Technical Manual: Plastic Pipe Used in Embankment Dams

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

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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:

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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.

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

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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.

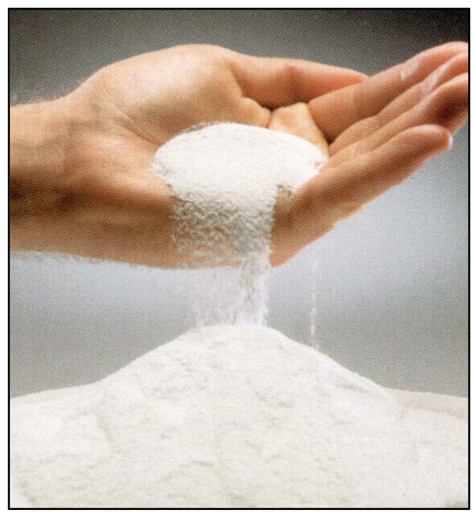


Figure 2.—Resin. Photo courtesy of Uni-Bell PVC Pipe Association Members and Associate Members.

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

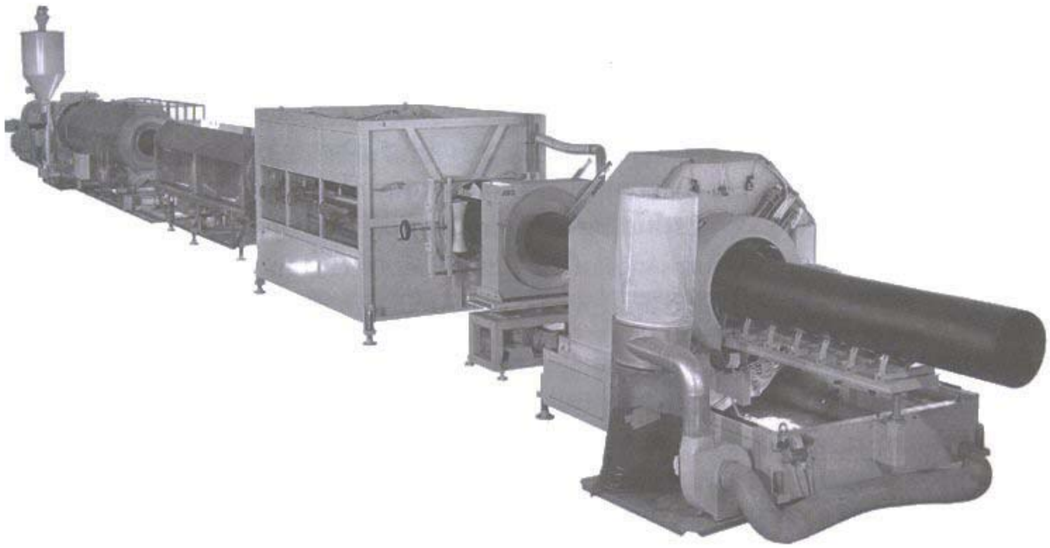


Figure 3.—Conventional extrusion line. Photo courtesy of the Plastic Pipe Institute.

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

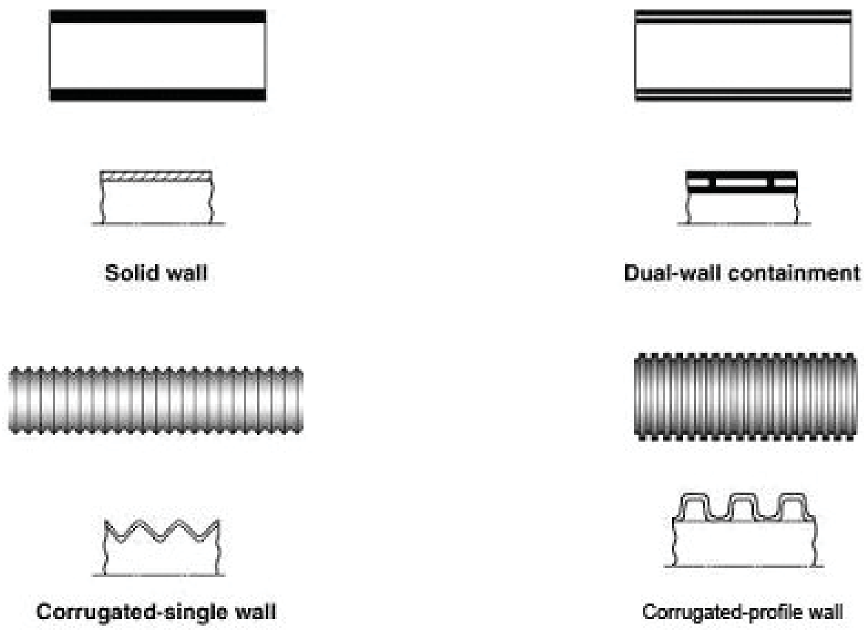


Figure 4.—Types of HDPE pipe walls.

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.

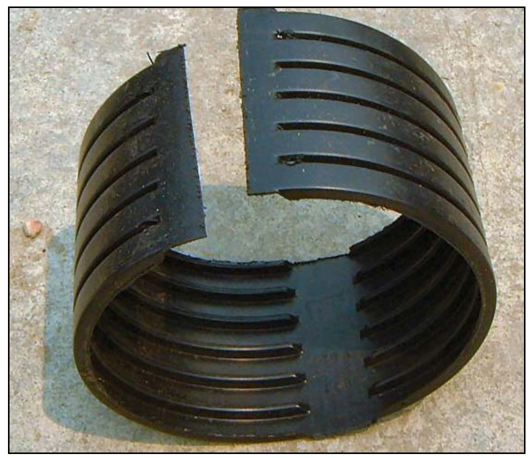


Figure 14.—External split coupler.

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.

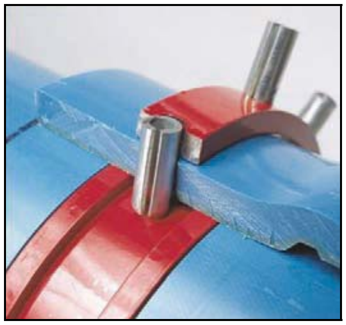


Figure 19.—This type of mechanically restrained joint is used to prevent overinsertion of bell and spigot gasket PVC pipe. Photo courtesy of Uni-Bell PVC Pipe Association Members and Associate Members.

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

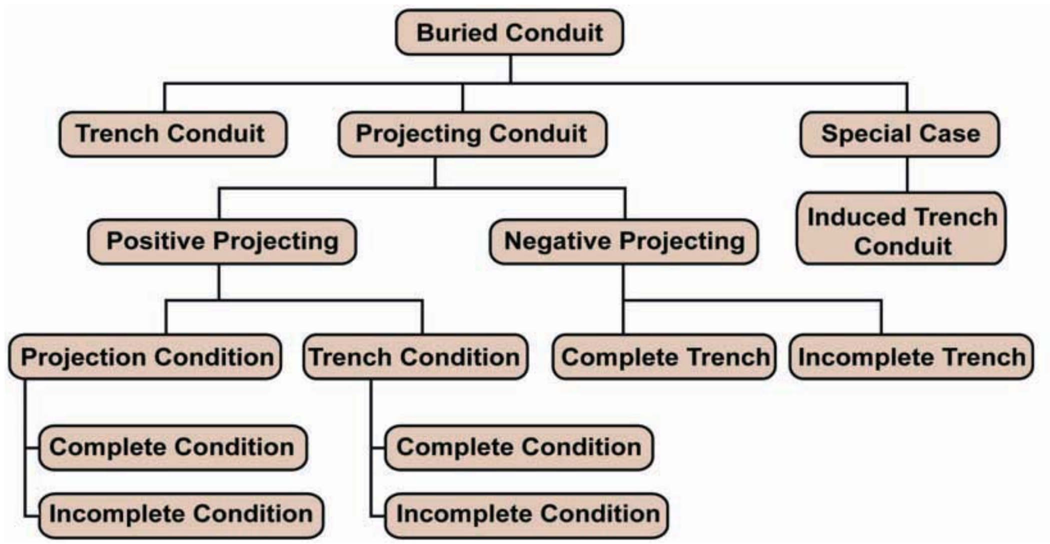


Figure 26.—Classification of buried conduits for the Marston theory (Spangler and Handy's *Soil Engineering,* 1982). Note: Use of incomplete trench and special case should not be used for embankment dam applications.

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

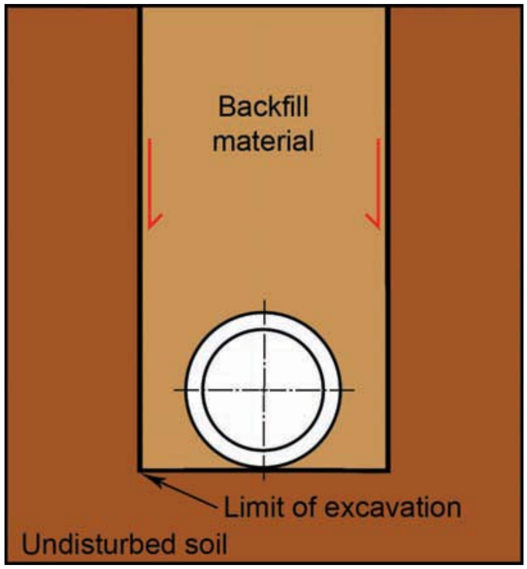


Figure 27.—Trench conduit. Backfill soil moves downward relative to the soil at the side of the trench. A drainpipe buried beneath natural ground is an example of a trench conduit. (Note: A trench conduit should not be used for an embankment conduit.)

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.

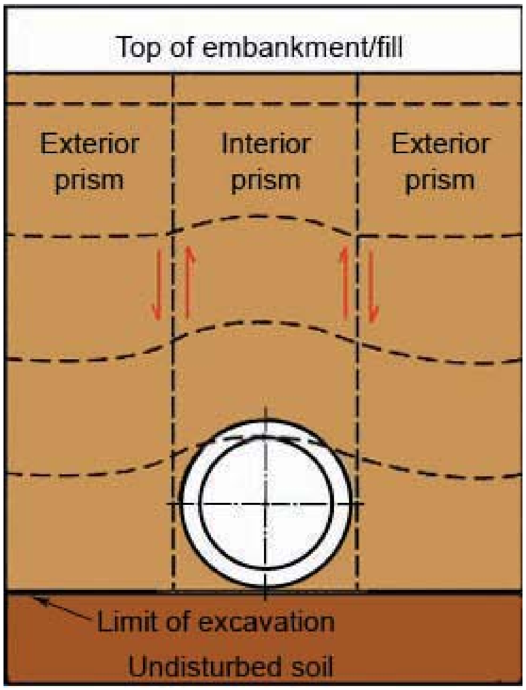


Figure 28.—Positive projecting conduit in a projection condition. The pipe is installed above the ground surface or compacted fill, with fill placed around and above the conduit. The exterior prisms settle more than the interior prism, causing load to be transferred to the interior prism. An embankment 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.)

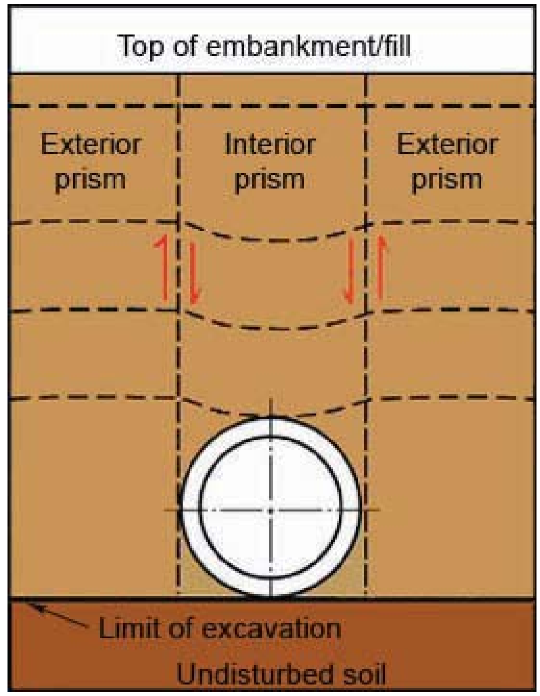


Figure 29.—Positive projecting conduit in a trench condition. If the foundation is yielding or the conduit deflects, the interior prism settles more than the exterior prisms. The soil load on the conduit is less than the weight of the soil above it. A drainpipe in an embankment dam is an example of a 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

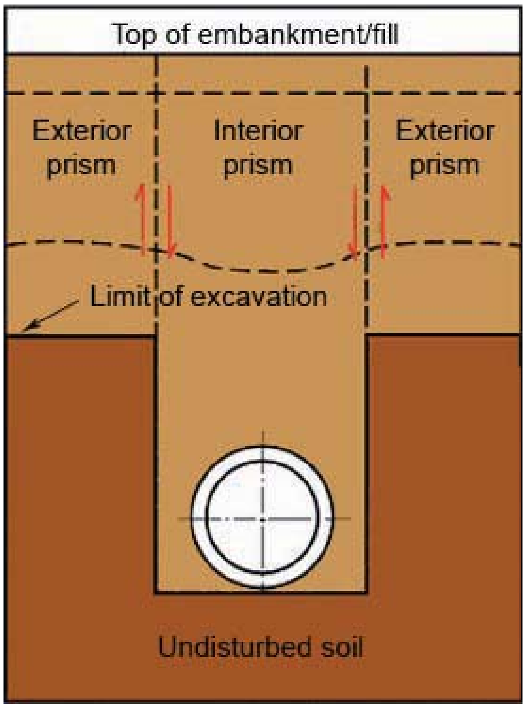


Figure 30.—Negative projecting conduit. The pipe is installed in a shallow trench such that the top of the pipe is below natural ground or compacted fill, and then covered with fill material. Negative projecting conduits should not be used for embankment conduits or drainpipes (Note: This figure is not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

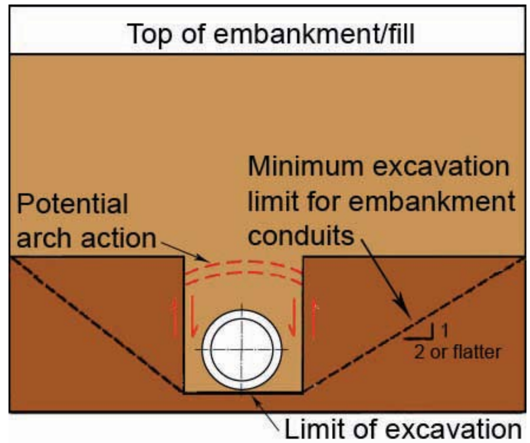


Figure 31.—Arching action of a negative projecting conduit. Negative projecting conduits should not be used for embankment conduits. For an embankment conduit, an excavation with 2:1 side slopes or flatter should be used. This causes the conduit to behave as a positive projecting conduit. (Note: This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

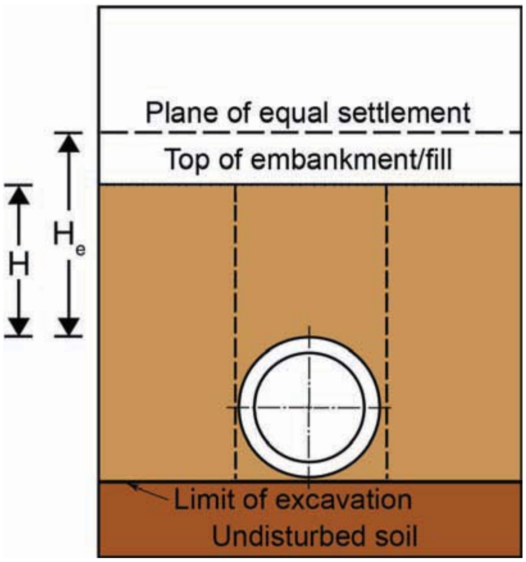


Figure 32.—Complete condition. The complete condition exists when the fill height (H) is less than or equal to the height to the plane of equal settlement (H_e). (Note: This figure is not intended to show all the design details required.)

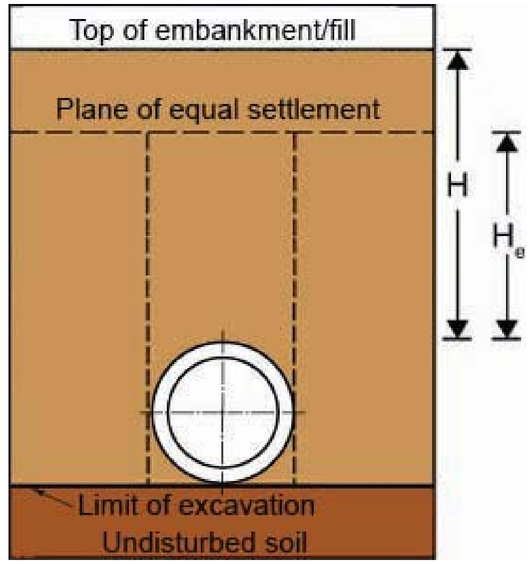


Figure 33.—Incomplete condition. The incomplete condition exists when the fill height (H) is greater than the height to the plane of equal settlement (H_e). (Note: This figure is not intended to show all the design details required.)

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.

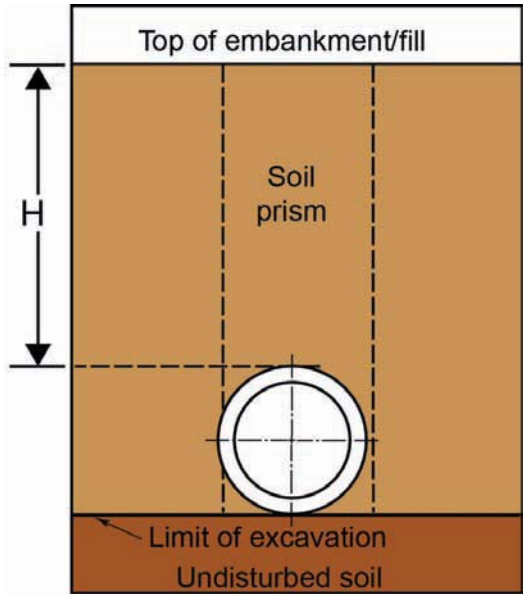


Figure 34.—The soil prism load is the weight of the soil directly above the conduit. (Note: This figure is not intended to show all the design details required.)

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

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

or

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

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 ϕ
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.

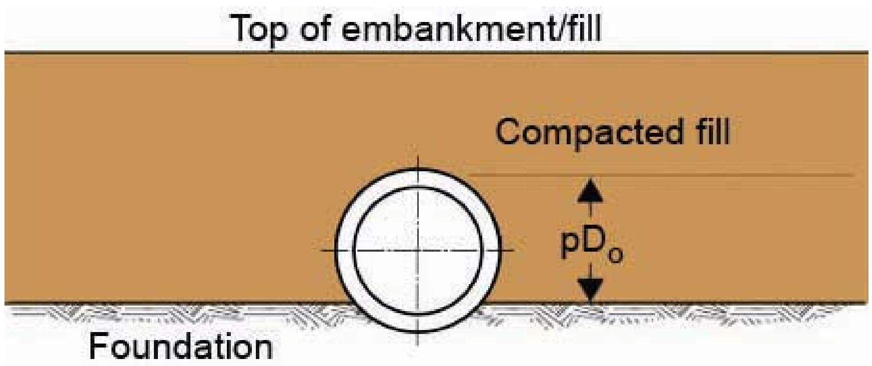


Figure 35.—Projection ratio, p = depth of the foundation material below the top of the conduit divided by the outside diameter of the pipe (D_O). (Note: This figure is not intended to show all the design details required.)

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:

$$P_s = \frac{W_c}{D_O} \tag{2-6}$$

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:

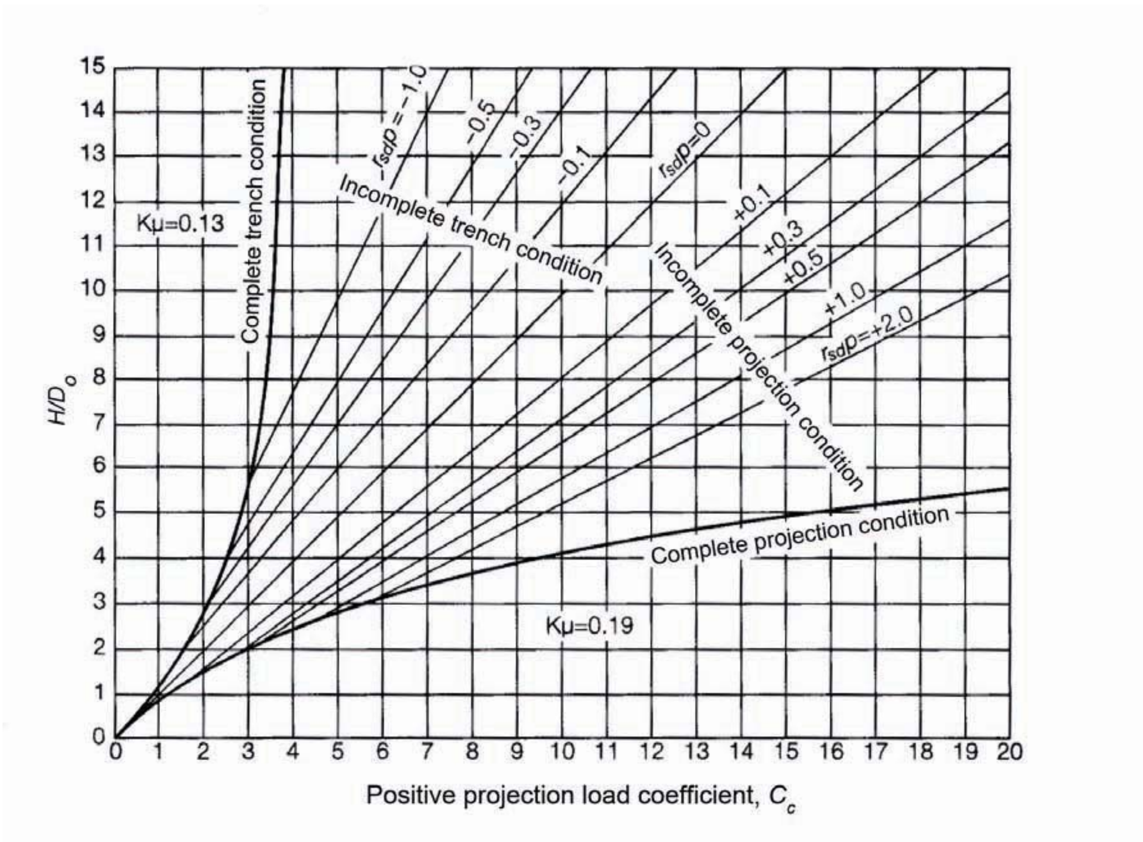


Figure 36.—Values for the positive projection load coefficient (C_c).

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

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.

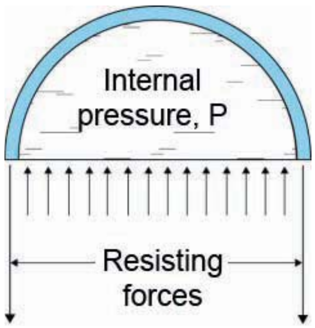


Figure 37.—Internal hydrostatic pressure.

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:

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

or

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

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:

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

or

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

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:

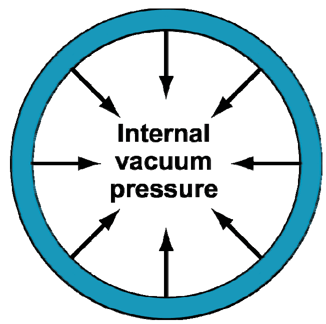


Figure 38.—Internal vacuum pressure.

$$P_V = \frac{12 \times W_V}{D_i} \qquad (2\text{-}15)$$

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.

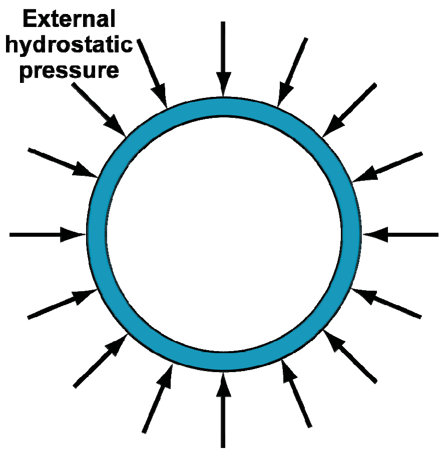


Figure 39.—External hydrostatic pressure.

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

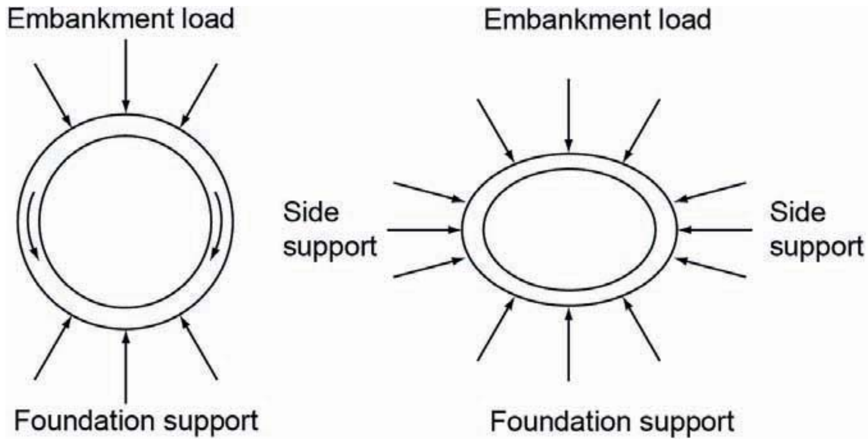
Flexible pipe design requires the load on pipe to be transferred to the soil surrounding the pipe. As the loading increases, the vertical diameter decreases and the horizontal diameter increases. Figure 41 illustrates the differences in load transfer for rigid and plastic pipe.

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

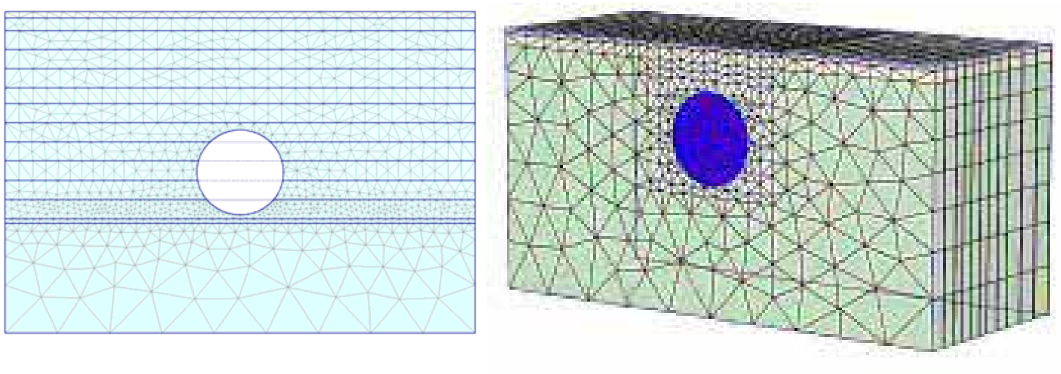
Figure 41.—Load is transferred differently for rigid and flexible pipe (Howard, 1996).

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:

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

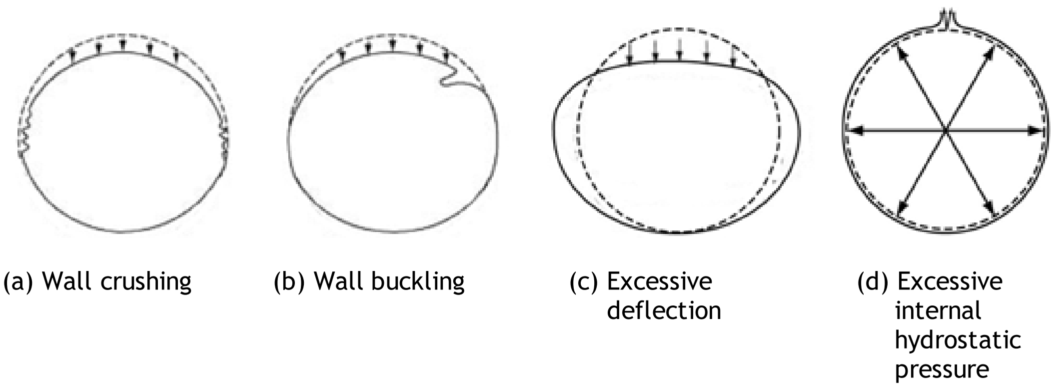


Figure 43.—Typical failure modes for flexible pipes.

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:

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

Corrugated single and profile wall pipe:

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

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 = Do/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 °F. 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:

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

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

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

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.

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

$$q_{ar} = q_a C \tag{3-22}$$

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.

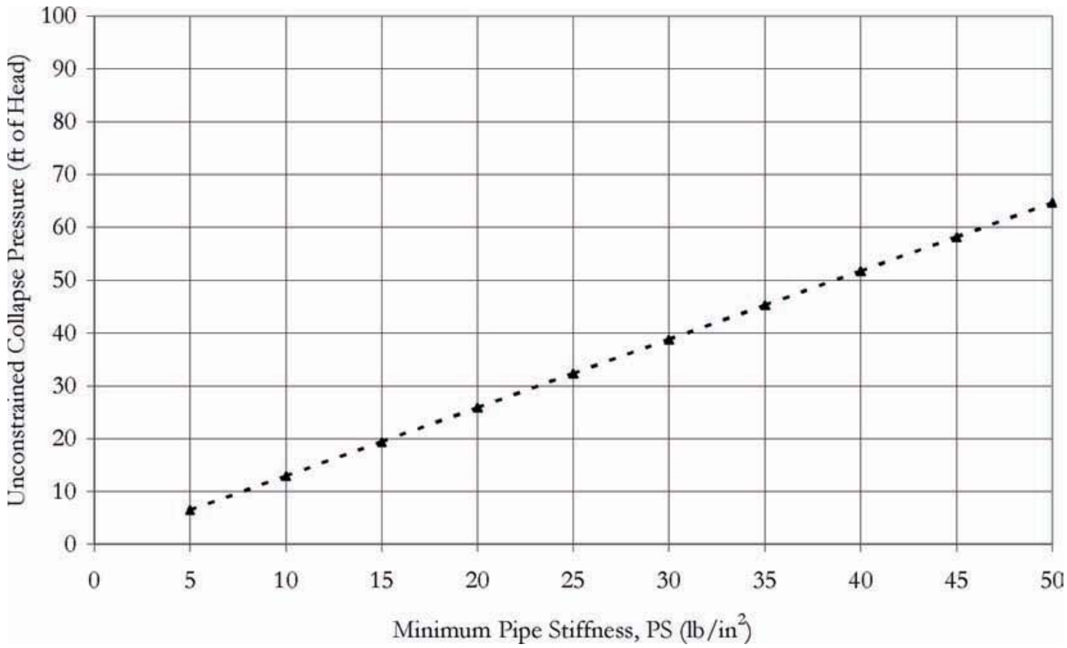


Figure 47.—Unconstrained collapse pressure vs. minimum pipe stiffness for single wall corrugated HDPE pipe.

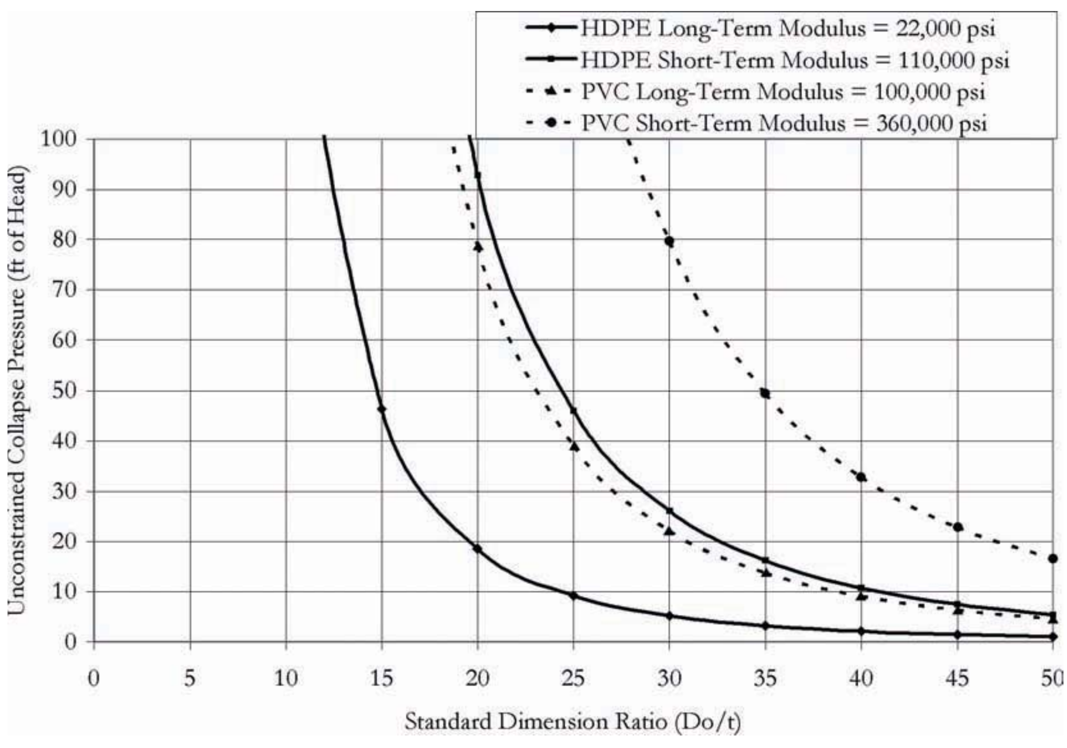


Figure 48.—Unconstrained collapse pressure vs. standard dimension ratio (SDR) for HDPE and PVC solid wall pipe. Plot is based on a minimum factor of safety of 1.0.

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 condui*t (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:

$$S_{EC} = E\alpha\Delta T \tag{3-23}$$

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.

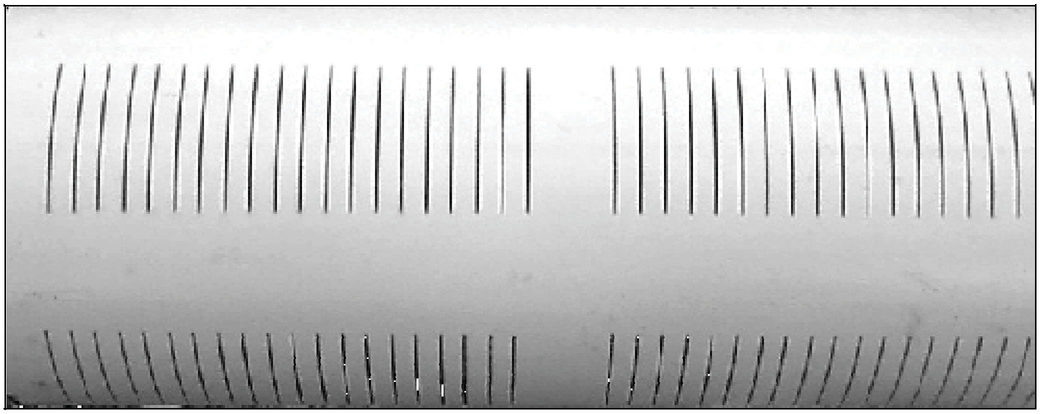


Figure 56.—Slotted PVC pipe.

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 10^{th} and 15^{th} 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:

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

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:

$$k(\mathrm{cm/s}) = C D_{15}^2 \tag{4-3}$$

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:

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

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.

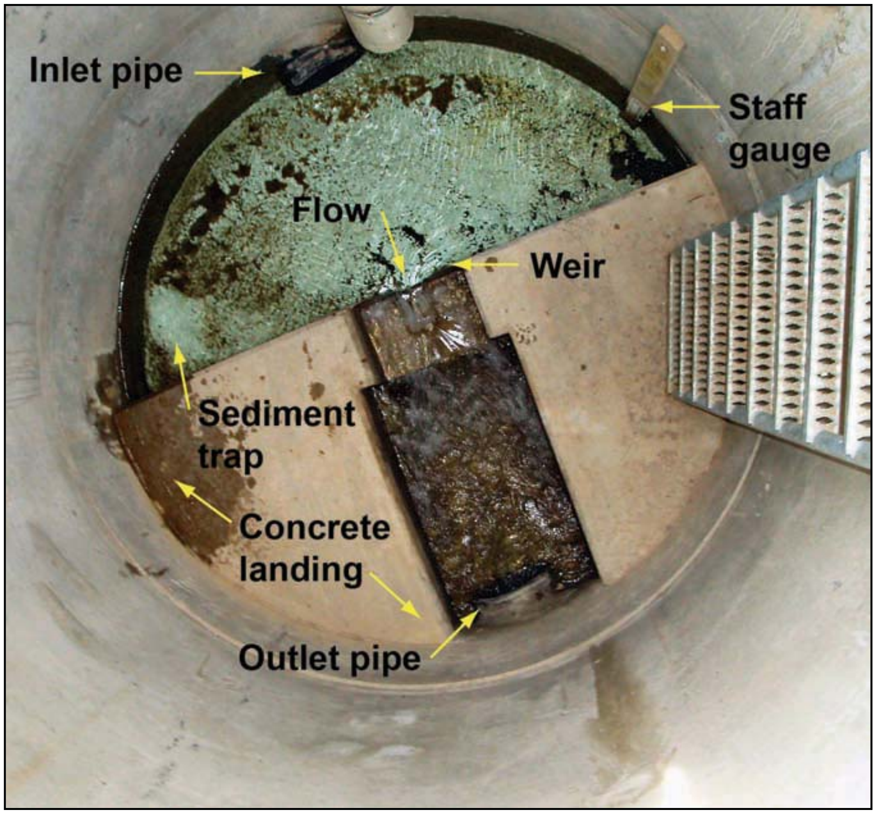


Figure 59.—Invert of an inspection well. The sediment trap is painted white, so any sediment can be easily observed. The dark material in the trap is algae.

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

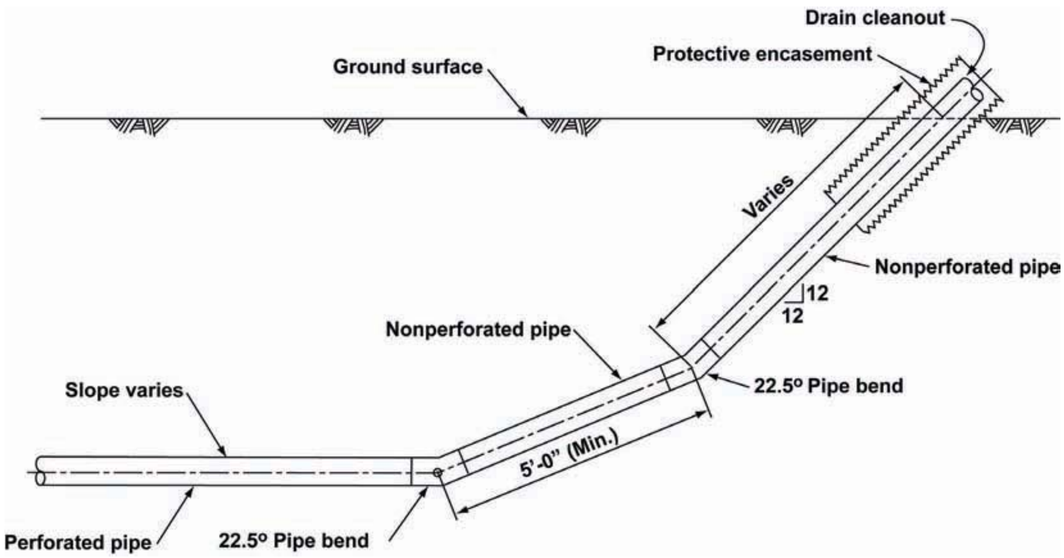


Figure 60.—Cleanout designed to accommodate CCTV inspection in pipes with diameters of 8 inches or larger.

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

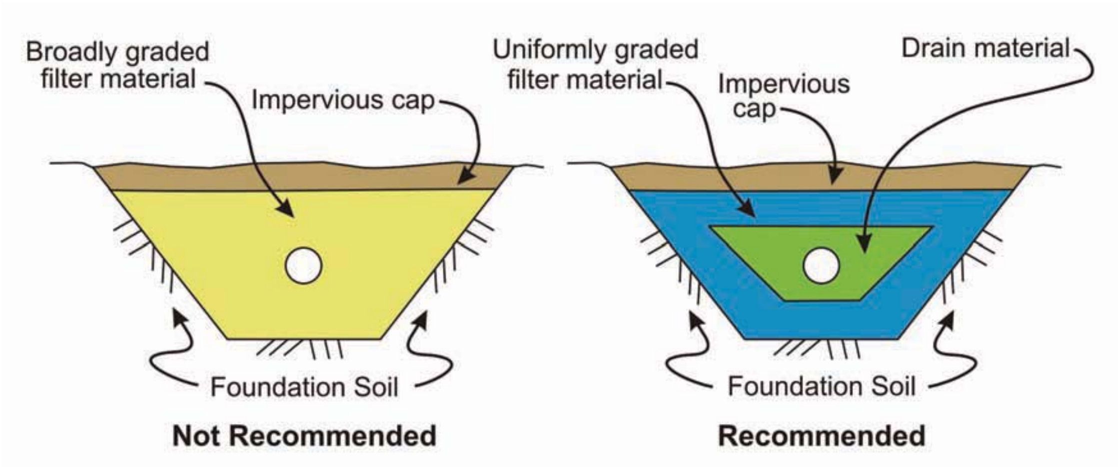


Figure 62.—Idealized cross sections of single (left) and double (right) stage drainpipes. The placement of the drain material around the pipe can result in a variety of geometries based on placement method. Minimum cover requirements should always be met independently of geometry.

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 ≤ 9 × D_{85}B			D_{15}F ≤ 4 × 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.

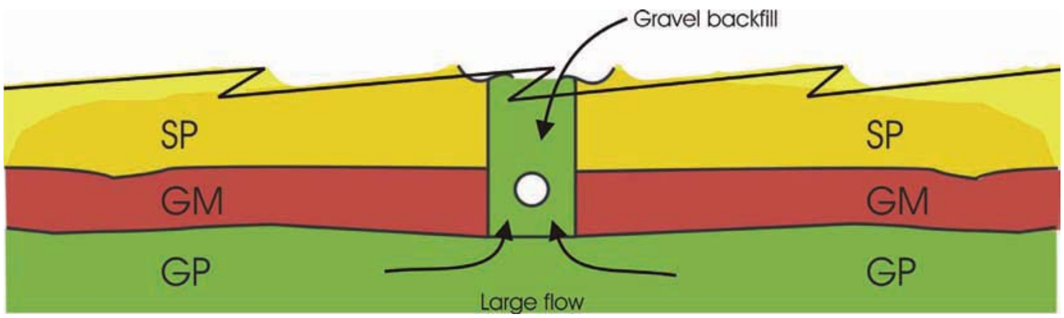


Figure 63.—Old drain.

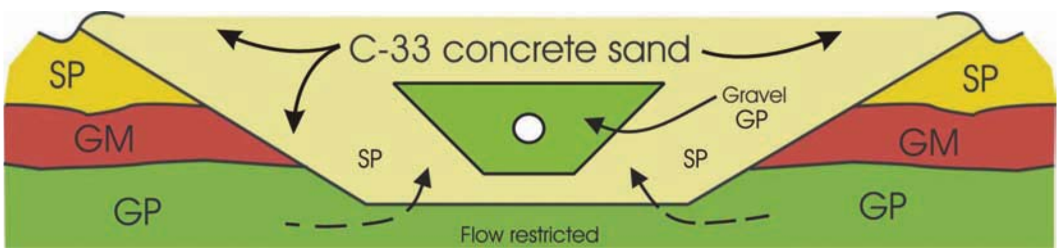


Figure 64.—Barrier condition introduced by a replacement drain resulting in poor seepage collection and high pore pressure.

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.