accordance with AWWA C906 or ASTM standards has not been reduced for surge pressure. Surge pressure is typically not a concern for drainpipes or embankment conduits. See section 2.2.1 for a discussion of surge pressure. The design of PE and PVC pipe for surge pressure is described in AWWA's *PE Pipe—Design and Installation* (2006) and Uni-Bell's *Handbook of PVC Pipe* (2001), respectively. The designer should not rely on the surrounding fill to resist internal hydrostatic or surge pressures.

The manufacturing process of solid wall plastic pipe controls either the outside or inside diameter of the pipe. Either SDR or standard inside dimension ratio (SIDR) is provided in the applicable ASTM or AWWA standard and by the manufacturer depending upon the manufacturing process. The outside diameter of a pipe is the same for the available range of SDR values in outside-diameter-controlled pipe while the inside diameter of the pipe is the same for the available range of SIDR values of inside-diameter-controlled pipe. The pressure rating for solid wall plastic pipe may be determined by one of the following formulas.

For outside-diameter-controlled pipe:

$$PR = PC = \frac{2(HDS)}{SDR - 1} \tag{3-10}$$

For inside-diameter-controlled pipe:

$$PR = PC = \frac{2(HDS)}{SIDR + 1} \tag{3-11}$$

where:

PR = pressure rating, lb/in^2
PC = pressure class, lb/in^2
HDS = hydrostatic design stress, lb/in^2
HDS = HDB/FS
where HDB = hydrostatic design basis, lb/in^2
FS = factor of safety (2.5 for AWWA C900 pipe, 2.0 for all others [ASTM; AWWA C901, C905, and C906])
SDR = standard dimension ratio
SDR = D_o/t
where D_O = outside diameter of the pipe, in
t = wall thickness of the pipe, in
$SIDR$ = standard inside dimension ratio
$SIDR$ = D_i/t
where D_i = inside diameter of the pipe, in
t = wall thickness of the pipe, in

The hydrostatic design basis (HDB) is an approximate measure of the amount of stress a plastic material can resist over a long time period. The hydrostatic design stress (HDS) is the maximum stress the plastic can resist over a long period of time with a high degree of certainty that it will not fail. The HDS is based on the HDB, which is reduced by a design factor or factor of safety. A complete description of HDB and HDS is included in ASTM D 2837.

Corrugated plastic pipe (single and profile wall) typically is not pressure rated and should not be used in sustained pressure applications. Due to the limited allowable pressure for watertight joints in corrugated plastic pipe (single and profile wall) and the variability in the types of joints, the manufacturer's recommendations should always be consulted.

All pressure ratings are determined in an environment of approximately 73.4 ºF. As the temperature of the water or surrounding soil environment increases, the pipe has a reduction in strength and stiffness. The pressure rating should be decreased by the factors shown in table 6 or by the manufacturer's recommended service factors. The temperature reduction factor is applied directly (by multiplication) to the calculated pressure rating.

Table 6.—Temperature reduction factors

Temperature, °F	PVC factor	HDPE factor
73.4	1.00	1.00
80	0.88	0.92
90	0.75	0.81
100	0.62	0.70
110	0.50	0.65
120	0.40	0.60
130	0.30	0.55
140	0.22	0.50

Source: AWWA (2002) and PPI (2006).

For embankment dam applications, the pipe temperature rarely exceeds 73.4 °F and a temperature reduction factor of 1.0 is used. As the pipe temperature falls below 73.4 °F, the pressure capacity of the pipe increases. The pressure rating (or pressure class) are considered to be the same when the pipe temperature is 73.4 °F.

3.1.5 Strain

Total strain in a pipe wall can be caused by two actions: (1) hoop stress due to internal or external pressure in the pipe wall and (2) flexure of the pipe as it deforms. Longitudinal strain is typically not a concern in buried pipe applications with mild and constant slopes since the load and surrounding support is relatively constant along the length of pipe. If a homogeneous wall is assumed and pressure concentrations are neglected, the following formulas can be used to estimate strain.

Strain due to hoop stress in the pipe walls:

$$\varepsilon_h = \frac{PD_M}{2A_{pw}E} \quad \text{(for single and profile wall corrugated pipe)} \qquad (3\text{-}12)$$

$$\varepsilon_h = \frac{PD_M}{2tE} \quad \text{(for solid wall pipe)} \qquad (3\text{-}13)$$

where:

ε_h = maximum strain in the pipe wall due to hoop stress, in/ in
P = design pressure, lb/in^2

D_M = mean pipe diameter, in
= $D_i + 2c$ (for corrugated pipe)
D_i = inside diameter of the pipe, in
c = distance from the inside surface to the neutral axis, in, (as supplied by the manufacturer)
A_{pw} = area of the pipe wall, in^2/in of pipe length
E = short-term modulus of elasticity of pipe material, lb/in^2 (see section 3.1)
D_M = D_O - t (for solid wall pipe)
where D_O = outside diameter of the pipe, in
t = wall thickness of the pipe, in

Strain from ring deflection:

Maximum strains due to ring deflection or flexure may be determined by assuming the pipe remains an ellipse during deflections. The resulting equations are:

$$\varepsilon_f = 6\frac{t}{D_M}\frac{\Delta Y}{D_M} \quad \text{(for single and profile wall corrugated pipe)} \tag{3-14}$$

or

$$\varepsilon_f = \frac{t}{D_M}\left(\frac{3\Delta Y / D_M}{1 - 2\Delta Y / D_M}\right) = \frac{1}{SDR}\left(\frac{3\Delta Y / D_M}{1 - 2\Delta Y / D_M}\right) \quad \text{(for solid wall pipe)} \tag{3-15}$$

where:
ε_f = maximum strain in the pipe wall due to ring deflection, in/in of pipe wall circumference
t = wall thickness of the pipe, in
D_M = mean pipe diameter, in
$\Delta Y/D_M$ = $\%\Delta\, Y/D$ = percent deflection expressed as a decimal
SDR = standard dimension ratio
D_M = $D_i + 2c$ (for single and profile wall corrugated pipe)
where D_i = inside diameter of the pipe, in
c = distance from the inside surface to the neutral axis, in
D_M = D_O - t (for solid wall pipe)
where D_O = outside diameter of the pipe, in
t = wall thickness of the pipe, in

In a buried pipe these strain components act simultaneously. The maximum combined strain in the pipe wall can be determined by summing both components.

$$\varepsilon = \varepsilon_f \pm \varepsilon_b \tag{3-16}$$

where:

ε = maximum combined strain in pipe wall, in/in of pipe wall circumference

ε_f = maximum strain in the pipe wall due to ring deflection, in/in of pipe wall circumference

ε_h = maximum strain in the pipe wall due to hoop stress

In calculating the maximum combined strain, the strain due to hoop stress in the pipe wall, ε_h, resulting from applied internal pressure, if any, should be added to the maximum strain due to deflection, ε_f. If the strain due to hoop stress in the pipe wall is due to external load or internal vacuum pressure, the strain due to hoop stress in the pipe wall from the applied internal pressure should be subtracted to obtain the maximum combined strain, ε.

Utah State University (USU) has conducted research on PVC pipes deflected in cross section up to 20 percent. These pipe have not experienced failure after years of deflection. Additional research by USU indicates PVC can withstand strains up to 100 percent. Thus, the allowable deflection for PVC pipe limits strain in standard PVC pipes to an acceptable value. Research by Janson (1991) showed that pressure rated HDPE pipe (solid wall) would not fail due to strain. Therefore, computation of strain and comparison to an allowable strain is not recommended for PVC pipe and HDPE made of the resins recommended in this document. A strain limit of 5 percent is recommended for single and profile wall corrugated HDPE pipe.

The maximum strain in the pipe should be limited to:

$$\varepsilon \leq \varepsilon_{all} \qquad (3\text{-}17)$$

where:

ε = maximum combined strain in pipe wall, in/in of pipe wall circumference

ε_{all} = allowable strain for the pipe material, in/in

An example of a flexible pipe design is provided in appendix A, example A-1.

3.2 Rigid Pipe

Rigid pipe, typically reinforced cast-in-place concrete pipe, is designed to transfer the load from the pipe wall to the foundation; the pipe wall is strong enough to take the load without deflecting in the cross section when the load transfer occurs.

Plastic pipe encased in reinforced cast-in-place concrete serves as only an interior form and water tight barrier. Plastic pipe surrounded by concrete does not become a rigid pipe. For guidance on the design of reinforced cast-in-place concrete conduits or precast concrete conduits, see chapter 4 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

3.3 Encased Plastic Pipe

Encased plastic pipe design applies to plastic pipe encased in concrete, flowable fill, grout in the annular space of a slipliner, and plastic pipe on a concrete cradle. The encasement provides uniform circumferential support to the pipe. Plastic pipe in this configuration should be designed as an encased pipe, rather than a flexible pipe, and cross-sectional deflection should be considered negligible. If the groundwater table is above the encasement, the potential exists to develop hydrostatic pressure between the encasement and the pipe through cracks, joints, imperfections in the encasement. Complete grouting of the annulus around a slipliner pipe is difficult and inspection is impractical. With these concerns, the structural design of encased plastic pipe should consider the wall crushing due to the soil load, internal hydrostatic pressure and wall buckling caused by external hydrostatic pressure. Example A-2 in appendix A demonstrates the principles used in encased plastic pipe design.

3.3.1 Wall crushing

Encased plastic pipe is analyzed for wall crushing due to the soil load using the equations for wall crushing described in section 3.1.1. Any support from the encasement or an existing pipe is ignored. If an encased conduit extends through an embankment dam, soil loads should be calculated for an embankment conduit in the positive projecting condition as discussed in section 2.1.2.

3.3.2 Wall buckling

The potential exists to develop an opening within the grouted annulus of a slipliner or between the concrete encasement and the plastic pipe. Therefore, plastic pipe should be designed to withstand external hydrostatic pressure on the pipe due to loadings from the reservoir or internal vacuum pressure. The pipe should be conservatively designed to withstand unconstrained buckling pressure by:

$$P_{CR} = \frac{3EI_{pw}}{\left(1-v^2\right)r^3} \text{ for all pipe} \tag{3-18}$$

$$P_{CR} = \frac{0.447PS}{\left(1-v^2\right)} \text{ for short-term loading}^1 \text{ of corrugated plastic pipe} \quad (3\text{-}19)$$

$$P_{CR} = \frac{2E}{\left(1-v^2\right)}\left(\frac{1}{SDR-1}\right)^3 \text{ for solid-wall pipe} \quad (3\text{-}20)$$

where:

P_{CR} = unconstrained collapse pressure, lb/in^2

E = modulus of elasticity of the pipe material,* lb/in^2 (see section 3.1)

I_{pw} = pipe wall moment of inertia, in^4/in of pipe length

v =Poisson's ratio (0.38 for PVC, 0.35 for short-term loading of HDPE, and 0.45 for long-term loading of HDPE)

r = mean pipe radius, in

PS = pipe stiffness, lb/in^2 (as determined in accordance with ASTM F 894 and D 2412)

$SDR = D_O/t$

where D_O = outside diameter of the pipe, in

t = wall thickness of the pipe, in

* A long-term modulus of elasticity and Poisson's ratio are recommended if the pipe is subject to the pressure in the normal operations. If the pipe is subject to the pressure for short time periods and infrequently, the short-term modulus of elasticity is recommended. The hydrostatic pressure from the maximum reservoir pool would be considered long term.

Research conducted by Ian Moore (El-Sawy and Moore, 1997) has shown that for plastic pipes fully encased in concrete, the unconstrained collapse pressure can be increased by an enhancement factor of 4 to 5 depending upon the pipe SDR and ovality. This assumes that the grouting process completely encases the pipe. However, for plastic pipe used in sliplining applications, a more conservative design is required for withstanding unconstrained buckling pressure, since complete grouting of the annulus (see section 3.5.4) can not be reasonably assured.

Pipes that are significantly out-of-round or deflected have less collapse (buckling) resistance than round pipes. Pipes that are out-of-round due to manufacturing or deflected due to external pressure from soil and wheel loads or internal vacuum pressure have a lower allowable buckling pressure due to an increase in the bending moment. The allowable buckling pressure for these out-of-round or deflected pipes should be reduced by the following factor:

---

[1] Equation 3-19 is to be used only for wall buckling due to short-term loads since the pipe stiffness (PS) is representative of short-term material properties.

$$C = \left[\frac{\left(1 - \frac{\%\Delta Y}{D}\frac{1}{100}\right)}{\left(1 + \frac{\%\Delta Y}{D}\frac{1}{100}\right)^2} \right]^3 \qquad (3\text{-}21)$$

$$q_{ar} = q_a C \qquad (3\text{-}22)$$

where:

C = reduction factor for buckling pressure
$\%\Delta Y/D$ = percent deflection
q_{ar} = reduced allowable buckling pressure, lb/ft^2 or lb/in^2
q_a = allowable buckling pressure, lb/ft^2 or lb/in^2

Figure 47 illustrates how pipe stiffness for single wall corrugated HDPE pipe relates to its ability to withstand unconstrained collapse pressure. A minimum factor of safety of 2 applied to the unconstrained collapse pressure is often recommended for external hydrostatic pressure or internal vacuum pressure (Chevron Phillips, 2002, p. 102). Figure 48 illustrates how standard dimension ratio relates to solid wall pipe. A more detailed wall buckling analysis may be completed as described by Watkins (2004).

An example of a encased plastic pipe design is provided in appendix A, example A-2. The Virginia Dam case history in appendix B illustrates how concrete encased plastic pipe can be damaged due to improper design and construction.

3.3.3 Internal hydrostatic or vacuum pressure

If the encased pipe is subject to internal pressure, the pipe should have a pressure rating as described in section 3.1.4. The pipe should also be designed to withstand unconstrained buckling pressure as described in section 3.3.2. Design for internal vacuum pressure is discussed in section 3.3.2.

3.4 Summary of Design Considerations for Flexible and Encased Plastic Pipe Design

A number of considerations must be taken into account when using a plastic pipe in an embankment dam, as discussed in previous sections of this manual. Table 7 summarizes these different design considerations and provides a reference to sections in the manual where additional information can be found.

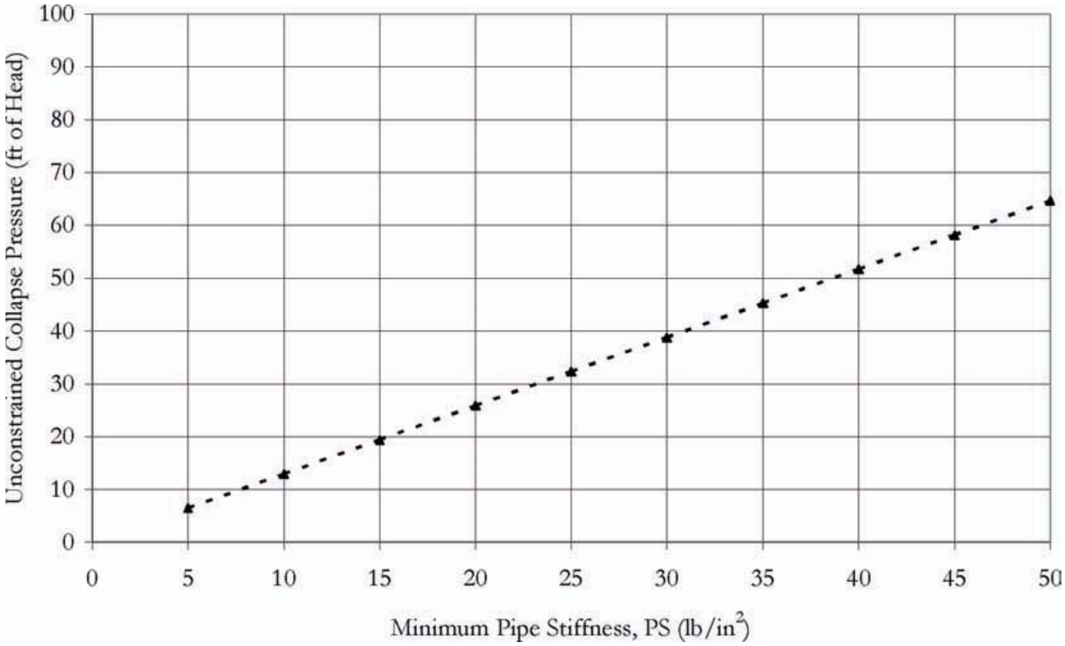


Figure 47.—Unconstrained collapse pressure vs. minimum pipe stiffness for single wall corrugated HDPE pipe.

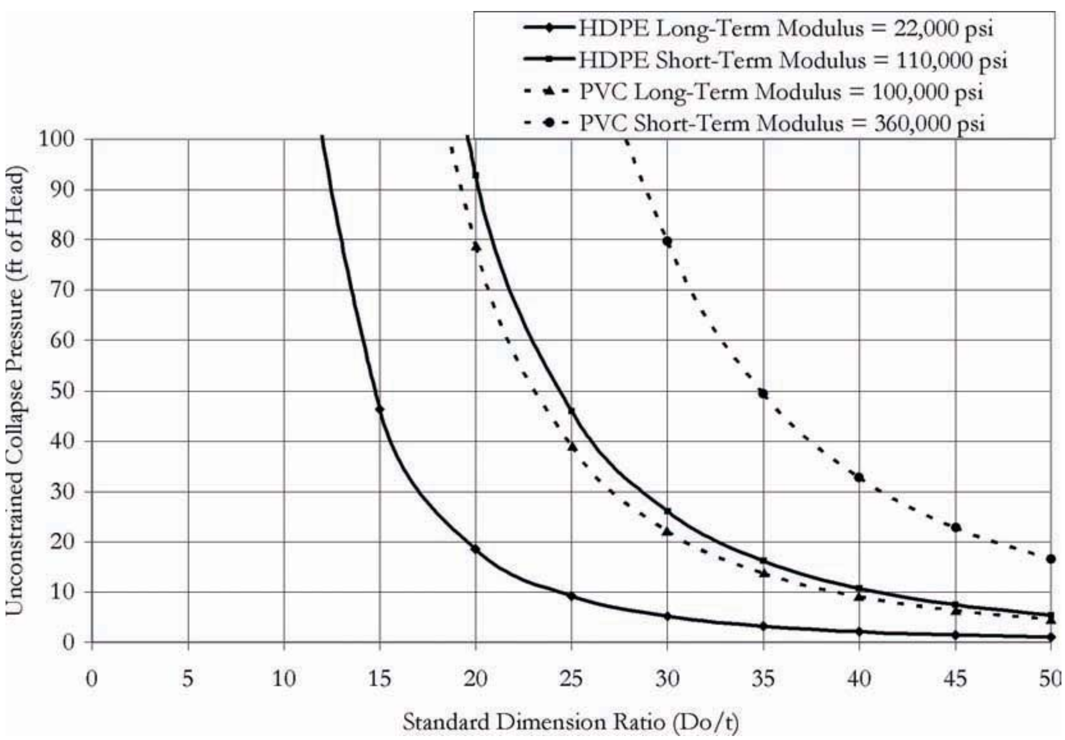


Figure 48.—Unconstrained collapse pressure vs. standard dimension ratio (SDR) for HDPE and PVC solid wall pipe. Plot is based on a minimum factor of safety of 1.0.

Table 7.—Summary of design considerations and required analyses for plastic pipe in embankment dams

Design consideration	Required analysis
Pipe diameter	Hydraulic design (section 3.9 for embankment conduits and section 4.1.2 for drainpipes) Access requirements (section 6.1 for embankment conduits and 6.2 for drainpipes)
Pipe material	Stress crack resistance (section 1.4)
Soil loading	Soil prism (section 2.1.1, and examples A-1 and A-2) Marston load (section 2.1.2, and example A-2)
Hydraulic loading	Internal hydrostatic pressure (section 2.2.1) Surge pressure (section 2.2.1) Internal vacuum pressure (section 2.2.2, and example A-3) External hydrostatic pressure (section 2.2.3, and example A-2)
Other loading	Construction (section 2.3)
Structural design	Flexible pipe (section 3.1, and examples A-1 and A-3) • Wall crushing (section 3.1.1) • Wall buckling—constrained (section 3.1.2) • Wall buckling—some siphons—unconstrained (section 3.3.2) • Deflection (section 3.1.3) • Internal pressure (section 3.1.4) • Strain (section 3.1.5) Encased pipe (section 3.3, example A-2) • Wall crushing (sections 3.3.1 and 3.1.1) • Wall buckling—unconstrained (section 3.3.2) • Internal pressure (sections 3.3.3, 3.1.4, and 3.3.2)

3.5 Embedment and Encasement Material Considerations

The embedment or encasement is the material immediately surrounding the pipe. The nature and placement of this material are critical to the structural performance of the plastic pipe installation. For instance, a properly shaped reinforced cast-in-place concrete encasement is required in significant and high hazard potential dams to facilitate the compaction of earthfill against the conduit to minimize differential settlement and the potential development of internal erosion. As discussed in sections 3.1 and 3.3, the type of embedment or encasement material dictates whether the plastic pipe is designed according to flexible or encased plastic pipe design procedures. In a flexible plastic pipe design, the pipe deflects into the embedment material. As the pipe deflects, load is transferred to the material surrounding the pipe, which results in a shifting of load away from the pipe. The embedment material should provide adequate strength, stiffness, uniformity of contact, and stability to minimize deformation of the pipe due to earth pressures. An encased plastic pipe design is necessary if stress redistribution is limited by concrete or grout, which limits deflection. An encased plastic pipe design is also necessary if the encasement material has an $E' = 0$ (fine grained soils with a high liquid limit), offering little or no stiffness compared to the pipe. The four most common embedment and encasement materials are soil, concrete, controlled low strength materials (flowable fill), and grout (contained within the annular space of a sliplined pipe).

3.5.1 Soil

Soil has been widely used as bedding and backfill for flexible pipe. However, soil can be problematic in obtaining adequate compaction under pipe haunches. The quality of the backfill and its placement, particularly in the haunch area (figure 49) and at the sides of the pipe, are the most important factors in limiting pipe deflection. Although flexible pipe is designed to deflect in cross section, excessive deflection can lead to unsatisfactory performance or structural failure. For this reason, the use of soil as bedding material for plastic pipe embankment conduits is only acceptable in low hazard potential embankment dams. Due to concerns with the potential for the development of internal erosion along embankment conduits in significant and high hazard potential dams, a reinforced cast-in-place concrete encasement should be used.

Soil stiffness is defined as the soil's ability to resist deflection. As discussed in section 3.1.3, soil stiffness is represented as E', the modulus of soil reaction. Loose soils have a relatively low E', while dense, well compacted soils have a high E'. Tables 4 and 5 in section 3.1.3 show average values of modulus of soil reaction.

Highly plastic soils, provide minimal resistance to pipe deflection. If the embedment material is primarily comprised of highly plastic soils, or soft organic material, the modulus of soil reaction should be assumed to be zero.

Research has shown that plastic pipe is generally resistant to structural failure. However, three conditions are known to cause pipe collapse: high internal vacuum pressure, excessive external hydrostatic pressure, and burial in loose or poorly compacted fine grained soils. Thus, the importance of having adequate compaction cannot be overemphasized. Section 5.2.4 in this document and section 5.3 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) discuss construction techniques that will ensure adequate compaction and pipe support.

If the pipe is not externally supported by embedment or if embedment provides little or no support, unconstrained pipe wall buckling may be a concern. External

Figure 49.—Compacting earthfill under the haunches of plastic pipe is very difficult and quality compaction can not be achieved.

pressures such as hydrostatic load from groundwater will also need to be analyzed using methods described in sections 2.2.3 and 3.3.3.

As with any type of pipe, the maximum size aggregate (MSA) of the backfill for plastic pipe should be considered. For drainpipe applications, the MSA is controlled by the processed drain material described in other sections of this document. For general backfill of embankment conduits, the MSA is a function of pipe diameter. For pipes less than 1 foot in diameter, the MSA should not exceed ¾ inch. For pipe greater than 1 foot in diameter, the MSA should not exceed 1.5 inches.

Backfill material should not be angular or subangular since sharp protrusions on the particles could puncture or gouge the pipe. Rounded and subrounded particle shapes are acceptable.

For dams, the type of backfill and bedding material for embedment used depends on where the pipe is located within the cross section, and whether the pipe is acting as an embankment conduit or a drainpipe. Section 4.3 and FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) provide guidance on selecting appropriate backfill.

3.5.2 Concrete

Concrete is often used as an encasement material for plastic pipe embankment conduits in significant and high hazard potential dams. There are three configurations: (1) reinforced concrete cradle up to springline (horizontal diameter) of the pipe, (2) reinforced cast-in-place concrete encasement, and (3) nonreinforced concrete encasement. In all cases, the use of concrete as encasement material limits deflection (longitudinal and cross-sectional) of the plastic pipe and changes the design approach. As discussed in section 3.3.2, if outward deflection is restrained, the plastic pipe has to buckle inward. The inward buckling takes more energy; thus the critical buckling pressure is higher when a pipe is encased in concrete.

3.5.2.1 Reinforced concrete cradle (encasement to springline)

There are arguments both for and against the use of concrete cradles. Continuous concrete cradles have been used to eliminate concerns of inadequate compaction beneath pipe haunches. The cradle also can provide anchor points for placement of pipe restraints, necessary to prevent pipe movement. Since a rigid cradle restrains cross-sectional deflection of the pipe, one concern has been the potential to create stress concentrations at the points where the pipe contacts the top of the cradle. Since the pipe does not deflect, it must be designed according to encased plastic pipe design principles (see to section 3.3). Additional research is necessary to better define the effect of stress concentrations on plastic pipe in concrete cradles (see research need EM-7 in chapter 8). Figure 50 shows an example of a concrete cradle used to support an HDPE pipe.

If contraction and expansion of in HDPE pipe after placement is an issue, flanges can be fusion welded onto the plastic pipe and encased by the cradle. When plastic pipe is used with bell and spigot joints, continuous concrete cradles are sometimes used to prevent excessive joint displacement and associated leakage potential.

Another concern when using a concrete cradle is that bond between the plastic pipe and the concrete cradle cannot be achieved due to material differences between the pipe and concrete and shrinkage of the concrete during curing. Thus, it is important that a diaphragm filter and drainage system be incorporated to prevent migration of fines along the conduit. For a discussion of filters, see chapter 6 of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

Until the additional research is completed as discussed in research need EM-7, concrete cradles beneath plastic pipe should not be used.

3.5.2.2 Reinforced cast-in-place concrete encasement (completely encased)

Many States and federal agencies require that on significant and high hazard potential dams, plastic pipe conduits be completely encased in reinforced cast-in-place concrete. The plastic pipe acts primarily as an interior form, as well as a watertight liner. When properly designed, the concrete encasement provides a good exterior shape to compact earthfill against the conduit. The reinforced encasement should be designed according to reinforced concrete design principals. Design of reinforced cast-in-place concrete is beyond the scope of this document. For guidance on

Figure 50.— Until further research is completed, concrete cradles beneath plastic pipe should not be used.

reinforced cast-in-place concrete design, see chapter 4 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

Although it is recommended that the strength of the plastic pipe be ignored when determining the structural strength of the reinforced concrete, the plastic pipe must still be designed to prevent collapse from both excessive hydraulic pressures and concrete pressures. Full reservoir head can be transferred through cracks to small annulus spaces in between the plastic pipe and reinforced concrete encasement.

With a reinforced cast-in-place concrete encasement, it is tempting to place the pipe and concrete in a vertical wall trench (causing the pipe to be negatively projecting). As discussed in section 2.1, negative projecting conduits should not be used due to the potential for soil arching above the conduit creating potential seepage paths through the embankment. To avoid installing a negative projecting conduit, the side slopes of the trench excavation must be sloped 2H:1V or flatter. The conduit then behaves as a positive projecting conduit. In addition, sloping trench sides facilitate compaction and bonding of the backfill with the sides of the excavation.

Special precautions are necessary to prevent floating the pipe during concrete placement. These precautions may include: (1) strapping the pipe to anchors and (2) welding or bolting end caps on to the pipe and filling it with water. Since each installation is unique, it is recommended that the pipe manufacturer or supplier should be contacted for recommendations. Often, they have installation manuals with specific instructions on how to prevent pipe movement during construction.

The reinforced cast-in-place concrete encasement should be completed within one monolithic placement around the pipe to prevent the need for construction joints. An alternative to a monolithic placement is the use of properly treated horizontal construction joints. Regardless of how the encasement is constructed, a diaphragm filter system is recommended. For guidance on filters, see chapter 6 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

3.5.2.3 Unreinforced cast-in-place concrete encasement (completely encased)

Complete encasement of conduits for plastic pipe in unreinforced cast-in-place concrete is sometimes used with low hazard potential embankment dams, but would not be acceptable in significant or high hazard potential dams. Concrete is strong in compression, but weak in tension. The reinforcement in concrete carries the tension. Also, reinforcement prevents encasement joints from opening. As with a continuous concrete cradle, complete concrete encasement eliminates compaction concerns under pipe haunches. However, when the concrete encasement is not reinforced, it is not structurally adequate to withstand large fill heights, without relying on the strength of the interior plastic pipe. Since both cross-sectional and longitudinal deflection of the plastic pipe is restrained, encased plastic pipe design principles should be applied. Additional research is proposed in section 8.1.2 (EM-9) to further investigate the use of unreinforced concrete encasement.

As discussed in section 3.5.2.2, the pipe should be designed for external hydraulic pressures, equal to the full reservoir head. Experience has shown that concrete encasements do not bond with the outer surface of the plastic pipe, allowing a potential seepage path for reservoir water between the pipe and concrete. When the pipe is unwatered, full reservoir head is transferred to the pipe exterior, and collapse can occur.

3.5.3 Controlled low strength material (flowable fill)

Controlled low strength material (CLSM), also known as flowable fill, is a self-compacted, cementitious material used primarily as an encasement in place of a compacted backfill. Use of low strength materials began in the early 1970's for trench backfill, pavement base, and other construction applications. More recently, CLSM has been used as an encasement material for flexible pipe embankment conduits in low hazard potential applications (figure 51). CLSM should not be used in significant and high hazard potential dams until further research is performed to evaluate the potential concerns discussed in section 3.5.3.2.

CLSM is defined as a cementitious material that is in a flowable state at the time of placement and has a specified compressive strength of 1,200 lb/in^2 or less at the age of 28 days. Most applications require unconfined compressive strength of 300 lb/in^2 or less, which is equivalent to a compacted fill. CLSM is made of Portland cement,

Figure 51.—The CLSM is typically transported to the construction site in ready mix concrete trucks. In this figure, CLSM is being used as a pipe encasement. CLSM should not be used for embankment conduits in significant and high hazard potential dams.

fly ash, water, fine aggregate and sometimes entrained air. The Portland cement type should be appropriate for most project requirements and site conditions. Typically, CLSM uses Type I or Type II Portland cement. The cement provides cohesion and strength. The fly ash improves flowability while reducing shrinkage and permeability. Water is necessary for flowability and hydration. Aggregate is the major constituent of CLSM. Usually ASTM C 33 fine concrete aggregate is used. If the fine aggregate does not meet ASTM C 33 (i.e., reactive, slaking), care must be taken to assess how the aggregate affects the CLSM performance. For example, aggregate with a higher fines content or a finer gradation will adversely affect the flowability.

The most critical parameter for use of CLSM as an embankment conduit encasement material is the strength. The strength must be kept low so that the CLSM can accommodate deformation of the pipe without creating large cracks and maintains a seal around the pipe. If the CLSM is too strong, the CLSM will be more brittle and may not accommodate deformation of the pipe. This could result in large cracks that may create seepage paths. The water content also affects the strength of the CLSM and the amount of bleed water that forms. Excessive bleed water may accumulate under the pipe, resulting in a void. Excessive water can cause shrinkage and cracking. Quality control is important for any successful application of CLSM. Proper attention must be given to uniformity of materials used in the mix design, equipment, and transportation of the mix to the project site. Specifications using ASTM standards C 33, C 150, and C 618 for concrete aggregate, cement, and fly ash respectively will help to ensure uniformity and control flowability and strength in CSLM mixtures (Brewer and Hurd, 1993, p. 29).

Air entrainment can be used to improve flowability, as well as to limit the maximum strength. Air entrainment is also used to reduce bleed and segregation. The additional cost of this or other admixtures must be weighed against the benefits.

Mixture proportioning of the CLSM is critical to achieving the required performance. The mixture must be appropriate for the project requirements and site conditions. The Department of Transportation or Department of Roads in some States has CLSM mix designs for backfilling culverts which might be applicable for use. Trial batching of the CLSM should be performed in a laboratory and at the batch plant. Due to differences in cement, fly ash, and aggregate sources, it is important to trial batch using the materials that will be used at the project. Field testing should verify that the mix and placement methods are within specification requirements for dams. Some considerations for material selection are sulfate resistance, improved flowability, thermal reduction, and bleed and segregation control. For instance, CLSM made with natural sand will be more flowable than the same proportioned CLSM made with manufactured sand. Trial batching needs to consider the time effects on the CLSM during placement and then with regard to strength gain. The trial batch should be evaluated with respect to how long the CLSM maintains flowability over time. Flowability should be retained long enough to complete the entire placement or lift. Another important consideration is that CLSM continues to gain strength with time. Compressive strength should be evaluated with trial batching over time. Strength tests should be made at least at 7, 28, and 90 days, but 3, 14, and 56 days would also be beneficial to evaluating performance. Although the CLSM is specified by a 28-day strength, additional strength gain beyond 28 days may result in the CLSM becoming stronger and more brittle than what is appropriate for the application. Testing durations longer than 90 days should also be considered when the project schedule allows for advance testing of the CLSM. Other considerations include evaluation of the bleed and shrinkage characteristics during trial batching.

CLSM should not be confused with lean mix concrete. Lean mix concrete is a term used for reducing the cement in a Portland cement concrete mixture and is usually designed using Portland cement concrete principles. CLSM is not designed by Portland cement concrete principles (Brewer, 1990, p. 109). CLSM is not designed to resist freezing or abrasion. This should not be a problem in dam construction, since most applications are buried. The designer should be aware that the quantity of CLSM used in low hazard potential applications is small and there may not be any significant cost savings compared to lean concrete. CLSM should also not be confused with soil cement, which is a much drier mix and requires compaction. Although internal concrete vibrators may be used to facilitate the flow of CLSM to ensure that no air is trapped under the pipe, compaction is not required. For additional basic information on CLSM (see ACI's *Controlled Low-Strength Materials*, 1999).

3.5.3.1 Design considerations for using CLSM

A concern with the use of CLSM as an encasement material is the hydrostatic pressure it exerts (ACI, 1999). CLSM is not self-supporting and places a load on the pipe. For large, flexible wall pipes, CLSM should be placed in lifts, so that lateral support can develop along the sides of the pipe before fresh CLSM is placed over the pipe. Since a plastic pipe will deflect into cured CLSM, loads are determined using the prism theory (section 2.1.1). Although it is tempting to place CLSM in a vertical walled trench, care should also be taken to adequately slope the sides of the pipe trench to make sure the conduit behaves as a positively projecting conduit.

As with other encasement materials, plastic pipe encased in CLSM should be checked for external water pressure and vacuum pressure as well as internal loading conditions.

As discussed in section 3.1.3, deflection is a function of the wall thickness and the soil structure interaction, and is calculated using the Modified Iowa Equation. Although CLSM and soil behave similarly, soil variables used in the Modified Iowa Equation are different for CLSM. Some research has shown that CLSM results in less horizontal deflection than for conventional backfill (Brewer and Hurd, 1993 p. 28). As shown in table 4 in section 3.1.3, E' for soils range from 50 for fine grained soils to 3,000 lb/in^2 for crushed rock. Laboratory tests have been performed to determine correlations between CLSM compressive strength, Young's modulus, and E' (Brewer, 1990, p. 118). For 100 lb/in^2 compressive strength CLSM, E' ranged from 1,000 to 1,800 lb/in^2, depending on the Poisson's ratio of the CLSM (which is a function of the aggregate filler). Also, E' for CLSM does not increase with depth of cover since CLSM does not consolidate. E' for CLSM also does not depend on compaction. Additional research is necessary to better define the modulus of soil reaction for CLSM (see chapter 8, research need, EM-3).

3.5.3.2 Problems with using CLSM

CLSM shows promise as an encasement material since it provides adequate support under pipe haunches, is easy to place, and does not require compaction. However, there are several uncertainties with the use of CLSM. Additional research is needed to evaluate the various performance considerations of CLSM before it can be recommended for use in significant and high hazard potential embankment dams. A number of research needs related to CLSM are proposed in section 8.1.2 (EM-3 through EM-8). Due to the number of uncertainties currently existing with the use of CLSM and until additional research is completed, it is recommended that CLSM only be used in low hazard potential embankment dam applications.

The main concern with CLSM is that shrinkage and cracking would create seepage paths. CLSM cracking can create localized stresses in the pipe wall. Shrinkage and cracking tendencies depend on the mixture proportioning, and need to be evaluated during the trial batching of the CLSM. There is also concern that the heat of hydration that is created when the cement and fly ash react could cause pipe deformation and affect the contact between the pipe and CLSM. In addition, bleed and segregation tendencies need to be evaluated since this may affect the contact of the CLSM with the pipe. In-place performance should be investigated with regard to the effects of bleed, shrinkage, and cracking. Also, the placement of CLSM in lifts should be evaluated to ensure that the cold joint between lifts does not create a seepage path.

CLSM is assumed to behave similarly to soil, allowing cross-sectional pipe deflection. The compressive strength of the CLSM influences the amount of pipe deflection. Laboratory testing is needed to determine recommended compressive strength for the use of CLSM as encasement in dam applications. Additional research is needed to better quantify the modulus of soil reaction for CLSM. Full-scale laboratory tests would also be useful in evaluating the response of plastic pipe encased in CLSM exposed to large vertical loads.

When using CLSM with plastic pipes, the absence of soil overburden will cause the pipe to float because the weight of the pipe does not offset the uplift forces of the CLSM. The designer will need to properly anchor the pipe to prevent floatation. The use of multiple lifts can reduce the uplift forces acting on the pipe by the CLSM.

3.5.4 Grout

From a structural design perspective, grout can be considered an encasement material. For example, when sliplining a deteriorating conduit, the presence of grout in the annulus between the slipliner and the existing conduit prevents the plastic pipe from deflecting. As the existing conduit continues to deteriorate, external soil loads can be transferred to the interior pipe/grout system. With deflection limited, the interior plastic pipe may be designed using encased plastic pipe design principles (section 3.3).

A common conservative approach is to not consider the existing conduit in calculations for the design of the plastic pipe slipliner. Also, when checking the buckling resistance of the slipliner, conditions during grouting and after grouting must be considered. During the grouting operation, the slipliner is not confined and its unconfined buckling resistance versus the grouting pressure must be checked. Once the pipe is grouted in place, the long term buckling resistance should be checked assuming the slipliner is exposed to external hydrostatic pressure, as discussed in section 2.2.3. External hydrostatic pressures can be transmitted to the plastic pipe through voids in the exterior pipe and shrinkage cracks in the grout. The plastic pipe must be designed to withstand full reservoir head and maximum embankment soil load. For additional guidance on grouting of the annulus, see section 12.1 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

The most common grout used for filling the annular space is cement grout. Cement grout is a mixture of cement, fly ash, and water. A superplasticizer is often utilized to facilitate pumping of the grout. The grout used as an encasement material typically has 28-day compressive strength of approximately 4,000 lb/in^2. For large diameter conduits, grouting is often done in stages to prevent floating the slipliner. Cement grouts can be ordered from the local ready mix company, making it readily available and inexpensive. For neat concrete grout (cement and water) and prepackaged dry grout mixed with water, a standard grout mixer/pump allows onsite mixing at the point of placement. Since specialized equipment is not required, the overall cost of using cement grout is low. One concern with cement grout is that the pressure used for pumping must be closely monitored to avoid danger associated with collapsing the plastic pipe. Also, verification of the complete filling of the annulus space is not achievable. Close attention to actual grout quality used must be made during construction.

Cellular grout should not be used in embankment conduit applications in significant and high hazard potential dams due to its porous nature and lack of strength as an encasement material. Cellular grout is a mixture of cement, water, and foam. And generally has a 28-day compressive strength of approximately 300 lb/in^2. Specialty, licensed contractors generally install cellular grout. The contractor has specialized equipment for generating the foam. The foam is usually introduced into the grout as it is being pumped, and this develops the most stable mixture.

3.5.5 Comparison of embedment and encasement materials

Many factors should be evaluated when choosing an embedment or encasement material. Table 8 compares the different types of embedment and encasement materials, and some of the issues that should be evaluated when making a selection. A thorough understanding of table 8 is crucial in the selection of the proper embedment and encasement material.

3.5.6 When to use flexible or encased plastic pipe design

The preceding sections have demonstrated that the behavior of plastic pipe depends on its surrounding medium, whether it is soil, concrete, CLSM, or grout. Table 9 summarizes the various situations where flexible and encased plastic pipe designs apply. When soil is the embedment material, the pipe can deflect, allowing the interior prism to settle more than the exterior prisms. Flexible plastic pipe design theory should be used (section 3.1). A flexible conduit surrounded by soil installed

Table 8.—Comparison of embedment and encasement materials

	Soil	Concrete	CLSM	Grout
Mechanical compaction required	Yes, placed in compacted lifts	Vibration required	No, consolidates under its own weight	No
Weather issues	Yes, cannot be placed in rain; special protection required in cold weather	Special protection required after placement in cold weather; surface integrity deteriorates in rain	Must be protected from freezing until it cures	Must be protected from freezing until it cures
Placement time	Slow, placed in lifts	In most cases, can be done with a single placement; use of reinforcement requires additional time; slow curing	Generally rapid placement; cures rapidly	Generally rapid
Permeability	Easily controlled—Depends on soil type	Low when not cracked	Similar to compacted granular fills; must use higher fines content to decrease permeability; shrinkage and cracking may be a concern	Low, with proper mix proportioning and injection methods
Pipe support	Difficult to compact adequately under haunches	Excellent	Excellent	Excellent
Homogeneous	No	Yes; with proper mix proportioning	Yes; with proper mix proportioning	Yes; with proper mix proportioning
Strength of pipe	Derives most strength by deflecting into soil	Reinforced: strength a function of concrete and reinforcement; Unreinforced: encased plastic pipe design may require lower SDR (stiffer pipe) than a pipe enclosed in well compacted soil	Derives strength from deflecting into flowable fill; mix design critical for proper performance; deflection less than with soil backfill	Encased pipe design necessary (deflection is limited)
Construction concerns	Pipe can move during compaction, difficult to restrain	Pipe restraints necessary to prevent pipe from floating; pipe must be designed to withstand the load concrete places on pipe during placement	Must prevent pipe from floating during placement; pipe must be designed to withstand load CLSM places on pipe during placement	Spacers required to keep pipe from floating and to promote even distribution of grout in annular space; bridging can be a concern
Primary concerns	Compaction under haunches is difficult to achieve	Construction joints can be source of seepage and stress concentrations	Potential for high permeability; potential for cracking and seepage paths. *E'* value is not well known.	Full encapsulation with grout is rarely achievable and cannot be confirmed in field

Table 9.—Flexible pipe design versus encased pipe design for plastic pipe, as a function of encasement material

Embedment/encasement material	Positive projecting conduits (Embankment conduits and drainpipes under embankment fill)	Trench conduits (Drainpipes not under embankment fill)	Flexible plastic pipe design? (sec 3.1)	Encased plastic pipe design? (sec 3.3)
Soil embedment	Prism (trench condition)	Prism	Yes	No
Reinforced concrete cradle to springline	Marston (projection condition)	Not applicable	No	Yes
Reinforced cast-in-place concrete encasement	Reinforced concrete design principals apply			
Unreinforced cast-in-place concrete encasement	Marston (projection condition)	Not applicable	No	Yes
CLSM placed in lifts	Prism (trench condition)	Not applicable	Yes	No
Grout	Marston (projection condition)	Not applicable	No	Yes

as a positive projecting conduit (i.e., located under embankment fill) is considered a *positive projecting conduit in the trench condition*, since the deflection of the conduit causes the interior prism to settle more than the exterior prisms (refer to figure 29). The soil load on a conduit in the trench condition is typically less than the weight of the fill above the conduit (soil prism load). Thus, soil prism load theory is conservative and should be used (section 2.1.1). A flexible conduit surrounded by soil that is installed as a *trench conduit* (i.e., located beneath natural ground), where the interior prism settles more than the exterior prisms (refer to figure 27), behaves similarly. The soil prism load should be also used.

When the plastic pipe is encased in a rigid material such as concrete or grout, deflection is limited. Encased plastic pipe theory should be used (section 3.3). An encased conduit installed as a projecting conduit (located under embankment fill) is considered a *projecting conduit in the projection condition*, since deflection is limited and the exterior prisms settle more than the interior prism (refer to figure 28). The soil load on the conduit is greater than the weight of the fill above the conduit. Thus, the soil prism theory underestimates the load. Marston load theory should be used in this situation (section 2.1.2).

If the plastic pipe is encased in CLSM placed in lifts, the CLSM behaves similarly to soil. Deflection occurs, and the interior prism settles more than the exterior prisms. Flexible pipe design theory and soil prism load theory should be used.

If the plastic pipe is encased in a single placement of CLSM, one of the concerns is that lateral support does not develop and a load is placed on the pipe until the CLSM cures. If there is no lateral support, flexible pipe design theory cannot be used; loads must be determined using encased plastic pipe design theory. Eventually, after the CLSM sets, flexible theory can be used, but during curing, which is the worst case situation, the conservative encased plastic pipe theory should be used.

3.6 Expansion and Contraction

All pipes expand and contract with changes in temperature. The designer needs to consider these changes and the effects on the selected length for installed pipe. Table 10 presents approximate coefficients of thermal expansion. In buried applications, the pipe will not typically experience significant changes in temperature, and thermal stress or dimension change will be minimal. However, changes in the ambient temperature prior to backfilling around the pipe may lead to excessive expansion or contraction. Plastic pipe may also experience a change in temperature once it is buried if the ambient temperature or temperature of pipe exposed to the sunlight is different than the buried condition. Contraction of the pipe can also occur if the water released is colder than the pipe's installation temperature during construction, or if the pipe is drained in the winter and open at the downstream end. If the conduit is empty during the winter and an air vent is provided at the upstream end, a cold draft can develop causing freezing conditions.

Table 10.—Coefficient of thermal expansion

Pipe material	Coefficient, in/in/°F
PVC	3.0×10^{-5}
HDPE	1.2×10^{-4}

Source: AWWA, 2002

Any change in pipe length due to thermal expansion or contraction depends on the pipe material's coefficient of thermal expansion and variation in the temperature. A pipe restrained or anchored at both ends will experience a change in stress with changing temperature due to expansion and contraction. The stress due to temperature change should be less than the allowable stress represented by the hydrostatic design stress for the plastic material. The longitudinal stress in the pipe wall due to temperature changes may be estimated by:

$$S_{EC} = E\alpha\Delta T \qquad (3\text{-}23)$$

where:

S_{EC} = stress due to temperature change, lb/in^2
E = short-term modulus of elasticity, lb/in^2
α = coefficient of thermal expansion, in/in/$^\circ$F
ΔT = change in temperature, $^\circ$F

The modulus of elasticity of plastic pipe is a function of the temperature. Heat transfer occurs at relatively slow rates through the wall of the pipe and temperature change does not occur rapidly. The average temperature is often recommended for use in determining the appropriate modulus of elasticity. The modulus of elasticity should be adjusted for temperature by the factors shown in table 7 in section 3.1.4.

3.7 End Restraint Design

Often, the friction between the soil and the plastic pipe or the slipliner and the grout surface provides enough restraint against the forces of expansion and contraction. However, if a structure such as an intake tower, principal spillway riser, impact basin, or manhole is intended to resist the expansion and contraction forces, the structural design and stability of the structure must consider these forces. If restraints are buried in the soil, the bearing capacity of the soil must resist the forces from expansion and contraction. The end thrust (force) due to expansion or contraction may be estimated by:

$$F = S_{EC} A_{pw} \tag{3-24}$$

The required area of an end restraint in soil may be determined by:

$$A_R = \frac{F}{q_{All}} \tag{3-25}$$

where:

F = force due to expansion or contraction of the pipe, lb
S_{EC} = stress due to temperature change, lb/in^2
A_{PW} = area of the pipe wall, in^2/in of the pipe length
A_R = required area of the end restraint, ft^2
q_{All} = allowable soil bearing capacity, lb/ft^2

3.8 Other Design and Construction Considerations

Other design and construction considerations are briefly discussed in the following sections. The pipe manufacture or supplier should be consulted for additional guidance. Often they have installation manuals with specific instructions on how to avoid problems with their particular brand of pipe.

3.8.1 Foundation problems

A nonuniform bedding can result from unstable or variable foundation materials, nonuniform compaction, collapsible soils, soft clays, and undermining or erosion from water flowing on the outside of the conduit. Fortunately, flexible pipe can deform away from many pressure concentrations. Axial bending is rarely a cause of failure in a flexible pipe. However, the designer should consider the possibility of joints opening due to soft foundations. For reinforced concrete encased plastic pipe, joint movement is generally not a concern because the concrete encasement and longitudinal reinforcement through the joint limits deformation.

For nonencased plastic pipe, foundation conditions must be carefully considered. Flexible pipe generally has enough flexibility to allow the pipe to conform to minor foundation movements without structural distress. However, bell and spigot joints can be susceptible to separation and leakage with foundation movement. Openings of pipe joints are often not uniform, and large concentrated openings can occur at isolated joints. See section 4.3.1 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) for guidance on joints. If soft, loose, expansive, or liquefiable soils are present where significant ground movement can be anticipated, butt fusion joints should be considered.

Differential settlement of a valve or other structures to which a pipe is rigidly connected can induce high bending moments and shearing forces. A support pad should be provided below the pipe and for at least two pipe diameters length under the connecting pipe. The support pad should be compacted soil or placed concrete. The designer must look at potential for settlement and determine whether the pipe and joints have adequate flexibility to withstand anticipated vertical movement. Pipe manufacturers can provide information on maximum allowable pipe joint deflection.

A mud slab can be used to protect the conduit foundation. A mud slab is a 2- to 6-inch layer of concrete typically placed over soft, wet soil or used to prevent degradation that can occur between the time the foundation is excavated and the concrete encasement is constructed. The mud slab is commonly placed within 24 hours of exposure of the foundation to protect the foundation from construction, erosion, and environmental causes.

3.8.2 Leak testing

Plastic pipe used for embankment conduits should be leak tested before being put into service. The purpose of a leak test is to find any defects before they result in leakage or rupture. Hydrostatic testing using water is the preferred method, although other methods are acceptable. Serious safety concerns exist if compressed air is used because failure of a portion of pipe or joint could be extremely hazardous to personnel. The pipe should be restrained against movement in the event of rupture before any pressure is applied. The manufacturers' recommendations should always be reviewed for guidance on leak testing of plastic pipe.

The most common method of leak testing for HDPE pipe is to butt fuse or mechanically join flanges on each end of the plastic pipe (these flanges can also be later used to attach a gate or valve to the pipe). End caps can then be bolted onto the flanges and the conduit filled with water to test for leakage (figure 52). A correctly made butt fused joint should not leak. Any joints showing leakage must be repaired. No repairs should be done while the pipe is under pressure. For sliplining applications where a plastic pipe has been inserted into an existing embankment conduit, repairs are not possible, and the entire slipliner will most likely need to be pulled back out and the repair made. For further guidance on leak testing of HDPE pipe, see ASTM F 2164.

Figure 52.—Leak test being performed on an HDPE slipliner for an outlet works renovation.

PVC pipe has been used for embankment conduit applications in low hazard potential dams. Hydrostatic testing is commonly performed to prove the integrity of the pipe. For further guidance on leak testing of PVC pipe, see AWWA's *PVC Pipe—Design and Installation* (2002).

All important data from the leak test should be recorded, including pressure, duration, ambient temperatures, leaks, and repairs. Drainpipes are assumed to operate in a nonpressurized condition and do not require leak testing.

3.8.3 Thrust blocks

Typically, plastic pipe used in embankment conduits or drainpipes does not require thrust blocks. When needed, a concrete thrust block is often used to transfer the tensile loading in the pipe into the surrounding soil. The tensile load in the pipe must first be transferred into the concrete. Since the concrete will not grip the pipe smooth surface, a branch saddle or flange must be fused to the pipe and embedded in the concrete. The concrete thrust block must be sized based on the bearing capacity of the soil. Additional information on thrust block design and construction is contained in ASTM F 1668 and NRCS's *Structural Design of Flexible Conduit* (2005).

3.8.4 Anchors and spacers

Plastic pipe can float during the installation process and must be anchored prior to placement of embankment or encasement material. This is particularly a concern with the use of CLSM. Because the difference in unit weights between CLSM and water is substantial, the uplift force of CLSM can be greater than two times the hydrostatic uplift. The problem with floating pipe is not new to the plastic pipe industry. Many manufacturers provide guidelines and recommendations for anchor spacing depending on the type of pipe and diameter. There are several options for anchoring plastic pipe. The pipe can be anchored with metal straps or rebar placed in an X pattern above the pipe and tied into wooden planks placed underneath the pipe at regular intervals. Another technique of filling the pipe with water will weight the pipe sufficiently to allow placement of concrete or CLSM to facilitate compaction of soil around the pipe and provide uniform support along the length and therefore minimizing deformation. However, in some applications, the additional weight of the water is not great enough to overcome the buoyancy of the pipe. For instance, grout has a density greater than water and the pipe would tend to float, even if it is filled with water. Designers should also be aware that in shallow burial applications, the cover over the top of the pipe must be sufficient to resist hydrostatic uplift pressure. Placing concrete and CLSM in lifts can minimize the effects of uplift.

The problem with floating pipe also applies to pipe used in a sliplining application. Pipe manufacturers often provide guidance on the proper location of spacers to maintain an even annular space around the pipe and facilitate a complete grout seal. A thorough discussion on the use of spacers in sliplining applications is provided in section 12.1.1 of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005)

3.8.5 Placement temperature

As discussed in section 3.6, plastic pipe expands and contracts radially and longitudinally with changes in temperature. Some of the problems associated with expansion and contraction can be overcome with special attention to placement temperature. Section 12.1.1 of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) provides guidance on techniques that can be used to avoid excessive expansion or contraction of the plastic pipe during construction. Although this discussion is primarily targeted toward sliplining applications, the information can apply to other plastic pipe installations as well.

Temperatures near or below freezing affect thermoplastic pipe by increasing stiffness, vulnerability to impact damage, and sensitivity to suddenly applied stress. Significant impact or shock loads against a PVC pipe that is at freezing or lower temperatures can fracture the pipe. Extra care must be used when placing a plastic pipe in freezing temperatures.

3.8.5.1 Soil as embedment material

In direct burial installations, embedment material friction normally restrains longitudinal pipe movement caused by seasonal temperature changes. If the pipe is not anchored at the ends to resist movement, a few inches at each end may expand or contract as the temperature changes. This zone will extend into the burial trench to a point at which the friction resistance of the embedment material is equal to the thermal force. When installing HDPE pipe that is warmer than the soil, a slightly longer length may be required to compensate for contraction of the pipe as it cools. The change in length as a function of temperature change can be estimated using methods described in section 3.6.

3.8.5.2 Concrete, CLSM, and grout as encasement material

The effects of heat of hydration on plastic pipe should be evaluated. A significant rise in temperature could allow the pipe to expand during curing of the grout, concrete, or CLSM. As the pipe cooled, it would shrink back to its original size, possibly leaving a gap between the encasement material and the plastic pipe. For sliplining applications involving embankment conduits, the annulus between the existing pipe and the new slipliner is typically very small (i.e., a few inches). The restricted size of the annulus results in a limited amount of grout surrounding the plastic pipe. Although no research has been done on the effects of heat of hydration caused by the grout as it cures within the annulus, it is suspected that an inadequate amount of heat is generated that would affect the thermal expansion properties of the slipliner. Additional research has been proposed in section 8.1.2 (EM-2) to evaluate the effects of heat of hydration. If the volume of grout is increased dramatically relative to the thickness of the plastic pipe, consider the use of additives such as flyash, or embedding small diameter pipes to circulate cold water in the grout mass to lower the heat of hydration. The volume of grout mass can be minimized by selecting the circulating cold water within the plastic pipe or using the largest possible plastic pipe diameter for sliplining. This reduces the grout mass in the annulus and the possible effects of the heat of hydration. Proper mix portions of the grout is usually the most effective method in controlling the heat of hydration.

3.8.6 Collapse of pipes due to grout pumping pressure

As discussed in section 2.2.3, external hydrostatic pressure can cause plastic pipe to collapse. The external hydrostatic pressure can be exerted by reservoir head or grout. The collapse pressure of plastic pipe should not be exceeded while grouting the annulus during sliplining. Most manufacturers provide information on the safe maximum differential pressures that can be applied to unsupported pipe without buckling or collapsing the pipe. The collapse pressure of the pipe may also be determined as described in section 3.3.2. Section 12.1.1.2 of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) provides several options for grouting the annular space that minimize the potential of pipe collapse. The Upper Wheeler Dam case history in appendix B illustrates the potential for pipe collapse during the grouting process.

3.8.7 Air venting

As with any conduit system, adequate air venting is critical. Extrusion welding is commonly used to join an air vent to HDPE conduits. For guidance on the location, airflow rates, and structural considerations of air vents, refer to *Air-Water Flow in Hydraulic Structures* (Bureau of Reclamation, 1980).

3.8.8 Seepage

Seepage along the contact between the embankment conduit and encasement material is a special design concern. Because there is no bond between the pipe and the embedment/encasement, a seepage path can develop. This problem can be worsened by radial expansion and contraction of the pipe with temperature change. Contraction can cause a void to develop between the pipe and embedment/ encasement and result in a seepage path along the outside of the pipe. Additional research has been proposed in section 8.1.2 (EM-1, EM-10 and EM-11) to further evaluate the concern.

This problem is of greater concern with plastic pipe because of the high coefficient of thermal expansion. The contraction could result from: (1) cooling of the concrete during curing, (2) venting of a drained outlet pipe in cold weather, and (3) change in temperature of the pipe after the pipe fills. The use of a diaphragm filter around the pipe mitigates some of this concern. Diaphragm filters are described in detail in chapter 6 of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

The Sediment Control Pond SP-4 Dam and the Sugar Mill Dam case histories in appendix B illustrate the concerns with seepage along the conduit.

3.9 Hydraulic Design of Embankment Conduits

The hydraulics design principles for HDPE and PVC pipe are well established. Many empirical formulas and equations are available to the designer to solve problems involved with flow through plastic pipe. Therefore, this section will not provide detailed hydraulic analysis, but will direct the reader to appropriate resources where this information can be found. The following recommended references provide guidance and sound engineering principles for the hydraulic design of outlet works, spillway conduits, power conduits, and siphons.

Recommended references include:

- AWWA's *PE Pipe-Design and Installation* (2006)
- AWWA's PVC *Pipe-Design and Installation* (2002)
- The Bureau of Reclamation's *Design of Small Dams* (1987a)
- FEMA's *Conduits through Embankment Dams* (2005)
- Natural Resources Conservation Service *Hydraulics* (1956)
- USACE's *Hydraulic Design of Reservoir Outlet Works* (1980)

For guidance on drainpipe hydraulics, see section 4.1.2. The reader should also consult the "additional reading" section of this document. Additional references are provided to further the reader's understanding of topics related to plastic pipe and hydraulics.

3.10 Renovation, Replacement, and Repair of Embankment Conduits

As a result of the advancing age of the nation's inventory of embankment dams, the deterioration of conduits through embankment dams is becoming a common deficiency that must be addressed. Designers must consider a wide range of factors before selecting the method best suited for a particular application.

For a detailed discussion of methods of renovation, replacement, and repair involving plastic pipe, see FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). Discussions within this reference include:

- *Renovation* (chapter 12).—Many embankment conduits are too small to enter for renovation. 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. Recently, sliplining small, inaccessible conduits using plastic pipe has become the renovation method of choice. Sliplining typically results in minimized excavation, shorter construction periods, and less construction cost. Sliplining usually requires access to both the upstream and downstream ends of the conduit, requiring draining of the reservoir or construction of a cofferdam.

- *Replacement* (chapter 13).—Removal and replacement of an existing conduit generally consists of draining the reservoir or constructing a cofferdam, 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. Plastic pipe used in this method for significant and high hazard potential embankment dams should be encased in properly shaped reinforced cast-in-place concrete to assure quality compaction of earthfill against the conduit. Plastic pipe used in low hazard potential embankment dams is sometimes not encased in reinforced cast-in-place concrete. However, use of a filter zone surrounding the conduit is a valuable defensive design measure, even for low hazard potential classification sites with favorable conditions. Some low hazard potential embankment dam designs may not employ a filter zone around the conduit, but eliminating this valuable feature should be carefully considered and justified. Filter diaphragms should only be eliminated when extremely favorable soil conditions, good conduit construction materials and methods, reliable construction practices, and favorable foundation conditions exist.

- *Repair* (chapter 14).—Damage to plastic pipe may occur from improper shipping, handling, or improper construction technique. Damage can be in the form of kinks, punctures, breaks, or abrasion. Plastic pipe that undergoes this type of damage cannot be repaired, and the damaged section of pipe should be removed and replaced.

See section 4.1.4 for guidance on the renovation, replacement, and repair of drainpipes.

Chapter 4

Drainpipes and Filters

Most modern embankment dams designed since about the mid 1970's include drainage and filter zones that are sized to protect against seepage-related failure modes without relying solely on a system of drainpipes. Drainpipes provide extra capacity in drain systems and provide added conservatism and redundancy to the design of these important features. Collecting and measuring seepage flows through and under embankment dams is an integral part of safe and reliable monitoring of embankment dam performance. This flow is typically collected and conveyed through filters and drainpipes as part of a embankment drain collection system. Collected seepage can be measured to detect changes in seepage flows that may indicate changes in the condition of the dam or foundation, or possible clogging of drains. Collected seepage can also be inspected for the presence of sediments that may indicate a possible loss of soil materials.

This chapter will present methodology for the structural design of the drainpipe and hydraulic design for the collection of water into the drainpipe. This chapter will also discuss the relationship between soil backfill and the drainpipe. Types of backfill are separated into several groups, including backfill for perforated and nonperforated pipe as well as impervious caps to prevent surface water infiltration. In the discussion for perforated pipe backfill, the issue of single versus two-stage filters is addressed including the recommended minimum thickness for those materials.

Placing drainpipes beneath embankment dams in inaccessible locations should be avoided. Drainpipes system designs should include inspection wells or cleanouts to allow easy access for inspection, cleaning, and maintenance. These features should be accessible without disruption of the embankment. Each drainpipe segment should be accessible from both ends.

Example A-4 in appendix A demonstrates the principles involved for drainpipe and filter design.

4.1 Drainpipes

Drainpipes as described in this document are structural pipes used to convey seepage water collected in a drain system to a discharge at some point downstream of the dam. The materials used for these pipes have changed over time. Early dam construction typically used rigid pipe (i.e., clay tile) with flexible plastic pipe becoming more popular since the 1980's. This section will address structural and hydraulic design for these flexible pipes. Figure 53 shows an example drainpipe construction using flexible plastic pipe.

A variety of materials and pipe cross sections are available for use as drainpipes. The most common materials are HDPE and PVC as described in other sections of this document. Commonly available cross sections include solid wall, corrugated single wall, and corrugated profile wall as described in section 1.2.1. Single wall corrugated plastic pipe is easily crushed during typical construction installations and should not be used. Solid wall pipe is available in PVC and HDPE materials. Solid wall PVC pipe should be a minimum schedule 80 gauge (schedule refers to the thickness of the pipe wall). While solid wall HDPE pipe offers sufficient strength, it is the most costly. Since quality of construction can vary, CCTV inspection should be performed to verify possible deficiencies within these pipes. The manufacturer's recommendations for installation should always be consulted. Improper installation can result in a number of deformations, punctures, etc. Good installation practice should always be used.

Corrugated metal pipes were commonly used in drainpipe systems at one time, but deterioration and subsequent piping of surrounding filters into the pipes has caused these materials to be regarded as a poor choice. Asbestos cement pipe was also used in many drainpipe systems, but the hazard from asbestos in manufacturing has caused this product to no longer be available.

4.1.1 Structural design

Drainpipes should be structurally designed by the design procedures described in chapter 3. The soil and hydraulic loadings on the pipe should be determined by the methods described in chapter 2. Drainpipes beyond the footprint of the embankment are typically trench conduits while those beneath the embankment are typically positive projecting conduits. For guidance on evaluating drainpipe configurations to accommodate CCTV inspection equipment, see section 6.2.

4.1.2 Hydraulic design

Determining the anticipated seepage that will be collected by a drainpipe and back-calculating the size of pipe required to carry that flow can be complicated. The Bureau of Reclamation has developed simple rules-of-thumb for sizing drainpipes based on the size of embankment and type of foundation soils in which the drain is embedded. Table 11 summarizes those recommendations. Smaller pipes can be justified by more detailed flow compilations. Drainpipes should be sized to maintain a piezometric surface below the top of ground in most situations.

Figure 53.—Installation of profile wall corrugated pipe for a drainpipe replacement during a modification of an embankment dam.

Table 11.—Drainpipe diameter based on dam size and foundation type

Dam height (ft)	Foundation type: Pervious < 15% fines (SP, GP)	Foundation type: Semipervious or impervious >15% fines (SM, GM, ML, CL, SC, GC)
< 30	min. 12 in	min. 8 in
30–100	12-18 in	min. 12 in
> 100	18-24 in	min. 12 in

While the load-carrying capacity of nonperforated pipe is well documented, the strength of perforated pipe is less commonly addressed. Since the corrugations carry the majority of the load for both single-wall and profile-wall HDPE pipe, perforations through the corrugation valley have negligible effect on pipe strength (less than 1 percent). However, for all types of solid-wall plastic pipe (PVC, HDPE, etc), perforations will reduce the load-carrying capacity (loss in strength proportional to perforation percent open area). Additional research (PM-3) is needed as proposed in chapter 8.

Solid and profile wall corrugated pipe have the additional benefits of a smooth interior, which increases flow capacity, and no interior corrugations to collect and trap soil particles (which should be trapped at the measurement point sediment trap). Joints for corrugated pipe are typically bell and spigot with a gasket. Solid wall

HDPE pipe has become popular recently for drainpipe applications due to its strength and leakproof butt fused joints. The two major limitations in using this type of pipe in drainage applications are its cost and lack of factory produced perforations (perforations have to be drilled or cut in the field).

Typically, perforations in drainpipes are available in three geometries:

- Circles, which are drilled
- Slots, which are made with a saw blade or furnished from a factory
- Well-screen configuration

A fairly wide range of slot widths can be chosen to meet filter criteria related to either surrounding drain material or constructed filter zones. For slot-shaped perforations, the controlling dimension is the slot width. Typically, the slot length is a function of the slot width since the size of the tool used to make the slot is a function of the desired width. The slot length is also a function of how far the manufacturer advances the tool into the pipe to make the perforation. The manufacturer will select a slot length that satisfies desired strength and inflow requirements for a particular pipe product. Slotted pipe has considerable higher capacity than pipe with circular perforations (see figures 54 and 55). Corrugated slotted HDPE pipe is readily available in a variety of pipe diameters (see AASHTO's M252 and M294). If desired, perforations can be drilled into solid wall HDPE pipe to provide a perforated pipe. PVC pipe is available with circular perforations as well as slots (figure 56). The use of geotextile socks surrounding perforated drainpipes should not be used (see section 4.2.1 for additional discussion).

Figure 54.—Profile wall corrugated HDPE pipe with slotted perforations.

Figure 55.—Profile wall corrugated HDPE pipe with circular perforations.

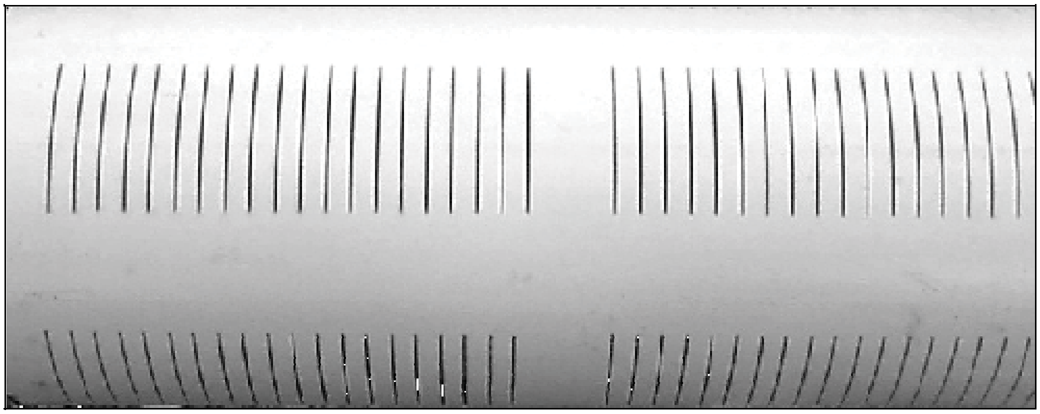


Figure 56.—Slotted PVC pipe.

Drainpipes can also be constructed using plastic well screen products. These pipes have the largest unit open area and highest capacity of the available products, and consequently have the lowest strength. They are usually more expensive than other pipe types.

Consideration should also be given to the amount of inlet open area of perforation for a unit length of pipe. As a rule, perforations should be used that incorporate the entire pipe circumference (AASHTO Class II Perforation, M252). These patterns typically consist of perforations on a 45- or 60-degree pattern (8 or 6 equally spaced perforations around the drainpipe, respectively). Perforation patterns that only utilize half of the pipe circumference (AASHTO Class I Perforations, M252) should not be used due to reduced collection capacity.

Another consideration is the percent of the openings that will be blocked by the surrounding filter material. NRCS Soil Mechanics Note No. 3 (1971, p. A-4) recommends that the effective open area for circular perforations be considered as 30 percent of the total perforation area in computing inflow capacity. For rectangular slots, the recommendation is to use 60 percent of the total area of slots as the available flow area. The rate of flow into any given pipe per foot can be obtained from the manufacturer literature.

Flow capacity and pipe size of drainpipes can be calculated using Manning's equation or Hazen-Williams equation, or obtained from table B-3 in the Bureau of Reclamation's *Design of Small Dams* (1987a). The depth of flow in the pipe is typically no more than a maximum 75 percent full so that flow does not become pressurized. Pressurization limits the effectiveness of the drain. The amount of flow into a plastic pipe is a function of the opening size and the number of openings per foot of pipe. Perforations or slots in drainpipes are sized to prevent surrounding drain materials from passing through them, which would result in a piping condition. If the size and number of perforations and slots limits capacity, a well screen type product should be considered. Since the requirement for soil retention is to prevent particles from passing through a perforation, care should be taken to not use slot length values as the maximum dimension. An inability to install the pipe uniformly (i.e., no sags within segments or from segment to segment) will reduce the flow capacity of the pipe (the pipe may flow full through sags). Even correctly installed pipes can develop sags after construction due to differential settlement.

Calculating the amount of water that can be collected from a foundation or embankment is not as simple as calculating flow in a pipe. Seepage analysis and collection prediction is complicated by lack of data and understanding of geologic conditions. Depending on site conditions and the complexity of the foundation, seepage analysis can be quite complicated, although lack of data in a small, simple foundation can be just as problematic (see USACE's *Seepage Analysis and Control for Dams*, 1993).

Depending on the site conditions and the complexity of the foundation, seepage analysis can be subject to significant errors. For instance, if high permeability lenses are ignored or not detected in an investigation, errors can be dramatic. In simple foundations with few strata, computations of flow quantities are more accurate.

The simplest way of calculating foundation flow contribution into a drainpipe is by using Darcy's Law:

$$Q = kiA \tag{4-1}$$

where:

Q = rate of flow into a drainpipe, ft^3/yr
k = coefficient of permeability of the surrounding filter or foundation, whichever is greater, ft/yr
i = hydraulic gradient, head loss outside the pipe divided by the distance over which that head loss occurs, ft/ft
A = filter or foundation area through which flow passes, ft^2

The coefficient permeability may be estimated from empirical relationships, presumptive values, laboratory tests, or field tests. Units for the coefficient of permeability should be consistent with other terms in the equation.

Empirical methods for estimating the permeability for filter materials and coarse-grained foundation soils with no fines are available. These methods are usually based on the grain-size distribution curve of the materials. Most empirical estimates use the effective grain size, or D_{10} size from a soil or filter's gradation curve. Some estimates use the D_{15} size, which is obtained similarly. The D_{10} and D_{15} sizes represent the particle size diameter (in millimeters) of the 10th and 15th percentile respectively, passing grain size of a material.

McCook (2002, p. 5) obtained an empirical relationship for coefficient of estimating the permeability of a soil based on its D_{10} size and porosity. The equation is:

$$k(\text{cm/s}) = 0.01047 e^{9.3071 \times \frac{\eta}{100}} D_{10}^{2} \tag{4-2}$$

where:

k = coefficient of permeability, cm/sec
e = base of the natural logarithms, 2.7183
η = porosity, percent of void volume, %
D_{10} = particle size diameter in millimeters of the 10th percentile passing grain size

Empirical methods for estimates for the coefficient of permeability for granular filters are also available from extensive testing performed by the Soil Conservation Service (now NRCS) Soil Mechanics Laboratories, reported in Sherard, et al. (1984). The study concluded that for clean sands and gravels in the tests, the D_{15} parameter from the grain size curves provided the best empirical estimate of permeability. The empirical relationship for that study is:

$$k(\text{cm/s}) = CD_{15}^{2} \tag{4-3}$$

where:

k = coefficient of permeability, cm/sec
C = constant ranging from 0.2 to 0.6, averaging 0.35
D_{15}= particle size diameter in millimeters of the 15th percentile passing grain size

The range for the C value is between 0.2 and 0.6, with an average value of 0.35. The range in values is based on the scatter in the data and the differences in materials tested. Generally, sands with rounded particles were slightly more permeable than those with angular shapes and have higher C values. See NRCS Soil Mechanics Note No. 9 (1984) for more details on this relationship.

A number of sources present presumptive values for soil permeability, such as table A1 in the Bureau of Reclamation's *Design Standard for Seepage Analysis and Control* (1987b), NRCS's Soil Mechanics Note No. 9 (1984), figure 7.6 in Holtz (1981), table 2.1 in Peck et al. (1974), and Sherard et al. (1984).

Common laboratory permeability tests can be found in ASTM D 2434, the Bureau of Reclamation's *Earth Manual* (1989) and USACE's *Laboratory Soils Testing* (1986). Foundation field testing can be done in single or multiple drillhole arrangements. While field tests give the most accurate prediction, they are also the most expensive. Typical testing methods are described in geotechnical engineering textbooks as well as the references previously mentioned.

Caution should be applied when using empirical, presumptive, and laboratory testing estimates of permeability. These methods tend to predict higher than actual permeability values, so actual seepage flows will be less because stratification and heterogeneity of foundation material are not considered. Naturally occurring soil deposits also almost always have greater horizontal permeability than vertical permeability. Estimates for k, i, and A should be on the high side in order to calculate a larger Q in the interest of not undersizing the pipe. Requiring drainage capacities 10 times greater than the calculated values is common.

Seepage analysis using computer software permits more detailed calculations, although the limitations of selecting permeability values described above also pertain to this method. Computer modeling allows the designer to utilize anisotropy in the calculations, which can have a significant effect on seepage calculations. If instrumentation data exists for an existing dam, model calibration should be done. This calibration consists of modeling known conditions for the existing structure. Once the model is constructed, analysis is made for a given reservoir elevation. The measured pressures in the instrument are compared to those produced by the model. If there is disagreement, permeabilities are adjusted until there is agreement. This trial and error method can be time consuming, and it should be noted since the number of unknowns exceeds the number of knowns, there are multiple valid solutions for any given model. Engineering judgment is required to discern whether a valid solution has been found. This calibration scheme aids in reducing uncertainty about the permeability and thus the flow.

The next design issue for drainpipes is determining perforation size. The U.S. Army Corps of Engineers (2004, p. B-4) recommends the following criterion:

$$\frac{\text{Minimum 50 percent size } (D_{50}) \text{ of filter material}}{\text{maximum opening of pipe drain}} > 1 \tag{4-4}$$

The Bureau of Reclamation has adopted two criteria for grain size of filter materials in relation to perforation openings in drainpipes (Bureau of Reclamation, 2007, p. 14). The first is for use with uniformly graded materials:

$$\frac{D_{85} \text{ of the filter nearest the pipe}}{\text{maximum opening of pipe drain}} > 2 \quad \text{(uniformly graded)} \tag{4-5}$$

This criterion applies to multistage filter/drain combinations surrounding the drainpipe.

The second criterion is based on recent studies by the Bureau of Reclamation (1997, p. 24) that indicate that single stage broadly graded sand and gravel filter combinations should have a smaller slot size to prevent plugging. The following criterion may be used:

$$\frac{D_{85} \text{ of the filter nearest the pipe}}{\text{maximum opening of pipe drain}} > 4 \quad \text{(broadly graded)} \tag{4-6}$$

Note: The maximum opening of drainpipe in equations 4-4, 4-5, and 4-6 is the diameter for hole perforations and the width for slot perforations.

The NRCS recommends the following criterion which is about two times larger than the previous two other criteria (NRCS, 1994, p. 26-5). For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

$$\frac{D_{85} \text{ of filter material}}{\text{perforation size}} > 1 \tag{4-7}$$

4.1.3 Inspection wells and cleanouts

Drainpipes should be installed with inspection wells or provided with cleanouts, so there is easy access for inspection and maintenance, and reasonable access without compromising the dam for repair, if necessary.

Inspection wells are commonly used along or at the end of drainpipes. They typically serve three functions; access to the drainpipe, inclusion of a flow measurement device such as a weir or flume, and inclusion of a sediment or stilling basin that collects any sediment that may be included in the drainpipe flow.

Inspection wells are typically manufactured from precast reinforced concrete and range in size from 8 to 12 feet in diameter. The base can be a cast-in-place (figure 57), or precast and set in place. Combined slab/ring units should not be used due to poor strength and difficulty in installation. Experience has shown that handling of the combined units has resulted in separation issues. Precast rings come in standard lengths and several may be required to reach the desired surface elevation. Figure 58 shows the bottom ring of an inspection well. The inlet and outlet opening sizes should be supplied to the manufacturer prior to fabrication, so proper reinforcement and opening size can be included at the factory. Opening sizes are larger than the maximum outside diameter of the drainpipe, so that the pipe can penetrate the well and the annulus can be dry packed with a lean concrete. The invert of an inspection well is shown in figure 59. See section 4.3.3 for a discussion of backfill around inspection wells.

Figure 57.—Placement of a cast-in-place base slab for an inspection well.

Figure 58.—The first ring of a drainpipe inspection well.

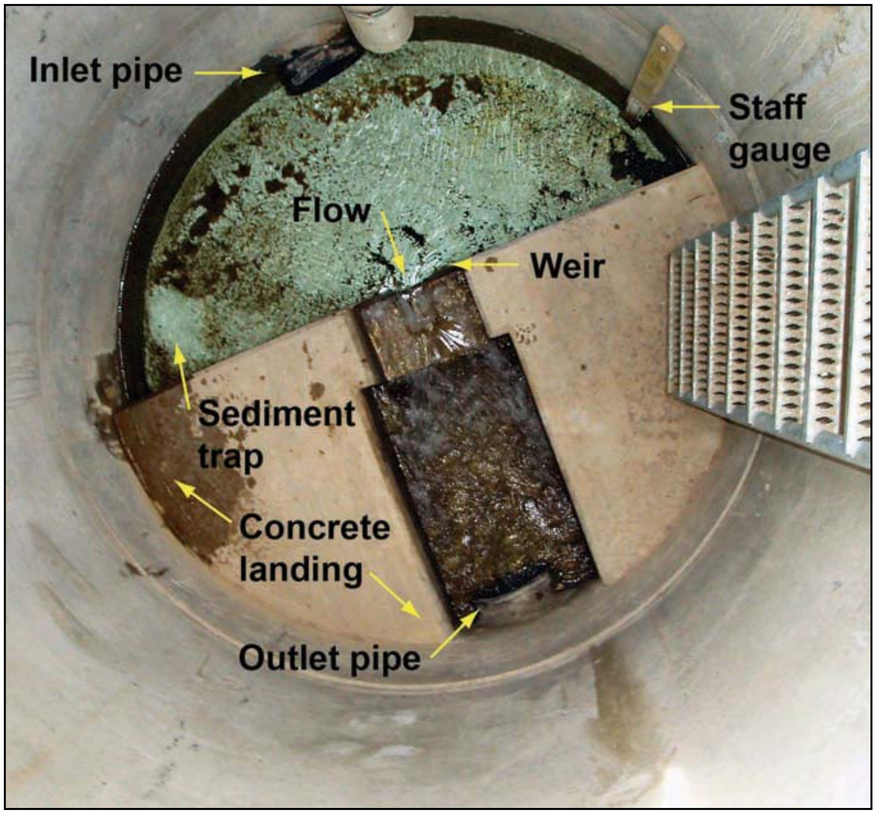


Figure 59.—Invert of an inspection well. The sediment trap is painted white, so any sediment can be easily observed. The dark material in the trap is algae.

When flow measurement and sediment traps are not required, a more economical cleanout type drainpipe access can be used. Cleanouts provide access for CCTV inspection equipment and cleaning tools and are used at the upstream end of drainpipes. These cleanouts consist of a "sweeping" end from its invert to the ground surface by two 22.5-degree bends. This results in the pipe daylighting at the ground surface at a 45-degree angle (figure 60). A protective encasement (typically CMP pipe) is placed around the plastic drainpipe to protect against vandalism and the elements. The encasement pipe should provide a minimum 6-inch free space between itself and the plastic pipe. The annular space between the two pipes is backfilled with gravel to provide support to the plastic pipe. The encasement pipe is embedded a minimum of 5 feet in the ground and a lockable protective metal cover is used to secure the end of the cleanout. Figure 61 shows an example of a cleanout.

Lateral cleanouts can also be used on long drainpipes. The layout of a lateral cleanout is the same as described above except the "sweep" consists of 22.5-degree bends that transition in both the horizontal (away from the drain alignment) and vertical (toward the ground surface) planes. An alternative to the sweep concept can be used for drainpipes of great length requiring intermediate cleanouts. A vertical riser consisting of nonperforated pipe of the same material and diameter is

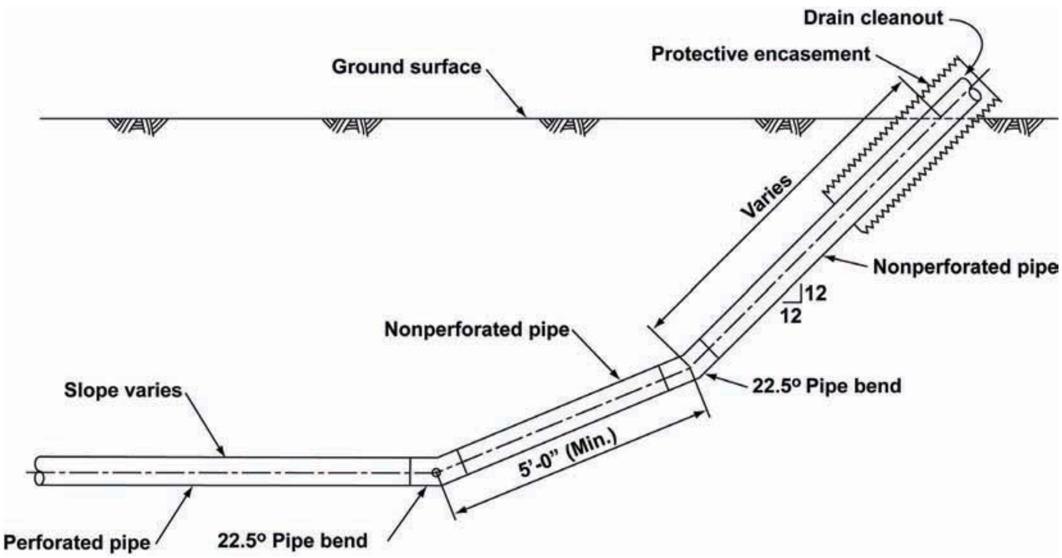


Figure 60.—Cleanout designed to accommodate CCTV inspection in pipes with diameters of 8 inches or larger.

Figure 61.—A drainpipe cleanout with a steel encasement and lockable protective cover.

connected to the drainpipe. The top of the riser is protected by a CMP pipe, lid, and lockable latch similar to end cleanouts. This alternative can be used for drainpipes with diameters greater than 12 inches. See section 6.2 for additional guidance on the design of drainpipes to accommodate CCTV inspection equipment.

4.1.4 Renovation, replacement, and repair of drainpipes

Experience has show that CCTV inspection often reveals damaged or collapsed drainpipes. In many cases, the drainpipes appear to have failed during original construction due to equipment travel over the drain alignment, inadequate pipe support, pipe material defects, or other factors. In other cases, the drains are in a state of failure due to the deterioration of the drainpipe, differential settlement along the alignment of the drain. Replacement of the drainpipe may be an appropriate response in these situations. Considerations for repairing or replacing the drainpipe include:

- *Failure mode.*—If considering replacement of a drain, the designer should consider relocating the drain alignment, elevations, outfalls, etc. to better address seepage conditions at the dam or to reduce or eliminate potential failure modes associated with the drainage feature.

- *Address why the drain failed.*—In repairing or replacing the drainpipe, the designer should consider reasons why it failed, such as poor construction practice, reasons related to the pipe material, or drainpipe plugging due to problems with the drain envelope material. The repair or replacement should be designed to address these issues.

- *Design considerations.*—Drain repairs or replacements offer excellent opportunities to provide additional access to a drain system. If practicable and reasonable, access points should be added at least every 500 to 1,000 feet to facilitate inspection, cleaning, maintenance, and monitoring activities. Access points should include features for monitoring flow and material movement within the drain system, personnel safety features, and access for CCTV inspection and cleaning equipment.

- *Quality assurance.*—Many drainpipe failures are the result of construction activities. Drainpipe repair or replacement projects should include provisions for thorough inspection during construction and following completion of construction. A CCTV inspection of the drain alignment at the completion of construction is required. As-built drawings with accurate surveys must also be completed as part of the modifications.

When damaged or collapsed existing drainpipes are encountered, complete removal and replacement may not be possible due to a large amount of fill over the pipe or cost constraints. When met with such a situation it may be possible to slipline the existing pipe. The simplest way to perform this sliplining is to insert a new pipe into the damaged pipe. Since joint offset, deformation, and cracking can lead to a significant reduction in interior cross section of the existing pipe, the new pipe may have to be significantly smaller than the existing pipe.

Generally, it is not practical to design the replacement pipe to meet filter criteria. Usually the intent of sliplining repairs is to provide structural support to the existing damaged pipe. Flow measurement and a sediment trap should be installed at the downstream end of sliplined drainpipe, so changes in flow and material movement can be monitored.

Introducing the new pipe into the existing pipe can be problematic. If both ends of the existing pipe are accessible this will make installation of the liner easier. Having to install the liner from one end will be much more difficult and if the amount of liner to be installed is large, it could be impossible.

Successful installation techniques when both ends of the drain are accessible include sending a fish line through the segment to be sliplined, attaching a torpedo to the fish, with the slipliner attached to the torpedo. The force required to pull this type of an arrangement may be large. Mechanical means may be required to make the pull, but care should be taken to not exceed the fish line or connection strengths of the apparatus. Breaking a fish line, or getting the torpedo stuck in the pipe can lead to a bigger problem than what was originally being corrected.

4.2 Filters

Properly designed filters adjacent to drainpipes serve two functions—allow foundation flow into the pipe, and prevent foundation and embankment soil from migrating into the pipe.

4.2.1 Zoning

Drainpipes have been designed in single and double stage configurations. A single stage system consists of one zone of filter material, usually sand, surrounding a drainpipe. The double stage system consists of a coarse drainage zone (gravel) surrounding the pipe and a filter (sand) zone surrounding the coarse element. Figure 62 illustrates these two types of design.

Single stage designs have been used on smaller jobs, such as low hazard potential dams in the interest of reducing costs and simplifying construction. Because the perforations in commonly available drainpipe are too large to meet infiltration criteria for typical sand filters, one of the following conditions must be met:

- The perforated pipe must be wrapped in a geotextile.
- Screen type pipe must be used.
- The trench must be lined with geotextile.
- A broadly graded filter must be used.

None of these approaches is entirely satisfactory. A geotextile used to prevent sand from infiltrating into perforations in the drainpipe can become clogged from ochre biofilm. Ochre formation results from microbial colonization by bacterial

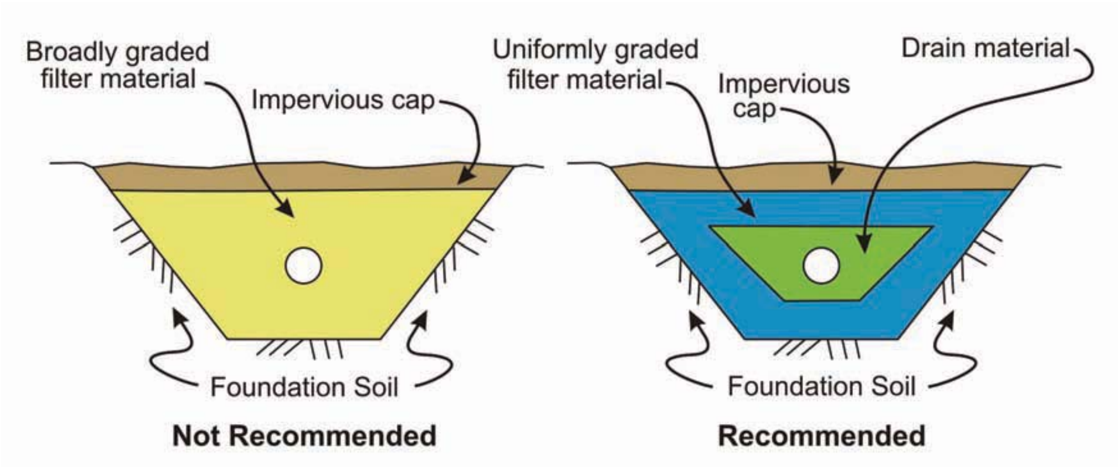


Figure 62.—Idealized cross sections of single (left) and double (right) stage drainpipes. The placement of the drain material around the pipe can result in a variety of geometries based on placement method. Minimum cover requirements should always be met independently of geometry.

consortia (biofilm) that may include various iron bacteria and its affinity to iron compounds (Mendonca, Ehrlich, and Cammarota, 2006, p. 34). Factors that influence the formation of ochre biofilm on geotextile include space between fibers, roughness of the fibers, and thickness of the geotextile. Geotextile wraps have also been known to clog when a filter seal forms caused by concentrated flow through the geotextile at perforations in the pipe. The concentrated flow transports (erodes) soil particles that concentrate on the face of the fabric. Clogging can more easily occur if the surrounding drainage medium contains some fines or the perforations are circular holes rather than slots. Most major design agencies, including the Bureau of Reclamation, Natural Resources Conservation Service, and the U.S. Army Corps of Engineers, do not permit use of geotextiles in critical drain applications due to the potential for clogging and particle migration around the edges of thin geotextile sections.

Poor performance of broadly graded filters have been noted in a number of case histories and laboratory tests. These filters may have small particle sizes that pass through the slots while larger particles may become lodged in the slots. Meeting the slot size requirements described in section 4.1.2 is difficult with broadly graded materials. Two stage filter/drain combinations have higher permeability and will be more efficient in collecting seepage than single stage filters. For these reasons, single stage filters should be avoided and high hazard potential dams and two stage filters are preferred by designers. However, for economy and simplicity, sometimes single stage drainage elements are used in low hazard potential dams. When considering this type of filter, consideration should be given to internal stability and plugging of perforations within the drainpipe. A number of methods are available to check for internal instability and are presented in the literature (Kenney and Lau, 1986; Laflaur, Mlynarek, and Rollin, 1989; Milligan 1986; Ripley, 1986). The designer should also be aware that a broadly graded sand and gravel filter has a lower permeability than a uniformly graded sand filter.

If single stage filters are used, slots should be no larger than the D_{50} of the filter. This can lead to small slot size, which requires the use of screen type pipe or custom made perforation. Caution should be exercised in the use of screen pipe due to its low strength.

4.2.2 Determination of filter gradation limits

Determination of the required gradation limits for filter and drain material is a function of the "base" material it is protecting. The current state of practice for these limits is that the material performs two functions. The first is that it prevents the movement of the base material into the filter and the second is that the filter be sufficiently permeable that pore pressures do not build up as a result of the filter itself. For a single stage drain, the base material is the foundation soil. For a two stage drain, the base material of the outer filter is the foundation soil and the base for the inner filter (gravel) is the outer filter. The details for this design are covered in chapter 6 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). See example A-3 in appendix A for an illustration of required calculations. Guidance can also be found in the NRCS (1994) design standard.

In lieu of complete filter design, experience has shown that fine concrete aggregate designated in ASTM C 33 meets the design requirements for many foundation materials with between 40 and 85 percent passing the No. 200 sieve. The No. 200 sieve must be restricted to meet the permeability requirement of the filter design. Table 12 gives the gradation for this material, which is commonly referred to as "C 33 concrete sand." Because foundation conditions differ from site to site, this filter should always be checked against the gradation of the base soil (foundation soil). Because foundation conditions differ from site to site, this filter should always be checked against the gradation of the base materials (foundation soil) before use.

In a similar manner, when modified ASTM C 33 concrete sand is used as a filter, there are standard materials that can be used as the gravel drain that surrounds the pipe. Several coarse aggregates in ASTM C 33 have been checked against modified C 33 concrete sand and are included in table 13. When using modified C 33 concrete sand, the coarse aggregates does not have to be checked since the filter size is fixed. Six materials have been included since not all materials will be available at all locations.

Based on the D_{85} size of these materials, the maximum slot size can be calculated as described in section 4.1.2 using the Bureau of Reclamation criteria (equation 4-4, 4-5, and 4-6, respectively). Table 14 summarizes the resulting perforation sizes.

Table 12.—Gradation of ATSM C 33 fine aggregate with additional requirement[1]

Sieve size	Percent passing, by weight
⅜-inch	100
No. 4	95-100
No. 8	80-100
No. 16	50-85
No. 30	25-60
No. 50	5-30
No. 100	0-10
No. 200[1]	0-2[2]

[1] Note qualifications of No. 200 sieve.
[2] 2% stockpile, 5% in-place. For discussion of material breakdown, see section 4.3.2.

Table 13.—Gradation for ASTM C 33 drain materials (percent passing by weight)

	No. 467	No. 57	No. 67	Blend 579*	No. 8	No. 89
Sieve size	$D_{15}F \leq 9 \times D_{85}B$			$D_{15}F \leq 4 \times D_{85}B$		
2-in	100	-	-	-	-	-
1½-in	95-100	100	-	100	-	-
1-in	-	95-100	100	90-100	-	-
¾-in	35-70	-	90-100	75-85	-	-
½-in	-	25-60	-	-	100	100
⅜-in	10-30	-	20-55	45-60	85-100	90-100
No. 4	0-5	0-10	0-10	20-35	10-30	20-55
No. 8	-	0-5	0-5	5-15	0-10	5-30
No. 16	-	-	-	0-5	0-5	0-10
No. 50	-	-	-	-	-	0-5

* This gradation is a blend, in equal parts, of gradations No. 5, 7, and 9 and is not an ASTM standard aggregate.

Table 14.—Maximum perforation dimension for ASTM C 33 Drain Materials*

	No. 467	No. 57	No. 67	Blend 579	No. 8	No. 89
USACE (mm)	0.53 in. (13.5)	0.41 in. (10.3)	0.35 in. (8.8)	0.28 in. (7.2)	0.23 in. (5.8)	0.16 in. (4.1)
Bureau of Reclamation (mm)	0.53 in. (13.4)	0.38 in. (9.6)	0.35 in. (9.0)	0.37 in. (9.5)	0.19 in. (4.8)	0.18 in. (4.5)

* The minimum dimension should be used. For circular perforation, that is the diameter; for slots, the width measurement should be used.

While some design standards allow for $D_{15}F \leq 9 \times D_{85}B$, in this instance (Bureau of Reclamation, 2007), other standards only allow $D_{15}F \leq 4 \times D_{85}B$ (NRCS, 1994). Table 13 illustrates which materials meet each standard. The Blend 579 material is a blend of the No. 5, No. 7, and No. 9 gradations from the C 33 specification. Although it is not a standard ASTM gradation, it is included since it allows a greater pipe perforation size as shown in table 14.

4.2.3 Flow capacity

When designing drainpipes or other drainage collection systems for pervious foundations where seepage is expected to be significant, consideration should be given to the permeability of the filter in relation to the permeability of the foundation. In situations where the foundation consists of interbedded silts, sands, and gravels, design criteria require sizing the filter for the silt sizes. This can result in a filter composed primarily of sand sizes being placed over the gravel layers that carry the majority of seepage. This filter then acts as a barrier to the flow in the gravel, resulting in poor seepage collection and high pore pressures. If this issue cannot be resolved by adjusting the filter design, additional water barrier elements upstream of the centerline of the dam (i.e., cutoff wall, upstream blanket, or reservoir liner) may be required. Figure 63 illustrates an existing (old) drain that produces a large flow, although it does not meet modern filter criteria. Figure 64 illustrates the barrier situation that can arise for a replacement drain when filter criteria are followed.

4.3 Backfill

The following sections address two types of backfill around drainpipes. The first describes backfill around nonperforated drainpipe, and the second describes backfill around perforated drainpipe. ASTM D 2321 also provides guidance on backfill for drainpipes installed in trenches with vertical sides.

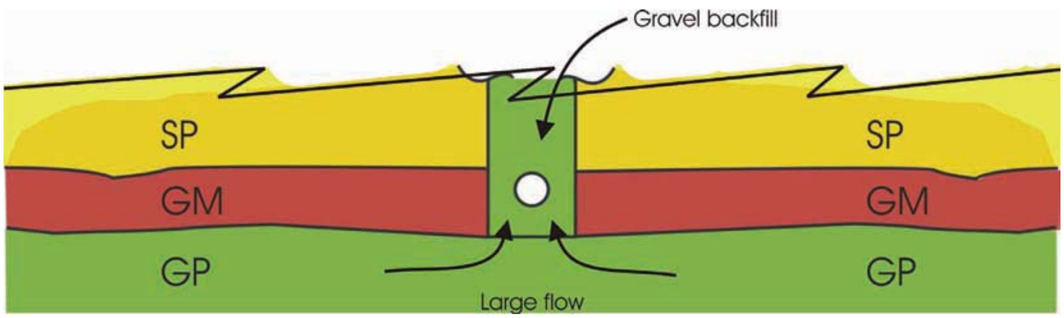


Figure 63.—Old drain.

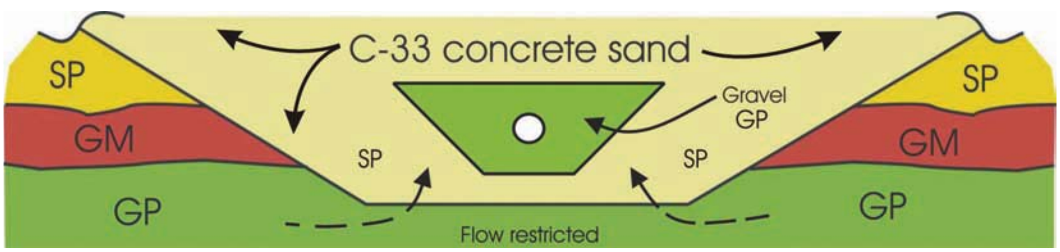


Figure 64.—Barrier condition introduced by a replacement drain resulting in poor seepage collection and high pore pressure.

4.3.1 Backfill for nonperforated drainpipe

For ease of construction and placement, backfill should be a material composed of natural gravel and sand, and free of silt, clay, loam, friable or soluble materials, and organic matter. Table 15 gives the gradation requirements for an acceptable material, although other granular gradation may also be satisfactory.

Table 15.—Example gradation for drainpipe embedment

Sieve size	Percent passing, by weight
¾-inch	100
No. 4	50-75
No. 50	10-25
No. 200	0-5

No backfill materials should be placed in the drain when either the materials or the foundation on which it would be placed is frozen or flooded. No brush, roots, sod, or other organic or unsuitable materials should be placed in the backfill.

Backfill should be carefully placed and spread in uniform layers. Backfill should be placed to approximately the same elevation on both sides of the pipe to prevent unequal loading and displacement of the pipe. The difference in elevation of the backfill on both sides of the pipe should not exceed 6 inches at any time. Adequate earth cover (minimum 2 to 4 feet) should be provided over the pipe to prevent damage to it from construction equipment loads. Figure 65 shows equipment passing over a pipe with a 4-foot cover of material, with no damage to the pipe.

4.3.2 Backfill for perforated drainpipe

Durability and material quality go hand in hand. Concerns with these characteristics are associated with material breakdown during construction. Once they leave the processing plant, the aggregate particles can break down during handling and placing procedures. Typically, loaders and possibly dozers place these materials in stockpiles in order to build larger piles. Then the materials are loaded into trucks, dumped onto the fill, bladed to a uniform lift thickness, and compacted. Each of these operations can cause individual aggregate particles to break down. This breakdown will lead to a change in gradation between the material produced at the sieving plant and what is in place in the dam. Typically, filters are required to have no more than 5 percent fines measured in the fill. Breakdown between the stockpile and fill is 1 to 2 percent, thus requiring 3 percent limit on the fines when measured in the stockpile. While it is beneficial to specify measurement in the stockpile for construction operations, testing of the fill should also be done in accordance with ASTM C 117 and ASTM C 136 to measure the amount of breakdown caused by placement operations. The amount of breakdown is a function of the durability of the raw material and the amount of handling between the plant and the fill. Breakdown is usually a greater concern for smaller grain sizes used for filters than it is for larger grain sizes, which are used for drain material.

As a minimum, the filter material should meet the durability requirements of concrete aggregate as defined in ASTM C 33 class designation 1N. In addition to the quality requirements of ASTM C 33, the material should be nonplastic. Since it is desirable that filter materials "flow" or self heal, adhesion such as plasticity or cementing is undesirable. Plasticity can be determined in accordance with ASTM D 4318 on material passing the No. 40 sieve. Nonplastic material is defined as having a plasticity index (PI) of zero as per the previous procedure. Additionally, the material should be free of cementing agents, such as, but not limited to, carbonate minerals, gypsum, sulfide minerals, and sand-sized volcanic (pyroclastic) ash. Cementing is indicated by cohesive behavior of granular material. Cementing agents can be detected by checking for reaction of the material to hydrochloric acid.

McCook (2005, p. 3) suggests performing compressive strength tests on samples of fine filter materials to determine if undesirable cementitious properties may be present in a given sample. The "sand castle" test proposed by Vaughan and Soares (1982, p. 29) may also be helpful for evaluating self-healing properties of sand filters. For small projects, it may not be feasible to determine aggregate quality by laboratory

Figure 65.—Construction equipment can travel safely over plastic pipe when adequate cover above the drainpipe is provided.

testing. In this instance, the designer should consider the mineralogy of the parent material. Aggregates that are derived from metamorphic and igneous based rocks will usually have higher quality than aggregates that come from sedimentary rocks.

For materials obtained from commercial sources, stockpiles should be examined for slope uniformity. Piles with irregular slopes or portions of near vertical surfaces indicate high fines content or possibly binders or cementing agents in the material.

4.3.3 Zoning design

Filter and drainage materials surrounding the drainpipe should have a minimum thickness equal to 12 inches. The ease of placement and inspection of the filter and drainage material around a drainpipe should serve as a guide to the designer on setting the thickness of these materials. The thickness may need to be increased for more difficult placements and inspection conditions. For two stage filters, care must be exercised to ensure the gravel stage is completely surrounded by the sand stage to ensure the foundation does not erode into the gravel stage.

A capping layer of relatively impervious material (>12% fines) should be used to differentiate groundwater and surface water flows in order to more completely understand the performance of an embankment dam. For this reason, only groundwater flow, or seepage through the dam, should be collected and measured. Drainpipes should be designed to isolate the surface flow (by use of drainage ditches) and infiltration by the use of a surface cap. Therefore, the drainpipe filter envelope should be capped with a relatively impervious layer to prevent precipitation from entering the drainpipe.

Relatively impervious material should also be used as backfill around inspection wells. This relatively impervious material, or "underground dam" acts as a barrier to flow in surrounding drainage materials. This barrier directs this flow into the drainpipe and through the measurement device in the inspection well.

4.3.4 Improving access

CCTV inspection equipment is sometimes limited by the length of cable tether and by sharp bends in the drainpipe. The same is true of drain cleaning equipment. In general, CCTV inspection equipment and cleaning equipment can travel up to about 1,000 feet from the access point (under ideal conditions), depending on the grade of the pipe and its smoothness. Some selected equipment may have extended capabilities, but 1,000 feet is a good rule of thumb. Some existing embankment dams have very long sections of drainpipe with no intermediate access points. Access is further complicated by sharp bends in the drainpipe alignment. Sediment accumulation, roots, organic debris, or damaged pipes can further limit access to the drainpipes. These problems may limit the possibility for inspection, monitoring, cleaning, or maintenance of significant portions of the drainage feature. For additional guidance on inspection and cleaning of drainpipes, see section 6.2.

The cost and feasibility of improving access to drainpipes warrant careful consideration of the need for such access. When evaluating the need to improve access to the drain system, the following factors should be considered, in addition to those considerations discussed in the previous section:

- *Constructability.*—Constructability has a major influence on the decision to provide additional access to an existing drain system. The location and configuration of drainpipes vary from dam to dam. Drain alignments near the downstream toe of embankments are generally much easier to access than alignments deeper under the dam. Some drain systems have multiple alignments parallel to the crest of the dam, possibly necessitating multiple access points. Others are constructed as a grid under the embankment. Access constructability considerations include:

    1. Potential to cause harm

    2. Depth of excavation

    3. Disruption to the embankment and foundation

    4. Need for unwatering and dewatering

5. Need for reservoir restrictions or need to schedule the work during the normal reservoir filling and drawdown schedule to facilitate construction

6. Number of access points needed

- *Alternatives for providing access to drainpipes*.—The selection of an alternative should be based upon evaluation factors listed under *Constructability* above. Possible alternatives include:

  1. *Construct or expose embankment drain outfall*.—Many structures have drainpipes with buried outfalls. If the embankment drain is located as a result of exploration, constructing an outfall would be considered the minimum access necessary to provide monitoring capability. This may be appropriate in cases where little or no history of flow is apparent in the pipe or surrounding area.

  2. *Construct access at junction of drain and outfall*.—Construction of an access point at the juncture of the outfall and drain can be accomplished either by casing the excavation, which may be appropriate if the junction is located well within the embankment, or by normal excavation if the junction is located near the downstream toe of the embankment. Once this access is established, additional inspection can be conducted, and intermediate access points can be located, if necessary.

  3. *Locate access at upstream terminal points of drains*.—The upstream end of the drainpipe can be utilized as an access point to the drainpipe. The advantages of constructing access at the upstream end can include:

     a. Shallower excavation

     b. Less reservoir loading at the point of excavation

     c. Once established, can be used to locate intermediate points of access

  Disadvantages include difficulty in locating the upstream end of the drainpipes. Generally, as-built drawings that accurately locate the elevation or alignment of the drainpipe do not exist. Locating the drainpipe often requires extensive exploratory excavation.

- *Other considerations*.—When implementing recommendations to provide improved access to drainpipes, the following items should be considered:

  1. *Flow and sediment monitoring*.—Access points should include provisions for measuring flow and monitoring sediment movement through the system.

The access points should include sediment traps and flow measurement devices.

2. *Material sampling.*—Collect and analyze samples of surrounding embankment, foundation, and drain envelope material when installing new access points to assess their erodibility and determine if filter criteria are met.

3. *Personnel safety.*—The access points should include provisions for safe entrance and egress for personnel to measure flow and sediment accumulation.

4. *Configuration.*—The design of improved access must take into consideration the size requirements needed to accommodate use of CCTV inspection equipment.

4.3.5 Abandonment/grouting of the drain system

Abandonment may be an appropriate alternative in cases where the drainage is not considered a critical feature in the performance of the dam, where historic flows have been small or nonexistent, and where the results of the examination reveal damage or failure of the drain system that could lead to a future "incident," and abandonment cannot cause harm.

Abandonment would likely be most appropriate in those cases where there is not a likely failure mode that would lead to failure of the embankment. Rather, this alternative could be selected to prevent development of an "incident," such as development of a depression over the alignment of the drain, and may also be an appropriate alternative when replacing a drain system.

When making the decision to abandon or grout the drain, the designer should consider temporary measures to evaluate the impact of plugging the drainpipe. The designer should assess all sections of the drain to make sure that plugging would not cause detrimental pressures to rise. One alternative would be installation of a packer to temporarily plug one or more sections of the drainpipe. This would allow evaluation of changes in seepage conditions prior to implementing permanent measures to plug the drains. An adequate length of time should be allowed for any changes in seepage to be monitored.

Options for plugging the drain system include filling the drain and outfall pipes with sand or grouting the drains. The sand alternative would have the advantage of being a less permanent measure, in that the sand could be jetted from the drain if changing conditions warrant such an action. However, grout may be easier to place and assure complete filling of the drain. The existing conditions within the drainpipe, as observed with CCTV inspection, may govern the alternative selected.

Chapter 5

Construction Guidance

This chapter describes appropriate construction practices for the placement of plastic pipe that functions as an outlet works, spillway, siphon, or drainpipe in an embankment dam. Good construction practice is critical in accommodating the flexible nature of plastic pipe to avoid creating inherent deficiencies that would result in deformation or failure. Key construction issues are foundation preparation, the placement and compaction of earthfill, the selection of filter and drainage media, and the placement of filter and drainage systems.

For guidance on transportation, handling, and storage of plastic pipe, the reader should consult the manufacturer's recommendations and guidance provided in publications such as AWWA's *PE Pipe—Design and Installation* (2006) and *PVC Pipe—Design and Installation* (2002), and the PPI's *Handbook of Polyethylene PIPE* (2006), AWWA's, and Uni-Bell PVC Pipe Association's *Handbook of PVC Pipe Design and Construction* (2001).

5.1 Embankment Conduits

If conduits are located on foundations that are not uniform or homogenous, differential settlement can lead to problems within the conduit. If foundations consist of low strength or highly compressible materials, unacceptable deformations and lateral movements can damage the conduit. 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. Plastic pipe that is used in the construction of new significant and high hazard potential embankment dams should be encased in properly shaped reinforced cast-in-place concrete to ensure quality compaction of earthfill against the conduit. Plastic pipe used in low hazard potential embankment dams is often not encased in reinforced cast-in-place concrete. However, use of a filter zone surrounding the conduit is a valuable defensive design measure, even for low hazard potential classification sites with favorable conditions. Some designs for low hazard potential embankment dams may not employ a filter zone around the conduit, but eliminating this valuable feature should be carefully considered and justified. Filter diaphragms should only be eliminated when extremely favorable soil conditions, good conduit construction materials and methods, reliable construction practices, and favorable foundation conditions exist. For detailed construction guidance involving conduits, see FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). Discussions within that reference include:

- Understanding the importance of excavation and foundation preparation for the installation of conduits. Special attention is needed for any excavations made transverse to the centerline or axis of the dam where the excavation backfill may be different in compressibility than the adjacent foundation materials.

- How the settlement of the dam near the conduit can create a hydraulic fracture mechanism.

- Recommendations for proper backfilling of embankment materials against the conduit. Problematic soils such as broadly graded soils and dispersive clays are defined, and the potential problems associated with them are included.

- 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 dam.

- The type and configuration of the filter zone depends 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.

5.2 Drainpipes and Filters

Construction of drainage systems consisting of drainpipes and filters is critical to the successful performance of embankment dams. Incorrect drainpipe and filter construction techniques can lead to contractor claims and unexpected performance during first filling, which can lead to embankment dam failure.

Corrugated plastic pipe is supplied in coils and straight tubes, depending on the pipe's wall thickness and stiffness. Manufacturers' recommendations should be followed for installation of plastic pipe. While axial bending or "snaking" may be permissible for less stiff products to accommodate directional changes in the alignment, it should not be allowed for products distributed as tubes. Some small changes in direction may be allowable for specially designed gasketed joints in PVC pipe, but this should be minimized to avoid the potential for joint leakage. Changes in directional alignment for tube products should always be accomplished using prefabricated fittings (figure 66), such as elbows or sweeping bends. See NRCS's

Structural Design of Flexible Conduits, chapter 52 (2005) or Uni-Bell's *Handbook of PVC Pipe* (2001), for additional guidance on longitudinal bending of plastic pipe.

Figure 66.—Prefabricated 22.5-degree bend for profile wall corrugated HDPE pipe.

At some sites, rodents, snakes, amphibians, and other animals may take up residence in drainpipes and other small outlets. This can be problematic if nests or blockages are built by the animals. To prevent animal entry into drainpipes, screens, bars, or flap gates can be installed at the downstream end. Caution should be used when installing screens, since they can become clogged with sediment or algae growth. Typically, clogging by algae growth is a function of the screen opening size where smaller openings lead to a greater chance of clogging. Additionally, the design of the end protection should allow easy access by CCTV inspection equipment.

The following sections will address critical construction techniques for both drainpipe placement and earthwork associated with the filter zones. Discussion is also presented that addresses issues associated with filter processing and handling prior to placement.

The Ganado Dam case history in appendix B illustrates the importance of proper drainpipe installation.

5.2.1 Foundation preparation

While foundation preparation is important for drainpipe installations, it is not as critical as it is for embankment conduits. The major issue related to foundation preparation for drainpipes is the same as for conduits (e.g., settlement). The likelihood for differential settlement is greater than for uniform settlement due to the heterogeneous nature of most foundations. This differential settlement can lead to sags and joint separation, even in well constructed pipe laid to the correct grade during installation. For a detailed description of foundation preparation, see chapter 5 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). Discussions in that reference, and variance, include:

- Proof rolling is recommended on soft foundations to help limit differential settlement and provide a uniform grade to lay the pipe on. Any offset in grade produced by the proof rolling can be corrected during the placement of the filter material.

- Rock foundations can be excavated by ripping or blasting. If blasting is used, care should be taken not to damage the foundation by incorrect blasting techniques. Line drilling is the preferred excavation method in this instance. For drainpipe installations, cleaning and backfilling of joints and fractures is not required. The amount of cleanup required in rock foundations for drainpipe is limited to sufficient effort to produce a foundation that is readily mapped for foundation acceptance requirements. Removal of sharp edges and other rough surfaces is not required since these irregularities will be covered with filter material.

- Rock foundations in material that is subject to slaking should be cleaned of slaked material no more than 24 hours prior to filter placement. Protective slabs (mud slabs) are not required for drainpipe installations.

- Soil foundations should be free of 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.

- Water control and removal are critical for both soil and rock foundations. As a minimum, the foundation should be free of water to enable the foundation mapping and acceptance to be performed. Placement of filter materials should not be made through standing water. Mud or other saturated soil should be removed prior to filter placement.

5.2.2 Placement around drainpipes

The following steps describe a common method for installation of drainpipe in a trapezoidal trench. Figure 67 shows a drainpipe being properly installed. Other methods have also been successfully used, but are not discussed in this manual. This installation method is for a two stage filter/drain system:

1. Excavate the trench as shown in figure 68.

2. Place the filter material on the trench invert and side slopes to the specified thickness and compact as shown in figure 69.

3. Place the drain material to a thickness of at least the crown of the pipe. In some instances, the entire thickness of drain material is placed as shown in figure 70.

Figure 67.—Trapezoidal side slopes used in drainpipe construction.

4. Excavate a small trench in the drain material to the invert elevation of the pipe as shown in figure 71.

5. Install the plastic pipe into the small trench, being careful to prevent debris or zone material from entering the pipe as shown in figure 72 (the use of temporary pipe caps is recommended). In inclement weather or other unsuitable situations, positive measures must be placed at the edges of the drain system to ensure that the materials are not contaminated. These may include for example, earthen berms, straw bales, and silt fences.

6. Place drain material around the pipe at select locations (approximately on 5-foot centers) as shown in figure 73. This material acts as an anchor so the pipe stays in place during the main backfilling. As an alternative, steel rods can be driven on either side of the pipe to anchor it. The drainpipe must be constantly monitored to ensure that its alignment remains as specified.

7. Place the remaining drain material over the pipe to the specified elevation as shown in figure 74, as this placement is made, ensure that drain material is placed in the haunch by hand labor using hand tools leaving no large voids or loose material. The drainpipe must be constantly monitored to ensure that its alignment remains as specified.

8. Place the remaining filter material to the specified grade with compaction as shown in figure 75. Note: Minimal compaction is needed for the preceding drain material placement. For guidance in compaction, see section 5.2.4.

9. Place the final miscellaneous fill or protective cap to the specified grade as shown on figure 76.

Figures 77, 78, and 79 show examples of filter and drain material placement around a drainpipe.

A number of poor practices are commonly encountered in pipe installation and should be avoided. These practices typically result in crown collapse or, in the worst case scenario, complete crushing of the pipe. They include, but are not limited to:

- Compaction of backfill using the backhoe bucket by "thumping" or setting the bucket on the backfill and lifting the end of the backhoe using the bucket.

- Wheel rolling either parallel or transverse to the pipe by any kind of construction equipment or vehicle.

- Haul roads or equipment crossing the pipe without sufficient cover. A minimum depth of 2 to 4 feet should be provided over the top of the pipe for H-20 highway truck loading (front axle load of 8,000 pounds and rear axle load of 52,000 pounds) in accordance with AASHTO standards (more depth may be required if recommended by the manufacturer). Construction equipment that exerts a loading on the top of the pipe larger than H-20 requires special consideration, and the contractor and dam owner should closely evaluate the proposed crossing method. See section 2.3 for additional guidance concerning loading from construction equipment.

- Not placing or fully compacting material under haunches of the pipe.

Placing material around plastic drainpipes can lead to damage and poor drainage when installed incorrectly. Manufacturers' literature typically describes how to install pipe in vertically sided trenches (often using a trench box such as the "doghouse" shown in figure 80). However, vertically sided trenches should not be used for drainpipe construction in significant and high hazard potential dams; a trapezoidal section is preferred. Low hazard potential dams often utilize vertically sided trenches. Pipe installed in vertical trenches encounters arching that occurs in the fill above the pipe. This arching reduces load not only on the pipe, but also in the adjacent and overlying fill. Since arching is less likely to develop in a trapezoidal trench and load on the pipe is greater, pipe installation and backfill are critical in order to offer haunch support to the pipe so its maximum strength can be developed.

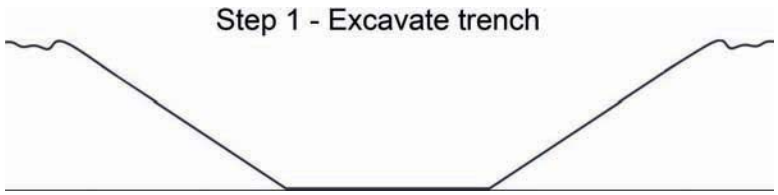


Figure 68.—Trench excavation.

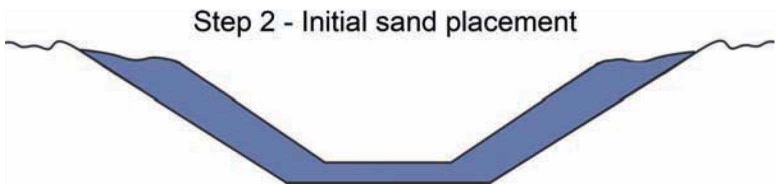


Figure 69.—Initial filter placement.

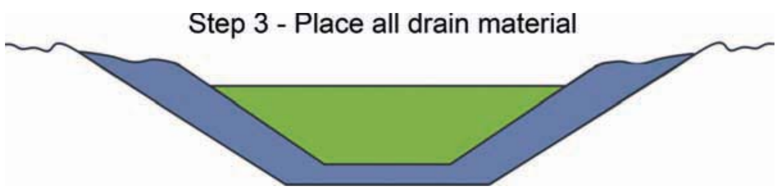


Figure 70.—Drain material placement.

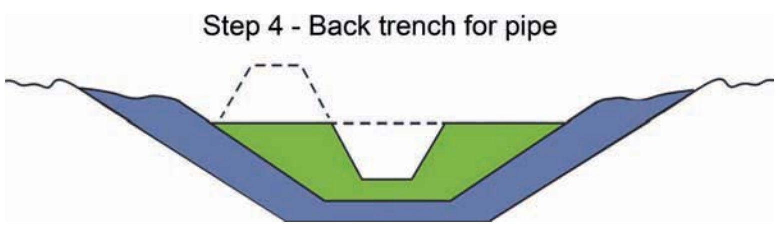


Figure 71.—Excavate for pipe.

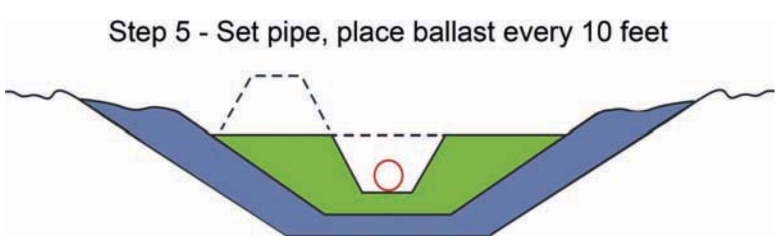


Figure 72.—Set pipe, place ballast.

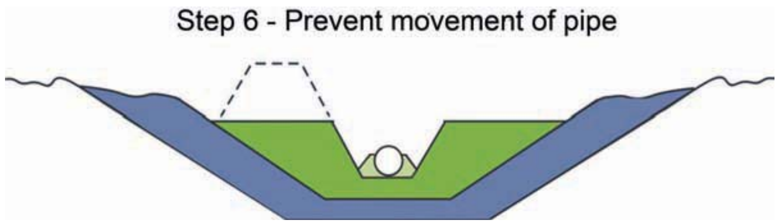


Figure 73.—Backfill haunch by hand.

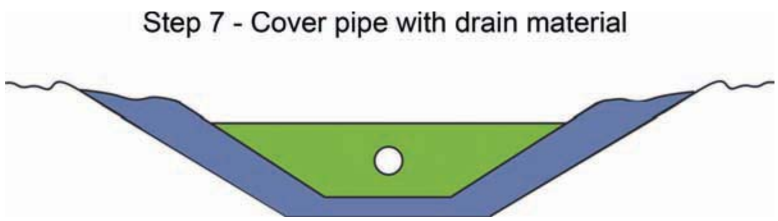


Figure 74.—Place remaining drain material.

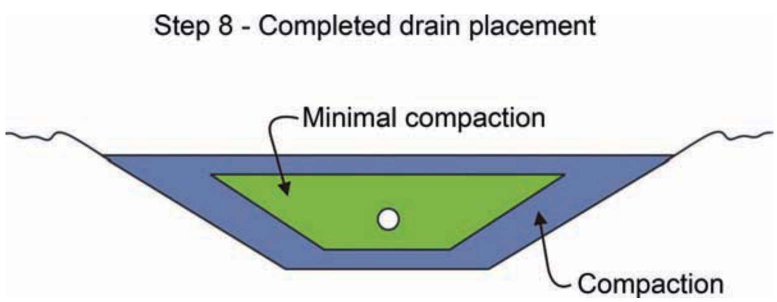


Figure 75.—Place remaining filter.

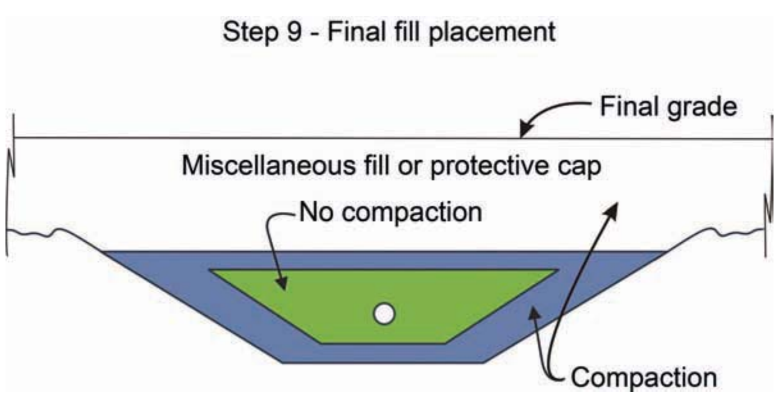


Figure 76.—Place the final miscellaneous fill or protective cap to the specified level.

Figure 77.—Initial filter placement in a trapezoidal trench.

Figure 78.—Drain material being placed over a drainpipe.

Figure 79.—Filter material being placed over drain material.

Figure 80.—A trench box or "doghouse" has been used to place material around drainpipes in vertically sided trenches in low hazard potential dams. Vertically sided trenches should not be used in significant and high hazard potential dam construction.

5.2.3 Segregation

Segregation during processing and placement is a common problem. Segregation may result in overly coarse filter/drain materials in contact with adjacent finer materials, which negates the effect of the filter. Incompatibility at the interface materials is the result. Many designers consider that segregation control during construction is the most important aspect of constructing a filter/drain. Segregation can have a significant bearing on the ultimate performance of the embankment dam. Figure 81 shows naturally occurring segregation.

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 sides of stockpiles or piles within the hauling unit. Material dumped from a truck, front loader, or other placing equipment almost always segregates, 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 satisfactorily controlled in several ways. First, the designer should specify a uniformly graded filter or drain. Secondly, construction techniques to control segregation should be specified and enforced. Use of rock ladders, spreader boxes, and "elephant trunks" for loading hauling units, and hand working the placed materials help prevent segregation. If material is dumped, limiting the height of drop helps. Placing filter/drain material with belly dumps sometimes adequately limits segregation during placement. Limiting the width of the belly dump opening by chaining or other means can 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 apprised of the importance of limiting segregation.

5.2.4 Compaction methods for backfill and filter and drain materials around drainpipes

Compaction of filter and drain materials should be adequate to produce sufficient density to preclude liquefaction, limit consolidation, and provide adequate strength. However, excess compactive effort can cause particle breakdown and reduce permeability. Therefore, the amount of compactive effort should be limited to that required to produce the required strength and consolidation parameters, yet not cause excess particle breakage and unnecessarily high densities which both reduce permeability. Thought should be given to the number of passes specified instead of just using what has been used previously. If two passes will get the required density, then four passes are not justified because they will reduce permeability by causing more particle breakdown and increased density. Also, the roller operator should be made aware that it is undesirable to continue to roll after the required passes have

Figure 81.—Naturally occurring segregated soil. During deposition, the gravel sizes were segregated from the sand sizes. Poor construction practices can lead to similar segregation. The deposit is a broadly graded mixture of silt, sand, and gravel and is internally unstable. Note that silt has eroded into the gravel sizes and coated the particles.

been made. The idea often exists that, if two passes are necessary, three are better. This may not be the case, and the contractor and his operators should be aware so that additional passes are not made to ensure no failing densities or to fill in operator slack periods. When the specified density is not achieved during construction, typically the cause is insufficient water content in the fill (dry of optimum). A contractor may be resistant to applying water to the fill and instead will prefer to make many passes in an effort to achieve density. This will lead to material breakdown and the required density will still not be achieved. Vigilance should be exercised in assuring the contractor has the fill at sufficient water content prior to compaction (figure 82). For a pervious filter this may require that the application of water immediately precede the roller and in many cases the roller literally follows the water truck.

Most current equipment used for compaction of granular material used today possess vibratory capabilities where dynamic loading is used to achieve density. In addition to equipment compaction, granular materials can also be densified by flooding (i.e., applying sufficient water in order to achieve 100 percent saturation).

The recommended minimum density ranges from 50 to 75 percent relative density (the lower value can be used for small structures in areas of low seismic activity and the higher value used for large structures or structures in higher seismic active areas). Relative density can be determined in accordance with ASTM D 4254. Whenever grain-size limits for filters/drains are specified, the grain-size tests should be made on materials compacted to simulate as closely as possible the grain sizes and soil structure after particle breakdown caused by construction. Ring permeability tests

Figure 82.—Water being added to filter immediately preceding compaction. Note that this placement is occurring during a heavy rain, and wetting of the filter must continue.

made at various levels in test fills are one way to obtain realistic permeabilities representing vertical permeabilities of compacted filters and drains. Laboratory procedures that closely duplicate field placement and compaction methods can also provide reasonable values for levels of permeability to be expected in filters and drains. If proposed materials do not have sufficient permeability after compaction, changes in grain sizes should be made that will provide the required permeability.

Also, designers should consider changes in layer thickness or geometries of drains that will increase discharge capacity to the required levels, while providing the needed filter protection.

In-place density tests should be taken to verify the required density is being met. The sand cone density test, such as ASTM D 1556, will meet this need. Nuclear testing (ASTM D 2922) can also be used, although it may underestimate density in sand filters. This underestimation can lead to lift rejection and direction to the contractor to perform additional compaction. This additional compaction can lead to additional breakdown of the material, increasing its fines content. Opinions vary on the efficacy of these two test methods. The reader is directed to chapter 6 of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) for a more detailed explanation of density testing methods and their shortcomings.

5.2.5 Borrow sources

In general there are two potential sources for filter/drain material: undeveloped sources and existing commercial sources. For small dams it may be cost effective to use commercial sources and for larger projects, more economical to develop a new source specifically for the job, if suitable undeveloped material exists near the job site. The availability and suitability of material must be factored into the design. For example, if suitable material is limited in quantity or expensive to obtain, it may be more economical to use thin or narrow zones (less than placement equipment width) and more intensive placement and inspection techniques to ensure construction of adequate filter/drain zones. On the other hand, if ample material is near the job site and can be economically developed, equipment width dimensions of filter/drain zones with less intensive placement and inspection techniques may be more cost effective. The designer must ensure that there is sufficient volume available to construct the work. Generally, it is prudent to have at least four times the volume of material available in borrow than is necessary to produce the final in-place volume of the filter/drain zones. For large jobs a sieve by sieve analysis should be made in order to find out which grain sizes are critical for a specific pit. If thinner zones are used, the dimensions must be checked for adequate hydraulic capacity, as discussed herein. Logical sources must be investigated and, for approved sources, appropriate information such as location, availability, ownership, drill logs, test pit logs, appropriate lab tests, and geotechnical considerations provided in the specifications. Figure 83 shows an example of a typical borrow area and processing plant.

5.2.6 Contamination

To avoid contamination of filter/drain zones with excess fines during construction, several techniques should be used. The zone should be maintained higher than the surrounding fill surface, and the fill should be placed to maintain drainage of surface water (and sediments) away from the filter/drain zones. This will prevent the flow of muddy water into the filter or drain. Traffic should be well controlled, with crossings limited to prepared roadways which will be removed entirely prior to placing of additional filter/drain materials (figures 84 and 85). Crossings should be

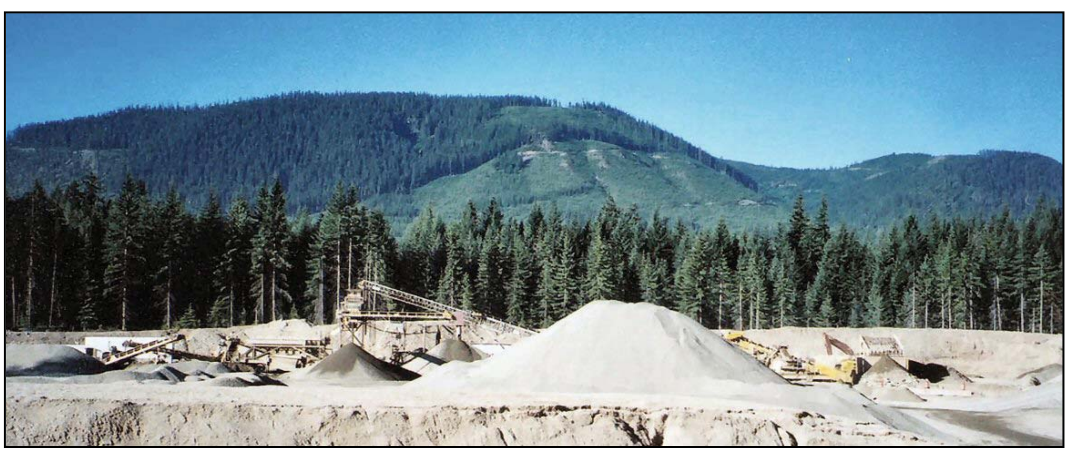


Figure 83.—Typical borrow area including processing plant. Produced material is in foreground and the plant is in the background.

Figure 84.—Roadway crossing over a filter. Photo courtesy of ASDSO (Hammer, 2003).

Figure 85.—Contaminated materials being excavated beneath a roadway crossing over a filter. Photo courtesy of ASDSO (Hammer, 2003).

staggered to remove any possibility of vertical transmissibility of the filter/drain zone being reduced. Durable materials should be specified, and compactive effort held to the minimum needed to obtain desired in-place density, to minimize particle breakdown during placement and compaction. Equipment for placement and compaction of filter/drain zones should be restricted to operation only on the filter/drain zones, or cleaned before moving onto the filter, to avoid unnecessary internal contamination. Commonly, equipment operators (spreading and compacting) will want to move off of the filter/drain zone when their operation is done for a particular lift. This will lead to cross contamination between zones and should be avoided. Operators should be instructed "once on the filter, stay on the filter." In cold climates, construction seasons are often short. When the construction season is terminated, the surface of the filter/drain zones should be covered (in addition to surface drainage requirements) and the covering material removed completely before the resumption of placement in the subsequent season.

Contamination can also occur during loading, hauling, placing, and compaction because these processes tend to cause breakdown of the materials, sometimes to the extent of causing the gradation to be out of specification requirements. Specifically, trucks used to haul high fines content material may end up with that material stuck in corners of the truck box. When these trucks are then used to haul filter or drain material, this material can dislodge and end up in the filter. Truck boxes should be clean of such material before hauling filter or drain materials. Contamination can occur in the stockpile. Dust abatement control procedures and use of equipment around the stockpile that is maintained in a clean condition will reduce this problem. Reprocessing or not using the bottom foot or so of the stockpile may be necessary, since this is where the greatest contamination of the stockpile generally occurs. Generally, the concern is for an increase in the fines content, that is, material finer than a U.S. Standard, No. 200 sieve, because these fine materials can drastically reduce the filter permeability. However, breakdown of any particle size can be detrimental because this may alter the ability to filter or be filtered.

Generally, the percent fines after compaction should not exceed 5 percent to ensure that permeability is not decreased to an unacceptable degree when tested in accordance with ASTM C 117 and C 136. To achieve this, the material has to contain less than about 2 or 3 percent fines in the stockpile, depending on the durability of the particles. Durability requirements equal to those used for concrete aggregate as described in ASTM C 33 Class Designation 1N, are preferred, and will usually ensure that the material can withstand necessary processes to get them in place and compacted without excessive breakdown, and will also help ensure long-term durability during operation. Making the specifications requirement for filter material gradation in place after compaction is necessary. In some cases, such as in the case when material is preprocessed in a prior contract, after-compaction requirements are not desirable. In these cases, specifying clean material (less than 2 or 3 percent fines in the stockpile) and adequate durability becomes even more important as well as thorough inspection to ensure the materials are not mishandled or over rolled.

5.2.7 Quality control and assurance

Individuals responsible for quality control and assurance should be experienced professional engineers with at least 2 years experience with design and construction of the type of drainpipe system being employed. Quality control and assurance should include the following:

For the subgrade:

- *Equipment.*—Visually inspect and verify soil processing, placement, and compaction equipment meet the requirements described in the specifications.

- *Weather conditions.*—Verify that soil placement, grading, or compaction does not occur during periods of freezing temperatures, if it is raining excessively, or if other detrimental weather conditions exist.

- *Subgrade preparation execution.* —

    1. *Subgrade preparation.*—(1) Ensure the elevation of the top surface of the subgrade is correct. (2) Verify the subgrade is smooth, free of voids, and composed of satisfactory materials. Also, verify the subgrade is compacted as specified. (3) Standard moisture and density tests are taken at the same location as the rapid tests so that results can be easily compared. Ensure that large equipment is turned off in the vicinity where sand cone tests are being performed.

    2. *Subgrade protection.*—(1) Ensure the contractor removes puddles and excess moisture from the soil surface prior to placement of additional soil, bedding, filter, or drain rock. (2) Look for areas of erosion after each rainfall. (3) Inspect for damage due to freezing and/or desiccation. (4) Ensure the contractor repairs damaged areas and reestablishes grades.

    3. *Subgrade repairs.*—If the subgrade does not conform to the specifications, the designer should assist in defining the extent of the area requiring repair. This should be done through the use of additional testing and visual inspection. Material from areas that are to be repaired should be removed and replaced. After repairs have been made, ensure retests are performed to check the repaired areas. In general, tests should be performed at the same frequency as the rest of the project. Additional testing should be performed in suspect areas.

For the bedding, filter, and drain materials:

- *Equipment.*—Verify equipment used to place and compact the materials are in accordance with the specifications and the pipe manufacturer's recommendations.

- *Delivery, storage, and handling.*—The inspector shall be present during delivery, unloading, and stockpiling and should verify the following:

    1. Materials have not been segregated or mixed with deleterious materials during shipping, storage, and handling.

    2. Deliveries are properly recorded.

    3. The correct material type and gradation have been delivered.

    4. The materials are stockpiled with proper protection and handling.

    5. Materials that have been contaminated are rejected before placement.

- *Weather conditions.*—Verify weather conditions are acceptable for material placement.

- *Material properties.*—Verify that material is sampled and tested in accordance with the specifications and test results not meeting the requirements specified result in the rejection of applicable materials.

- *Installation execution.*—

    1. Oversize and deleterious material which could damage the performance of the system has been removed prior to placement.

    2. Drainpipes are not being damaged or moved out of alignment by placement equipment. Placement equipment should be observed from the front side as material is being spread over the plastic pipes.

    3. Excessive fines have not been generated as a result of handling and placement of the drainage materials.

    4. Wind-borne and water-borne fines do not contaminate the drainage system after placement.

    5. Erosion controls are placed such that drainage systems are not contaminated by fines.

6. Watch for ponds of water on top of the drainage system which may be an indication that an excessive amount of fines have contaminated the drainage materials.

For the drainpipe:

- *Equipment.*—Verify equipment used to place and cover pipe is in accordance with the specifications and the manufacturer's recommendations.

- *Delivery, storage, and handling.*—The inspector should be present during delivery and unloading and should verify the following:

    1. Pipe and appurtenances are not damaged during shipping, storage, and handling.

    2. Deliveries are properly recorded.

    3. The correct material type, strength, and pipe sizes have been delivered.

    4. The size, number and location of pipe perforations are as specified.

    5. Pipes with gouges deeper than 10 percent of the wall thickness are rejected or repaired before use.

    6. Out-of-round pipe which cannot be properly joined together is rejected.

- *Weather conditions.*—Verify weather conditions are acceptable for pipe placement.

- *Material properties.*—Verify that pipe is sampled and tested in accordance with the approved manufacturer's quality control manual and test results not meeting the requirements specified results in the rejection of applicable pipe.

- *Installation execution.*—

    1. *Pipe.*—Verify the following during pipe placement:

        a. Pipe is carefully carried or pulled to the place of installation.

        b. Defective or damaged pipe is not used.

        c. Pipe is not laid when trench conditions or weather is unsuitable.

        d. Pipe is not installed if standing or flowing water is present.

        e. Pipe and accessories are carefully lowered into the trench.

f. Pipe is placed at the lines and grades indicated in the plans and specifications. Verify the contractor does not lay pipe on blocks to produce the specified grade.

g. Specified bedding is used and the bedding is graded to provide proper support of the pipe.

h. The full length of each section of pipe rests solidly upon the pipe bedding layer with recesses excavated to accommodate couplings and joints.

i. Compaction requirements are being met for bedding layers and haunch areas located around the pipe.

j. Continually monitor the pipe for alignment and shape deformation during placement of "fill materials." Correct immediately any problems identified.

k. Partially perforated pipe is installed with the perforations facing down unless otherwise specified.

l. Pipe and fittings are free of dirt, oil, or other contaminants.

m. The interior of pipe and accessories are thoroughly cleaned of foreign matter before being lowered into the trench.

n. Pinch bars and tongs for aligning or turning pipe are used only on the bare ends of pipe.

o. Bell and spigot connections are seated properly with no foreign material introduced into the connection.

p. If piping is butt fused, the fusion is allowed to set for the required cure time and within the recommended temperature range.

q. All required leak tests are performed successfully prior to backfilling.

r. When work is not in progress, open ends of pipes, fittings, and valves are securely plugged or capped so that no trench water, earth or other substance enters the pipe and fittings.

s. The entire length of the drainage system is CCTV inspected initially when about 3 to 5 feet of fill is placed over the pipe and again prior to cleaning and completion.

Chapter 6

Inspection

Periodic inspection of the condition of plastic pipe is essential in detecting problems and evaluating its long-term safety and reliability. Periodic inspection may reveal trends that indicate more serious problems are developing. However, plastic pipe used in embankment conduits and drainpipes is often not inspected as part of an overall inspection of the embankment dam and appurtenant features. Generally, structural defects and deterioration develop progressively over time. A trained and experienced inspector can identify defects and potential problems before existing conditions in the dam and conduit become serious. On occasion, situations can arise suddenly that cause serious damage in a short period of time. Chapter 7 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) provides guidance concerning modes of failure involving embankment conduits. This chapter will address the inspection of plastic pipe used in embankment conduits and drainpipes.

If changes are made in the field during construction and not accurately recorded, confusion may result during the inspection. Once plastic pipe is buried, it is difficult to find, making it difficult to service the pipe and more likely that unintentional damage will result from nearby digging. Accurate as-built construction drawings will facilitate the inspection process. Plastic pipe cannot be located using common metal pipe detection systems. Acoustical methods have been problematic when used on plastic pipes.

6.1 Embankment Conduits

For detailed guidance involving the inspection of embankment conduits, the reader is directed to chapter 9 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). Discussions within this reference include:

- *Preparing for and performing an inspection.*—Good planning and preparation will ensure the successful outcome of an inspection. The inspection team should know what to look for and evaluate as the inspection progresses. The inspection team must keep proper documentation using written records and photographs. This documentation provides valuable information on changing conditions that could indicate a serious problem is developing. Confined space precautions and proper ventilation must be considered for any man-entry in an embankment conduit.

- *Specialized inspections*.—Specialized inspection includes the use of a dive team, climbing team, remotely operated vehicle (ROV), or closed circuit television. Specialized inspection is required for embankment conduits that are inaccessible for man-entry.

Figure 86.—A butt fused, solid walled HDPE pipe joint in an outlet works slipliner as viewed using CCTV inspection equipment.

Figures 86 and 87 show examples of CCTV inspection of an HDPE sliplined outlet works conduit. Figure 88 shows CCTV inspection of a white HDPE pipe. Manufacturers are moving away from white HDPE pipe and are using gray or black pipe to provide better viewing of the interior of the pipe. Inspections have found that white reflected too much light, and gray or black provides a better picture.

Figure 87.—Looking upstream toward the control gate in an HDPE sliplined outlet works conduit using CCTV inspection equipment.

Figure 88.—CCTV inspection of a newly installed slipliner for an outlet works renovation.

6.2 Drainpipes

Before the 1990's, most drainpipes were too small in diameter to allow for adequate inspection and typically were not inspected. However, the use of CCTV equipment has allowed for inspection of many previously inaccessible drainpipes. Unfortunately, in many existing dams, unless new access is provided, the drainpipes will likely remain uninspected, since older designs often have excessively long reaches of drainpipe or sharp bends. New drainpipe installations should always be designed to accommodate CCTV inspection equipment. The designer needs to consider the proper locations for inspection wells and cleanouts (figures 89 and 90). The reader is directed to chapter 9 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) for a complete discussion of CCTV inspection.

The Bureau of Reclamation conducted a CCTV equipment performance study (Bureau of Reclamation, 2004) in order to assist their designers with the proper design of drainpipes to better accommodate CCTV equipment. This study evaluated the influence of drainpipe diameters, bends, invert slopes, and invert conditions on CCTV inspection equipment. The study was performed using varying configurations of profile wall corrugated HDPE drainpipe (figure 91).

The study was based on the assumption that a camera-crawler would travel up the pipe from a downstream location. Drainpipe designs that provide an upstream (upslope) access location from which the camera-crawler can enter allow for

Figure 89.—Inspection well provides access to the drainpipe for CCTV inspection.

Figure 90.—Cleanout provides access to the drainpipe for CCTV inspection.

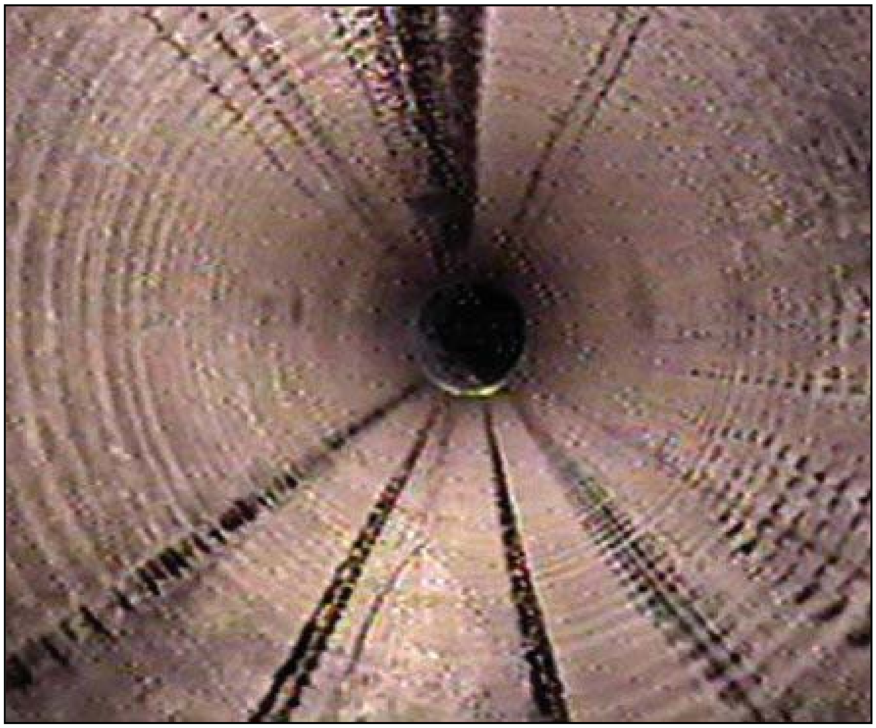


Figure 91.—Profile wall corrugated HDPE pipe has corrugated exterior and smooth interior surfaces.

improved cable tether pulling capacity, since the camera-crawler can move more easily downward (downslope) on a sloping decline. Sloping declines generally do not result in camera-crawler traction issues. For the camera-crawler backout process, the transport vehicle had a free-wheeling clutch mechanism on the track unit that allowed for high speed retrieval either manually or by a cable take up reel. Although not tested in this research program, an upstream access location would also benefit camera-crawler navigation around pipe bends and allow for the use of steeper invert slopes because the effect of cable drag would be lessened. Providing upstream access locations would be especially important where steeper invert slopes may be required, such as on abutments. The following summarizes the conclusions from the Bureau of Reclamation's study and provides recommendations concerning the layout of drainpipe systems to accommodate inspection using CCTV equipment:

- *Pipe diameters.*—The minimum recommended pipe diameter to successfully accommodate CCTV equipment is 8 inches. Although camera-crawlers are available for pipes smaller than 8 inches, they are very limited in cable tether pulling capacity and generally do not have sufficient traction for use in drainpipe inspection. In addition, the cameras typically only have a fixed lens, and the transport vehicle is not steerable. Camera-crawlers used in pipes with diameters between 8 and 12 inches generally have cameras with some pan, tilt, and zoom capabilities but generally are not steerable. Camera-crawlers used in pipes with diameters of 15 inches or larger are steerable, have a greater cable tether pulling capacity, and have cameras that can provide a wider array of optical capabilities, including pan, tilt, and zoom. Where practical, the use of pipes with diameters 15 inches or larger is strongly encouraged. This allows for the use of more powerful and versatile camera-crawlers. The selection of larger pipe diameters allows for some accommodation of sediment accumulation on the pipe invert. Larger diameters also increase the likelihood of camera-crawlers getting past many types of obstructions that may exist in the pipe.

- *Pipe bends.*—The maximum recommended horizontal bend angle to successfully accommodate CCTV equipment is 22.5 degrees. In pipes with diameters of 8 and 10 inches, some camera-crawlers encounter difficulties navigating bends of 45 degrees or greater because the camera cannot clear the pipe crown as it travels through the bend, and drag friction on the tether cable reduces pulling capacity. Sweeping bends should always be used to facilitate camera-crawler navigation. For best practice in pipes of all diameters, a series of 22.5-degree bends is recommended. Each 22.5-degree bend should be connected to a minimum 5-foot length of pipe to allow the camera-crawler to navigate around the sweeping bend and provide adequate crown clearance.

- *Invert slope inclination.*—The maximum recommended invert slope inclination to successfully accommodate CCTV equipment is 5 degrees. The difference in invert slope inclination between flat and 10 degrees can reduce cable tether pulling capacity by as much as 70 percent depending upon the pipe diameter, degree of pipe bend, and the invert condition. Flat to 5-degree invert slopes would appear to be the most reasonable inclination. Slopes with inclinations greater than 10 degrees are not recommended, due to the significant loss of traction that occurs when camera-crawlers are pulling long cable tethers. If slopes greater than 5 degrees are required, upstream access locations should be provided within the pipe.

- *Distance between manholes or access entry locations (cleanouts).*—The maximum distance between manholes or access entry locations should be between 500 and 2,000 feet, but depends highly upon the pipe diameter, bends, invert slopes, and invert conditions. The designer needs to take these limitations into account when selecting the appropriate distance between manholes or access entry locations. In pipes with diameters of 8, 10, and 12 inches, the maximum distance should not exceed about 1,000 feet. This assumes that access is available on both ends of the pipe. If access will only be available on the downstream end of the pipe, then the maximum distance should be limited to about 500 feet. In pipes with diameters of 15 and 18 inches, the maximum distance should not exceed about 2,000 feet. This assumes that access is available on both ends of the pipe. If access will only be available on the downstream end of the pipe, then the maximum distance should be limited to about 1,000 feet.

The results of this study are also considered applicable for pipes constructed with PVC.

The primary cause of pipe failure is not always known. Reasons for failure could be singular or a combination of events. Failures observed are not indicative of all plastic pipe installations, but do assist in the understanding of drain reliability issues. The Bureau of Reclamation has been performing CCTV inspection of drainpipes as part of their dam safety program since about 1999. In performing these inspections a database has been developed to track the problems found within various types of drainpipe materials (Cooper, 2005). The results of these inspections show that early installations of single wall corrugated HDPE drainpipe experienced dimpling shape deformation, and/or failure in about half of all drainpipes inspected. Shape deformation ranged from minor to extensive. Figure 92 shows a single wall corrugated HDPE drainpipe experiencing buckling. Joint offsets and separations were observed in about 10 percent of all HDPE drainpipes inspected. Joint offsets and separations ranged from minor to extensive. Figure 93 shows a single wall corrugated HDPE drainpipe joint that has experienced an extensive separation and has allowed materials surrounding the drainpipe to enter through the separated joint.

HDPE is not the only type of plastic pipe that has experienced problems. Figures 94 and 95 show examples of PVC drainpipe that have failed. These pipes were damaged during construction.

The Davis Creek Dam case history in appendix B illustrates how CCTV can be utilized to inspect drainpipes.

Figure 92.—Single wall corrugated HDPE drainpipe experiencing failure due to buckling.

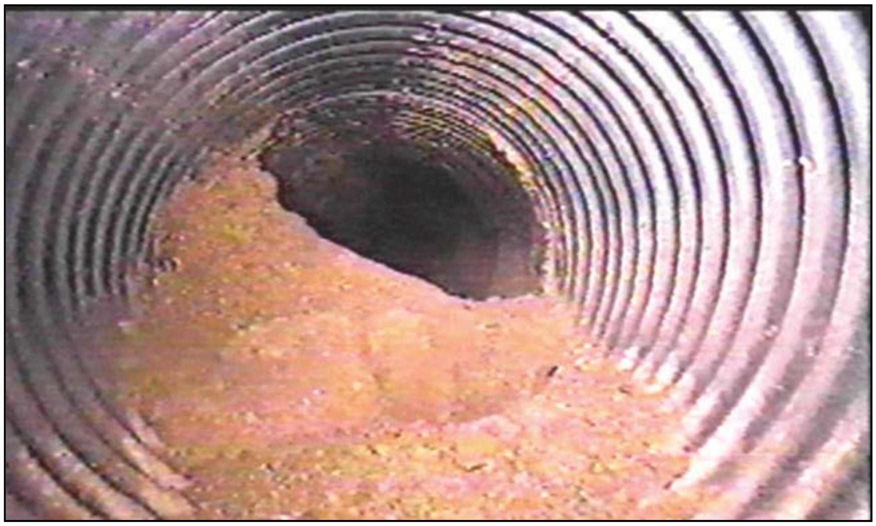


Figure 93.—This HDPE drainpipe has experienced joint separation allowing backfill materials to enter the drainpipe.

While the nature of plastic pipe minimizes the likelihood of plugging mechanisms (i.e., soluble encrustants, biofouling, etc.) developing within the pipe, they can still occur. For instance, calcite precipitates out of solution and forms deposits where ion concentrations in the seepage increase to the point where it exceeds the solutioning capacity of the water (Bureau of Reclamation, 2004, p. 173). This can occur at slots, perforations, and joints in the pipe. Figures 96 and 97 show examples of calcium carbonate that has precipitated out of solution as the mineral, calcite. Biofouling is the result of certain life process activities of bacteria. Bacterial growth can occur anaerobically (without oxygen) or aerobically (with oxygen). Plastic pipe lacks any nutrients in its composition, but fungi may still grow upon pipe surfaces, feeding upon nutrients that may settle or be deposited on the surface by seepage and serve as a physical support for the life cycle. Such surfaces are generally not attacked and may suffer only slight surface etching (PPI, 2000, p. 2). Bacterial growths can be soft and easily removed or can become hard and mineralized. Iron bacteria form the most common bacterial deposit (figure 98). Iron bacteria are often characterized by orange, red, brown, or black slime, unpleasant odor in water, and an oil-like film on water. Other microflora can exist in drainpipes, such as sulfate-reducing bacteria, sulfur-oxidizing bacteria, heterophic bacteria, and algae. Sampling and testing may be required to assist in planning the best course of action in dealing with plugging mechanisms.

Sediments are often encountered during a CCTV inspection (figure 99). Sediments may be transported by seepage flowing within the drainpipe and could be an indication of internal erosion occurring within the dam or foundation. Sampling and petrographic examination of the sediments may be required to assist in evaluating evidence of internal erosion.

Figure 94.—Slotted PVC pipe used for toe drain has experienced longitudinal cracking. The cracking occurred during construction.

Figure 95.—PVC pipe used for toe drain has experienced transverse cracking. The cracking occurred during construction.

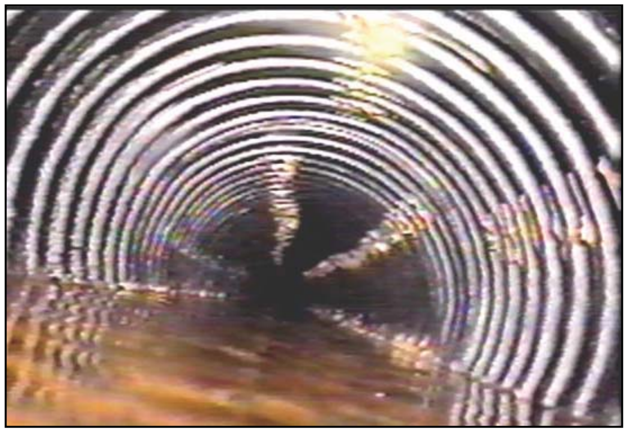


Figure 96.—Calcite deposits have blocked many of the slots in this HDPE drainpipe.

Figure 97.—Calcite deposits have formed at joints and perforations in this HDPE drainpipe.

Figure 98.—Iron bacteria have partially blocked the perforations in this HDPE drainpipe. Note that the only open perforation passing seepage is in the lower left corner of figure.

Figure 99.—Sediment deposit on the invert of a HDPE drainpipe.

HDPE drainpipe has been used in many drainpipes constructed or modified after about 1980. HDPE drainpipe, while lightweight and easily handled and installed, has experienced a significant number of shape deformation and failure instances. Many of the HDPE drainpipe failures may be related to stress cracking or improper installation of the pipe. Stress cracking is a failure mechanism which develops over time at stresses less than the yield strength. In the past, HDPE drainpipe resins have differed in the amount of SCR. Proper installation of HDPE drainpipe requires good compaction and quality control of the backfill to insure good support under the haunches. If the drainpipe is not well supported by the backfill, the drainpipe will deflect excessively and stresses will be concentrated at the crown, invert or springline. These stress concentrations can lead to premature failure. Other failures could be the result of isolated point loads from construction loading, such as equipment crossings.

The following guidelines are recommended for inspection of plastic drainpipe:

1. A preliminary CCTV inspection should be performed when 3 to 5 feet of backfill has been placed over the drainpipe. The purpose for this inspection would be to identify and repair any abnormalities, cracks, bulges, etc. early before construction is completed.

2. Another CCTV inspection should be performed when the final backfill loading over the drainpipe is completed. CCTV inspection should be performed prior to the contractor pulling the torpedo-shaped plug or pig through the drainpipe and prior to any cleaning. The purpose for this inspection would be to identify any abnormalities, cracks, bulges, etc. that may have developed since the preliminary inspection. CCTV inspection could replace the need for pulling the plug or pig through the drainpipe.

3. Subsequent periodic inspection should be performed based on the performance of the drainpipe, changes in the characteristics or quantity of the flow, or other events.

Cleaning can remove biofouling, mineral encrustation, roots, and soil deposits. Caution must be exercised in each step of the cleaning process to prevent damage. Before attempting to remove soil deposits, the engineer should consider if they are benign, like those that enter a pipe during the development of the filter (filter set), or if their removal could initiate more soil movement and make the condition worse. The fundamental guiding principle for any type of drainpipe cleaning should be "do no harm."

After a newly installed drain system has been through first filling of the reservoir, sometimes soil material is seen in the pipe invert or in a downstream sediment trap during CCTV inspection. Concern may arise that this material could be from the foundation, but it is possible it could be from the filter and drain material itself. In order to address this concern, the method of filter and drain processing and construction quality will have to be determined. During material processing three methods can be used to produce a material of the desired gradation; crushing, dry screening, and washing. Details of the process used to produce the material in question will be needed to determine the likelihood that fine grain material exists within the filter and/or drain materials. Note that it is possible to produce filter and drain materials with small amounts of fines and still meet the specification requirements.

As an example, consider a borrow area material consisting of silt, sand, and gravel that will used to produce a filter composed of sand and drain material composed of gravel. Upon entering the processing plant the material is dry screened separating the gravel from the silt and sand. The silt and sand go onto to further processing to remove the silt, typically by using washers. This is the 'wet' side of the plant. Meanwhile, the gravel continues for additional dry screening until the final drain material gradation is achieved. This is the 'dry' side of the plant. Complications can arise on this side of the plant in that fine material, perhaps with some plasticity, can adhere to the gravel particles. For this reason drain material should be visually examined during construction for fines adhesion to the gravel particles. Experience indicates that sometimes a large amount of effort is needed to remove the fines contamination. This material can then show up in the drain system as described above. While this example describes one way fine grain material can show up in a drain, others can occur also. For this reason it should be expected that some finer material will flush out of newly constructed drain systems and plans should be made from the beginning of the job to clean the system after initial reservoir filling. This cleaning should occur after the reservoir has reached its maximum normal level so the drain system is wetted to the maximum extent.

No clear guidance exists on how to tell the difference between fines that flush out of a newly constructed drain system and material that may be coming from the foundation. However, material derived from the filter and drain materials should have a uniform appearance and should occur uniformly along pipe segments which have flow. The amount of material should also be modest, at the most, no more than one cup over a 100-foot drain length. The best indication though of the source of the material is re-examination in the second year. If no material is found in year two, then it most likely was from the filter/drain materials themselves.

Cleaning of drain systems is not yet routine (figure 100). Most drains have never been cleaned, and based on their performance, cleaning may not always be justified. CCTV inspection should precede any cleaning attempt, to ensure that cleaning will not degrade existing conditions. The proper method of cleaning a drainpipe varies according to the conditions within the pipe and the structural integrity of the pipe. Commercially available water-jet cleaning is most often used that utilizes high-pressure water spray from a nozzle. The orientation of individual jets on the nozzle of the cleaner can also be varied, depending on site conditions. In some cases, low pressure/high volume flow is best suited for sediment removal and high pressure/low volume flow is best suited for root or mineral encrustation removal. Sometimes both methods may be required at a particular site. The condition of the pipe is paramount for any cleaning attempted, and this may actually govern the cleaning method selected at a given site

Typically, it is difficult to ascertain how effective the cleaning has been, due to limited instrumentation and variations in drain flows caused by factors other than the reservoir, such as infiltration from precipitation. However, follow-up CCTV inspection after cleaning that used high pressure jet washing has shown that the biofouling and mineral encrustation was generally removed from the interior surface and some improvement of discharge from the drainpipe was often observed. No determination can usually be made as to the extent of the plugging mechanism remaining in the backfill materials surrounding the exterior of the pipe.

Any cleaning system used should always be proven effective in a similar situation and on similar pipe materials. If a new cleaning system is used, it should be tested on a piece of pipe similar to the drain to be cleaned to ensure the process will not damage the pipe.

The recommended process for drain cleaning generally includes the following six steps:

1. Record all pertinent information, including measuring drain outflows, reading all piezometers and observation wells, and walking the alignment of the drain to observe the preexisting conditions.

2. Perform an initial CCTV inspection to document existing conditions.

Figure 100.—Operator water jetting a drainpipe.

3. Test the cleaning system on the first short segment of pipe.

4. In cleaning the remainder of the pipe, use care to observe the entire process, including advancement rates, effluent, etc. Steps 2 through 4 may require an iterative process to ensure that cleaning procedures are not damaging previously uninspected portions of the drainpipe.

5. Reinspect the pipe using CCTV and record all other pertinent information again. This could be completed by the contractor doing the cleaning, if they have appropriate CCTV inspection equipment.

6. Document all information for use in future cleanings, if needed, and information beneficial to an evaluation of the cleaning by others. Disseminate copies of the cleaning report to the engineer and other appropriate parties.

For guidance on improving drainpipe access for inspection and cleaning activities, see section 4.3.4. For additional guidance on cleaning, see section 9.6 in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

Chapter 7

Plastic Pipe Used in Tailings Disposal Facilities and Slurry Impoundments

The mining industry constructs dams for waste disposal, water supply, water treatment, and sediment control. Tailings dams are used for the disposal of "metal and nonmetal" mine waste or tailings. Slurry impoundments or coal waste impoundments are used for the disposal of fine waste from the processing (i.e. removing impurities) of coal. Tailings dams and slurry impoundments differ in many ways from traditional water storage embankment dams. For a discussion of the differences, see the *Introduction* of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005). This chapter will focus on the use of plastic pipes in tailings and slurry impoundments. Plastic pipe has been used since about 1980 in mining-industry dams for decant pipes, for internal-drain collector pipes, and for delivery pipes for slurry or tailings disposal. A decant pipe typically serves the functions of removing clarified water from the impoundment, controlling the normal water level, and drawing down the pool level following rainfall events.

The main reasons why plastic pipe has come into use, versus other types of pipe, in mine-waste disposal applications include its resistance to chemical attack, its capability to be constructed with watertight joints, its ease of construction, and its ability to tolerate deformation. More specifically, consider that:

- The drainage and the seepage from mine waste impoundments can cause chemical deterioration of pipe materials due to its acidity or alkalinity. Waste from the processing of materials such as coal and phosphate, for example, can be highly acidic.

- Some tailings dams are required to have impervious liners due to the acidity of the leachate. Plastic pipe has been used in these applications with a watertight seal provided by a boot at the point where the pipe penetrates the liner. In these cases, the potential for seepage along the pipe, and the development of a problem associated with such seepage is limited, provided careful attention is paid to design, construction, and monitoring. Some designers of tailings disposal facilities consider it best practice to avoid penetrations through embankments, whether the impoundment is lined or unlined, and use barge-mounted pumps to control the water level.

- When pressure-testing became more prevalent for decant pipes – such as in coal slurry impoundments – the fused joints of plastic pipe were able to meet the testing requirements.

- Designers have considered HDPE pipe to be beneficial for the type of foundation conditions and construction practices found at mining impoundments. The locations of these impoundments are limited to areas near the processing plants, meaning that designers need to deal with varied, and often times less than ideal, foundation conditions. Furthermore, decant pipes are extended as the impoundment is expanded and the pipes can become relatively long, sometimes exceeding 1,000 feet. Over such lengths, a flexible pipe can tolerate some differential movement due to varying foundation and installation conditions. Additionally, in the coal fields, many slurry impoundments have underground mining in their vicinity and the possibility of subsidence, or mining-induced ground movement, needs to be considered.

HDPE pipe has been the type of plastic pipe most commonly used for decant and internal drain conduits in mining applications. Decants are typically solid wall pipes in the range of 18 to 36 inches in diameter. SDR values are commonly in the range of 11 to 21.

Internal drains are used within tailings disposal facilities for various purposes such as to improve stability by lowering the phreatic surface; reduce the potential for the tailings to flow by promoting consolidation; lower the hydraulic head on an under-liner to minimize seepage through the liner for environmental protection; and/or limit settlement after the surface of the tailings is capped and reclaimed. The collected seepage may be acidic, or for example, with gold tailings, contain cyanide from the processing of the gold ore. Profile wall corrugated HDPE pipe has been used for drainpipes in this type of application. The pipe has a corrugated wall on the exterior and smooth interior with slots in the recesses of the corrugations. This pipe is typically joined with snap couplings and surrounded by drainage aggregate. Pipe diameters in the 4- to 6-inch range are typically used for lateral drains that connect to main drains that may be 12 to 18 inches in diameter. The mains may be solid wall HDPE pipe. Numerous installations have been in operation for tens of years and are approaching 200 feet in height.

Another mining-industry application for corrugated plastic pipe is in the heap leaching process, where large piles of ore are leached with various chemical solutions to extract valuable minerals such as gold and copper. Perforated pipes installed under the heap collect the solution for processing of the metals. Heap leach pads can be over 300 feet high.

The design of tailings or slurry impoundments differs in many respects from the design for traditional embankment water dams. Basically, embankment dams are designed to store water, while tailings facilities are designed to form a basin for the deposition of fine material. The tailings/slurry disposal facility designer can take advantage of this difference by, for example, minimizing the amount of free water that is stored and by using drainage to develop as "dry" a tailing/slurry deposit as is practical. Dryer soils are inherently more stable than saturated soils.

The solids that settle out of suspension in a tailings/slurry impoundment can have a significant effect on seepage. Tailings or slurry is typically discharged from the upstream slope of the dam and the larger, sand-size particles tend to drop out near the discharge point. The finer particles settle out farther back in the impoundment, with the finest particles settling at the back end. A delta, which slopes back away from the dam, is formed by the settled material and a pool of free water accumulates at the back end of the reservoir. The effect of this typical configuration is that the free water must seep through the settled fines before seeping through the dam. Additionally, an objective in many tailings/slurry impoundment designs is to minimize the amount of free water. This is done both by site selection (i.e., minimizing the contributing watershed), diversion ditches, and the use of decants or pumping. If the impoundment is operated in a manner where ponded water can rise above the settled fines and contact the upstream slope of the dam for a period sufficient to develop steady-state seepage pressures, then the facility should be designed like a traditional water dam.

The characteristics of tailings/slurry impoundments sometimes allow designers to employ some differences in the design of tailings/slurry impoundments versus traditional embankment dams. In slurry impoundments, for example, plastic pipes that are not encased in concrete have been commonly used. Problems with seepage along the pipes has not been a significant occurrence in these impoundments, likely because of the effect of the settled fines in limiting seepage along the pipe combined with the long lengths of the decant pipes.

Another area where differences may be found, related to plastic pipe usage, is in the design of underdrains. For example, perforated or slotted plastic pipe is commonly used within underdrains, but the drainage aggregate used to surround the pipe may contain somewhat more than the recommended maximum of 5 percent minus No. 200 sieve material. This may be done, in cases where the seepage amounts and gradients are limited, to control the costs of installing drains to cover extensive areas. Where a less stringent requirement such as this is considered, however, tailings/slurry impoundment designers need to ensure that appropriate testing and analysis is performed—and construction and performance is carefully monitored—by engineers knowledgeable of the potential problems. Design alternatives must still ensure that filter criteria are met or critical exit gradients are not exceeded.

An important consideration in the design of underdrains for tailings/slurry impoundments is that the chemical characteristics of the seepage may affect the performance of the drainage system. Potential chemical reactions of the seepage solution and the drain materials must be fully evaluated and thoroughly understood. The potential may exist for clogging, cementation, crystal growth, or biological growth to render the drains inoperable. There are many examples of drains that have failed by these mechanisms.

A unique situation with mine-waste disposal dams is that they are raised as needed to provide additional capacity. In some mine waste disposal impoundments, a decant pipe is initially constructed with inlets positioned at various intervals to handle the entire life of the facility (see figure 101).

The inlets are blocked off as the level of waste accumulates in the impoundment. More commonly, the decant pipe is extended as the dam is raised with new inlets being installed and the old inlets being blocked off. In some cases, the amount of fill over the pipe is limited by installing a completely new decant pipe at a higher elevation when the embankment is raised, and the lower decant pipe is abandoned by filling it with grout.

This method of construction, where mine waste dams are raised on a nearly continuous basis over the 20 or 30 year life of the facility, presents an opportunity to monitor the performance of conduits that is not available in traditional dams. That is, monitoring can be performed, and design assumptions can be verified, as the height of fill over the pipe is raised and well before the fill reaches a critical height. Additional research is proposed in chapter 8 (PM-4) to study this method of construction.

Designers have proposed fill heights in the range of 200 feet over plastic decant pipes in mine waste dams. Due to the limited experience with plastic pipe under high fills, and because of uncertainties in deflection analysis, such as appropriate E' values, the Mine Safety and Health Administration has generally only accepted designs for high fill heights contingent upon the mining company monitoring the pipe deflection at various amounts of cover and over time (figure 102). Based on the monitoring data, the value of E' is back-figured and an estimate is then made of the performance of the pipe under additional fill. Initial measurements are taken when the pipe is installed to establish baseline values. Subsequent deflection measurements start at a fill height where there is confidence that pipe performance will be adequate. Thereafter, the monitoring depends on how fast the fill level rises. Monitoring is done at predetermined increases in fill height and at predetermined time intervals, such as annually. Deflection monitoring has been accomplished by pulling devices, such as mechanical deflectometers (measures the vertical diameter of the pipe), sonar devices, and video cameras, through the pipe. Results of monitoring have shown, in some cases, that deflection was at or approaching the amount considered allowable (typically 7.5 percent). Once the allowable deflection is approached, or other signs of distress are evident, the decant pipe is replaced by another pipe. The new pipe is installed higher in the dam cross section, and the

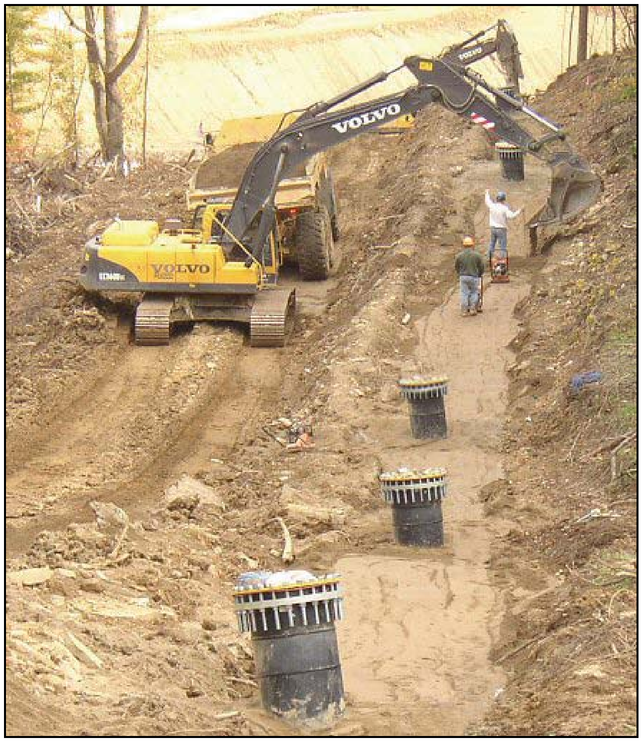


Figure 101.—Decant pipe with multilevel inlets.

Figure 102.—Deflection monitoring of a decant pipe.

original pipe is abandoned by being filled with grout. Designers have come to realize that it is costly to measure deflection every year and at certain fill levels, so they have moved toward installing new pipes at a higher elevations more frequently, so that the amount of fill on the active pipe, before it is grouted full, is limited.

Problems with the use of plastic pipe in mining dams are indicated by two of the case histories in appendix B. In the "Sediment Control Pond SP-4" case history, a 30-foot high dam failed during first filling due to seepage along a spillway pipe. This failure was attributed to inadequate compaction of the backfill and/or inadequate contact between the backfill and the pipe. In contrast to this failure, HDPE pipes have been used extensively as decants for coal slurry impoundments with no known failures from seepage along the pipe. In these applications the pipes have not been encased in concrete, but have typically been backfilled using hand-held equipment, such as pogo-sticks or rammers. The lack of problems with this type of construction may be due to a combination of factors which include: the impact of the lower-permeability settled slurry that acts to restrict seepage; the long length of the pipes - normally several hundred feet long; and the potential for the slurry fines to choke-off seepage paths along the pipe. The other problem, highlighted in the "Virginia Dam" case history in appendix B, involved a plastic pipe encased in concrete. In this case, the plastic pipe did not have adequate resistance to buckling and collapsed due to external hydrostatic pressure between the pipe and the concrete encasement.

Aside from the cases indicated above, the biggest performance concern with plastic pipe at slurry impoundments has been with pipe deflection. Excessive deflection or deformation has occurred in some cases and been attributed to soft or inadequately compacted areas in the backfill, or to stress concentrations from oversized rock in the backfill.

As stated elsewhere in this document, the "best practice" in installing a decant through a dam is to use a properly shaped reinforced cast-in-place concrete conduit through the impervious zone, so that the outside of the conduit can be battered to allow rubber-tired equipment to compact backfill directly against the conduit. This eliminates the problem with poor compaction in the haunch area. As indicated by Dr. Ernest Selig, "Apparently no amount of haunching effort can provide good soil support to the region about 20 degrees from the invert" (Selig, 1996, p. 6).

This "best practice," however, creates a dilemma in the case of mine waste impoundments. As previously explained, there are benefits to having a conduit that can tolerate some deformation in these impoundments. Furthermore, tailings and slurry impoundments do not typically have an "impervious core" and the added cost of reinforced cast-in-place conduits is not as suitable for the shorter life of these conduits, as compared to traditional embankment dam conduits. And while the absence of significant problems does not rule out future problems, the record does provide some indication that alternatives to concrete encasement may be reasonable in mine waste impoundment applications. The following recommendations are provided for installing conduits in tailings and slurry impoundments:

1. Although extensive problems have not been encountered with decant pipes through slurry impoundments, good conduit design and installation practices need to be followed. Slurry impoundment designers should recognize that the large body of evidence indicates adequate compaction cannot be achieved in the haunch area by conventional hand compaction methods. Using these methods, full contact between the pipe and the backfill cannot be ensured.

2. Decant pipes should be provided with an adequately designed seepage diaphragm and filter. The diaphragm should be extended far enough out from the pipe to intercept areas where cracks may occur due to hydraulic fracture or differential movement of backfill/embankment materials. See chapter 6 of FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) for guidance on filter diaphragm design.

3. The seepage diaphragm should not be considered as an adequate defense, by itself, against problems with seepage along the pipe. 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 pipe. The seepage diaphragm is intended to collect the limited seepage that occurs through well-compacted and suitable backfill and intercept particles that are being transported by water. The diaphragm could be overwhelmed and rendered ineffective by excessive seepage.

4. If the pipe is not to be encased in reinforced cast-in-place concrete, with battered sides that allow compaction by heavier equipment, then an alternate construction method, which provides for adequate backfill density in the haunch area, and full contact between the backfill and the pipe, needs to be specified.

5. Use of an alternate construction method should only be considered in slurry or tailings impoundments where it can be shown that the combination of hydraulic gradient and backfill material characteristics indicate adequate protection against internal erosion and piping. For example, slurry/tailings is typically discharged along the upstream face of the embankment resulting in the fine waste settling out and the free water collecting at the back end of the impoundment. However, if the slurry/tailings are discharged from the back end of the impoundment, free water would pond directly against the upstream slope of the dam. In this case, the seepage benefit from the settled fines would not be realized and the pipe should be designed as for a traditional embankment dam.

6. If the pipe is not encased in reinforced cast-in-place concrete, the installation options appear to be shaping the bedding, or the use of flowable fill. As indicated elsewhere in this document, many questions (see chapter 8, research items EM-3 through EM-8) need to be answered concerning the performance of flowable fill with respect to shrinkage, cracking, deformation properties, and stress concentrations at the pipe/backfill interface before it can be recommended for use.

7. Shaping the bedding to conform to at least the bottom one-third portion of the pipe is one technique that slurry impoundment designers have used to address the problem with compaction in the haunch area. However, the practice of shaping the bedding has concerns associated with it. Perfect contact between the shaped bedding and the pipe is not achievable. Designers have used a thin layer of expansive material, such as bentonite powder, to compensate for small irregularities between the shaped bedding and the pipe. An extreme level of care and attention to detail, with close supervision by a professional engineer knowledgeable of the potential problems, would be required. Further research (EM-7) is needed on this method, as proposed in chapter 8, before it can be recommended for use.

8. Specifications should include a detailed step-by-step procedure for installing the pipe and for achieving full contact between the conduit, bedding, and backfill. The type of equipment to be used to achieve the specified backfill densities should be specified. Construction related to critical piping installations should require full-time observation by experienced, qualified, and knowledgeable personnel.

9. Whatever pipe installation method is 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 the required backfill moisture/density specifications have been met and that full contact between the conduit and the backfill has been achieved. The inspector should periodically remove a portion of the compacted backfill and making use of a knife, or whatever device is necessary, ensure that the adjacent backfill is in intimate contact with the conduit and that no voids are present, especially along the bottom half of the conduit. The engineer should be required to inspect and accept the conduit bedding and backfill installation before the embankment fill is placed over the conduit.

10. The designer(s) should always provide reasonable accommodations for inspection using CCTV in their designs.

As indicated elsewhere in this document, nonencased plastic embankment conduits are not recommended for traditional water-retention dams with significant or high hazard potential. Designers of slurry tailings disposal facilities and impoundments should only specify nonencased plastic decant pipe where it can be shown, based on the conditions which are unique to slurry/tailings structures, that potential problems, such as with internal erosion along the pipe, are precluded.

Chapter 8

Research Needs

The National Dam Safety Program (NDSP), which was formally established by the Water Resources and Development Act of 1996, includes a program of technical and archival research. Research funding under the NDSP has addressed a cross section of issues and needs, all in support of ultimately making dams in the United States safer.

This chapter identifies research needs that are related to the performance of flexible plastic pipe within embankment and tailings/slurry impoundments. The authors considered these research needs to be good candidates for NDSP research funding.

8.1 Research Items

Research is needed in two categories: the performance of pipe material and the performance of embedment/encasement material.

8.1.1 Pipe material (PM)

Research is needed for the performance of pipe material includes:

- *PM-1*.—Determine minimum pipe resins and grades needed for dam related applications using laboratory tests or a review of existing research.

  A variety of formulations are used to produce the different kinds of plastic pipe. Even within a particular pipe category, such as HDPE, a number of resins are available. Some agencies require specific resins be used in the interest of obtaining sufficient pipe strength and resistance to aging. Currently there are no guidelines for chemical composition or grade for plastic pipe used in dams. A literature review would be done to determine standard practice and the associated issues with practitioners. At the completion of the literature review some laboratory testing may also be needed. Testing would focus on stress crack resistance, which in turn determines design life.

- *PM-2*.—Service life as it relates to the wear surface.

HDPE and PVC are reported to have better abrasion resistance than many other pipe materials. Manufacturers have conducted tests to measure pipe performance when subjected to abrasive forces. A literature review of research conducted by manufacturers is necessary to help determine the design life of plastic pipe that will be subjected to abrasion.

- *PM-3.*—Strain effects of perforations/slots on various types of pipe sections-solid wall, corrugated wall (single and profile wall).

  Circular perforations and slots have the potential to weaken pipe. Laboratory tests would be used to assess how number, type, location and size of holes/slots affect strain under load in the pipe.

- *PM-4.*—The performance of plastic pipe under staged loading associated with the type of stage construction unique to mine-waste dams.

  In tailings dams, the height of fill over a pipe may increase gradually over a period of 20 years or more. Testing should be performed, and field measurements collected and analyzed, to determine the affect on pipe performance of this type of staged construction. Issues include the allowable deflection, which is considered prudent under these conditions, and whether the short-term or long-term pipe modulus should be used in design.

- *PM-5.*—New and promising plastic pipe products.

  Perform a market survey and technical evaluation of all new plastic pipe products currently available. Evaluate potential applications within embankment dams including the advantages and disadvantages associated with new each product.

- *PM-6.*—Evaluate the watertightness and long term suitability of new joining systems for PVC pipe in dam applications.

  Newer joining systems have recently become available for PVC pipe. These newer joining systems include splined, heat fused, and mechanical joints. These types of joints are currently being used on water distribution and sewer installations, but have not been used in dam applications. Some manufacturers may have conducted tests for these new systems, and a literature search is needed. Additional laboratory testing may also be necessary.

8.1.2 Embedment/encasement material (EM)

Research is needed for the performance of embedment/encasement material includes:

- *EM-1*.—Investigate the interface bonding between plastic pipe and the encasement material.

  A bond between encasement materials and plastic pipe cannot be achieved due to material differences. Designers use a downstream filter to control internal erosion of embankment soil along this interface. The use of materials such as chemical grout and bentonite that could be used to form a seal between the plastic pipe and the encasement material would be evaluated for cost effectiveness and functionality using laboratory testing.

- *EM-2*.—Investigate the effects of the heat of hydration on plastic pipe using laboratory testing.

  Determine if heat of hydration from the curing of grout, concrete, or CLSM causes a significant rise in temperature which could cause the plastic pipe to expand.

- *EM-3*.—Quantify the Modulus of Soil Reaction for CLSM.

  The soil structure interaction between a flexible pipe and CLSM backfill is not clearly understood. Since the strength of CLSM is somewhere between soil and concrete, the reaction is somewhere between flexible and rigid restraint, respectively. Design of flexible pipe, as described earlier in this document, requires that the designer know the strength and modulus of soil reaction for the backfill. Limited laboratory tests have been completed (Brewer, 1990) which have provided estimates of the Modulus of Soil Reaction for CLSM. Additional testing is necessary to better define the Modulus of Soil Reaction for a wider variety of CLSM mixes and materials.

- *EM-4*.—Quantify compressive strength for CLSM used as backfill in dam applications by literature review and laboratory testing.

  CLSM is assumed to behave similar to a soil, allowing pipe deflection. The compressive strength of the CLSM will dictate the amount of pipe deflection. Laboratory testing is needed to determine recommended compressive strength for the use of CLSM as backfill in dam applications and changes with time as the CLSM ages. Full-scale laboratory tests would also be useful in evaluating the response of plastic pipe encased in CLSM exposed to large vertical loads.

- *EM-5*.—Evaluate the behavior of plastic pipe fully encased in CLSM.

  A laboratory testing program would confirm the assumption that CLSM behaves like a soil. A testing program would confirm this assumption and explore additives, which could make CLSM more flexible.

- *EM-6*.—Evaluate the shrinkage, permeability, and cracking potential for different CLSM mixes.

  A laboratory testing program would be used to measure the shrinkage, cracking potential, and erosion resistance for a variety of CLSM mix designs. The mix design would consist of evaluating a number of materials for improved performance of the mix. An example additive would be the use of nonshrink cement to see if shrinkage can be eliminated.

- *EM-7*.—Evaluate the response of plastic pipe partially encased with reinforced cast-in-place concrete (cradle) or CLSM.

  Some designers are using plastic pipe partially encased by a cradle. There are concerns about the effect of stress concentrations in the pipe at the top of the cradle and the failure mechanism with deflection limited to the top half of the pipe. Full scale laboratory testing would be used to determine if CLSM and concrete could be used as cradle material.

- *EM-8*.—Investigate if CLSM should be placed in lifts so lateral support can develop.

  Concerns exist that use of a single placement of CLSM can lead to pipe collapse, as lateral support has not developed prior to pipe being loaded vertically. There are also concerns with the placement of CLSM in lifts. In addition, the heat of hydration of the CLSM may heat the pipe, thus reducing its strength and potentially contributing to pipe collapse. Full scale laboratory testing would be used to evaluate CLSM placement methods.

- *EM-9*.—Evaluate the response of plastic pipe fully encased in nonreinforced concrete.

  For the unreinforced concrete case, a laboratory testing program would determine if the plastic pipe does perform as a rigid pipe, as currently assumed, or is there some deflection. Testing would also determine if there is a minimum concrete strength at which the pipe behaves rigidly.

- *EM-10*.—Investigate the use of self-consolidating concrete (SCC).
  SCC is a high-performance concrete that can flow easily into tight and constricted spaces without segregating and without requiring vibration. Determine if SCC can be economically used as an encasement material to improve consolidation under the haunches of circular pipes.

- *EM-11*.—Investigate the use of the "cut-earth cradle" method for installing plastic pipe.

Obtaining adequate density in the haunch area of nonencased circular pipes is a problem. The cut-earth cradle method was developed in an attempt to address this concern. The cut-earth cradle method involves compacting the backfill to the level of the springline of the conduit and excavating a cradle through the compacted backfill to conform to the shape of the pipe. An expansive material, such as powdered bentonite is used to compensate for small irregularities in the contact between the backfill and the pipe. The effectiveness of this technique would be evaluated and limitations and guidelines for use in dam construction developed.

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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.

Abrasion (ASTM, 2002): A rubbing or wearing away.

Additive (PPI, 2006): A substance added in small amount for a special purpose such as to reduce friction, corrosion, etc.

Allowable strain: A change in pipe dimension relative to the original dimension that provides an adequate factor of safety against unacceptable performance or failure.

Angle of friction (ASTM, 2002): Angle whose tangent is the ratio between the maximum value of shear stress that resists slippage between two solid bodies at rest with respect to each other, and the normal stress across the contact surface.

Anisotropy: Exhibiting properties with different values when measured in different directions. For soils, typically the horizontal permeability is greater than the vertical permeability due to layering introduced during deposition.

Antioxidant: A plastic additive to extend the temperature range and service life.

Arching: The condition in which vertical pressures within backfill in a trench is reduced because of the transfer of stress at the backfill/excavation surface interface.

Backfill (FEMA, 2005): Soil or concrete used to fill excavations.

Bead: Small ridge formed around the circumference of a polyethylene pipe joint as the two pipe ends are brought together during the butt fusion process.

Bell and spigot gasket joint: See **Joint, bell and spigot gasket**.

Biofouling: An accumulation and growth of deposits or contamination linked to microbial activity.

Borrow (AGI, 1987): Earth material (sand, gravel, etc.) taken from one location (such as a borrow pit) to be used for fill at another location; e.g. embankment material obtained from a pit when there is insufficient excavated material nearby to form the embankment. The implication is often present that the borrowed material has suitable or desirable physical properties.

Branch saddle: A fitting which is bonded to the exterior of a pipe to assist in transferring tensile loads in the pipe to a concrete thrust block.

Branching: The growth of a second chain of a polymer out of another one by replacement of a hydrogen atom on a monomer by a free-radical reaction, or by a condensation or other chemical reaction with a reactive group on a monomer.

Breakdown: Undesired alteration of soil gradation by mechanical action such as loading, pushing, and compacting.

Broadly graded: A soil consisting of a wide range of particle sizes where $c_u \geq 5$.

Buckling: See **Wall buckling**.

Butt fusion (PPI, 2006): A method of joining polyethylene pipe where two pipe ends are heated and rapidly brought together under pressure to form a homogeneous bond.

Butt fusion joint: See **Joint, butt fusion**.

Camera-crawler (FEMA, 2005): 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.

Carbon black (PPI, 2006): A black pigment produced by the incomplete burning of natural gas or oil, that possesses excellent ultraviolet protective properties.

Carrier pipe: The interior pipe of a dual containment pipe system.

Cell classification: A method used to classify thermoplastic compounds based on the material's composition and select properties.

Centralizer: Provides support to the carrier pipe within the containment pipe.

Chimney filter: See **Filter, chimney**.

Closed circuit television (CCTV) (FEMA, 2005): A method of inspection utilizing a closed circuit television camera system and appropriate transport and lighting equipment to view the interior surface of conduits.

Coefficient of internal friction (ASTM, 2002): The tangent of the angle of internal friction.

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.

Colorant: A plastic additive used to provide color.

Complete condition: A loading condition for an encased plastic pipe when the embankment height is less than or equal to the height of the plane of equal settlement. The frictional forces between the interior and exterior prisms extend to the top of the embankment.

Compound: A mixture of ingredients before the final processing into a completed product.

Conduit (FEMA, 2004): A closed channel to convey water through, around, or under an embankment dam.

Containment pipe: The outer pipe of a dual containment pipe system.

Contamination: The introduction of unwanted material into a dam, typically during construction, such as tracking core material onto a filter.

Controlled low strength material (CLSM) (FEMA, 2005): A self-compacting, cementitious material typically used as a replacement for compacted backfill around a conduit.

Corrosion (ACI, 2000): Disintegration or deterioration of a material by electrolysis or chemical attack.

Corrugated metal pipe (CMP) (FEMA, 2005): A galvanized light gauge metal pipe that is ribbed to improve its strength.

Coupling agent: A plastic additive to improve the properties of the plastic material.

Coupling: A mechanical device that serves to connect the ends of pipes.

Crosslinking: Chain-reaction polymerization which results in chemical links (bonds) between individual polymer chains.

Cured-in-place pipe (CIPP) (ASTM, 2003): 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.

Dam (FEMA, 2005): 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.

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: A structure used in mining operations to draw water off a reservoir after the heavier materials have settled out.

Deflection (FEMA, 2005): The decrease in the vertical diameter of a pipe due to load, divided by the nominal diameter, expressed as a percent.

Deformation (ACI, 2000): A change in dimension or shape due to stress.

Design (FEMA, 2005): An iterative decisionmaking process that produces plans by which resources are converted into products or systems that meet human needs or solve problems.

Designer (FEMA, 2005): 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.

Dimension ratio: See **Standard dimension ratio.**

Dimpling: Localized instability (buckling) resulting in a wavy checkerboard appearance on the inner surface of a pipe wall.

Double stage filter/drain: A system consisting of a coarse drainage zone (gravel) surrounding the pipe and a filter (sand) zone surrounding the coarse element.

Drainpipe: A system of pipe within a embankment dam used to collect seepage from the foundation and embankment and convey it to a free outlet.

Dual wall containment pipe: A pipe systems that provides a secondary containment pipe around the carrier pipe. Any leakage from the carrier pipe will be safely contained within the containment pipe.

Durability (ACI, 2000): The ability of a material to resist weathering, chemical attack, abrasion, and other conditions of service.

Embankment dam (FEMA, 2005): Any dam constructed of excavated natural materials, such as both earthfill and rockfill dams, or of industrial waste materials, such as a tailings dams.

End restraint: A structural member designed to resist the anticipated expansion/contraction forces caused by temperature change in a plastic pipe.

Engineer (FEMA, 2005): A person trained and experienced in the profession of engineering; a person licensed to practice the profession by the appropriate authority.

Environmental stress cracking (ASTM, 2001): The development of cracks in a material that is subjected to stress or strain in the presence of specific chemicals.

Extender: A plastic additive used to reduce cost.

External hydrostatic pressure: Pressure on the outside of the pipe due to water surrounding the pipe in the voids or in the soil surrounding the pipe.

Exterior prism: The soil adjacent to the soil directly above the buried conduit (the interior prism).

Extrusion joint: See **Joint, extrusion**.

Fibrous reinforcement: A plastic additive used to improve the strength to weight ratio.

Filler: A plastic additive to improve properties of the resin.

Filter (FEMA, 2005): A zone of material designed and installed to provide drainage, yet prevent the movement of soil particles due to flowing water.

Chimney (FEMA, 2005): 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 (FEMA, 2005): 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 (FEMA, 2005): 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.

Filter collar: See **Filter, collar**.

Filter diaphragm: See **Filter, diaphragm**.

Filter material (NAWIC, 1986): Granular material that has been graded to allow water to pass through it while retaining solid matter.

First filling: The initial filling of the reservoir behind a dam. Also, used to describe refilling of a reservoir after a modification has been made to a dam.

Flanged joint: See **Joint, flanged**.

Flexible pipe: A pipe that derives its load carrying capacity by deflecting at least 2 percent into the surrounding medium upon application of load.

Fold-and-formed pipe (FFP): A thermoset system where a plastic pipe manufactured in a folded shape of reduced cross-sectional area is pulled into an existing conduit and subsequently expanded to the internal shape by heat and pressure.

Gasket: A flexible material used to form a water-tight seal between two components.

Geosynthetic (ASTM, 2004): A planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure or system.

Gradation (ASTM, 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.

Grout (FEMA, 2005): 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.

Haunch: The area beneath a pipe between the springline and the invert.

Hazard (FEMA, 2004): A situation that creates the potential for adverse consequences such as loss of life, property damage, or other adverse impacts.

Hazard potential (FEMA, 1998): The adverse incremental consequences that result from the release of water or stored contents due to failure of the dam or misoperation of the dam or appurtenances. Impacts may be for a defined area downstream of a dam from flood waters released through spillways and outlet works of the dam or waters released by partial or complete failure of the dam. There may also be impacts for an area upstream of the dam from effects of backwater flooding or landslides around the reservoir perimeter.

- **Low (FEMA, 1998)**: Embankment dams assigned the low hazard potential 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 owners' property.

- **Significant (FEMA, 1998)**: Embankment dams assigned the significant hazard potential classification are those 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 (FEMA, 1998)**: Embankment dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

Hazard potential classification: A system that categorizes 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 dam (i.e., safety, structural integrity, flood routing capacity).

Heterogeneous: Consisting of parts or aspects that are unrelated or unlike each other. In relation to earth materials, soils that consist of any combination of clays, silts, sands, gravels, cobbles, and boulders.

High density polyethylene plastic (HDPE) (ASTM, 2001): Those linear polyethylene plastics, having a standard density of 0.941 g/cm^3 or greater.

High hazard potential: See **Hazard potential, high**.

Homogenous: Consisting of a single material of uniform properties. In relation to earth dams, a design of uniform cross section.

Hoop strain: Strain in the pipe wall due to internal or external pressure.

Hoop stress: The tensile stress in the wall of the pipe in the circumferential orientation due to internal hydrostatic pressure.

Hopper (NAWIC, 1986): A storage bin or a funnel that is loaded from the top and discharges through a door or chute in the bottom.

Hydraulic loading: Loading on the pipe due to internal pressure, water hammer, internal vacuum pressure, or external hydrostatic pressure.

Hydrostatic design basis (HDB) (ASTM, 2001): One of a series of established stress values specified in ASTM D 2837 for a plastic compound obtained by categorizing the long-term hydrostatic strength determined in accordance with Test Method D 2837.

Hydrostatic design stress (HDS) (ASTM, 2001): The estimated maximum tensile stress the material is capable of withstanding continuously with a high degree of certainty that failure of the pipe will not occur. This stress is circumferential when internal hydrostatic water pressure is applied.

Hydrostatic pressure: The force per unit area due to water within the pipe (internal) or surrounding the pipe (external).

Igneous (AGI, 1987): Said of a rock or mineral that solidified from molten or partly molten material, i.e. from a magma; also, applied to processes leading to, related to, or resulting from the formation of such rocks.

Inclination: The degree of deviation from a horizontal.

Incomplete condition: A loading condition for an encased plastic pipe when the embankment height is greater than the height of the plane of equal settlement. The frictional forces between the interior and exterior prisms do not extend to the top of the embankment.

Inspection (FEMA, 2005): The review and assessment of the operation, maintenance, and condition of a structure.

Inspector (FEMA, 2005): 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.

Interior prism: The prism of soil directly above the buried conduit.

Internal erosion (FEMA, 2005): 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. Note: For a complete discussion of internal erosion and backward erosion piping, see FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

Internal hydrostatic pressure: Pressure inside the pipe (typically no more than the pressure due to the full reservoir).

Internal vacuum pressure: Negative internal pressure inside the pipe.

Invert (FEMA, 2005): The bottom or lowest point of the internal surface of the transverse cross section of a conduit.

Joint (ASTM, 2001): The location at which two sections of conduit or pipe and a fitting are connected together.

> **Bell and spigot gasket (ASTM, 2001)**: A connection between piping components consisting of a bell end on one component, an elastomeric gasket between the components, and a spigot end on the other component.
>
> **Butt fusion (ASTM, 2001)**: A joint in which the prepared ends of the joint components are heated and then placed in contact to form the joint.

Extrusion (ASTM, 2001): A joint formed by a process whereby heated or unheated plastic forced through a shaping orifice becomes one continuously formed piece.

Flanged (ASTM, 2001): A mechanical joint using pipe flanges, a gasket, and bolts.

Mechanical (ASTM, 2001): A connection between piping components employing physical force to develop a seal or produce alignment.

Lean concrete: Low strength concrete (low cement content) used for non-structural applications such as fill, or as a sub base for concrete pavements.

Liquefaction (AGI, 1987): In cohesionless soil, the transformation from a solid to a liquid state as a result of increased pore pressure and reduced effective stress.

Load coefficient: A coefficient used in calculating the soil load on buried conduits to account for the load transfer between the prism of soil directly above the pipe and the adjacent soil.

Long-term modulus of elasticity: A material property describing the stress/strain behavior of a material in the linearly elastic region after exposed to a long period of time (50 to 100 years).

Low hazard potential: See **Hazard potential, low**.

Lubricant (ASTM, 2001): A material used to reduce friction between two mating surfaces that are being joined by sliding contact.

Marston load theory: A theory on the magnitude of soil load on a buried conduit based on the construction method and relative settlements of the soil directly above the pipe, soil adjacent to the pipe, and soil adjacent to the soil directly above the pipe.

Material quality: Physical properties of soil related to strength, absorption, density, etc.

Maximum dimension: In relation to opening sizes in perforated pipe the diameter for circular holes and the length for slots.

Maximum size aggregate (MSA): The smallest sieve through which 100 percent of the aggregate sample particles pass.

Mechanical joint: See **Joint, mechanical**.

Metamorphic (AGI, 1987): Pertaining to the process of metamorphism or to its results.

Metamorphism (AGI, 1987): The mineralogical, chemical, and structural adjustment of solid rocks to physical and chemical conditions which have generally been imposed at depth below the surface zones of weathering and cementation, and which differ from the conditions under which the rocks in question originated.

Miscellaneous fill: Earthfill that does not serve a specific function such as drainage, filtering, or water barrier.

Modulus of elasticity: A material property describing the stress/strain behavior of a material in the linearly elastic region.

Modulus of soil reaction (E'): Measure of the stiffness of the material which surrounds the pipe.

Multistage filter: A filter consisting of more than one zone, such as a sand filter zone and gravel drain zone.

Negative projecting conduit (Spangler and Handy, 1982): A conduit installed in a relatively narrow and shallow trench with its top at an elevation below the natural ground surface and which is then covered with an embankment.

Nuclear testing: Of or relating to the nuclear density test as described in ASTM D 2922.

Opening size: The minimum dimension of a perforation in a pipe. For circular perforations it is the hole diameter, for slots, it is the slot width.

Outlet works (FEMA, 2004): A dam appurtenance that provides release of water (generally controlled) from a reservoir.

Out-of-round: The allowed difference between the maximum measured diameter and the minimum measured diameter (stated as an absolute deviation).

Particle breakdown: Undesired alteration of a soil grain by mechanical action such as loading, pushing, and compacting.

Perforation: A hole or pattern made by or as if by piercing, drilling, or sawing.

Permeability (k): The rate at which water passes through soil in accordance with Darcy's law.

Pipe stiffness: The inherent resistance of a flexible pipe to load.

Plane of equal settlement: A location above a pipe where the accumulated strain and settlement in the exterior prisms equal that of the interior prism. Above this plane, the interior and exterior prisms settle equally and no shear or friction forces are transferred between the prisms.

Plastic (ASTM, 2001): A material that contains as an essential ingredient one or more organic polymeric substances of large molecular weight, is solid in its finished state, and, at some stage in its manufacture or processing into finished articles, can be shaped by flow.

Plastic pipe (ASTM, 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.

Plasticity index: Numerical difference between the liquid limit and the plastic limit.

Poisson's ratio (ν) (ASTM, 2002): Ratio between linear strain changes perpendicular to the direction of a given uniaxial stress change.

Polyester (PPI, 2006): Resin formed by condensation of polybasic and monobasic acids with polyhydric alcohols.

Polyethylene (FEMA, 2005): A polymer prepared by the polymerization of ethylene as the sole monomer.

Polyvinyl chloride (PVC) (FEMA, 2005): A polymer prepared by the polymerization of vinyl acetate as the sole monomer.

Positive projecting conduit (Spangler and Handy, 1982): A conduit installed in a bedding with its top projecting above the natural ground surface and which is then covered with an embankment.

Preservative: A plastic additive used to prevent bacterial attack.

Pressure pipe (ASTM, 2001): Pipe designed to resist continuous pressure exerted by the conveyed medium.

Pressure rating (PR) (ASTM, 2001): The estimated maximum water pressure the pipe is capable of withstanding continuously with a high degree of certainty that failure of the pipe will not occur.

Profile pipe: Pipe that has smooth interior and corrugated exterior surfaces.

Projecting conduit: A conduit covered by fill material such as embankment material.

Projection condition: A projecting conduit above which the exterior prisms settle more than the interior prism.

Projection ratio (*p*): The ratio of the vertical height of the top of the conduit above the embankment subgrade to the outside conduit diameter.

Proof rolling: A process accomplished by the application of heavy construction or compaction equipment on an excavation invert in order to locate low density areas.

Quality assurance (**FEMA, 2005)**: 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 (FEMA, 2005): 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.

Renovation (FEMA, 2005): The repair or restoration of an existing structure, so it can serve its intended purpose.

Repair (FEMA, 2005): The reconstruction or restoration of any part of an existing structure for the purpose of its maintenance.

Resin (ASTM, 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).

Rigid pipe: A pipe, typically reinforced concrete, designed to carry loads without support from the surrounding medium.

Ring strain: Strain in the pipe wall due to deflection or deformation from external loads.

Rock ladder: A device that lifts aggregate vertically by the use buckets attached to a belt.

Sand (ASTM, 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.

Sediment trap: An area, such as a pool, behind a weir or flume where the inflow velocity is reduced sufficiently that any soil particles included in the flow will settle out.

Sedimentary rock (AGI, 1987): A rock resulting from the consolidation of loose sediment that has accumulated in layers; e.g. a clastic rock (such as conglomerate or tillite) consisting of mechanically formed fragments of older rock transported from its source and deposited in water or from air or ice; or a chemical rock (such as rock salt or gypsum) formed by precipitation from solution; or an organic rock (such as certain limestones) consisting of the remains or secretions of plants and animals.

Seepage (ASTM, 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 process of separating coarser soil from finer soil, typically during construction activities.

Service life (FEMA, 2005): Expected useful life of a project, structure, or material.

Settlement ratio (AWWA, 1995): The relationship between the pipe deflection and the relative settlement between the prism of soil directly above the pipe and the adjacent soil.

Short-term modulus of elasticity: A material property describing the stress/strain behavior of a material in the linearly elastic region immediately upon a change in load.

Significant hazard potential: See **Hazard potential, significant**.

Single stage filter/drain: A system consisting of one zone of filter material, usually sand, surrounding a collector drainpipe.

Siphon (FEMA, 2005): 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.

Sliplining (FEMA, 2005): The process of inserting a new, smaller-diameter lining or pipe into an existing larger-diameter conduit.

Slot: A long, narrow aperture or slit.

Slow crack growth (PPI, 2006): The slow extension the crack with time.

Soil (ASTM, 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 prism theory: The soil load on a buried pipe is weight of the soil directly above the pipe.

Soil prism: The soil directly above the pipe.

Soil-cement: Highly compacted mixture of soil/aggregate, portland cement, and water. Soil-cement differs from portland cement concrete pavement in several respects. One significant difference is the manner in which the aggregates or soil particles are held together. A portland cement concrete pavements mix contains sufficient paste (cement and water mixture) to coat the surface area of all aggregates and fill the void between aggregates. In soil-cement mixtures, the paste is insufficient to fill the aggregate voids and coat all particles, resulting in a cement matrix that binds nodules of uncemented material.

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.

Spreader box: A device used in construction to deposit fill uniformly over the ground surface.

Stabilizer: A plastic additive to prevent degradation.

Standard dimension ratio (SDR) (FEMA, 2005): Ratio of the average specified outside diameter to the minimum specified wall thickness for outside diameter controlled plastic pipe. Also referred to as dimension ratio (DR).

Standard inside dimension ratio (SIDR): A specific ratio of the average specified inside diameter to the minimum specified wall thickness for inside diameter-controlled plastic pipe.

Stockpile (NAWIC, 1986): Material dug and piled for future use.

Strain (ASTM, 2001): The change per unit length in a linear dimension of a body, that accompanies a stress. Strain is a dimensionless quantity which may be measured in percent, in inches per inch, in millimeters per millimeter, etc.

Stress crack resistance (SCR): Resistance to cracking from tensile stresses; a failure that develops over time at stresses less than the yield strength. Stress cracking is a macro-brittle cracking phenomenon that occurs at a constant stress significantly less than the yield or break stress of the material. Stress cracking is initiated at an internal or external "defect" in the material such as an inclusion or scratch.

Surge pressure (water hammer): A surge in pressure caused by a sudden change in water velocity. Typical causes include the sudden starting or stopping of a pump, sudden valve movement, or air movement in a pipeline. The surge may damage or destroy pipelines and pumps if severe enough.

Thermoplastic (ASTM, 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, 2001): A plastic that, when cured by application of heat or chemical means, changes into a substantially infusible and insoluble product.

Tailings (FEMA, 2005): The fine-grained waste materials from an ore-processing operation.

Toe drain: Typically a pipe used to collect water at the downstream toe of a dam.

Trench condition: A projecting conduit above which the interior prism settles more than the exterior prism.

Trench conduit (Spangler and Handy, 1982): A conduit installed in a relative narrow trench excavated in passive or undisturbed soil which is then covered with earth backfill.

Undisturbed soil: In situ or in place soil unaltered by human activity.

Uniformly graded: A soil consisting of a small range of particle sizes where $c_u < 5$.

Void (FEMA, 2005): A hole or cavity within the foundation or within the embankment materials surrounding a conduit.

Wall buckling: Collapse of the pipe due to excessive external pressure or internal vacuum pressure.

Wall crushing: Failure of the pipe wall due to excessive wall stress from loads on top of the pipe.

Water content (ASTM, 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 hammer: See **Surge pressure**.

Zoning: The cross sectional area of an embankment divided into zones that serve different purposes such as core, shell, chimney filter, etc.

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Appendix A

Example Calculations

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A-1 Flexible pipe design (for a drainpipe)

Description

A 12-inch diameter HDPE (ASTM D 3350 cell class 345464C) solid wall pipe will be used as a drainpipe (figure A-1). The maximum height (H) of fill over the pipe is 15 feet. The pipe will be embedded in a well graded sandy soil that is compacted to 50 percent relative density.

Assumptions

The following assumptions are made for this example:

- The pipe is considered a projecting conduit within the footprint of an embankment dam and classified as a positive projecting conduit in a trench condition. Since the pipe is embedded in coarse grained soil rather than encased in grout or concrete, flexible pipe design will be used. The soil prism theory is used as recommended in section 2.1.1. Table 9 in section 3.5.6 describes the applicable soil load conditions.

- The total unit weight of soil (γ) is 115 lb/ft^3 with an E' of 2,000 lb/in^2 (table 4, section 3.1.3, using 75% of this value is 1,500 lb/in^2).

- The short-term modulus of elasticity (E) of the HDPE is 140,000 lb/in^2 and the long-term modulus is 30,000 lb/in^2 (table 3, section 3.1). The modulus of elasticity and allowable compressive stress depend upon the type and classification of plastic.

- The pipe meets ASTM D 3035 and has an D_O = 12.75 inches.

- The allowable long-term compressive stress (σ) (1/2 the hydrostatic design basis of 1,600 lb/in^2) of the HDPE (cell class 345464C) is 800 lb/in^2.

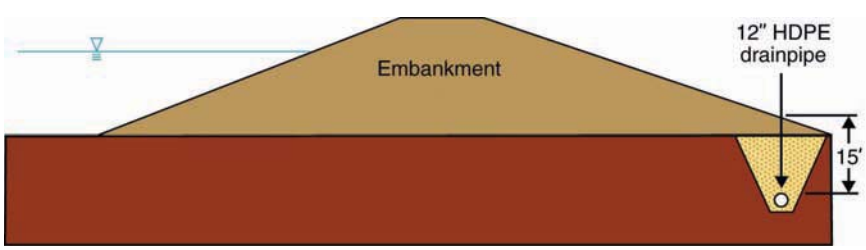

Figure A-1.—Cross section of an embankment dam and drainpipe (filter not shown).

Calculations

Assume pressures from wheel (p_w) and internal vacuum (p_v) are zero.

<u>Soil Load</u>

The load due to overlying soil should be determined by the soil prism theory.

The soil load on the pipe by the soil prism theory is:

$$P_s = \gamma H = 115(15) = 1{,}725\ \mathrm{lb}/\mathrm{ft}^2 \tag{2-1}$$

<u>Wall Crushing</u>

The resistance to wall crushing of the plastic pipe is evaluated by:

$$T_{pw} = \frac{PD_O}{2} = \frac{\left(1725/144\right)(12.75)}{2} = 76.36\ \mathrm{lb}/\mathrm{in} \tag{3-1}$$

The required wall cross-sectional area is determined by:

$$A_{pw} = \frac{T_{pw}}{\sigma} = \frac{76.36}{800} = 0.095\ \mathrm{in}^2/\mathrm{in} \tag{3-2}$$

The area of a pipe wall may be computed as:

$$A_{pw} = \frac{\left(D_o - D_i\right)}{2} \text{ or t (for solid wall pipe)} \tag{3-3}$$

where:

$D_i = D_o - 2t$

Solving for t:

$t = 0.095$ in

So the minimum wall thickness, t, is 0.095 inches. The minimum wall thickness of a 12-inch HDPE pipe meeting ASTM D 3035 with an SDR of 26 (maximum recommended SDR) is 0.490 inches.

Wall Buckling

Plastic pipe embedded in soil may buckle due to excessive loads and deformations. The total soil load must be less than the allowable buckling pressure. The long-term modulus of elasticity is recommended since the soil load is a permanent load. The allowable buckling pressure may be determined from:

$$q_a = \frac{1}{FS}\left(32R_w B' E' \frac{EI_{pw}}{D_o^3}\right)^{1/2} \tag{3-4}$$

$$B' = \frac{4\left(h^2 + D_o h\right)}{1.5\left(2h + D_o\right)^2} = \frac{4\left(15^2 + {}^{12.75}\!/_{12}(15)\right)}{1.5\left((2)(15) + {}^{12.75}\!/_{12}\right)^2} = 0.665 \tag{3-6}$$

$$q_a = \frac{1}{2.5}\left((32)(1)(0.665)(1500)\frac{(30,000)\left({}^{0.490^3}\!/_{12}\right)}{12.75^3}\right)^{1/2} \tag{3-4}$$

$$= 22.8 \text{ lb/in}^2 = 3288 \text{ lb/ft}^2$$

The soil pressure of 1,725 lb/ft^2 is less than the allowable buckling pressure of 3,288 lb/ft^2 for a 12-inch diameter pipe with an SDR of 26.

Deflection

Since this pipe is a drainpipe for a filter, the recommended allowable deflection is 7.5% (see section 3.1.3). The deflection may be estimated from the following equation for solid wall pipe:

$$\frac{\%\Delta Y}{D} = \frac{(D_L P_s + P_W + P_V)K(100)}{\left[\left(\frac{2E}{3(SDR-1)^3}\right) + 0.061E'\right]}$$
$$= \frac{\left((1.5)\left(1725/144\right) + 0 + 0\right)(0.1)(100)}{\left[\left(\frac{(2)(140,000)}{3(26-1)^3}\right)\right] + (0.061)(1500)} \qquad (3\text{-}8)$$
$$= 1.84\% < 7.5\%$$

where:

K = 0.1 as recommended in section 3.1.3

Conclusion

A 12-inch diameter, HDPE with ASTM D 3350 cell class 345464C resin, and SDR of 26 is recommended for the drainpipe.

A-2 Encased pipe design (for an embankment conduit)

Description

An existing 20-foot high embankment dam has a 24-inch diameter CMP outlet works conduit (figure A-2). The conduit does not have a gate and is not considered a pressurized conduit. The foundation consists of stiff clay. The existing conduit will be sliplined with an HDPE (ASTM D 3350 cell class 345464C) pipe with an outside diameter, D_O, of 18-inches. The annulus between the existing conduit and the HDPE slipliner will be grouted. The liner pipe will be designed to withstand 18-feet of hydrostatic pressure.

Assumptions

The following assumptions are made for this example:

- Since the annulus of the sliplined pipe will be grouted, the soil load will be assumed to act on the encased HDPE pipe. The grouted annulus is assumed to prevent deflection. Therefore, the HDPE liner will be considered a projecting conduit in the positive projecting condition.

- The CMP will continue to deteriorate and will not support the load.

- A solid wall HDPE pipe will be used as the slipliner pipe.

- The total unit weight of soil (γ) is 110 lb/ft^3.

- The short-term modulus of elasticity of the HDPE (cell class 345464C) is 140,000 lb/in^2 and the long-term modulus is 30,000 lb/in^2 (table 3, section 3.1).

- The allowable long-term compressive stress (σ) (1/2 the hydrostatic design basis of 1,600 lb/in^2) of the HDPE (cell class 345464C) is 800 lb/in^2.

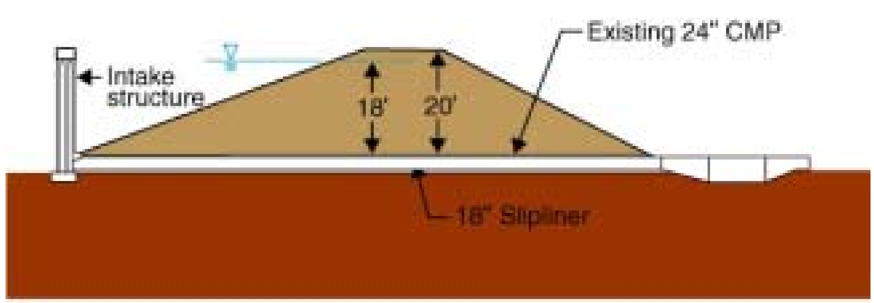

Figure A-2.—An 18-inch diameter solid wall HDPE slipliner installed in an existing 24-inch diameter CMP outlet works conduit.

- The settlement ratio (r_{sd}) is assumed to be +0.5 since the fill around the pipe was a compacted earth fill (table 2, section 2.1.2).
- The projection ratio, p, is 1.0 (figure 35).

Calculations

Soil Load

By prism method: The soil load on the pipe using the soil prism load is:

$$P_s = \gamma H = (110)(20) = 2{,}200 \text{ lb/ft}^2 \qquad (2\text{-}1)$$

By Marston method: The Marston soil load for a positive projecting conduit in the projection condition may be determined from the following:

$$W_C = C_C \gamma D_O^{\ 2} \qquad (2\text{-}2)$$

C_c is determined from figure 30 for H/D_o=(20)/(18/12)= 13.3. The load is based on an incomplete condition with r_{sd}=+0.5. The value of C_c is approximately 20 from figure 36.

$$W_c = (20)(110)\left(18/12\right)^2 = 4{,}950 \text{ lb/ft}$$

The pressure on the top of the pipe may be determined by:

$$P_s = \frac{W_c}{D_O} = \frac{4950}{18/12} = 3{,}300 \text{ lb/ft}^2 \qquad (2\text{-}6)$$

The Marston soil load is recommended for conduits encased in grout as shown in table 9 in section 3.5.6 and discussed in chapter 3.

Wall Crushing

The thrust in the pipe wall is

$$T_{pw} = \frac{PD_o}{2} = \frac{\left(3300/144\right)(18)}{2} = 206.25 \text{ lb/in} \qquad (3\text{-}1)$$

The required wall cross-sectional area is:

$$A_{pw} = \frac{T_{pw}}{\sigma} = \frac{206.25}{800} = 0.2578 \ \mathrm{in}^2/\mathrm{in} \tag{3-2}$$

The area of a pipe wall may be computed as:

$$A_{pw} = \frac{(D_o - D_i)}{2} \text{ or } t \text{ (for solid wall pipe)} \tag{3-3}$$

So the minimum wall thickness, *t*, is 0.2578 in.. The minimum wall thickness of an 18-inch HDPE pipe meeting ASTM D3035 is 0.554 inch with an SDR of 32.5.

<u>Wall Buckling</u>

The external hydrostatic pressure on the pipe is

$$P_G = \gamma h_w = (62.4)(18) = 1{,}123 \ \mathrm{lb/ft^2} \tag{2-16}$$

The long-term modulus of elasticity is recommended since the 18-feet of hydrostatic pressure act on the pipe throughout its design life.

The unconstrained buckling pressure of the pipe with an SDR of 32.5 is:

$$P_{CR} = \frac{2E}{(1-v^2)}\left(\frac{1}{SDR-1}\right)^3 = \frac{(2)(30000)}{(1-0.45^2)}\left(\frac{1}{32.5-1}\right)^3 = 2.41 \ \mathrm{lb}/\mathrm{in}^2 = 2.41(144) = 346 \ \mathrm{lb}/\mathrm{ft}^2 \tag{3-20}$$

where:

v = 0.45 for HDPE as recommended in section 3.3.2

The unconstrained buckling pressure of a SDR 32.5 HDPE pipe is less than the external hydrostatic pressure of 1,123 lb/ft^2.

Check the unconstrained buckling pressure of an SDR 17.

$$P_{CR} = \frac{(2)(30000)}{(1-0.45^2)}\left(\frac{1}{17-1}\right)^3 = 18.3 \ \mathrm{lb}/\mathrm{in}^2 = 18.3(144) = 2{,}644 \ \mathrm{lb}/\mathrm{ft}^2$$

A factor of safety of 2.0 is applied to the unconstrained buckling pressure.

$$\frac{P_{CR}}{1.5} = \frac{2,644}{1.5} = 1,763 \; {}^{\text{lb}}\!/_{\text{ft}^2} > 1,123 \; {}^{\text{lb}}\!/_{\text{ft}^2}$$

Conclusion

An 18-inch diameter, HDPE with cell class 345464C resin, and SDR of 17 is recommended.

A-3 Siphon Design

The following assumptions are made for this example:

Description

A 10-inch diameter, solid wall, HDPE pipe will be installed as a siphon over the crest of an embankment dam (figure A-3). The siphon head (*H*) is 14 feet. The siphon will provide additional drainage capacity and allow lowering of the reservoir.

Assumptions

- The siphon operates for short periods on an infrequent basis. The short-term modulus of elasticity will be used for the buckling analysis.
- The short-term modulus of elasticity (*E*) of the HDPE is 140,000 lb/in^2 and the long-term modulus is 30,000 lb/in^2 (table 3, section 3.1). The modulus of elasticity and allowable compressive stress depend upon the type and classification of plastic.
- The allowable long-term compressive stress (σ) (1/2 the hydrostatic design basis of 1,600 lb/in^2) of the HDPE (cell class 345464C) is 800 lb/in^2.
- The outside diameter of a HDPE pipe meeting ASTM D 3035 is 10.75 inches.
- The soil load will be determined by the soil prism theory.
- The internal vacuum pressure is 14 feet of head = 6 lb/in^2.

  $$P = \gamma H = \frac{(62.4)(14)}{144} = 6 \text{ lb/in}^2$$

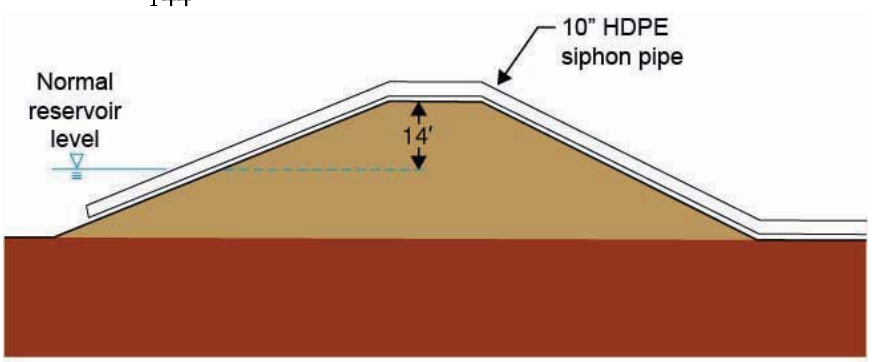

Figure A-3.—This figure illustrates a siphon extending over the crest of an embankment dam. Alternative siphon designs may want to consider the addition of earthen ramps over the siphon or embedment into the crest of the dam to facilitate vehicular traffic on the dam crest.

- The Poisson's ratio (v) for HDPE = 0.45.

Calculations

Soil Load

The pipe is on top of the embankment and does not have a soil load.

Wall Crushing

This is not an issue since there is not a soil load.

Deflection

Deflection is not determined since there is not a soil load.

Wall Buckling

The maximum SDR (minimum wall thickness) of a 10.75-inch HDPE pipe meeting ASTM D3035 is 32.5.

The unconstrained buckling pressure of the pipe with an SDR of 32.5 is:

$$P_{CR} = \frac{2E}{\left(1-v^2\right)}\left(\frac{1}{SDR-1}\right)^3 = \frac{2(140,000)}{\left(1-0.45^2\right)}\left(\frac{1}{32.5-1}\right)^3 = 11.2\,{}^{\text{lb}}\!/_{\text{in}^2} \qquad (3\text{-}20)$$

where:

v = 0.45 for HDPE as recommended in section 3.3.2

A factor of safety of 1.5 is applied to the unconstrained buckling pressure.

$$\frac{P_{CR}}{1.5} = \frac{11.2}{1.5} = 7.5\,{}^{\text{lb}}\!/_{\text{in}^2} > 6\,{}^{\text{lb}}\!/_{\text{in}^2}$$

The unconstrained buckling pressure of an SDR 32.5 HDPE pipe is greater than the internal vacuum pressure of 6 lb/in^2.

Strain

Strain is not evaluated since there is not a soil load.

Conclusion

A 10-inch diameter, HDPE with cell class 345464C resin, and SDR of 32.5 is recommended.

A-4 Toe drain design (filter and drain)

Description

This example will illustrate the design of a toe drain system utilizing the guidelines presented in this document as well as judgment required by the designer beyond these guidelines. The example is derived from the case history of the Keechelus Dam modification completed in 2002 and includes a portion of the modified dam's entire drainage system. For brevity, filter materials at other toe drain locations are not included in this example. The filter used for the toe drain was designated Zone 2B and the drain material Zone 3. The foundation for the toe drain is an alluvial fan deposit in a glacial environment during the Quaternary (Qaf).

Zones 2B and 3 were designed in accordance with Bureau of Reclamation's *Embankment Dams*, Design Standards No. 13, Chapter. 5, "Protective Filters," 2007.

The gradation data of the base material (foundation), Qaf, was determined from laboratory testing on 17 samples obtained from drillholes and test pits. The statistics for the proportions of gravel, sand, and fines is shown on figure A-4. As a rule, grain size distribution for soils are not uniformly distributed due to the inherent heterogeneity of soil. Figure A-4 illustrates the simple statistics (minimum, maximum, mean, 1st std deviation) to indicate the nature of the material. The actual gradation curves for the 17 samples are shown on figure A-5.

Filter Design

The first step in sizing the filter is to mathematically regrade the base material (finer limit only) to the minus No. 4 sieve. The regraded limits are shown on figure A-5 in red. An outlier was identified and eliminated from the dataset as shown on the figure. The percent passing the No. 200 sieve is determined as 32.3% by using the finer side of the regraded curve. Based on the percent passing the No. 200 sieve, the base material is classified as "category 3" and protection against particle movement is controlled by:

$$D_{15}F \leq 0.7mm + \frac{(40-a)(4D_{85}B-0.7mm)}{25}$$

where:

D_{85}B = finer side of regraded gradation = 0.78 mm

a = percentage of soil passing No. 200 sieve = 32.3%

D_{15}F ≤ 1.4 mm (particle movement limit)

D_{15}F ≥ D_{15}B (permeability limit)

where:

D_{15}B: coarser side of regraded = .05 mm

$D_{15}F \geq (5)(0.05) = 0.25$ mm
0.25 mm > 0.10 mm so 0.25 mm controls
$D_{15}F \geq 0.25$ mm (permeability limit)

The $D_{15}F$ limits for the range of acceptable filter gradations are 0.25 mm. to 1.4 mm. (i.e., 0.25 mm. ≤ $D_{15}F$ ≤ 1.4 mm), and are shown on figure A-6.

In order to maximize the permeability of the filter, the filter's upper limit is set near the upper end of this range (the trial is shown on figure A-7).

The next step for sizing the filter is selecting the degree of uniformity of the gradation. Based on the trial $D_{10}F$ of approximately 0.5 mm, the $D_{85}F$ upper limit for segregation is 20 mm (as shown on Table 2, Bureau of Reclamation, 2007) and is indicated on figure A-6. Uniformity in this example is found by matching the coefficient of uniformity (c_u) of "concrete sand," as shown in table A-1. Since "concrete sand" (ASTM C 33, fine aggregate) has shown good performance in the field (does not segregate), it is used as a guide for this selection (this procedure is not in Bureau of Reclamation's design standard).

Table A-1.—Selection of coefficient of uniformity for Zone 2B

Degree of Uniformity	"Concrete Sand"	Zone 2B
D_{60} / D_{10}	4.0	3.8 (1.63/0.425)

The third step is an estimation of the gradation limits (band width) for the filter. Since the $D_{10}F$ at this stage of the design is less than 20 mm, no requirement is given for the width of the prescribed gradation range (limits of gradation). Again, recognizing that "concrete sand" is limited to ranges no greater than 35 points, this limit is set for this filter (this step is not in Bureau of Reclamation's design standard). The trial gradation is given in table A-2 and plotted on figure A-7.

The permeability of this filter is checked against the foundation permeability later in this example.

Table A-2.—Gradation limits for Zone 2B

Sieve size	Percent passing, by weight
¾-inch	100
⅜-inch	90 - 100
No. 4	65 to 100
No. 8	40 to 75
No. 16	10 to 45
No. 30	0 to 15
No. 50	0 to 3
No. 100	0 to 2
No. 200	0 to 1

Drain Design

A drain material will be used to surround the perforated drainpipe in the downstream toe drain (the envelope). This material is bounded by two surrounding materials; perforation size of the pipe and the D_{85} size of the filter. Since this is a two stage filter and it is assumed the drain material will be uniformly graded, the following relationship is used for the perforation constraint (also see section 4.1.2).

$$\frac{D_{85} \text{ of the filter nearest the pipe}}{\text{perforation opening of pipe drain}} > 2 \text{ uniformly graded}$$

Assuming a perforation width = 10 mm (HDPE 12-inch diameter, ADS N-12 pipe, circular perforation, ADS Product Note 3.106 (2003):

$D_{85}E \geq (2)(10.0) = 20.0$ mm,
where:
$D_{85}E \geq 20.0$ mm (slot limit)
$D_{85}E$: finer side of envelope

Next, material size is determined against the base (filter). The first step in sizing the envelope against the filter is to determine the category of the filter. Since Zone 2B contains less than 15% fines as, shown on figure A-7, it is a "category 4 soil." The envelope criteria is:

$D_{15}E \leq D_{85}F$ (particle movement limit),

where:

D_{85}F: finer side of filter = 3.4 mm

D_{15}E ≤ 13.6 mm (particle movement limit)

D_{15}E ≥ D_{15}F (permeability limit), but, not less than 0.10 mm.

D_{15}E ≥ (5)(0.6) mm,

D_{15}E ≥ 3.0 mm (permeability limit)

D_{15}F: finer side filter = 0.6 mm

The D_{15}E limits for the range of acceptable envelope gradations are 3.0 mm. to 13.6 mm (i.e., 3.0 mm ≤ D_{15}E ≤ 13.6 mm), and D_{85}E limit is minimum 20.0 mm, for pipe perforation size. The limits are shown on figure A-7. The uniformity of the envelope (C_u = 2.80) was slightly more uniform than the filter.

A summary plot of the selected filter and drain materials is shown on figure A-7. The gradation specification is given for the Zone 3 drain material in table A-3.

Table A-3.—Gradation limits for Zone 3

Sieve size	Percent passing, by weight
¾-inch	100
⅜-inch	90 - 100
No. 4	65 to 100
No. 8	40 to 75
No. 16	10 to 45
No. 30	0 to 15
No. 50	0 to 3
No. 100	0 to 2
No. 200	0 to 1

Permeability Check

The permeability of the filter needs to be checked to see if its permeability is less than some material found in the foundation. Since this example consists of a relatively pervious foundation including highly pervious layers there is a concern that the filter could act as a barrier to these zones.

Examination of 17 gradations for the foundation materials range from GW–GM material at the upper bound, as shown in figure A-5, to SW-SM material near the midpoint of the gradation band. GW–GM materials are estimated to have a permeability in the range of 10,000 to 1,000,000 ft/yr (Bureau of Reclamation, 1987a) and the SW-SM soils are in the range of 100 to 30,000 ft/yr. The finer side of the filter (figure A-7) classifies as an SP and its range of permeability is 50 to 150,000 ft/yr. This indicates that the permeability of the filter equals the foundation permeability somewhere between the mid point and upper limit of the gradations.

Assuming the four coarsest samples produce a permeability greater than 150,000 ft/yr, then about 25% (4/17) of the foundation would be blocked by the filter. An increase in pore pressure would then be expected until equilibrium is reached. Since 25% is a relatively small portion of the foundation, the filter is deemed adequate. Additionally, in this instance because of site topography, the toe drain was 10 to 18 feet deep. Due to this deep burial there was no concern about excessive uplift pressures

Borrow Area

A local borrow site is available for use in producing filter and drain materials. Twelve test pits and three drillholes were used for characterization of this pit. The test pits ranged from 4.5 to 30 feet deep and the drillholes were about 40 feet deep. Laboratory analysis on forty-five samples (figure A-8) indicates that the material within the borrow site consists primarily of a Silty Sand (SM) to a Well Graded Gravel with Sand (GW)s. The percent of oversize material ranges from a trace to 30 percent cobbles and a trace to 20 percent boulders with maximum dimension of six feet. Simple statistics were produced on all samples (similar to figure A-4 described earlier) which results in the following categorization; gravel content varies from 0 to 76 percent, but generally ranges from 10 to 56 percent; sand content varies from 1 to 91 percent, but generally ranges from 31 to 76 percent, and fines content varies from 1 to 99 percent, but generally does not exceed 27 percent. The specific gradations of the borrow area (by grain size) are plotted on figure A-8.

Figure A-9 is a plot of the filter and drain materials (average) designed in the previous sections along with the average gradation of the borrow material. This plot is used to compare the available grain sizes against those that are required to produce the filter and drain. Note: "Oversize" material (material larger than 3-inches) has been removed from the data set. This is done because that material is not usable for the products in question. Additionally, a more detailed analysis was done for comparison of material demands from all produced materials against available material within the pit (not covered in this example). The demand is calculated by computing the weight required, per sieve, and deducting it from the available weight on a per sieve basis. Performing the analysis on a per sieve basis will illustrate which sieve (grain sizes) are used most and which are used least. Estimation can then be made on the amount of waste (from washing operations) and the amount of by-products produced (material that passes through the plant but is not used for any of the final products).

Figure A-9 illustrates that all grain sizes are available to produce the two required products. That is, no supplemental material will have to be brought in to complete the material. The figure also indicates that since the pit contains an average of 14% fines a washing operation will be required. Note that the 14% fines content is a product of scalping the sample to a minus 3-inch material and a number of samples that had very high fines content, as shown in figure A-8. At this site these samples were near the ground surface and were stripped prior to production. Examination of figure A-8 indicates that 7% fines (average) is a better indication of the actual amount of fines that would be expected during production.

Figure A-9 also indicates that a large amount of oversize material would be surplused (by products) from the screening operation. Once this was identified this material was specified for use as riprap, riprap bedding and slope protection. The more detailed analysis ('per sieve' analysis) also indicated that the medium sand sizes would be used most extensively. In order to reduce the total yardage that would have to be processed through the plant, a crushing operation was recommended.

Reference

Bureau of Reclamation, *Design of Small Dams*, 1987a.

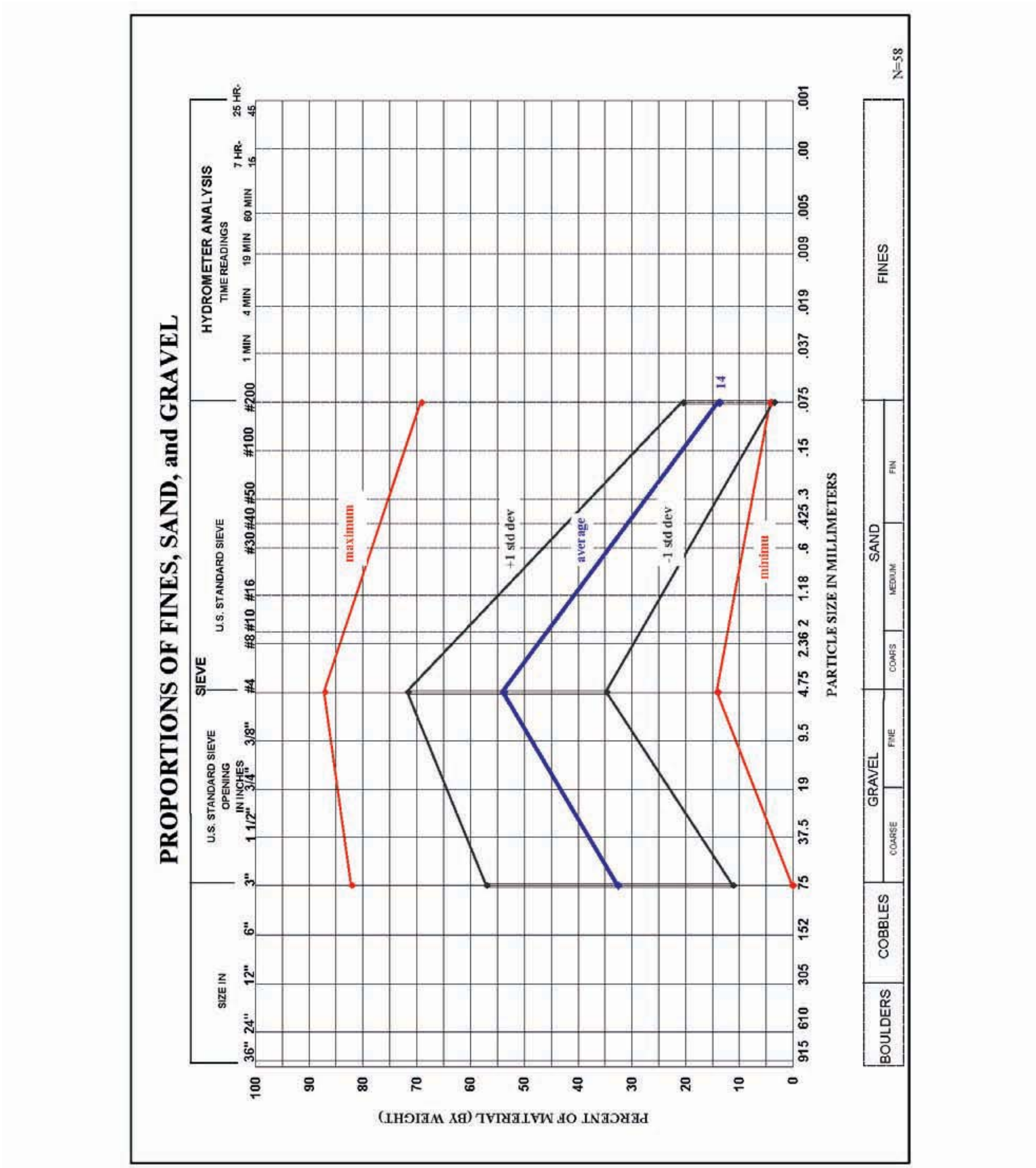

Figure A-4.—Proportion of fines, sand, and gravel for foundation soils.

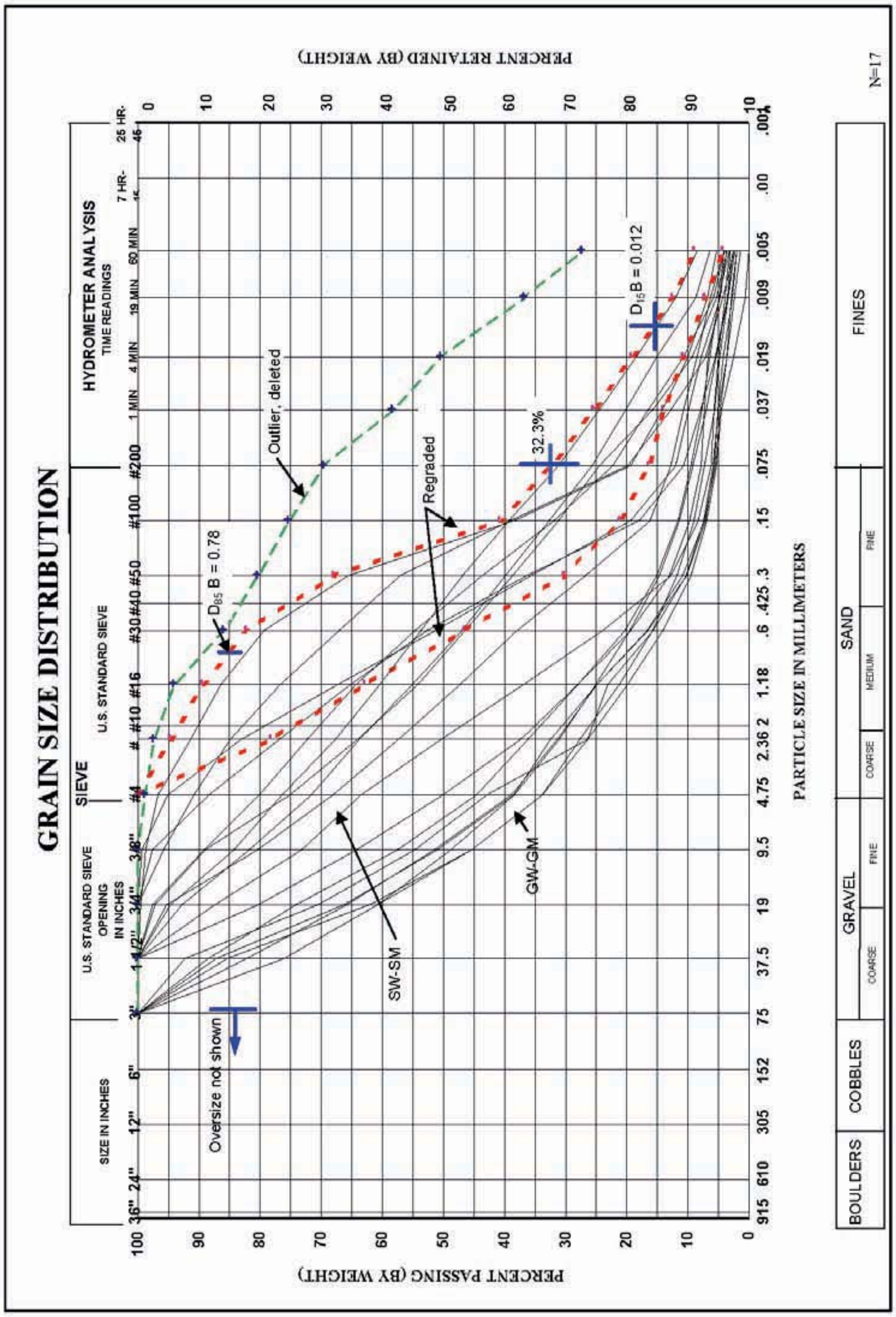

Figure A-5.—Individual gradations for foundation soil.

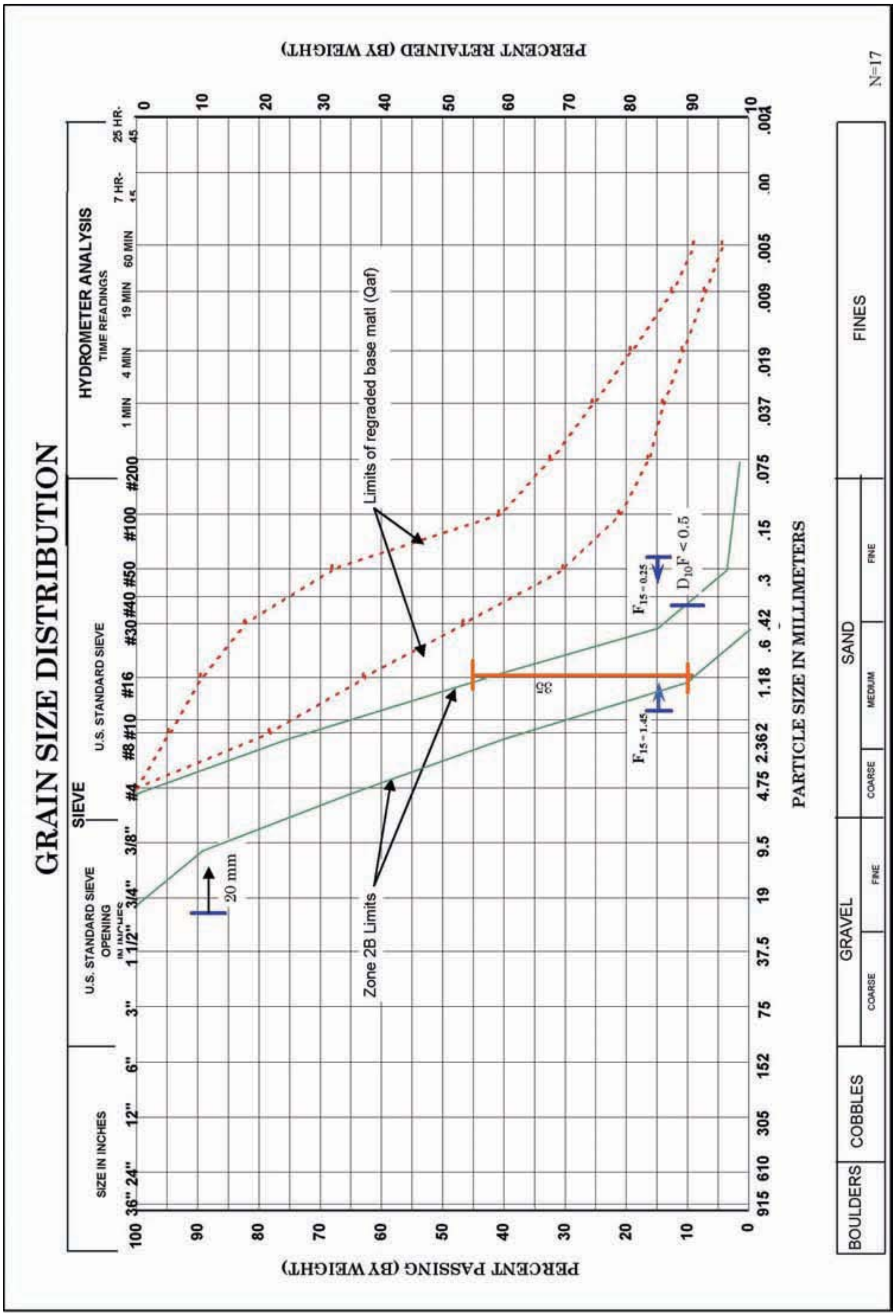

Figure A-6.—Design of filter material.

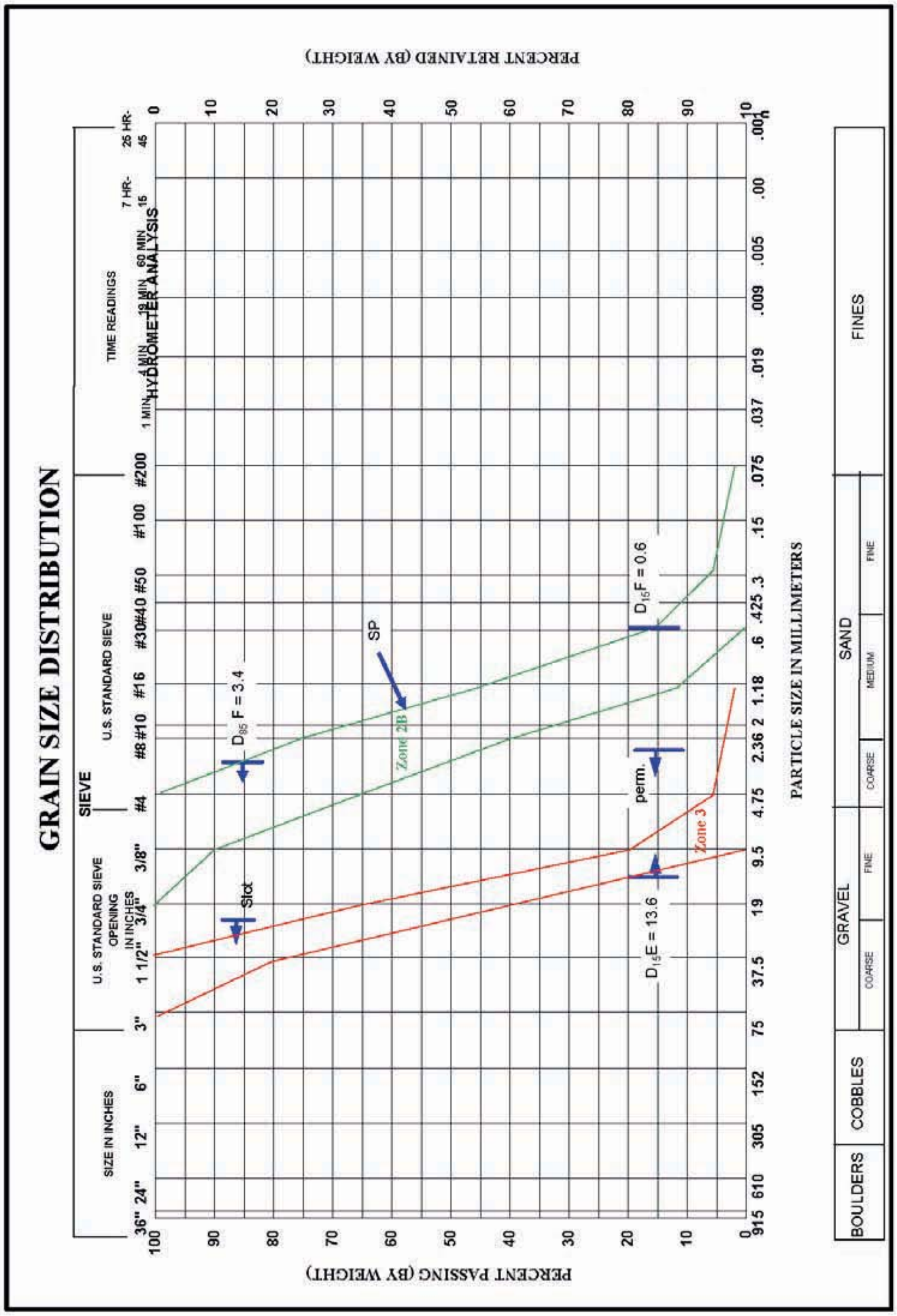

Figure A-7.—Design of drain material.

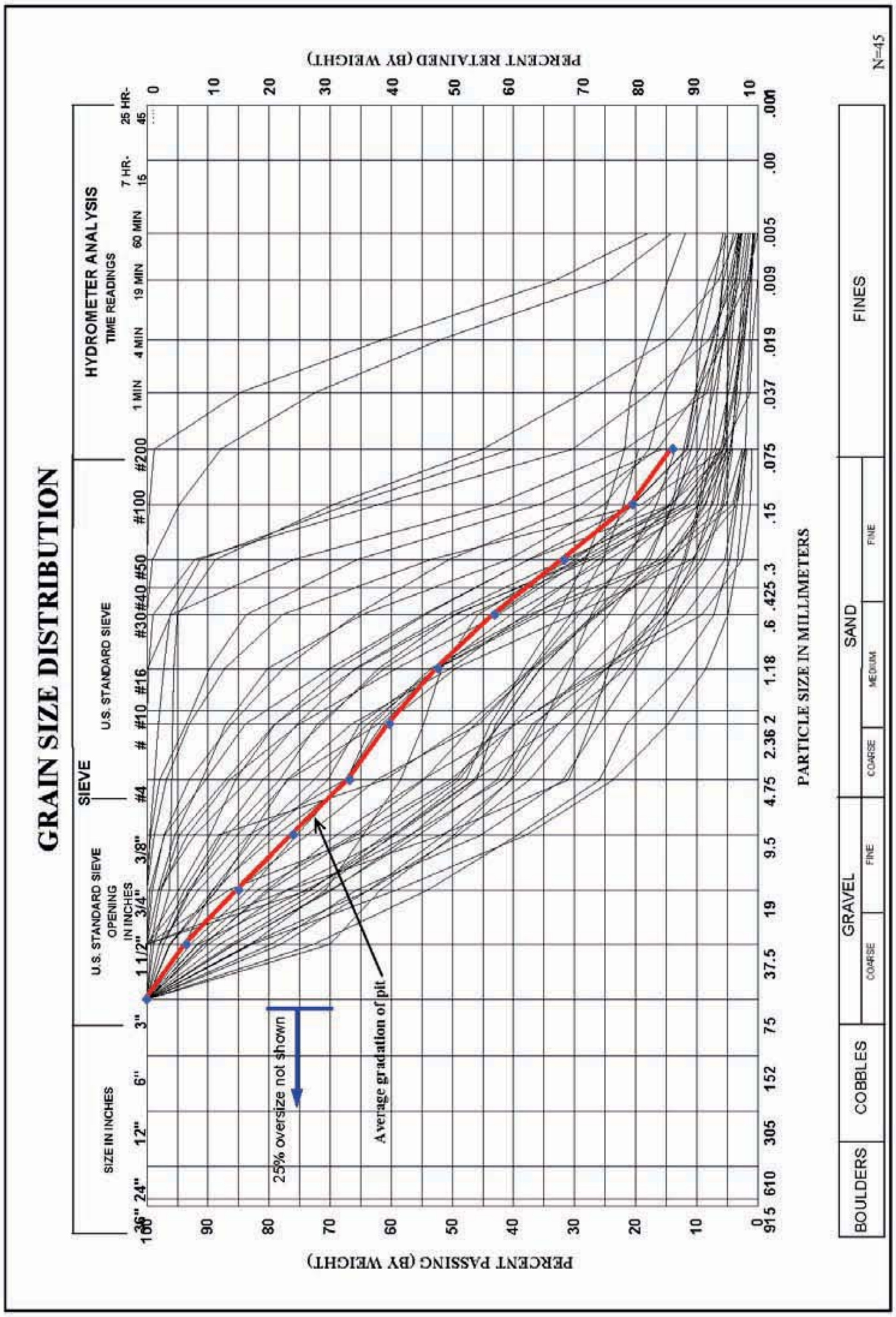

Figure A-8.—Individual gradations of borrow material.

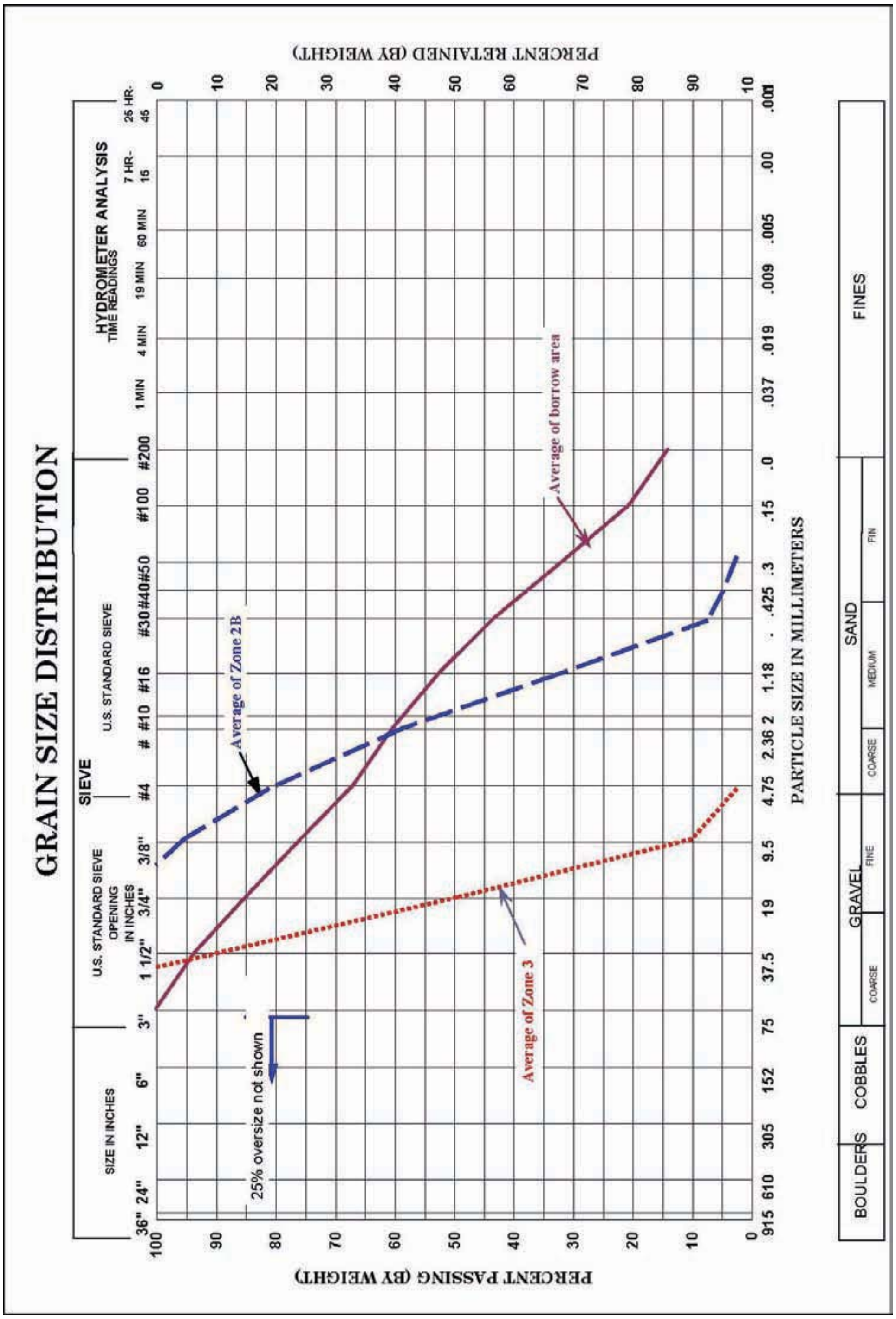

Figure A-9.—Average of filter, drain, and borrow materials.

Appendix B

Case Histories

Index

Additional case histories involving plastic pipe used in dams are available in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005).

Project: Davis Creek Dam

Location: Nebraska

Summary: CCTV inspection of a toe drain

Davis Creek Dam was completed in 1992 and is located about 6 miles southeast of North Loup in central Nebraska. The dam is a homogenous earthfill embankment with a structural height and crest length of approximately 110 feet and 3,000 feet, respectively. The toe drain system consists of two toe drains, one to the right of the outlet works centerline and another to the left of the outlet works centerline. The right and left toe drains consist, respectively, of approximately 1,200 feet of 8-inch-diameter and 1,440 feet of 12-inch-diameter perforated, corrugated polyethylene pipe. Flow from the right toe drain is measured by a V-notch weir, located about 30 feet to the right of the outlet works centerline in inspection well No. 7. Figure B-1 shows the toe drain layout. Flow from the left toe drain is measured by a V-notch weir located at the end of a weir box. The weir box is on the ground surface several hundred feet to the left of the outlet works centerline. The toe drains meet at the location of the toe drain outfall manhole, station 98+95, where they flow into the Jack Canyon drainpipe. The Jack Canyon drainpipe was constructed to carry toe drain discharges and surface runoff. The Jack Canyon drainpipe extends for about 1,100 feet and consists of 18-inch-diameter perforated, corrugated polyethylene drainpipe.

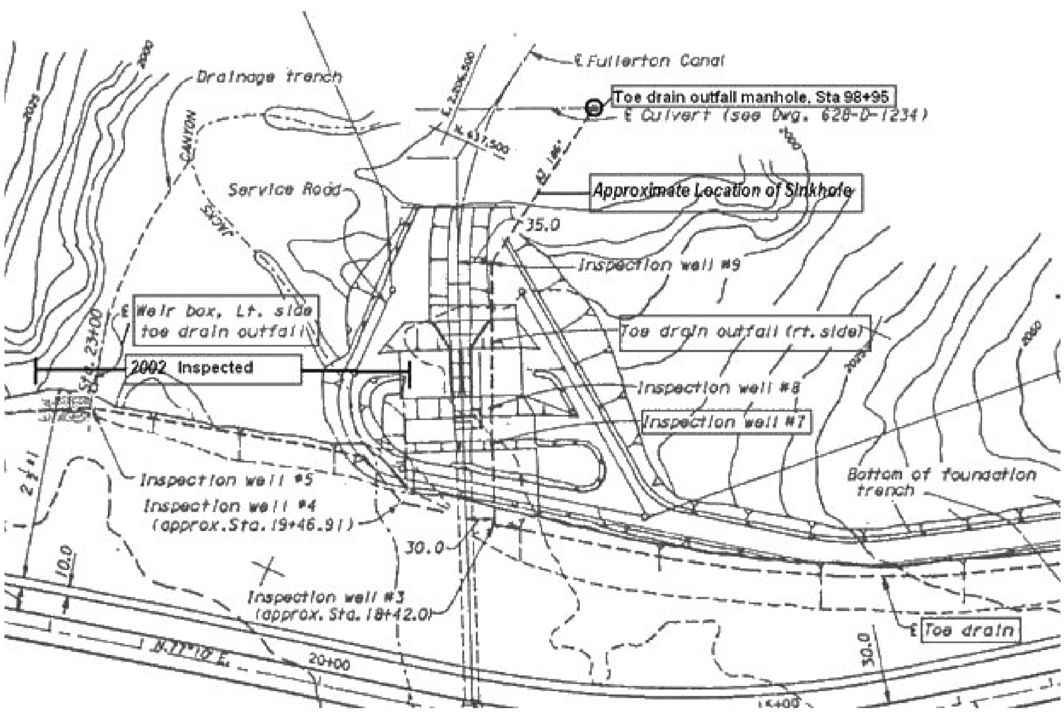

Figure B-1.—Locations of observation wells and cleaned reaches.

In the spring of 1994, a sinkhole 8 to 10 feet deep and approximately 20 feet wide developed above the 12-inch-diameter nonperforated, corrugated polyethylene outfall pipe. The sinkhole was located along the right outfall about midway between inspection well No. 9 and the Jack Canyon diversion drain culvert outlet transition. Drain rehabilitation in the fall of 1994 and the spring of 1995 consisted of replacing the 12-inch diameter outfall drainpipe with a 12-inch diameter perforated pipe placed within a gravel envelope.

In November of 2000, the Bureau of Reclamation performed a CCTV inspection of the toe drains at Davis Creek Dam as part of routine drain maintenance. Observations from the CCTV inspection showed areas of pipe buckling, other potentially damaged areas of pipe, and sediment deposition. Figure B-2 shows a typical amount of sediment deposition that was seen in the toe drain.

Based on CCTV inspection, selected reaches of the Davis Creek toe drains were cleaned using sewer cleaning equipment in January 2002. The reaches cleaned were located in the left toe drainpipe from stations 19+46.91 to 23+00 and from stations 23+00 to 26+00; however, care was taken not to wash out any of the materials from the locations where the pipe was damaged.

In February 2002, the Bureau of Reclamation inspected the cleaned reaches of pipe, including the short reach of the Davis Creek toe drain outfall replacement pipe and stations 98+95 to 99+12 of the Jack Canyon drain. The inspection of the left toe

Figure B-2.—The typical amount of sediment deposition observed in toe drain during the November 2000 inspection.

drain at Davis Creek was within the 12-inch-diameter pipe. The camera-crawler was inserted into the manhole at station 23+00, and proceeded downstream to station 19+46.91. The camera-crawler was then backed out and turned around in order to proceed upstream. The camera-crawler proceeded upstream to approximately station 23+25. Originally, it was intended to inspect the entire cleaned reach to station 26+00, but the camera-crawler was unable to proceed when it came across a section of buckled pipe that was previously reported during the 2000 inspection. Figure B-3 shows the results of drain cleaning and the pipe damage that halted the camera-crawler. This photograph was taken in the Davis Creek toe drain at approximately station 23+25. The fine materials previously seen on the pipe invert have been removed.

Lessons learned:

- In the short term, the cleaning was effective in removing most of the deposited sediments within the cleaned reaches. The long-term efficiency of the cleaning operation is unknown. No additional damage occurred inside the drainpipe because of the pressure jetting. Decreases in toe drain flows were not seen before cleaning, nor were higher flows seen immediately following cleaning. However, it should be noted that only a portion of the toe drain was cleaned. If the entire length were cleaned, the discharge rate might have increased. Also, it

Figure B-3.—A buckled left toe drain pipe at approximately Sta. 23+25 stopped the camera-crawler. A cleaning removed fine materials from the pipe invert.

is possible that the sediments in the toe drain are not controlling toe drain flows.

- In a 1994 field examination, it was concluded that the sinkhole developed from material being transported into an open, collapsed pipe. The collapse of the pipe could have occurred either from equipment load during construction or from earth pressure on the outside of the pipe. A CCTV inspection immediately following or during construction would have been helpful in pinpointing the cause of the pipe failures. Even though the cause of the sinkhole could not be pinpointed, the CCTV inspections in 2000 and 2002 were helpful in viewing the condition of the drainpipe and supporting the conclusion from the 1994 exam. Both inspections noted some pipe failures that could facilitate the development of sinkholes.

Reference:

Bureau of Reclamation, *Drainage for Dams and Associated Structures*, 2004.

Project: Ganado Dam

Location: Arizona

Summary: Toe drain installation in an embankment dam modification

Ganado Dam, originally constructed in the early 1900's, was raised 5.5 feet in 1943 for an approximate total height of 28 ft. Constructed of locally available dispersive soil on a dispersive foundation, numerous dam safety issues and poor performance led to the reservoir being drained in 1982.

The dispersive properties of the embankment and foundation led to a number of dam safety deficiencies including:

- Internal erosion initiated by seepage concentration through transverse cracks in the embankment.

- Erosion of dispersive material into porous regions of the dam foundation that can lead to piping.

- Erosion of dispersive material along structures that can lead to enlarged concentrated flow paths.

- Soil erosion and rilling of the downstream face of the embankment.

Most dispersive soils can be field identified by characteristic erosion features as shown on figures B-4 and B-5. This case history is an excellent example that even a low head structure was unable to store water due to the highly erosive nature of the soils.

The dam was rebuilt in its entirety, including embankment, outlet works, and spillway. Additionally, a toe drain system was added. Prior to draining the reservoir, seepage through and under the dam led to standing water downstream of the embankment. The redesign of the embankment included a cutoff trench in the foundation and the inclusion of chimney and blanket filters as shown in figure B-6. Since the foundation consisted of alternating clay, silt, and sand layers the intent was to include a cutoff of sufficient depth to engage at least several of the sand layers. This design feature minimized seepage and pore pressures in the downstream area of the dam.

The toe drain consisted of a 12-inch profile wall corrugated HDPE pipe surrounded by a two stage drainage system (gravel envelope surrounded by a sand filter). The cross section is shown in figure B-7. Profile wall corrugated pipe was selected due to its greater strength and smooth interior (single wall corrugated interior pipe can trap

Figure B-4.—Subsurface fissure located at the downstream toe of the dam east of the outlet works. The subsurface soil had a very high moisture content.

Figure B-5.—Erosional features typically associated with dispersive soils as seen at Ganado Dam. The common name for such features is "jughole."

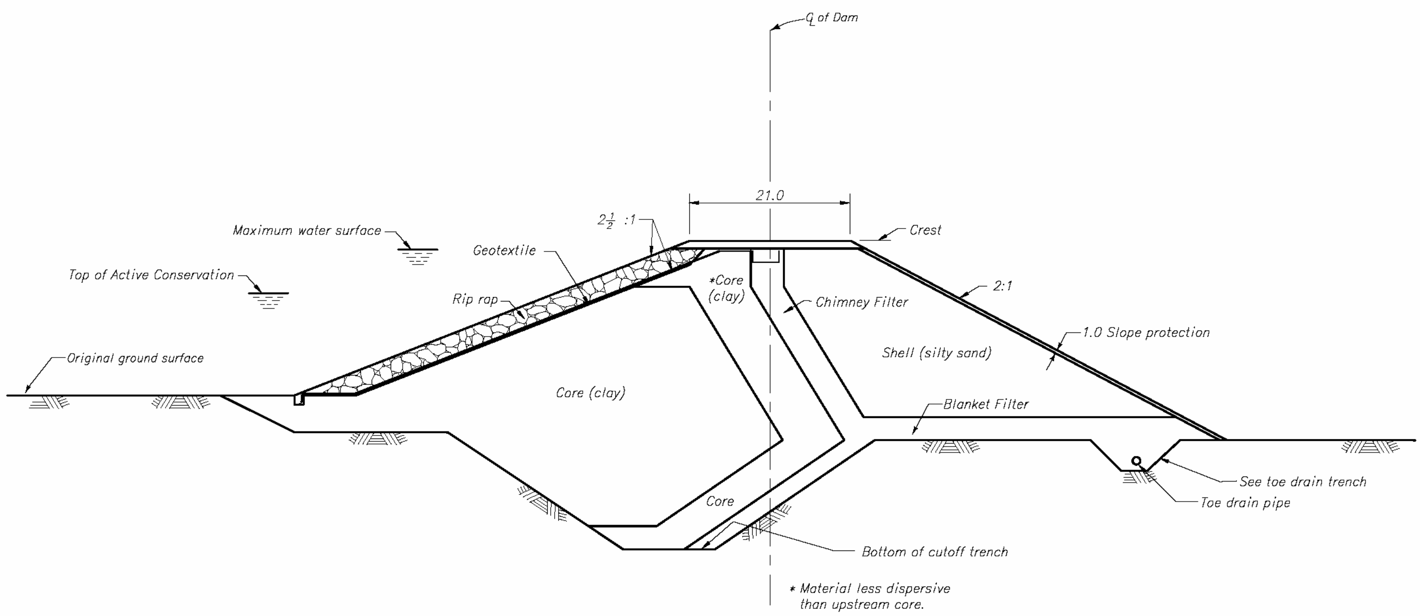

Figure B-6.—Modification cross showing embankment zones and toe drain system.

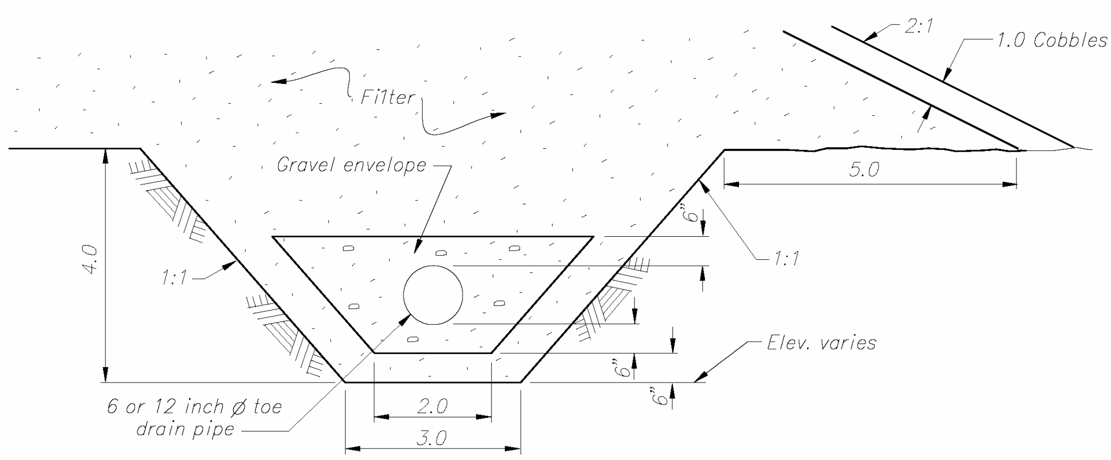

Figure B-7.—Cross section of toe drain.

sediment which may enter the pipe and is more difficult for CCTV examinations). The pipe diameter was selected at 12-inches as a minimum for camera crawler access since the predicted flows were expected to be quite small. Perforated pipe was used along the toe drain trench and nonperforated pipe was used for the outfalls. Clean outs and inspection wells were also included in the design for access to the toe drainpipe along with measurement devices and sediment traps. This arrangement allows for complete access to the toe drain system for monitoring and inspections.

Lessons learned:

An effective foundation cutoff minimized the amount of seepage past the dam allowing a minimal toe drain installation. The available on-site sand source, although abundant in quantity, was composed mostly of No. 100 fine sand (locally known as blow sand). While acceptable as a filter, this was on the fine side of the criteria and was marginal in meeting the permeability requirement of the design standard. This was judged not an issue since the cutoff and very low permeability of the core rendered the cross section nearly impervious. In fact there is excellent seepage attenuation through the cross section and the toe drains are all dry.

As recommended in the Bureau of Reclamation's *Protective Filters* (2007) a filter compatibility test was performed. The design standard recommends that specific filter material be tested for specific sites when dispersive base soils are present. This check was done for the Ganado work and the prescribed filter was found to be adequate.

The profile wall corrugated HDPE pipe was easily installed and capable of withstanding construction installation loads.

Reference:

Bureau of Reclamation, *Design Summary—Ganado Dam*, 1998.

Bureau of Reclamation, *Embankment Dams*, Design Standards No. 13, Chapter. 5, "Protective Filters," 2007.

Project: Sediment Control Pond SP-4 Dam

Location: Mississippi

Summary: A breach occurred due to internal erosion along an HDPE pipe spillway

Sediment Control Pond SP-4 Dam failed on February 13, 2004 when a 26-foot wide breach occurred at the location of the pond's spillway pipe. The failure occurred approximately 77 days after the facility began to impound water. Approximately 439 acre-feet of water were released as a result of the breach. No injuries or significant damage occurred.

The low hazard potential pond was constructed to control runoff and collect sediment from upstream mining operations. The earthen embankment had a crest width of 20 feet, maximum height of 29.5 feet, and an overall length of approximately 2,700 feet. The upstream and downstream slopes of the homogeneous embankment were sloped at 3 horizontal to 1 vertical. The embankment was constructed of clay and silty-clay soils. The PI values of soil used to backfill the pipe were in the range of 9 to 12. The majority of the embankment was constructed of CL soil with a PI in the range of 10 to 18.

The spillway pipe consisted of a drop inlet structure having a 60-inch diameter polymer-coated corrugated metal pipe riser and a 36-inch outside-diameter, SDR 17, HDPE conduit. Two slide gates on the side of the riser allowed for low level discharge. The length of the HDPE conduit was approximately 125 feet. Joints were butt fused. The pipe discharged into a plunge pool.

The pipe was installed by compacting fill to the bottom elevation of the pipe and then shaping the bedding by hand excavation, for a depth of approximately 6 inches, to conform to the shape of the pipe. The bedding was reportedly shaped until the workers achieved what they considered "reasonable contact." A transit and plywood template was used to maintain alignment and grade control. Workers reportedly rolled the pipe into and out of the cradle excavation several times to check if "full" contact was achieved between the pipe and the bedding. Backfill was then compacted in the haunch area in 6-inch lifts using powered hand-tampers. A walk-behind sheepsfoot roller was used to compact the remainder of the backfill to approximately 2 feet above the pipe. The fill was then raised above the pipe as the rest of the embankment was raised.

A seepage diaphragm was constructed approximately 25 feet downstream of the centerline of the embankment. The sand diaphragm was approximately 21 feet wide, 12 feet high, and 3 feet thick at the base (2 feet thick at the top). The diaphragm extended approximately 7 feet above the pipe, 10.5 feet to either side of the pipe's centerline, and less than 2 feet below the pipe. The pipe was located in the fill portion of the dam. Approximately 8 feet of foundation soil had been removed and replaced with compacted fill. The diaphragm extended less than 1.5 feet into this replacement fill.

A sand filter zone extended downstream of the diaphragm as bedding for the spillway pipe and to act as a drainage outlet for seepage collected by the diaphragm. This bedding layer was approximately 6 feet wide and extended approximately 1 foot below the pipe. A 3-foot "gravel plug" was constructed at the downstream end of this sand layer. A woven geotextile separated the "gravel plug" from a layer of riprap that lined a plunge pool at the pipe outlet.

The embankment had been completed on November 28, 2003. During January 2004, the slide gates were opened on three occasions to release water from the reservoir. On January 20, 2004, a member of the pump crew noticed vibrations at the downstream end of the spillway pipe while treating the pond with a flocculent. Until February, the highest reported water level in the reservoir was only 4.5 feet above the invert of the transport section of the spillway pipe. However, on February 5, 2004, over 4 inches of rain fell in the area.

A routine embankment inspection was performed on February 13, 2004 at 3:30 p.m. At this time, the water level had risen to the point where it was 2.5 feet below the top of the riser pipe, and 9.5 feet below the crest of the dam. The corresponding head on the invert of the transport section was approximately 8.5 feet. This was the highest water level that the reservoir had experienced. Nothing unusual was noted during this inspection. No seepage was observed around the periphery of the decant pipe at that time.

The failure occurred 5 hours later (figures B-8, B-9, and B-10). Witnesses indicated that, just prior to the failure, they observed a stream of water, described as being about 10 inches in diameter, exiting at the downstream toe of the embankment adjacent to the spillway pipe. At the same time, a vortex was observed in the reservoir near the point where the pipe intersected the upstream slope. Failure of the embankment occurred approximately 20 minutes after the water flow was first observed. At the time of the failure, no water was flowing through the spillway pipe.

The following postfailure observations were made:

- The fused pipe joints were intact, and their workmanship appeared to be of high quality.

- The seepage diaphragm that had been constructed 25 feet downstream of the embankment centerline was completely washed away in the failure.

Figure B-8.—Looking upstream through the breach. Embankment was 18 feet high at breach.

Figure B-9.—Breach showing 36-inch diameter HDPE pipe and riser.

Figure B-10.—View looking downstream along the pipe. The breach was 26 feet wide.

- The pipe bedding and backfill soil was indicated to be nondispersive based on pinhole tests (ASTM D 4647), crumb tests (ASTM D 6572), and double hydrometer testing (ASTM D 4221).

- The conditions observed immediately prior to the failure indicated that a direct flow path was present along the outside of the pipe or through the pipe backfill. As indicated below, several explanations are possible for the presence of a preferential flow path:

    1. Apparently the procedures used in installing the pipe did not achieve adequate contact between the bedding/backfill and the pipe, and/or adequate compaction of the backfill immediately around the pipe. Based on the low permeability of the backfill when compacted to the specified density, voids or zones of poorly compacted material were likely present in portions of the haunch areas, allowing the flow path to develop as quickly as it did.

    2. Observations of the compacted fill surface that remained after the failure show the surface prepared for placement of the pipe was highly compacted and did not bond properly to subsequent lifts. Sheepsfoot roller impressions were visible in the lift surfaces remaining after the failure. This condition is favorable to hydraulic fracturing, which could also explain the internal erosion flow path.

3. The sand used to construct the filter diaphragm was much finer and more poorly graded than ASTM C 33 concrete sand. The gradation used for the filter diaphragm is shown in figure B-11 (plotted on the same graph with ASTM C 33 sand). The finer sand used to construct the diaphragm was probably more likely to crack and to sustain an open crack because it would likely have poor self-healing characteristics and a high potential to bulk during placement.

4. The intent of filter diaphragms is to intercept any flow through preferential flow paths, and hydraulic fracture cracks. To effectively accomplish this, the diaphragm must extend well beyond the portion of the fill that could be affected by poor construction or hydraulic fracture. This filter diaphragm may not have been deep enough to encompass voids beneath the conduit or hydraulic fracture cracks at the contact between the conduit backfill and the remainder of the embankment. Current design criteria used by some agencies require the filter diaphragm to extend a distance equal to at least two times the outside diameter of the pipe below the pipe (NRCS, 2007). This would equal a distance of about 6 feet, but this diaphragm extended less than 2 feet below the conduit.

- Either the diaphragm or its outlet drain was overwhelmed by a large quantity of flow through defects under the haunches of the pipe, or flow bypassed the diaphragm. The head likely had reached a point where the gradient was sufficient to cause the uncontrolled flow along the pipe to carry away the embedment material, causing the soil above the pipe to collapse and erode, and the breach to develop.

Lessons learned:

- In the installation of a circular pipe, full contact between the pipe and the backfill is difficult to achieve, and compacting backfill in the haunch area is particularly difficult because the energy of backfill efforts can easily lift the pipe.

- If a pipe is not encased in concrete, then construction procedures, such as the shaping of the bedding for at least the lower third of the pipe diameter, must be used to ensure that full contact is achieved between the pipe and the surrounding soil and that the soil in the haunch area is adequately compacted.

- Filter diaphragms should have dimensions both horizontal and vertical that are extensive enough that flow in the vicinity of the conduit cannot circumvent the diaphragm, particularly under the diaphragm. The filter diaphragm did not extend deeply enough below the conduit according to current criteria used by many agencies.

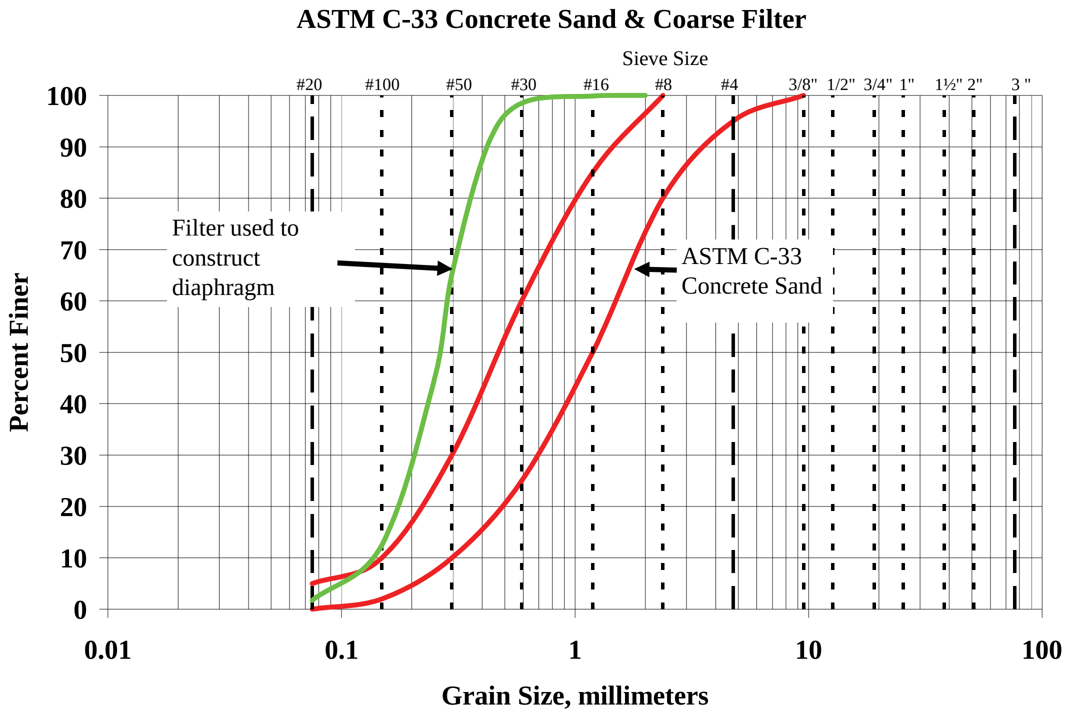

Figure B-11.—The gradation used for the filter diaphragm.

- The sand material used for the diaphragm was too fine and poorly graded to furnish properties considered desirable for filter diaphragms. Those properties are self-healing and a lack of bulking characteristics. ASTM C 33 concrete sand has been found to be an excellent filter for this purpose, but the filter used was significantly finer and more poorly graded than C 33 sand. The result was probably that the filter diaphragm could support an open crack and consequently could not fulfill the most important diaphragm function of collecting and filtering flow in the crack.

- The designer should monitor pipe installations to ensure that the specifications and the intent of the design are complied with, and that construction difficulties are adequately accounted for in the design requirements and construction specifications.

- The downstream area where the pipe exits the structure should be monitored closely for unusual quantities of seepage and evidence of internal erosion, especially during first filling of the reservoir.

References:

Mine Safety and Health Administration, *Investigation of Embankment Failure - Sediment Pond SP-4*, Report No. MW04-024, 2004.

Natural Resources Conservation Service, "Filter Diaphragms," *National Engineering Handbook*, Part 628, Chapter 45, 2007.

Project: Sugar Mill Dam

Location: Georgia

Summary: Poor construction practices lead to internal erosion along a siphon spillway

Sugar Mill Dam is a residential subdivision that was developed in the early 1990's in north Fulton County, Georgia (Atlanta metropolitan area). A central amenity of the development was an existing lake impounded by an old earthen embankment with inadequate spillway capacity.

In addition to widening the earthen emergency spillway, five PVC siphon pipes (ranging from 6 to 24-inches in diameter) were installed in a trench excavated through the crest of the embankment and terminating in a new wall at the toe of the dam (figure B-12). The design called for the pipes to be bedded in concrete. Control valves were installed in the siphons at the top of the dam, inside of manhole structures.

In 2002, about ten years after construction of the siphon spillway system, the dam owner noted water flowing out of a hole in the embankment adjacent to the siphons, approximately 15 feet downstream of the valve manhole.

Figure B-12.—In the early 1990's the spillway capacity of the dam was increased by construction of a system of 5 PVC siphons embedded in concrete in a shallow trench through the dam.

The owner contacted the designer for guidance. An internal CCTV inspection of the siphons found no problems with the PVC pipes, and the designer recommended that a filter drain system be constructed to control the seepage along the pipes. However, this did not work and the seepage situation continued to get worse. In 2003, the owner attempted to operate the siphon spillways during a storm, and found that the manholes were full of water and that the seepage flow along the siphons had substantially increased.

The designer suspected that flow was occurring under the pipes and recommended exploratory “surgery” in an attempt to locate the source of the seepage. After removal of the backfill over the pipes (figure B-13), a small hole drilled through the concrete between the siphons (figure B-14) revealed no voids and additional excavation was required. After portions of the siphon pipes and concrete bedding were removed it was found that the original contractor had not achieved adequate placement of the concrete bedding, and there were extensive voids under the center of each siphon pipe. Constant flow through these voids had caused internal erosion of the underlying embankment soils.

These sections of the siphons were replaced with new PVC pipe and the bedding was replaced with a higher slump concrete than was used originally (figure B-15).

Figure B-13.—After about 10 years of operation, seepage was observed on the downstream slope of the dam in the vicinity of the siphons, and the overlying embankment material was excavated to expose the pipes to determine the source of the seepage.

Figure B-14.—A small hole drilled through the concrete bedding between the siphons did not encounter voids under the concrete, even though the designer suspected that seepage was occurring directly under the pipes.

Figure B-15.—After removal of portions of the PVC siphons, it was determined that the original concrete bedding had been improperly placed, resulting in voids under the centers of the pipes. Portions of the siphons were replaced, and the bedding was replaced with a high slump concrete.

Lessons learned (adapted from Wilson and Monroe):

- Internal erosion of soils is a real-life occurrence.
- Neglecting minor details during construction can result in development of failure mechanisms.
- Successive attempts were made to find the source of the seepage problem in a cost effective manner.
- Good communication between the contractor, owner, and designer can result in a cost effective solution to a major problem, saving dollars for the owner and improving the safety of the dam.

References:

Sugar Mill Community Association, Minutes of Board of Directors meetings: April 18, 2002; May 7, 2002; and January 14, 2003.

Wilson, Charles and Joseph Monroe, *Dam Surgery—Repairs to Sugar Mill Dam, Fulton County, Georgia*, ASDSO Southeast Regional Conference, 2004.

Project: Upper Wheeler Reservoir Dam

Location: Washington

Summary: Collapse of HDPE pipe during grouting operation

In 1992, construction on Upper Wheeler Reservoir Dam included sliplining an old concrete box conduit with an HDPE pipe. The project also included extending the outlet downstream. The problem arose while grouting the annular space between the original box conduit and the new HDPE pipe.

The grouting operation consisted of pumping grout in from the downstream end of the box conduit, forcing the grout upstream. The contractor was successful in only grouting the lower 120 feet of the existing box conduit. When they reached the halfway point, the sides of the old box conduit failed at the lower end, resulting in the loss of about 5-6 cubic yards of concrete. Because the contractor could no longer continue the grouting operation from the downstream end, the equipment was relocated to the upstream end of the conduit. At the time, no one was aware that the pipe had collapsed. The fact that they could not release any water was the ultimate "smoking gun." Figure B-16 shows the grout tube in the original box conduit at the toe of the dam and the new outlet extension. Note: The State of Washington requires HDPE to be encased in concrete.

Figure B-16.—Looking upstream towards toe of dam: View of old box conduit and new downstream concrete encasement, during grout operation. Grout pipe is shown in top of photo.

Figure B-17.—Looking downstream from toe of dam after excavation of collapsed portion of HDPE pipe (located in original box conduit, upstream of new concrete encasement).

Calculations completed after the failure (figure B-17) showed that the HDPE pipe's resistance to external hydraulic pressures was much lower than the grouting pressures that were used. Due to the location of the grout pipe, high grout pressures were necessary to push the grout upstream.

Lessons learned:

- Get a specialty contractor experienced in grouting.
- Reviewers should get the specialty subcontractors documentation of the suitability of the grouting scheme and independently check their calculations of pipe stresses.

- If possible, the grouting of the downstream end of the pipe should be done using some form of slickline grout pipe inserted from the upstream end of the pipe.

- Avoid pumping grout up a pipe.

- If practical, use a low-density grout and fill the pipe to be encased with water during grouting.

Reference:

State of Washington Department of Ecology.

Project: Virginia Dam

Location: Virginia

Summary: Collapse on a HDPE pipe encased in concrete due to external hydrostatic pressure

A plastic pipe that was encased in concrete collapsed during first filling of a slurry impoundment in 1996. The decant conduit consisted of a 48-inch diameter, SDR 32.5 HDPE pipe encased in unreinforced concrete. The encasement had been formed around the pipe during conduit construction and the concrete had been placed in several separate sections along the pipe. Blocks had been placed under the pipe to hold it in position during the concrete placement. The encasement was square and 72 inches on each side. The concrete thickness was 16 inches above the pipe, 8 inches below the pipe and 12 inches at the springline (figure B-18).

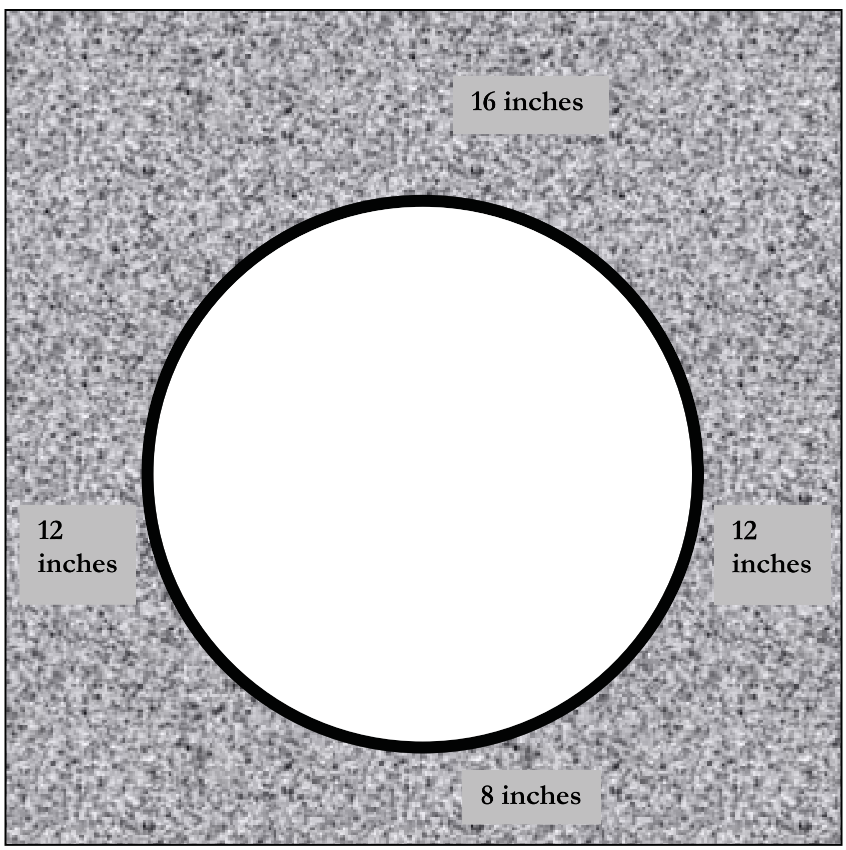

Figure B-18.—Position of 48-inch diameter, SDR 32.5 HDPE pipe in unreinforced concrete encasement.

Construction was completed in early May, 1996, and the reservoir began to fill in mid-May. No problems were evident up to and including an inspection of the pipe on May 27, 1996. On June 5, 1996, when the pool level had risen by approximately 50 feet, a discharge of approximately 300 gallons per minute was observed from the decant pipe, even though the pool was still three feet below the riser inlet elevation. Man-entry inspection of the pipe revealed that for a distance of approximately 25 feet, the bottom of the pipe had deformed upward, with the bottom of the pipe contacting the top of the pipe in one area. See figures B-19 and B-20. While inspecting the deformed area from the upstream end, running water could be heard entering the pipe farther downstream. Two days later, the pipe had deformed for a distance of approximately 250 linear feet. Eventually, the HDPE pipe became deformed along most of its encased length.

The plastic pipe had collapsed as a result of being subjected to hydrostatic pressure between the pipe and the concrete encasement. Possible entry points for the water included the joints between concrete placements, the contact area between the concrete and the blocks used to position the pipe, and cracks in the concrete. At the point where the collapse first occurred, the pipe was subjected to a potential head from the pool of approximately 81 feet, or a hydrostatic pressure of 35 lb/in^2.

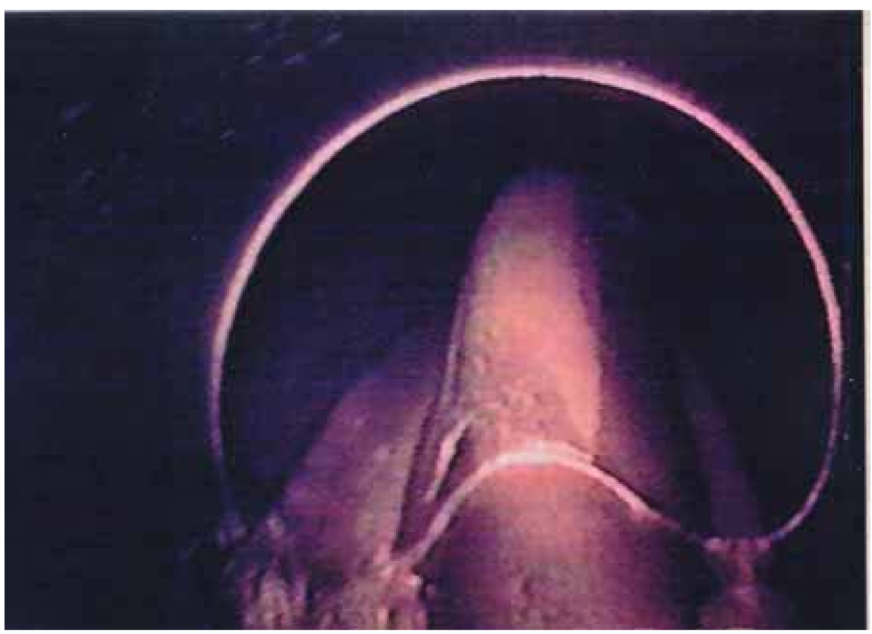

Figure B-19.—View of interior of deformed HDPE pipe from CCTV camera.

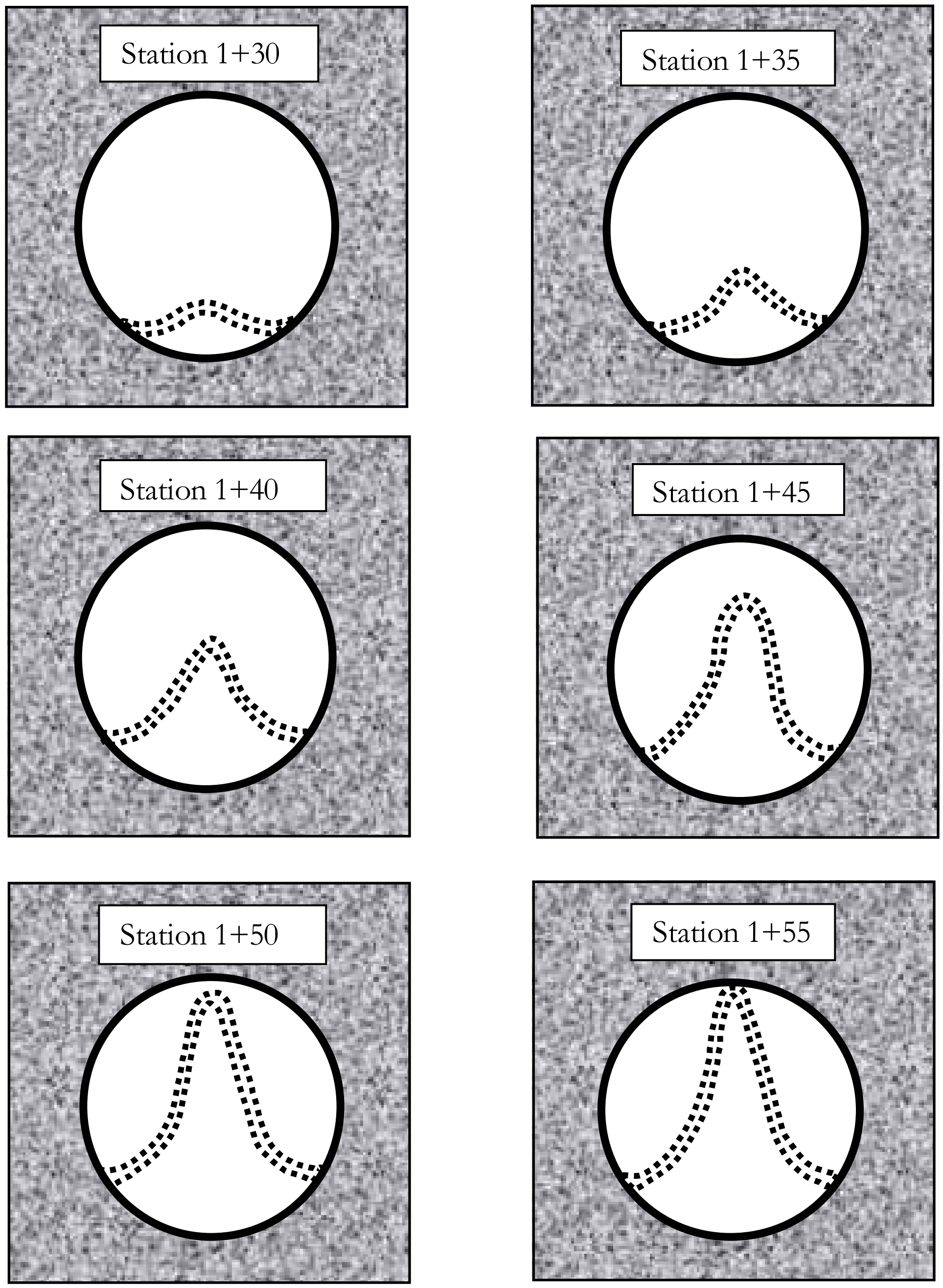

Figure B-20.—Approximate upward distortion of bottom of HDPE from outside hydrostatic pressure between the pipe and its concrete encasement.

Based on information supplied by HDPE pipe manufacturers, unrestrained SDR 32.5 pipe can collapse as a result of a short-duration external hydrostatic pressure of less than 7 lb/in^2, and a long-duration hydrostatic pressure of less than 3 lb/in^2. In a study by Jenkins and Kroll (1981), samples of SDR 32 polyethylene pipe which were encased in grout collapsed when subjected to short-term hydrostatic pressures, at the interface between the pipe and the grout, in the range of 32 to 34 lb/in^2.

As a result of the problem with the pipe, the reservoir was lowered by pumping, an open channel spillway was excavated, and the conduit and annulus were filled with grout and abandoned.

Lesson learned:

- Plastic pipe encased in concrete or grout must have sufficient strength to resist the external pressures to which it may be subjected. This includes pressures during the placement of the concrete or grout, as well as potential pressures from the reservoir. In this case, the concrete encasement had been evaluated for the earth loads, but the plastic pipe had not been designed to withstand hydrostatic pressure acting between the pipe and the encasement.

- Designs for encased pipes need to take into account that the pipe may become out-of-round or a flat spot may be created during the construction process. The floatation forces created during concrete pouring, for example, can cause deflection of the pipe and/or local deformation where the pipe is restrained. If a pipe is deflected or otherwise out of round, its resistance to collapse from outside hydrostatic pressure is reduced (Watkins, 2004).

Reference:

Jenkins, C.F. and A.E. Kroll, "External Hydrostatic Loading Polyethylene Pipe," *Proceedings of the International Conference on Underground Plastic Pipe*, ASCE, pp. 527-541, 1981.

Watkins, Reynold King, *Buried Pipe Encased in Concrete*, ASCE, 2004.

Project: Wheatfields Dam

Location: Arizona

Summary: Sliplining a deteriorating outlet works conduit using HDPE dual-wall containment pipe

Wheatfields Dam is an earthfill embankment located on the Navajo Indian Reservation in Arizona. The dam is an offstream storage facility used for irrigation and recreational purposes. Wheatfields Lake has a storage capacity of 3,880 acre-feet at the top of active conservation, elevation 7,296.6. Wheatfields Dam impounds flows from a small drainage basin on an unnamed tributary of Wheatfields Creek and diverted flows from Wheatfields Creek via a diversion canal.

The embankment is homogenous earthfill consisting of silt, clay, gravel, and cobbles and was constructed to crest elevation 7,302.1 in 1963. The dam has a crest length of 1,600 feet and a maximum structural height of 66 feet, a 36-foot crest width, and upstream and downstream slopes of roughly 3H:1V. A highly traveled two-lane paved highway crosses over the crest of the dam.

Appurtenant structures at the dam include a spillway and an outlet works. The spillway is an unlined trapezoidal cut excavated through a shallow ridge at the north end of Wheatfields Lake. The outlet works is located in the central portion of the embankment. The outlet works foundation consists of Pleistocene age alluvial red silty clay. The outlet works has two intake structures for low-level and irrigation releases. The low-level intake structure consists of a trashracked concrete box with an inlet sill elevation of 7,261.0 and is controlled by a 24-inch diameter slide gate. The low-level intake structure connects to approximately 290 feet of 24-inch diameter CMP that extends downstream to an exit portal. The irrigation intake structure consists of a trashracked concrete box with an inlet sill at elevation 7,292.0 and is controlled by a 24-inch diameter slide gate. The irrigation intake structure connects to approximately 30 feet of 24-inch diameter CMP that extends vertically downward to a location where it merges with the low-level outlet works. The irrigation intake was abandoned after the sliplining of the CMP. The outlet works has a computed discharge capacity of 41 ft^3/s (through the low-level intake only) when the reservoir water surface is at the spillway crest, elevation 7,296.6. The gate stem to the low-level gate is broken, making it inoperable. Figure B-21 shows the general configuration of the existing outlet works.

Modifications were required to the outlet works to address dam safety deficiencies and operational and maintenance issues. The primary deficiency involved separation of pipe joints and deterioration of the interior surface of the CMP. The existing condition of the conduit and the possibility of internal erosion of embankment materials either into or out of the conduit prompted concerns about increased risk

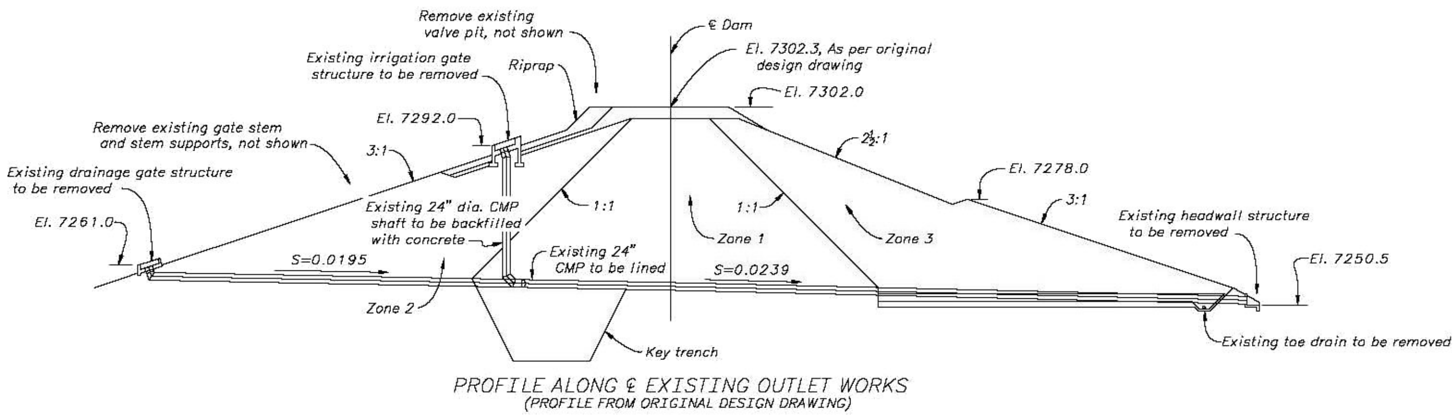

Figure B-21.—The profile of the existing outlet works prior to modifications.

for the development of a serious dam safety failure mode. As part of the modifications, new operational capabilities were added to supply pressurized flow to the outlet works to meet the need for future irrigation downstream from the dam.

After evaluation of a number of alternatives, sliplining of the outlet works conduit was selected as the best solution for eliminating the dam safety issues associated with the outlet works and accommodating future needs. Since the modified outlet works will have pressurized flow, it will be controlled by a rate-of-flow control valve located in a new downstream control structure during normal operations. For operations that require faster drawdown of the reservoir, the flow will be controlled by a ball valve located in the downstream control structure. The outlet works conduit will remain pressurized during the irrigation season. During the winter months, the upstream slide gate will be closed, and the liner will be drained. The ball valves will be left open in the winter to prevent freezing.

A dual-wall HDPE containment pipe was selected for sliplining. Dual-wall containment pipe provides structural integrity to the outlet works in addition to addressing potential internal erosion concerns by preventing seepage either into or out of the conduit. The potential for internal erosion along the outside of the CMP is addressed by using sufficient pressures during the grouting process to encourage grout travel through any existing small openings in the CMP and the construction of a downstream filter and drainage system.

A dual-wall HDPE containment pipe consisting of a 14-inch outside-diameter pipe in a 20-inch outside-diameter HDPE pipe was selected for sliplining and grouting into the existing 24-inch diameter CMP. In addition to sliplining, the upstream intake structure was removed and replaced, and a new downstream control structure was constructed. Figure B-22 shows the general configuration of the modified outlet works.

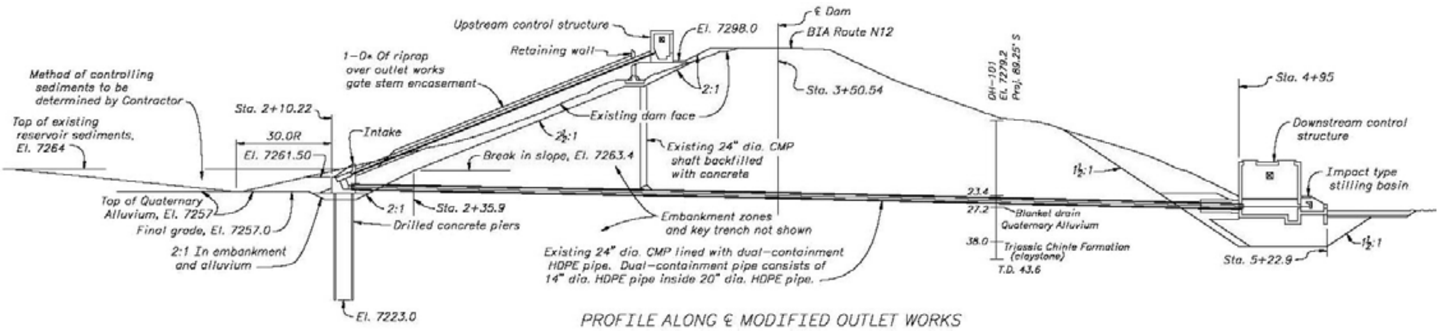

Figure B-22.—The profile of the modified outlet works.

The controlling hydraulic design factor of the modified outlet works was the maximum diameter of dual-wall containment pipe that could be inserted into the existing CMP. During initial design, the outside diameter of the HDPE lining was made about 10 percent smaller than the pipe to be lined. However, to facilitate installation and annular grouting, a dual-wall containment pipe consisting of a 14-inch outside-diameter carrier pipe (approximate inside diameter 12.9 inches) inside a 20-inch outside-diameter containment pipe (approximate inside diameter 18.5 inches) was selected. The inside dimensions of the existing CMP were carefully measured using a CCTV crawler-camera with a template attached to it to be sure there were no obstructions or deformations within the CMP that would prevent a 20-inch-diameter pipe from being installed.

The following loading conditions and methods of analysis were used for design of the slipliner:

- External loading equal to the maximum embankment load with no consideration given for the existing CMP and no internal pressure. Maximum embankment depth was assumed to be 50 feet. The HDPE pipe was analyzed for wall crushing, wall buckling, and ring deflection.

- Internal loading equal to the hydrostatic loading with the reservoir at the dam crest without side support from the surrounding embankment. Maximum hydraulic head rounded to 50 feet. Seventy percent of the strength of the internal 14-inch diameter pipe was used in the pressure calculations. This criterion was based on the manufacturer's recommendation for the dual-wall containment pipe. The strength is reduced because the fusion welds of the containment pipe cannot be inspected from the interior of the pipe. The welds will be visually inspected by CCTV before the pipe is grouted into the CMP.

The following material properties were used in design:

- Hydrostatic design stress = 800 lb/in^2

- Standard dimension ratio = 26
- Linear thermal expansion coefficient = 1.2 x 10^{-4} in/in/°F
- Short term modulus of elasticity = 100,000 lb/in^2
- Compressive yield strength = 1,500 lb/in^2

Construction began in January 2005. The reservoir was drained prior to the start of construction. The existing outlet works conduit was pressure washed and the cleaning verified by CCTV inspection. During construction, it was decided to use two separate pipes to form the dual-wall containment pipe rather than using a fabricated prejoined system. A McElroy track-star 500 Series fusion welder was used to butt fusion weld the sections of HDPE pipe together. A John Deere 230LC excavator (figure B-23) and Case 580-super L backhoe were utilized for moving the HDPE pipe from the staging area and for placing the pipe onto the fusion machine. To facilitate butt fusion and avoid construction congestion, the 14-inch diameter pipe was joined together on the downstream side of the dam and the 20-inch diameter pipe was joined together on the upstream side of the dam. The ¾-inch diameter HDPE grout lines were attached to the exterior surface of the containment pipe using a Munsch MA-40-B hand extrusion welding gun. Spacers were placed on 8-foot centers to center the 14-inch diameter pipe within the 20-inch diameter pipe. Additional ¾-inch diameter pipe was extrusion welded onto the bottom quadrant of the containment pipe to act as centering skids.

The excavator was used for guiding the 20-inch diameter pipe into the CMP at the upstream end of the conduit (figure B-24), while the backhoe pulled the pipe using a specially designed steel pulling head attached to the pipe pulled from the downstream end. The installation process was reversed for pulling the 14-inch diameter pipe into the 20-inch diameter pipe. After installation of both pipes, water from the reservoir was used to separately fill each pipe for hydrostatic pressure testing.

Bulkheads were constructed at the upstream and downstream ends of the outlet works conduit for grouting of the annular space between the 20-inch diameter pipe and the existing 24-inch diameter CMP. A grouting subcontractor was used for the grouting operations. Two identical grout plants were made available onsite, with one plant serving as the backup in case it was needed. Grouting was performed through four ¾-inch diameter HDPE pipes extrusion welded to the crown of the 20-inch dual-wall containment pipe of different lengths. The four lengths are 25.5 feet, 60.5 feet, 90.5 feet, and 120.5 feet. One additional ¾-inch diameter HDPE pipe was also welded to the crown of the 20-inch dual-wall containment pipe and used as an air vent. An initial grout mix of 4,000 lb/in^2 with 0.6:1 w/c (water-cement ratio by volume) and super plasticizer was used. After grouting operations began, it was

Figure B-23.—HDPE pipe being unloaded at the site using a John Deere 230LC excavator.

Figure B-24.—Guiding the 20-inch pipe into the upstream end of the CMP.

determined that the grout mix could not be injected into the ¾-inch diameter grout pipe at the prescribed grout pressure of 5 lb/in^2. The grout mix was changed to 0.8:1 w/c with the amount of superplasticizer increased and the pumping pressure slowly increased to 25 lb/in^2. Additional modifications to grout mix and pumping pressure were required to maintain a constant injection rate. The entire grouting process took about 8 hours for injection of 340 bags of cement.

The modifications, including sliplining, construction of new upstream and downstream structures, and other toe drain modifications, were completed in September 2005. A CCTV inspection was performed after the completion of construction and showed no problems.

Lessons learned:

The color of the installed HDPE dual-wall containment pipe was white. This was selected to improve inspection using CCTV equipment. However, it was later found that the white causes too much contrast and gray or black is better suited for CCTV inspection.

Grout pipes should be 1-inch diameter rather than ¾-inch diameter to facilitate grouting of the annulus.

References:

Bureau of Reclamation, *Wheatfields Design Summary*, 2006.

Project: Worster Dam

Location: Colorado

Summary: Renovation of an existing outlet works using an HDPE slipliner grouted in place

Worster Dam and Reservoir, also known as Eaton Dam and Reservoir, are located in Larimer County northwest of Fort Collins, Colorado. The dam and reservoir is located on Sheep Creek, a tributary to the north fork of the Cache La Poudre River in a mountainous area about five miles south of the Colorado-Wyoming border.

Worster Dam is a concrete face rockfill dam constructed in the early 1900's. The exact year of construction is not known. The dam is about 72 feet in height with a crest width of about 12 feet and a crest length of over 700 feet. The impounded reservoir has a maximum storage capacity of about 3,750 acre-feet.

The outlet works consists of reinforced cast-in-place concrete arched pipe with a central gate chamber. The inlet conduit consists of a 36- to 38-inch wide and 37-inch high reinforced concrete arch. The inlet conduit then connects to a central gate chamber. The gate chamber is approximately 7 feet in height, approximately 3 to 6½ feet wide and about 10 feet long. Flow was controlled by two 36-inch diameter slide gates housed in the gate chamber. The gate stems extend vertically through the embankment with the operators located at the dam crest. The gate chamber discharges to a larger 48-inch by 48-inch reinforced concrete arch conduit and directly to the stream.

An evaluation of the existing outlet works was performed by Woodward-Clyde Consultants (heritage firm to URS Corporation). The evaluation found that the central gate chamber appeared to be in relatively good condition. However, the evaluation also found that the outlet works conduit upstream and downstream of the gate chamber was composed of extremely poor quality concrete. Areas of the inlet structure had exposed steel and the steel reinforcement was severely corroded. Furthermore, much of the cement paste had been dissolved by the aggressive water.

Renovation of the outlet works consisted of replacing the inlet structure and guard gate, demolishing the slide gates in the central chamber, and sliplining the outlet works with an HDPE pipe. The new HDPE outlet works conduit was designed to withstand the full reservoir for external water pressure (buckling) and internal reservoir pressure. The upstream portion of the outlet works conduit was lined with a 30-inch diameter SDR 17 HDPE pipe grouted with a 2,000 lb/in^2 cement grout. The downstream portion of the conduit was lined with 42-inch diameter SDR 17 HDPE pipe grouted in place. The 30-inch and 42-inch pipes were connected using an eccentric reducer. The upstream end of the inlet structure was connected to a steel elbow using a Dresser coupling and a steel elbow with air vent was installed at the inlet with a 30-inch diameter reducing thimble.

The HDPE pipe was welded and assembled prior to installation. The conduit was then pushed/pulled into place using two Caterpillar D8 dozers. The integrity of the installed pipe was verified by pressure testing. Once a satisfactory condition was established, bulkheads were constructed at the upstream and downstream ends of the conduit. Grout discharge pipes of various lengths were installed on the downstream and upstream bulkheads and within the existing casing for the central grade chamber valve stems. In all, eight grout introduction points were established and utilized during the grouting process.

Flange ends were installed at the inlet and outlet, and the outlet pipe was filled with water. A pressure gauge was installed to monitor external grout pressures to verify that external pressures did not exceed the design pipe load.

The grout was slowly introduced at the downstream end. A combination of sand cement grout and neat cement grout with a superplasticizer were introduced at various times throughout the grouting process. The mix water was obtained from the reservoir. The mix water was near freezing because the source was a recent snowmelt. The cold grout resulted in constriction of the HDPE pipe. During the grouting process, the pipe water pressure began to increase. Grouting activities were suspended to allow the grout pressures to dissipate. Much to the surprise of the construction team, the water pressure continued to rise as the pipe continued to contract in response to the low grout temperature. Eventually the pressure became sufficiently high that the end couplings slipped, relieving the pressure. The internal water pressure returned to near zero pressure, the pressure remained stable, and grouting resumed. The temperature in the pipe stabilized and no further increases or decreases within the pipe pressure were observed. Once the grout was allowed to set up for 72 hours, the flanges were removed from the pipe and the outlet works was placed into service.

Lessons learned:

The following lessons were applicable to the construction described above.

- Filling a pipe with water is an effective means to reduce risk of pipe collapse during grouting.

- Continuous monitoring of the internal pipe water pressure should occur.

- Plans should be made to consider and monitor the grout mix temperature. The subject project observed a contraction of the pipe due to the low water pressure. The opposite could occur in which the grout temperature may cause expansion of the HDPE pipe, resulting in a negative pressure in the pipe.

- All of the eight grout introduction points were utilized during the grouting process. The success of the grouting project was based on the ability to introduce grout at these eight locations and fewer locations would have resulted in a failure to establish a complete grout seal around the pipe.

Reference:

Christopher N. Hatton, P.E., Woodward-Clyde Consultants, 2005.